EVALUATION OF THE USE OF RECYCLED CONCRETE AGGREAGTE IN STRUCUTRAL CONCRETE

LIM WEE FONG

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Executive Summary

In response to the government's call for sustainable construction, there is a need to divert the construction and demolition (C&D) waste away from the landfills and an urgency to source for alternative materials to replace natural aggregates, which are mainly imported from neighbouring countries. One feasible solution is to produce structural concrete using recycled concrete aggregate (RCA) derived from C&D waste. Due to the fact that the characteristics of RCA are different from those of the natural coarse aggregate (NCA), the use of RCA in structural concrete has always been met with scepticism.

This report describes a research study that carried out to investigate the feasibility of using a proposed rational concrete mix design method to overcome the barriers that restrict its use in structural concrete. First of all, laboratory tests were performed to determine the properties of RCA and the results were either used to compare with the corresponding properties of NCA or to check for their conformities with the specifications and standards. Tests were carried out to measure its geometrical, mechanical & physical, thermal & weathering properties and chemical composition. Due to the presence of attached mortar, RCA tends to lose out in the mechanical and physical properties but gain in shape index and surface texture. On the other hand, RCA is stable volumetrically and it can also be considered as an inert material in a concrete mix since it will not react readily with the other concrete components.

Concrete mix designs were formulated for the full range of RCA contents (up to 100% replacement level) at different water-to-cement (W/C) ratios. A design slump of 125mm was used as the acceptance criteria for specimen preparation. For the same W/C ratio, the control mix (concrete mix with 0% RCA content) was used as a basis for comparison. The compressive strength test results illustrated that the use of RCA in low- and medium-strength concrete (between W/C ratio 0.67 and 0.35 respectively) has

negligible effect on their performance. Conversely, there was an insignificant decrease in compressive strength with the increase in RCA content for high-strength concrete (W/C ratio < 0.45).

Based on the results, concrete with W/C of 0.45 and 30%, 50% and 100% RCA content were utilized to produce specimens for the in-depth study. Tests were carried out to evaluate the hardened concrete properties which include the compressive strength, elastic modulus, flexural strength, indirect tensile strength, drying shrinkage, creep, drying and wetting, water absorbability, initial surface absorption, water permeability, chloride ingress and sulphate resistance. The test data obtained in this study demonstrated that comparable strength properties of RCA concrete were achievable with respect to the control mix. Though RCA had some influences in the drying shrinkage, creep and durability characteristics of concrete, the disparities were commonly confined within narrow bands. Thereby, these findings suggested that it is feasible to produce RCA concrete with the proposed rational method which can be used in structural applications.

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t_i	Instant immediately after loading in creep testing	90
$\varepsilon_c(t)$	Total creep strain at any time *t*	90
$\varepsilon(t)$	Total measured creep strain at any time *t*	90
$\varepsilon_i(t_i)$	Instantaneous elastic strain recorded immediately after loading	90
$\varepsilon_{sh}(t)$	Drying shrinkage strain at any time *t*	90
Ø	Creep coefficient at any time *t*	93
σ_c	Constant stress applied to the concrete for creep testing	93
$E(t_0)$	Elastic modulus of concrete for the instant t_0	93
t_0	Instant before loading for creep testing	93
K	Coefficient of permeability in metres per second	104
e	Depth of water penetration of concrete in metres	104
h	Hydraulic head in metres	104
t_p	Time under pressure in seconds for water permeability test	104
v	Fraction of the volume of concrete occupied by pores	104

Chapter 1 Introduction

1.1 Background

As a key segment of Singapore's economy, the construction industry plays an important role in providing the infrastructure and buildings to support the economic growth and social development. However, there is now global recognition of the importance of sustainable development, especially in the light of continued population growth and the adverse effects of climate change. Sustainable development means that economic growth and social development should be undertaken in a manner that safeguards the needs of the future generations. In fact, the construction industry has great potential to achieve the goal for sustainable development and to help to mitigate the effects of climate change by playing an active role in shaping a sustainable environment for Singaporeans, now and the future.

Under the Sustainable Singapore Blueprint, we recognize the need to optimize the use of natural resources, and to expand the renewable resources. This promotes resource efficient buildings design and the use of recycled waste materials, which is an important component of the sustainable development journey.

Among the waste materials, it has been estimated that about 1.5 million tonnes of C&D waste is generated annually. C&D waste is mainly the by-product of the construction activities such as demolition of buildings, concreting and road works. The disposal of the C&D waste posed a major environmental problem in land-scarce Singapore. This triggers a need to look into the possible use of C&D waste for more beneficial applications. On the other hand, lack of natural resources has resulted in a strong dependency on importing of natural aggregate, which is the main constituent for the production of concrete. As such, there is an exigency to source for alternative materials to replace the imported natural aggregate.

To alleviate these problems, plants are being set up to process the C&D waste in order to produce RCA, which contain mainly aggregates and cementitious materials. State-of-the-art equipments are used to process the C&D waste which involved crushing, screening as well as removal of ferrous metals and foreign materials. Nonetheless, there are limitations to the use of the processed waste and it is mostly utilized in low economical value applications such as backfilling materials for temporary road access in construction sites, fabrication of non-structural concrete precast products and construction of road sub-base. In fact, using RCA in structural concrete is another option whereby the application of C&D waste can be widened and its economical value greatly enhanced.

1.2 Objectives of study

In response to the government's directive towards sustainable development, Samwoh Corporation Pte Ltd (Samwoh), a leading integrated construction company and green products supplier embarked on an ambitious and forward-thinking demonstration project to build the first structural building in the region using concrete with up to 100% RCA content. Prior to the construction of building, a study, which is awarded the MND research grant by the Ministry of National Development of Singapore, was carried out by Samwoh, Building & Construction Authority (BCA) and Nanyang Technological University (NTU) to investigate the feasibility of using RCA in structural concrete. The objectives of the study are as follows:

a) To evaluate the properties of RCA used in this study
b) To formulate the concrete mix designs with a proposed rational method
c) In-depth evaluation of the properties of RCA concrete

1.3 Scope of works

In order to accommodate the above-mentioned objectives in this study, the experimental programme was outlined and carried out in three stages. The first stage details the investigation on the properties of RCA. The second stage describes the work carried out in the laboratory trials on the concrete design mixes and the last stage covers the laboratory testing on RCA concrete.

a) Stage 1 – Conformance of properties of RCA to the relevant standards/specifications

C&D waste can be specified and used with the same confidence as primary materials in a variety of low and high value applications. One of the main barriers often quoted about recycled materials in general relates to quality. To address this issue, relevant tests were carried out to verify the conformity of RCA against the standard requirements or the corresponding properties of NCA were used as a basis for comparison. As tabulated in Table 1.1, the qualities of RCA and NCA were evaluated based on the proposed key properties.

Table 1.1 Testing on properties of coarse aggregate

<table>
<tr><td>1) Constituent materials</td><td>8) Los Angeles abrasion</td></tr>
<tr><td>2) Gradation</td><td>9) 10% fine value</td></tr>
<tr><td>3) Particle density</td><td rowspan="3">10) Alkali-silica reaction
i) Chemical test
ii) Mortar bar test</td></tr>
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<tr><td>6) Aggregate crushing value</td><td>11) Total chloride content</td></tr>
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</table>

b) Stage 2 – Formulation of concrete mix designs for the incorporation of RCA

Various concrete mix designs were developed to encompass a wide range of W/C ratios, containing different RCA replacement levels. With the adoption of the proposed rational mix design method, NCA was directly substituted by RCA in accordance to the mass of coarse aggregate in a particular concrete mix. On the other hand, the contents for the rest of the constituent materials remained constant and considered as non-variables, except for the dosage of water-reducing agent. Trial mixes were performed to determine the exact dosages of water-reducing agent for the various W/C ratios, coupled with the full scope of RCA replacement levels (up to 100% RCA content). Slump value determined by slump cone test was used as the acceptance criteria for specimen preparation. After which, the compressive strength of the control mixes and RCA concrete were determined on 100mm cubes after water-curing for 3, 7 and 28 days. The test results were used as a basis for the selection of a suitable concrete grade, with the inclusion of various RCA contents, to be applied in the structural building. Making use of the chosen concrete mixes, specimens were prepared for further studies.

c) Stage 3 – In-depth evaluation of concrete with different RCA replacement levels

The control mix and RCA concrete were evaluated for their performance in term of engineering properties and durability characteristics. Table 1.2 highlighted the series of hardened concrete properties to be assessed in this study.

Table 1.2 Testing on hardened concrete properties

1) Compressive strength	6) Drying and wetting
2) Elastic modulus	7) Water absorbability
3) Strengths in tension i) Flexural strength ii) Indirect tensile strength	8) Initial surface absorption 9) Water permeability
4) Drying shrinkage	10) Chloride ingress
5) Creep	11) Sulphate resistance

Based on the scope of works, the research timeline was planned and presented in Table 1.3.

Table 1.3 Research timeline

Milestones	Oct 08 – Dec 08	Jan 09 – Mar 09	Apr 09 – Jun 09	Jul 09 - Sep 09	Oct 09 – Dec 09	Jan 10 – Mar 10	Apr 10 – Jun 10	Jul 10 -- Sep 10	Duration (Month)
1. Stage 1 – Conformities of RCA properties	■	■							6
2. Stage 2 – Formulation of concrete mix designs			■	■					6
3. Stage 3 – In-depth study of RCA concrete					■	■	■	■	12

1.4 Overview of thesis

Chapter 2 of the thesis cites the past research works related to the use of RCA in concrete which includes the properties of RCA, different mix designs to optimize the replacement level of RCA in concrete and its impacts on the properties of fresh and hardened concrete.

After which, the approaches deployed to achieve the objectives of this study are covered in Chapter 3. The formulation of concrete mix designs and procedures for specimen preparation are discussed in details. On the other hand, the test methods engaged to evaluate both the properties of RCA and the hardened concrete with the inclusion of RCA are clearly specified too.

In Chapter 4, the test results are presented and discussed in three main categories. Firstly, the results of the properties of RCA are underlined and compared to the corresponding properties of NCA. Next, the results of the strength properties, drying shrinkage and creep of the RCA concrete are analyzed. Lastly, the effects of RCA on the durability characteristics of concrete are highlighted.

Chapter 5 sums up the findings from this study with the research limitations and significance also being mentioned. Last of all, recommendations are stated for enhancements to the current research and extensions for the future research.

Chapter 2 Literature Review

Truly, the replacement of NCA with RCA in concrete has aroused great interests in different regions globally and extensive laboratory studies have been carried out to investigate the feasibility of this concept. Typically, the properties of RCA, the concrete mix designs with the incorporation of RCA, properties of fresh and hardened RCA concrete are evaluated and discussed in the publications. Based on the aforementioned categories, the study findings extracted from the research works done by others are compared and presented in this chapter. Thereby, the undesirable outcome can be avoided and the favourable results can be served as guidance to this research study.

2.1 Properties of RCA

2.1.1 Purity

RCA is defined as a relatively high quality material comprising mainly crushed concrete and some foreign materials. Magnets, air knives (directed blasts of air) and hand picking are usually used between the crushers to remove impurities (Dhir & Paine, 2007). BS 8500-2 (2006) defines the type of aggregates that are suitable for use in concrete and it also defines the compositional limits for RCA (Table 2.1). The standard allows certain amount of masonry, asphalt and other foreign materials to be present in RCA. Overall, the maximum allowable contaminant is 16.5% (by mass) and in other words, RCA must be predominately composed of at least 83.5% (by mass) of crushed concrete.

Various agencies and standardization bodies of different countries have been actively involved in the drafting of requirements of constituent materials for RCA. As shown in Table 2.1, the requirements listed in BS 8500-2 (2006) has a similar criteria for acceptance compared to Highways Agency (1998). As compared, EHE (2008) has a relatively stringent requirement with at least 90% of the constituent materials making up of crushed concrete. Among all, Japan has the least tolerance for the contaminants in recycled aggregate H, which is produced through advance processing, as less than 5% of foreign materials are allowed to be presence in the sample (Japan Industrial Standard (JIS), 2005).

Table 2.1 Requirements of constituent materials for RCA

Contaminant	Requirements (% by mass)			
	BS 8500-2 (2006)	Highways Agency (1998)	JIS A5021 (2005)	EHE (2008)
Max. masonry content	< 5%	< 5%	< 2%	< 5%
Max. fines	< 5%	< 5%	< 1%	< 1.5%
Max. lightweight material	< 0.5%	< 0.4%	< 0.1%	< 1%
Max. asphalt	< 5%	< 5%	Included in masonry	< 1%
Max. other foreign material	< 1%	< 1%	< 1.6%	< 1%
Max. acid-soluble sulphate (SO_3)	< 1%	Not quoted	Not quoted	< 0.8%

These deleterious substances that may be found in RCA are undesirable and harmful to the development of good bond between the aggregate and hydrated cement mortar or through the development of chemical reactions between the aggregate and cement mortar. The adverse effects of individual substances are briefly discussed (Sagoe-Crentsil & Brown, 1998).

- Asphalt content – Bituminous aggregate has a general effect of reducing concrete strength in a similar way to the effect of low-strength lightweight aggregate on concrete mix.
- Lightweight material – For example, wood and paper, may become unstable in concrete subjected to cyclic wetting and drying. Hence, lightweight material (≤ 1000 kg/m^3) is limited to less than 0.5% by mass in BS 8500-2 (2006).
- Acid-soluble sulphate content – Typically, the acid-soluble sulphate content of RCA falls in a range of 0.17-1.04% (Commonwealth Scientific Information and Research Organization (CSIRO), 2000).

Though the value exceeds the requirement specified in BS 8500-2 (2006) and higher than that for NCA, the sulphate may not be in a readily soluble form. The common sulphate-based products such as plaster, which contains gypsum, may give rise to expansive disruption of concrete. Up to 6% of gypsum contents have shown to give delayed ettringite formation in laboratory conditions (Dhir et al, 2001) whereas gypsum contents of 6% or higher would fail the acid-soluble sulphate content of 1%.

Buyle-Bodin & Hadjieva-Zaharieva (2002) found out that industrially produced RCA could contain various impurities, including negligible lightweight material, other foreign materials, 3.5% of asphalt and 7.1% of bricks which slightly exceeded the allowable limit for masonry content. Exteberria et al (2006) has proven that RCA produced from recycling plant could be of good quality with high content of crushed concrete (consists of 49% and 43% of original aggregate with and without adhered mortar respectively), 1.6% of ceramic and 5.3% of bituminous aggregate.

2.1.2 Particle size distribution

Neville (1995) pointed out that the main factors governing the desired aggregate grading are: the surface area of the aggregate which determines the amount of water necessary to wet all the solids, the relative volume occupied by the aggregate, the workability of the mix and the tendency to segregation. These factors represent the important characteristics of fresh concrete and affect also the properties of hardened state in terms of strength, shrinkage and durability. Thus, acceptance criteria should be established to ensure that RCA meets the statutory and regulatory requirements for particle size distribution before it is used for concrete batching. Dhir & Paine (2007) suggested that to achieve the most desirable grading curves and shape for RCA, a series of successive crushers and screening should be used; with oversized material returned to the respective crusher. Furthermore, the size of crusher openings and screening filters should also be controlled in order to obtain the required coarse aggregate grading for concrete. Most of the studies have managed to obtain the sieve analysis results which corresponded to the standard grading of concrete aggregate for respective regional requirements (Limbachiya et al, 2000; Sagoe-Crentsil et al, 2001; Buyle-Bodin & Hadjieva-Zaharieva, 2002; Topcu & Sengel, 2004; Yang et al, 2008).

2.1.3 Volume of attached mortars

The amount of surrounding mortar will vary depending on the method by which RCA was produced. The size of RCA is a critical factor in determining the amount of mortar adhered to the particles. Hasen & Narud (1983) investigated the volume percentage of old mortar which was attached to gravel particles in each grade and size fraction of RCA on a representative number of samples by means of a linear traverse method, similar in principle to the method which was described in ASTM C457-71. The test results have shown that the mortar contents were about 30%, 39% and 60% by volume for the size fraction of 16-32mm, 8-16mm and 4-8mm respectively. It might be inferred that a recycled and graded coarse aggregate would contain an average of perhaps 40% by volume of mortar from old concrete, which comprised about 41% to 51% of mortar by volume, and the quality of the original concrete has negligible effect on the volume of attached mortar. Ravindrarajah & Tam (1985) determined the mortar contents by means of an electronic digital planimeter, the four exposed surfaces were traversed to measure the areas of mortar and granite from the original concrete. Based on the method adopted, RCA was found to contain an average of 50% by volume of mortar from the original concrete, which contained about 42% to 46% of mortar by volume. Significant values for the coefficient of variation indicated the randomness of the distribution of mortar content in RCA. Buyle-Bodin & Hadjieva-Zaharieva (2002) highlighted that the average proportion of old cement mortar is 28±5% with the fine fraction presented a higher proportion than the coarse fraction. Yang et al (2008) reported that the content of old cement mortar on the surface of recycled aggregate was 2.2% by mass of original recycled aggregate. The value was the difference between the weight of original recycled aggregate and the weight of recycled aggregate immersed in hydrochloric acid solution for 24 hours expressed over the weight of original recycled aggregate.

The substantial difference in the volume of attached mortar among the studies may be attributed to the type of crushers deployed and scale of crushing. Both Hasen & Narud (1983) and Ravindrarajah & Tam (1985) were only using jaw crusher in laboratory scale to obtain the RCA for the preparation of concrete specimens. Conversely, the RCA used by Yang (2008) was derived from the commercial processing plant whereby C&D waste was properly crushed using an impact crusher. Furthermore, dust in RCA was removed by an air blower and cleansing process was iteratively carried out until achieving the required quality of RCA for structural applications. In fact, Waste Management & Recycling Association of Singapore (WMRAS) and Waste & Resources Action Programme (WRAP) of United Kingdom suggest that in order to produce a quality aggregate product, it is important to identify the appropriate production processes and equipment required. Typically, a proper recycling plant should be equipped with input, crushing (such as jaw, cone and impact crusher), sorting, transfer, output and separation device.

2.1.4 Density

The porosity of RCA increases significantly with the amount of coated mortar over the particles. Hence, the specific gravity and bulk density of RCA are relatively lower when compared with corresponding properties of NCA. Ravindrarajah (1987) found out that specific gravity value is between 2.36 and 2.44 for RCA as compared to 2.67 for NCA, which was about 13.6% more than RCA. Results indicated that RCA has a lower specific gravity in saturated surface-dry (SSD) condition by 9% as compared to that of NCA in SSD condition. It was also noted that the percentage reduction in specific gravity in SSD condition was more than the percentage reduction in apparent specific gravity. This might be due to the greater porosity of RCA (Bairagi et al, 1993). Chen et al (2002) recorded a specific gravity in SSD condition of 2.28 for RCA, which was 13% less than that of NCA. The bulk density, ranging from 1.241 to 1.252, was also lighter than that of NCA. Padmini et al (2009) has verified that due to the higher surface area available for equal volume of aggregate, the presence of parent mortar was more for smaller sized RCA which resulted in higher reduction in specific gravity; whereas for granite aggregate, there was no variation in its specific gravity for various aggregate sizes, namely 10mm, 20mm and 40mm. On the other hand, RCA, with a range of specific gravity value of 2.38 to 2.56, was much lighter than granite with a specific gravity of 2.8.

2.1.5 Water absorption capacity

The major difference between RCA and NCA is the presence of old cement mortar attached to the aggregate. The porosity of adhered cement mortar directly influences the properties of aggregate. In general, the water absorption capacity of RCA is highly variable and greater than 1% whereby NCA rarely contains more than this amount of surface moisture. Topcu & Guncan (1995) reported that there was a significant difference between the water absorption capacity of RCA and NCA with a value of 7% and 1.5% respectively. However, Limbachiya et al (2000) observed that the water absorption capacity of RCA was only 2 times higher than that of NCA, i.e. 5% for the former and 2.5% for the latter. Ryu (2002) investigated five different types of recycled aggregate and the water absorption capacities fell within the range of 3.3-5% which were much higher than the corresponding property of NCA with a value of about 1%. Shayan & Xu (2003) determined that the water absorption of RCA to be about 5% in SSD condition as compared to that of crushed basalt of 1% and this trend coincided with the findings of other researches. Levy & Helene (2004) obtained a slightly higher water absorption capacity in SSD condition for RCA with a value of 5.6% but relatively lower absorption capacity in SSD condition for granite (0.8%). In general, RCA has about 2 to 7 times higher water absorption capacity than NCA.

Though there is hardly any requirement regarding the water absorption of aggregate, it should be necessary to establish new specifications to aggregate, especially RCA, used in the blend. The research published by Building Contractors Society of Japan (BCSJ, 1978) pointed out that the water absorption capacity for coarse aggregate should be in the range of 3.6-8%. On the other hand, EHE (2008) suggested that the water absorption capacity of coarse aggregate for structural concrete application should be capped at 5%. In Japan, it has a very stringent performance-related requirement whereby the water absorption capacity of high-quality recycled aggregate H for concrete should not exceed 3% (JIS, 2005) while RILEM (1994) also recommended the same maximum water absorption capacity for RCA which contains at least 80% of NCA. Nonetheless, in Hong Kong, the water absorption rate for RCA has a wide scope, ranging from maximum 5% for structural concrete to maximum 10% for other applications (Tam et al, 2007).

2.1.6 Particle shape and texture

For the crushed NCA, the particle shape is influenced by the nature of the parent material, type of crusher and its reduction ratio, i.e. the ratio of the size of material fed into the crusher to the size of the finished product (Neville, 1995). Particle shape is related to two relatively independent characteristics: angularity and sphericity.

- Angularity is a function of the relative sharpness of the edges and corners of the particles. It can be defined numerically but it is normally expressed descriptively: rounded as opposed to angular.
- Sphericity is a function of the ratio of the surface area of the particle to its volume. It can also be defined numerically or expressed in descriptive terms on the basis of the relationship between the particle

dimensions: equidimensional, elongated and flaky. In practice, the Flakiness Index and Elongation Index, in accordance to BS 812-105.1 (1989) and BS 812-105.2 (1990) respectively, are usually used to determine the amount of elongated or flaky particles within a particular sample. It is essential to minimize the presence of elongated or flaky particles in concrete batching as there is a tendency for flaky aggregate to be oriented in one plane, with bleeding water and air voids forming underneath, which will affect the durability of concrete. On the other hand, elongated particles tend to be more brittle and affect the strength of concrete.

In the case of RCA, the key factor to produce good particle shape is the crushing processes. Due to the several stages of crushing and the use of impact crushers, RCA has a more angular shape, which means lower Elongation and Flakiness Index, than NCA and this is beneficial for the performance of concrete. Bairagi et al (1993) proved that RCA has fairly low elongation index (19) and flakiness index (12) as compared to that of NCA with a value of 30 and 19 respectively. Ravindrarajah (1987) and Topcu (1997) agreed with the results by reporting that RCA was more angular in shape than NCA. At the same time, Limbayachi et al (2000) and Gutiérrez & Juan (2004) also highlighted that RCA has an equidimensional and cubical shape compared to NCA.

Apart from the particle shape, the surface texture of aggregate is also an important physical property as it affects the bonding between the aggregate and cement mortar. Nevertheless, there is no recognized method of measuring the surface roughness and usually, it is determined by visual estimation with good reliability (Neville, 1995). The surface texture of NCA is usually rough, granular or crystalline in nature. Most of the researches have made the same visual observations on RCA, revealing that it has generally rough and porous surface texture due to the presence of a coating of mortar over the granite particles (Ravindrarajah & Tam, 1985; Bairagi et al, 1993; Topcu, 1997; Limbachiya et al, 2004).

Both the shape and surface texture of aggregate influence considerably the strength of concrete, especially the flexural strength rather than the compressive strength and the effects of shape and texture are particularly significant in the case of high-strength concrete (Neville, 1995). Thus, aggregates with good physical properties are desired so that the workability of fresh concrete as well as the engineering properties of hardened concrete is not badly affected.

2.1.7 Mechanical properties

The strength and other related characteristics of aggregate particle are dependent upon the properties of the constituent minerals, bonding between the grains and the porosity of the particle. Unfortunately, there is no truly satisfactory test for measuring the strength of the individual particle and the required information has to be obtained usually from indirect tests: crushing value of bulk aggregate and force required to produce ten per cent fines from the bulk aggregate. In addition to strength, toughness and hardness are the other mechanical properties which are of interest. Toughness is defined as the resistance of a sample of rock to failure by impact and the impact value test is usually adopted to determine the degree of fragmentation of aggregate under impact load. However, these crushing values can only be served as a useful guide as there is no direct correlation between the results and the performance of the aggregate in concrete or the strength of concrete. On the other hand, hardness is defined as the resistance to wear and the Los Angeles (LA) abrasion test is frequently used to determine the abrasion and attrition of aggregate subjected to a charge of steel balls in rotating drum. These test results show good correlation not only with the actual wear of aggregate when used in concrete, but also with the compressive and flexural strengths of concrete made with the given aggregate (Neville, 1995; Young et al, 1998).

In the case of RCA, all the researchers have made the similar observations with higher aggregate crushing value (ACV), aggregate impact value (AIV) and LA coefficient but lower ten per cent fine value as compared to NCA. The prime reasons for this uniform trend were mainly due to the weak mortar component adhered on the surface of aggregate and the poor mortar-aggregate bond in RCA. However, the extent of inferiority in the mechanical strengths of RCA varies among the studies and this might be attributed to the manufacturing and processing of the C&D waste.

Hansen & Narud (1983) accounted that the ACV and LA coefficient of RCA ranged from 20.4% to 29.6% and 22.4% to 41.4% respectively while the same properties for natural gravel were in the range of 14.5-21.8% and 18.8-25.9% respectively. On average, both ACV and LA coefficient of RCA were 40% higher than that of natural gravel. Ravindrarajah (1987) showed that the ACV, AIV and LA coefficient of granite were 16.9%, 14.6% & 18.1% respectively whereas the relevant properties for RCA ranged from 25% to 31%, 20% to 33% and 28% to 40% respectively. RCA was at least 50% higher in ACV as well as about 90% higher in AIV and LA coefficient compared to granite. In contrast, Bairagi et al (1993) observed that RCA, with ACV, AIV and LA coefficient of 23%, 29% and 27% respectively, was just about 20% to 30% higher in values when compared with NCA, having a value of 19%, 23% and 21% in ACV, AIV and LA coefficient respectively. Likewise, the attached cement mortar on aggregate has resulted in a lower ten per cent fine value for RCA. Limbachiya et al (2004) reported that the ten per cent fine value of RCA (160kN) was relatively lower than gravel (289kN) by 45%. The evaluation by Kou & Poon (2008) revealed that the ten per cent fine value of RCA ranged between 110 and 145kN while granite has a value of 159kN for the similar property and the difference was about 20%. Despite the negative outcomes on the mechanical properties of RCA, most of the test results indicated that RCA was satisfactory in its quality as specified in standards and should be accepted for concrete production.

2.1.8 Chemical contents

Although aggregate is commonly considered to be an inert filler in concrete, such is not always the case. Certain aggregates can react with Portland cement, causing expansion and deterioration. The most common reaction is that between the alkalis (Na_2O and K_2O) from cement or from other sources with hydroxyl and certain active siliceous constituents that may be present in aggregate (ACI Committee 201, 1977). Since RCA contains alkali-rich hydrated cement, there is a tendency for the expansive alkali-silica reaction (ASR) to take place in the Portland cement concrete containing RCA. Laboratory tests should be performed on RCA to evaluate its potential in the occurrence of ASR when in contact with cement. Among the available test methods, the following useful test procedures are recommended.

- Petrographic examination: ASTM C295 (2008) provides a list of the types of minerals involved in ASR and describes the procedures to recognize these constituents. Recommendations are available which show the tolerance levels of reactive minerals as determined petrographically.
- Mortar bar test: ASTM C227 (2010) is the one most generally relied on to indicate potential alkali reactivity. Acceptance criteria are given by ASTM C33 (2008) for evaluating these test results. The procedure is useful for both the evaluation of aggregates and of specific aggregate-cement combinations.
- Chemical test: ASTM C289 (2007) is used primarily for a quick evaluation and the results being obtainable in a few days as compared to three to six months or more with the mortar bar test.

Acceptance criteria for this test are given in ASTM C33 (2008) or elsewhere.

- Length change: ASTM C1293b (2008) is generally considered the most accurate and effective test in predicting the field performance of aggregates. In this test, the specimen is cast with equivalent total alkali content in the concrete mixture of 5.25 kg/m^3. However, this relatively long period (1 to 2 years) for conducting the test has been the major drawback for the test and has limited its use somewhat.

Researches have shown that in most cases, total equivalent sodium oxide, Na_2Oeq, values for Portland cement concrete containing RCA, which was evaluated using ASTM C1293b (2008), were below the recommended limit of 3.5kg/m^3 (Dhir & Paine, 2004). As a result, RCA could be regarded as a normal reactivity aggregate.

RCA may be contaminated with residual quantities of sulphates from contact with sulphate-rich soils or plasterboard inclusions. Some of the standards have specified that the maximum permissible limit for acid-soluble sulphate content of RCA to be less than 1% (BS 8500-2, 2006; CSIRO, 2000). Gutiérrez & Juan (2004) demonstrated that the acid-soluble sulphate content, ranging from 0.1% to 0.42%, was within the limit as specified in the standards. Even for the total sulphate content of RCA, ranging from 0.15% to 0.58%, it was still less than the recommended limit.

Besides, excessive chloride ions may be present in RCA due to the marine exposure of demolished structures or the use of marine/estuarine aggregate in original concrete. Chloride contributions from RCA for use in this calculation are measured by an acid-soluble test (BS EN 1744-5, 2006) that provides a worst-case value (Dhir et al, 2001) and probably overestimates the availability of chlorides – thus providing a margin of safety (CEN/TC 154/SC 2, 2004). CSIRO (2000) reported that the typical chloride content of RCA was between 0.002% and 0.011% by mass of

RCA and it also suggested that a maximum chloride ion content of 0.15% by mass of RCA can be applied to ensure satisfactory concrete quality. Likewise, Gutiérrez & Juan (2004) has a close observation of the total chloride ion content with a value between 0.001% and 0.005%.

In general, the degree of contamination and potential reactivity of RCA should not exceed limits permitted for NCA. In addition, RCA must neither contain reactive contaminants nor react with cement or reinforcing steel.

2.2 Concrete mix designs

For the design of concrete mixes incorporating RCA, the starting point is a mix containing NCA only. This may be a well established mix design in use at a commercial mixing plant or it may be a specifically designed mix. The aim is to produce a mix in which a proportion of NCA is replaced with RCA (The Highway Agency et al, 2007). Most importantly, the same design principles and application rules as those stated for conventional concrete should also be applied to RCA concrete. Nevertheless, due account must be taken in mix design since it is a known fact that the properties of RCA are somewhat different from those of NCA. In view of this, numerous mix design methods have been proposed by researchers with the attempts to demonstrate that other than the use in non-structural concrete, applications of RCA could be enhanced with the production of structural concrete. Ultimately, the researchers aimed to achieve equivalent performance of RCA concrete compared to control mix.

Due to the absorptive nature of RCA, the workability of fresh concrete is badly affected. The simplest and most direct way to address this issue was to increase the free water content by 5% to 10% (Casuccio et al, 2008; Gomes & Brito, 2009; Tabsh & Abdelfatah, 2009). As a result, the rest of the hardened concrete properties were changed implicitly. In turn, a proportional increase in both cement and water content were proposed so as to resolve the concern in workability and at the same time, to preserve the hardened concrete properties, especially the mechanical strength (Ravindrarajah & Tam, 1985; Sagoe-Crentsil et al, 2001; Fong et al, 2004). However, the additional cement content could affect the long-term stability of concrete such as shrinkage and creep. These properties were mainly influenced by the volume of the hydrated cement mortar rather than the role of aggregate in concrete being primarily that of restraint. Though different methods were being suggested to deal with this distinctive property of RCA, the past works did not discuss in details regarding the other key factors that affect the effective W/C ratio of concrete. These include the moisture state of aggregates prior to batching and the time allowance allocates for the extra free water content to be absorbed fully by RCA to achieve SSD state.

As to promote the practical application of RCA in concrete production, Poon et al (2004) carried out a study aims to increase the insight effects of the moisture state of RCA on fresh and hardened concrete properties. In the study, the moisture states of NCA or RCA were controlled at three states, i.e., air-dried, oven-dried and SSD. The results showed that oven-dried aggregate led to a higher initial slump and high rate of slump loss while air-dried and SSD aggregates did not exhibit any abnormality in initial slumps and slump losses. On the other hand, RCA concrete prepared with air-dried aggregate acquired the highest compressive strength. Whilst RCA containing SSD aggregate imposed the largest negative effect on concrete strength, which might be attributed to 'bleeding' of excess water in the pre-wetted aggregates in the fresh concrete. Poon et al (2004) concluded that for large scale of production of normal-strength RCA concrete, the optimal moisture condition of aggregates is in air-dried moisture state. The RCA in SSD state should only be used in special situation.

Unlike cement, aggregate and water, chemical admixtures are not an essential component of the concrete mix but they are an important and increasingly widespread component in modern concrete technology. Their main influence is on the properties of the fresh concrete whereby the rates of the early hydration reactions are altered or the rheological properties of the fresh concrete are regulated. Chemical admixtures are commonly classified by their function in concrete. Among these classes, high range water-reducing agent (also known as superplasticizer) is readily applied in RCA concrete, owing to its ability to improve the workability at a given W/C ratio. Superplasticizer, ranging from 1% to 3% by mass of cement, has been utilized in some research works and it was proven to be effective in achieving the designed slump values without increasing the water content (Park, 2001; Juan & Gutiérrez, 2004; Kheder & Al-Windawi, 2005).

Other than using the chemical admixtures, the researchers have shown great interests in the use of blended Portland cement with mineral additive to improve the performance of RCA concrete. The mineral additive content can be in the range of 20% to 70% of the cementitious material in the concrete. Most of the additives used are byproducts of industrial processes and the motivation for their application is economical, ecological and technical. The most commonly used mineral additives are:

- Fly ash (FA) – Byproduct obtained in the burning of coal for energy production.
- Ground granulated blastfurnace slag (GGBS) – Byproduct obtained in the production of steel.
- Silica fume (SF) – Byproduct obtained in the production of silicon metal or ferrosilicon alloys.

Levy & Helene (2004) made use of blended Portland cement, consisted of 35% blastfurnace slag, to prepare concrete specimens containing different RCA contents. Though the water absorption and total pore volume did not show favourable results, the carbonation depth of 100% RCA concrete was comparable with the control mix. This behaviour justified that carbonation depth depended strongly on the chemical composition of the concrete and not only on the physical aspects. Ting (2004) replaced 20% of the cement with fly ash in the FA-series recycled aggregate concrete. Poorer performances were observed in the mechanical properties with the inclusion of fly ash as compared to the other two series without fly ash. This observation was expected as the mechanical strength tests were carried out at 28 days and the effects of fly ash were expected to be better for longer time frame. Nevertheless, the sorptivity test indicated that a lower permeability was acquired as compared to the other two series without fly ash. Li et al (2009) presented the strength results for various combination of Portland cement with blastfurnace slag, fly ash and silica fume. Among the mixes, combination of Portland cement with fly ash and silica fume were more effective to produce higher strength RCA concrete which was primarily attributable to the higher packing density. In general, a satisfying strength could be achieved with the inclusion of the pozzolanic powders.

2.3 Properties of fresh concrete

Although fresh concrete is only of transient interest, its workability is vital as it will determine the degree of compaction and the extent of surface finishability. In turn, the former will seriously affect the strength of concrete of given mix proportions due to the presence of voids while the latter will decide on the permeation and durability of concrete as well as the aesthetic aspects of the finished products. For a given mix proportion, the primary factors that influence its workability are mainly the W/C ratio, aggregate-to-cement ratio, maximum size of coarse aggregate and water content. In practice, the slump cone, compacting factor and Vebe test are widely adopted to evaluate the workability of a mix proportion. Among the tests, the slump cone test is used extensively in site work as it gives a rapid and reliable workability of a mix of given nominal proportions.

In addition, the rate of slump loss is another essential property to be taken note of as there is always an interval between the mixing and placing of the concrete. The free water in the mix is reduced progressively due to the evaporation of mixing water, absorption into the aggregates, hydration process and the concomitant formation of hydration products. Climate is the external contributing factor for the rate of slump loss and it is of greater concern in hot climates. Nonetheless, the retarder is usually blended with the concrete mix to overcome the slump loss issue through the delay of the settling of concrete by slowing down the early hydration reaction.

In consequence of the properties of RCA, it will always pose a challenge to incorporate RCA in a concrete mix. Due to its absorptive nature, fairly low slump values of RCA concrete, without adjustments to the mix proportions, are definitely a major setback in application. Dhir et al (1998) observed a downward trend in workability with an increase in RCA content and concrete with 100% RCA content as aggregate has a slump value typically 10 to 15% lower than that of control mix. Moreover, the loss of workability, which was measured by compacting factor test, was of a uniform nature for the control mixes and the uniformity was relatively less significant for concrete produced with 100% RCA. Poon et al (2007) showed that the initial slump of RCA concrete was significantly affected by the moisture condition of aggregates. Higher initial free water content was required to achieve the targeted slump values with the increase of the replacement level of RCA used in a SSD state. Yang et al (2008) reported that the initial slump of fresh concrete slightly decreased with the increase of the replacement level of RCA. However, as the replacement level of RCA increased, higher water absorption was expected and in turn, the workability of fresh concrete was reduced with the elapse of time. Hence, it could be concluded that the type and replacement level of RCA have much more significant effects on the loss in workability than the initial slump of fresh concrete.

2.4 Engineering properties of hardened concrete

2.4.1 Compressive strength

The strength in compression is the most important engineering property of concrete because it not only reflects its mechanical quality but also qualitatively related to other desirable characteristics. Thus, the influence of RCA on the compressive strength of hardened concrete is crucial as it is always associated with other properties, especially the durability characteristics. As a result, extensive works have been carried out to investigate the compressive strength of concrete with the incorporation of RCA.

Bairagi et al (1993) highlighted that the maximum reduction in strength was 15% in which up to 50% of NCA was replaced by RCA. Correspondingly, when the replacement level was greater than 50%, the maximum loss in strength was around 40%. Topcu & Guncan (1995) also reported that there was a reduction in strength by about 40% at 30% RCA replacement level and increased further to more than 80% when RCA was used a sole aggregate in the concrete. However, with a better understanding of the behaviour of RCA over the years, it is possible to design the concrete containing low dosage of RCA without affecting its compressive strength. Dhir et al (1998) suggested that the cut-off point for RCA replacement level was around 30% by mass of coarse aggregate in a particular concrete mix. Both Price (2002) and Limbachiya et al (2004) agreed that the effect of RCA on strength was insignificant at levels lesser than 30% by mass but thereafter a gradual reduction occurred with increasing RCA content. Dhir et al (2001) also attempted to establish ceiling strength for various replacement levels of RCA at different W/C ratios. The general trend indicated that up to 30% RCA content has no effect on ceiling strength but as the RCA content increased beyond that, a reduction in strength took place. In addition, for concrete made with 100% RCA, the start of the ceiling strength effects were observed at a slightly higher W/C ratio than that containing up to 30% RCA. Nevertheless, the factors that arose to such improvement in the compressive strength of RCA concrete are unclear since the literature did not discuss in details.

On the other hand, as there are always variations with the incoming wastes, the quality of RCA is usually doubtful as the mix proportions of the original concrete are unknown. In view of this issue, studies have been carried out to evaluate the effect of variations in the strength of original concrete on the performance of hardened concrete. Dhir et al (1998) reported that there was insignificant variation in strength for a given RCA content when concrete was produced using RCA obtained from different sources. Likewise, Kou & Poon (2008) observed a similar trend with almost identical compressive strength test results obtained for concrete containing different sources of RCA at different replacement levels. Furthermore, in order to ease the concern over the long-term strength development of RCA concrete, the compressive strength test was performed on the concrete specimens at stipulated intervals for a period of 5 years. When compared with the control mix, the compressive strengths of the three types of RCA concrete were reduced by about 17% to 22% at 28-day. Nonetheless, the gain in strength by RCA concrete was far more outstanding than that of control mix after 5 years. In short, the difference in the strength of original concrete would not affect the long-term strength development of RCA concrete.

Nonetheless, at full replacement level, Hansen & Narud (1983) found out that low-strength concrete of uniform quality could always be produced from RCA, regardless of the strength of the original concrete from which RCA was derived. Conversely, RCA of non-uniform strength would create large fluctuations in the compressive strength test results of medium- and high-strength structural concrete. Ryu (2002) made the same observations whereby the strength characteristic of the concrete was not affected by the strength of RCA for high W/C ratio but the compressive strength reduced significantly for low W/C ratio of the concrete blended with different types of RCA.

Therefore, it could be concluded that the source and strength of original concrete has negligible effects on the strength of RCA concrete. In fact, it was the mix proportions and the replacement levels of RCA which governed the compressive strength of hardened concrete.

2.4.2 Elastic modulus

In material, a strain is always associated with any stress. For concrete, the relation between stress and strain over their full range is of vital interest in structural design. Like any other structural materials, concrete exhibits a certain degree of elasticity. In practice, since concrete is loaded to a range of about 40% of its strength, it is common to define its modulus of elasticity in terms of its secant modulus of elasticity, which is taken to this range. Given the fact that this property is affected by the modulus of elasticity of the aggregate and by the volumetric proportion of aggregate in the concrete, the substitution of NCA with RCA in concrete will raise a concern over its elasticity.

Bairagi et al (1993) investigated the effect of different replacement levels of RCA on the elastic modulus of concrete at three different W/C ratios. For the same W/C ratio, all the stress-strain curves for different replacement levels of RCA followed similar trends but the curvature of each curve progressively increased with increase in replacement level and thus, it reflected that there was a reduction in modulus of elasticity for higher replacement level. However, the maximum drop in elastic modulus was about 30% when NCA was fully replaced by RCA and it was a general trend that observed with the different W/C ratios. Here, it clearly illustrated that the quality of aggregate, rather than the W/C ratio, was the contributory factor for the fluctuations in elastic modulus of concrete. Juan & Gutiérrez (2004) noted that elastic modulus was one of the most affected properties of RCA concrete, even with low replacement levels. With the same mix proportions, concrete produced with 20% to 100% RCA content has given the average values of 10% to 40% lower in modulus of elasticity than the corresponding property of control mix. Kheder & Al-Windawi (2005) examined the relationships, in term of elastic modulus, among the control mix, concrete made of RCA and mortar. For all the W/C ratios investigated, the elasticity moduli of the control mix and RCA concrete exceeded that of mortar by about 40% and 10% respectively;

whilst the modulus of elasticity of control mix was about 20% to 25% higher than that of RCA concrete. Kou & Poon (2008) replaced the NCA with three different sources of RCA in concrete and the test results indicated the quality of original concrete has minor influence in the elastic modulus of concrete. However, there was a reduction in elasticity as the percentages of RCA increased and as compared to the control mix, the elastic modulus of 100% RCA concrete was lower by about 23%. On the other hand, there was a significant gain in elastic modulus from 28 days to 5 years for all the mix proportions, ranging from about 20% to 40%. Interestingly, RCA concrete has a higher rate in gaining as compared to that of control mix and in fact, it was the concrete with 100% RCA replacement experienced the largest gain in elastic modulus over 5 years.

2.4.3 Strength in tension

Concrete is usually assumed to be about 10% as strong in tension as it is in compression. For building structures, concrete is not normally designed to resist direct tension as the resistance to bending is well taken care by the addition of reinforcement bars. Nevertheless, the value of tensile strength is useful for the estimation of the load under which cracking will develop. The absence of cracking is of considerable importance in maintaining the structural health of a concrete building as cracks will facilitate the ingress of aggressive agents, leading to deterioration and ultimately the failure of concrete. In practice, flexural strength and splitting tension test are commonly performed to evaluate the concrete strength in tension.

Concrete is a composite and its properties depend largely on the properties of the component phases as well as the interaction between them. The bond between aggregate and cement mortar is an important factor in the strength of concrete, especially the flexural strength, since the interfaces are the weakest link in concrete which play a vital role in the process of failure. This process is strongly related with the characteristics of the aggregates and with the relative differences in strength between the matrix and inclusions. This phenomenon will be even more prominent in the case of RCA concrete as there are additional deficiencies in the mortar-aggregate bond.

Bairagi et al (1993) has carried out the full-range study on flexural and tensile strength of concrete with different levels of RCA replacement at various W/C ratios. For the replacement level of 25% and 50%, on average, the relative strengths of 94% to 90% in tension and 94% to 87% in flexure were attained for all the mix designs. Thereafter, as compared to the control mixes, a gradual reduction in flexural strength occurred in the range of 18% to 26% for the replacement level of 75% and 100% respectively whereas the splitting tensile strength has a drastic drop of 29% to 40% for the corresponding replacement levels. In order to investigate the influence of contaminants on the performance of concrete, Park (2001) has collected three different sources of RCA which comprised different maximum aggregate sizes and levels of contaminants. At a constant W/C ratio and mix proportion, 30% by mass of NCA was substituted by the different types of RCA for the preparation of concrete specimens. Overall, all the 28-day splitting tensile strength and flexural strength of concrete produced with different types of RCA were about 10 to 20% lower than the corresponding properties of control mix. At the same time, there was a marginal fluctuation of about 10% in the strength properties among the concrete containing various types of RCA. In the subsequent work, the same type of RCA was used for concrete production but at different replacement levels. The test results showed that concrete produced with 100% RCA content have the lowest flexural strength but the same splitting tensile strength as compared to the other replacement levels. On the whole, both properties suffered minor reductions in strength of up to 10% when compared to that of control mix. To study the effect of mortar strength on the mechanical properties of RCA concrete, Kheder & Al-Windawi (2005) compared the flexural strength and splitting tensile strength of the control mix, RCA concrete and mortar. The test results indicated that the flexural strengths of both control mix and RCA concrete were always lower than that of mortar. Conversely, the splitting tensile strength of control mix was always greater than that of mortar while the corresponding property of RCA was better than that of mortar in the high W/C mixes and became inferior in the low W/C mixes. Based on these findings, the flexural strength was relatively more sensitive to the presence of coarse aggregate whereas the splitting tensile strength was probably more dependent on the W/C ratios. On the other hand, since the splitting tensile strength and flexural strength test results of RCA concrete remained lower than that of control mix by about 25% and 20% respectively, it was evident that the mortar's strength in tension could not contribute directly to make these properties of RCA concrete matched that of control mix. The test results obtained by Tabsh & Adbelfatah (2009) also demonstrated that the influence of coarse aggregate in splitting tensile strength was predominant when the W/C was high but this effect diminished as the W/C was reduced. Nonetheless, the loss in splitting tensile strength of concrete due to the use of RCA was ranged between 10% and 25% as compared to that of control mix. Kou & Poon (2008) agreed with the previous works carried out by other researchers that the quality of original concrete hardly has any effect on the strength development of concrete as the fluctuations in the splitting tensile strength of concrete containing different sources of RCA were capped at 10%. As for the long-term strength development, similar trend was noted as that of the compressive strength and elastic modulus. Though RCA concrete has relatively lower splitting tensile strength at the early stages as compared to that of control mix, it showed continuous and significant improvement over time. After 5 years, concrete made of 100% RCA content has the highest gain in strength and it even attained the highest splitting tensile strength among the mix proportions while the control mix has the lowest strength in value.

2.4.4 Drying shrinkage

Though the resistance to deformation makes concrete a useful material, the volume changes of the concrete itself can have important implications in use. Whether subjected to loading or not, concrete contracts on drying due to environmental conditions and undergoes shrinkage when the water diffuses outwards from concrete. Hence, it is essential that the magnitude of drying shrinkage is taken into account for design considerations so as to avoid the formation of excessive cracking in the structural members.

Although the mechanism of volume changes in concrete due to the variations in moisture content is not fully understood, some of the key factors that affect the extent of shrinkage have been highlighted. One of them is the properties of aggregate used in concrete production since aggregate particles not only dilute the cement mortar but reinforce it against contraction. In general, the elastic property of aggregate determines the degree of restraint offered.

Hansen & BØegh, (1985) reported that after 440 days of exposure to air at 25 °C and 40% relative humidity, drying shrinkage of concrete made with RCA and natural sand was 40% to 60% higher than drying shrinkage of original concrete from which RCA was produced. The variation of drying shrinkage with time for RCA concrete and control mix were monitored for 365 days by Sagoe-Centsil et al (2001). The drying shrinkage of test specimens increased with time and stabilized at about 91 days. Though both reference and RCA concrete displayed similar trends with regard to the rate of shrinkage, strains associated with RCA concrete was 15% higher than the control mix. The research works conducted by Park (2001) illustrated that there was a slight increase of about 7% in drying shrinkage of RCA concrete compared to that of control mix when the replacement was less than 50%. Thereafter, there was a substantial increase of about 16% and 25% in drying shrinkage of concrete produced with 70% and 100% RCA content respectively. The drying shrinkage test results obtained by Juan & Gutiérrez (2004) were in line with observations made by the preceding researcher. The use of up to 50% of RCA slightly increased the shrinkage by 6% while the drying shrinkage of concrete produced with 100% RCA content could be 60% more than the corresponding property of control mix. The effect of RCA on the relationship between compressive strength and drying shrinkage of concrete has also been evaluated. The results demonstrated that the influence of strength on shrinkage was negligible. Yang et al (2008) found out that the unrestrained shrinkage strains of concrete containing different replacement levels of RCA exhibited the similar behaviour and most shrinkage strains occurred in the first 10 days of testing, followed by an increment at a decreasing rate. In addition, a lower shrinkage strain was developed in RCA concrete than in control mix in the early age but the shrinkage strain of RCA concrete was much larger than that of control mix in the long term.

2.4.5 Creep

Creep is defined as the time-dependent strain under a sustained stress and it can be of the same order of magnitude as drying shrinkage. This increase in creep strain can be several times as large as the elastic strain on loading. Experiments have shown that creep continues for a very long time and detectable changes have been found after as long as 30 years. It has been estimated that 75% of 20-year creep occurs during the first year. Albeit that the influence of creep on the ultimate strength of a simply supported and reinforced concrete beam subjected to a sustained load is insignificant, but considerably increment in deflection may be a critical design consideration. Another instance of the adverse effects of creep is its influence on the stability of the structure through increase in deformation and consequent transfer of load to other structural components. Thus, even when creep does not affect the ultimate strength of the component in which it takes place, its effect may be extremely serious as far as the performance of the structure as a whole is concerned (Feldman, 1969).

The strength of concrete has a considerable influence on creep and in turn, it follows that creep is closely related to the W/C ratio. Besides, the elastic modulus of aggregate also controls the amount of creep that can be realized and concrete made with different aggregates exhibit creep of varying magnitude.

Ravindrarajah & Tam (1985) had prepared test specimens with NCA and RCA. They were moist-cured for 28 days and allowed to dry in an unsaturated laboratory environment (30°C and 77% relative humidity). At maturity, the specimens were loaded at a load intensity corresponding to about 25% of the ultimate strength. The RCA concrete showed higher creep strain than the corresponding original concrete after monitoring for a period of 56 days at an identical applied stress. Similarly, the specific creep of concrete exhibited the same trend as that of creep strain. In order to achieve the equivalent 28-day compressive strength, Limbachiya et al (2000) reduced the W/C ratio by increasing cement content with the increase in RCA replacement level. The test specimens were loaded at a constant stress of 40% of the cube strength after 28-day initial curing. As compared to the control mix, the test results revealed that the increase in creep coefficient values could be as high as 65% and 33% for Concrete Grade 50 and 60 respectively when NCA was fully replaced by RCA. In general, the creep coefficient values decreased with the increase in the strength. The concrete samples produced by Gómez-Soberón (2002a) were tested at 50% relative humidity and 20°C for ages up to 270 days. Loaded at a stress factor of 0.35, the total creep strains of concrete containing less than 30% RCA content did not differ much from that of control mix. However, there was drastic increase in the total creep strains when the RCA replacement factors were beyond 30%. The test results attained by Limbachiya et al (2004) agreed with the research works carried out by Gómez-Soberón (2002a). The creep coefficient value of concrete made with 30% RCA was similar to that of control mix. Thereafter, there was a significant increase in the creep coefficient values of about

13% and 54% for concrete produced from 50% and 100% RCA respectively. RCA concrete with constant W/C ratio and the designed workability was maintained by the addition of water-reducing agent. The concrete specimens were tested at a controlled condition of 23°C and 65% relative humidity. The creep strains were obtained by deducting the deformation due to shrinkage and instantaneous deformation caused by the compression load from the total deformation. At 35% compressive strength level, it was observed that the creep strains of RCA concrete with a 20%, 50% and 100% substitution percentage was found to be 35%, 42% and 51% higher than the corresponding property of control mix for a period of 180 days.

2.5 Durability characteristics of hardened concrete

Durability of concrete is defined as its ability to withstand the weathering action, chemical attack, abrasion or any other process of deterioration during the specified or traditionally expected service life. Durable concrete will retain its original form, quality and serviceability when exposed to its environment. However, there is hardly any concrete which is totally impermeable and erroneous anticipation in its exposure to the environmental conditions could be costly as routine maintenance is required. Thus, for many conditions of exposure of concrete structures, both strength and durability have to be considered explicitly at the design stage.

Incipient damage may have developed within the concrete for a period of time before the first visible signs become apparent. Once its integrity is compromised, concrete becomes vulnerable to other kinds of degradation that it might have withstood beforehand. The external factors and internal causes within the concrete itself are the contributory factors for the degradation of concrete. The various actions can be chemical, mechanical or physical.

- Mechanical action – The damage is caused by impact, abrasion, erosion or cavitation.

- Chemical action - The internal causes of deterioration include the alkali-silica reaction and alkali-carbonate reaction while the external factors occur mainly through the action of aggressive ions, namely chlorides, sulfates or of carbon dioxide and etc.
- Physical action – The causes of deterioration include the effects of high temperature or of the differences in thermal expansion of aggregate and of the hardened cement mortar.

With the exception of mechanical damage, all the adverse influences on durability involve the transport of fluids through the concrete. Principally, there are three fluids pertinent to durability which can enter concrete: water (pure or carrying aggressive ions), carbon dioxide and oxygen. They can move through the concrete in different means and it is the resistance of concrete to the penetration of these fluids that affects its durability. As for the movement of fluids through concrete, three mechanisms have been distinguished, namely permeability, diffusion and sorption.

- Permeability – The process by which a fluid flows through concrete as a result of the action of a pressure gradient.
- Diffusion – The process by which a fluid moves under a differential in concentration. It diffuses through vacant pores or pore solution in saturated pores.
- Sorption – The process which is driven by a moisture gradient and involves the capillary movement in the concrete pores which are open to the ambient medium. Nonetheless, the conditions of the concrete will determine the capillary suction whereby no sorption of water is experienced in completely dry concrete or in saturated concrete but can take place only in partially dry concrete.

Indeed, the penetrability of concrete is voiced down to the nature of the pore system within the bulk of the hardened cement mortar and the interfacial transition zone between the cement mortar and the aggregate. Therefore, apart from the good mix design and use of raw materials complying with the requirements, it is necessary to stress that good quality control and workmanship are absolutely essential to the production of durable concrete. There are numerous factors which influence the ability of concrete to resist deterioration, some of which are highlighted in Table 2.2.

Apart from the deficiencies in the engineering properties, durability of RCA concrete is another area of concern. Since RCA has long been confronted for its excessive volume of attached mortar and high porosity, these features have restricted the use of high dosage of RCA in structural concrete applications. Its inclusion in structural concrete production may have detrimental effects in the aspects of the penetrability of hardened concrete and in turn, affecting its durability. Therefore, extensive research works have been carried out worldwide to address these issues and they mainly emphasize on the water and air permeability of RCA concrete as well as its resistance to the aggressive ions from external sources.

Table 2.2 Factors that influence concrete in durability (ACI, 1977)

Factors increasing deterioration	Factors decreasing deterioration
Higher temperatures	Lower W/C ratio
Increased fluid velocities	Proper cement type for the appropriate circumstances
Poor consolidation of concrete	
Poor curing of concrete	Lower absorption
Alternate wetting and drying	Lower permeability
Corrosion of reinforcing steel	

2.5.1 Initial surface absorption test

For practical purposes, it is the absorption characteristics of the surface zone of concrete that are of greatest interest since it offers protection to reinforcement. Hence, tests measuring the surface absorption have been developed. Among these test methods, initial surface absorption test (ISAT) provides data for assessing the uniaxial water penetration characteristics of a given concrete surface. The applied pressure of

200mm head of water is worse than the severest weather exposure due to driving rain. Nonetheless, as the results are of little relevance to behaviour under high water pressures, the test method cannot be used to assess the permeability of a hardened concrete. Indeed, the test results are more readily to be related to the quality of surface finishing and to the durability of the surface under the effects of natural weathering. The factors that affect the surface absorption of concrete are listed in Table 2.3.

Table 2.3 Factors that influence surface absorption of concrete (BS 1881-208, 1996)

Moisture conditions	Surface finish and type	Cracking
Concrete mix	Curing	Water type
Aggregate	Age of concrete	Temperature

Comparing with the control mix, Limbachiya et al (2000) found out that up to 30% RCA content has no influence on initial surface absorption measured at 10 minutes (ISAT-10) and thereafter, ISAT-10 increased with RCA content. However, further analyses of the ISAT results showed that the decay in the absorption rate with time increased with the increase in RCA content. As expected, ISAT-10 value and rate of decay of both the control mix and RCA concrete improved with increasing design strength. These results for high-strength concrete were in line with those observed for medium-strength concrete. At concrete strength of 30N/mm^2, both control mix and concrete containing 30% RCA has the same ISAT-10 value. Beyond that, there was an increase of about 14% and 60% in ISAT-10 value for concrete made with 50% and 100% RCA respectively (Limbachiya et al, 2004).

2.5.2 Water permeability

In order to cause any rapid deleterious effects on concrete, surface attack is insufficient as it is a slow process. Thus, the ease with which water can flow into and through concrete is of great importance. As concrete is a heterogeneous material, its permeability is reliant on the permeability of aggregate used and the degree of hydration of cement mortar. Nevertheless, the permeability of concrete is not a simple function of its porosity, but depends also on the size, distribution, shape, tortuosity and continuity of the pores. The use of low W/C ratio, dense and well-cured concrete can lower its permeability drastically as the formation of continuous pore system, through which bulk flow can occur, is inhibited.

Tested in accordance with ASTM C642 (2003), Levy & Helene (2004) observed that both the water absorption and total pore volume of concrete produced with 20% RCA were lower in values than that of control mix. Thereafter, the two test results increased gradually as the RCA replacement levels increased and exceeded the values of control mix. Goncalves et al (2004) measured the capillary absorption of hardened concrete, following the procedures described in RILEM TC 116-PCD (1999), and noted that RCA effectively induced the water uptake of concrete as compared to control mix. However, the various replacement levels of RCA have acquired quite similar sorption coefficient values for the same cement content. As expected, with the increase in cement content, the sorption coefficient values for both the control mix and RCA concrete were greatly reduced. Berndt (2009) examined the coefficient of permeability of concrete with the experiment setup given in ASTM D5084 (2003). Though the full substitution of NCA by RCA in concrete has resulted in increased permeability of about 55% from 1.1×10^{-10} cm/s to 1.7×10^{-10} cm/s, the coefficient of permeability was less than 3×10^{-10} cm/s and this was acceptable for durable concrete. Gomes & Brito (2009) evaluated the permeability of concrete by immersion test (LNEC E393, 1993) and capillary test (LNEC E394, 1993) whereby the former test measured the open porosity of the material while the latter test measured the capillary absorption by the pressure differential. First, the immersion test results showed that the water absorption capacity of RCA concrete increased approximately linearly with the increased in the replacement level. A maximum water absorption capacity of about 18% was obtained which was about 37% higher than the value obtained for control mix. In terms of water absorption by capillary test, a trend of progressive increased in the water absorption coefficients with the RCA replacement levels was obtained. As compared to the control mix, a maximum of about 17% increase in the coefficient value for concrete made with 100% RCA was observed too.

2.5.3 Air permeability

Gases, especially carbon dioxide and oxygen, can penetrate and move through concrete by diffusion. Lawrence (1984) has reviewed the derivation and the measurement of diffusivity of concrete, expressed in square metres per second. He has shown that the diffusivity is linearly related to the intrinsic permeability of concrete, expressed in square metres. The relation can be exploited to establish the value of diffusivity from tests on permeability, which are easier to perform.

Basically, air permeability is greatly affected by curing conditions, especially in concrete of low and moderate strength. The concrete cured in water exhibited much lower air permeability coefficient as compared to the corresponding property of the concrete cured in air. On the other hand, the air permeability of concrete is also strongly affected by its moisture content. Oven-dried concrete shows nearly 2 orders of magnitude higher in gas permeability coefficient than the concrete near saturation condition (Neville, 1995).

Limbachiya et al (2000) plotted the intrinsic air permeability test results of concrete with various levels of RCA content against the corresponding 28-day cube strength. The trends showed that for replacement level up to 30%, there was no detrimental effect on air permeability, regardless of concrete strength. However, the intrinsic air permeability increased significantly with RCA content beyond this level. The test performed by Buyle-Bodin & Hadjieva-Zaharieva (2002) also found out that the air permeability coefficient of the concrete was doubled when NCA was fully substituted by RCA. On the contrary, Goncalves et al (2004) reported that at the same cement content, the differences in oxygen permeability for various replacement levels of RCA was negligible. However, the oxygen permeability decreased with the increase in cement content.

2.5.4 Chloride ingress

Under most conditions, concrete provides adequate protection of embedded materials against corrosion due to its high alkalinity and relatively high electrical resistivity in atmospheric exposure. Furthermore, a protective passivity layer of oxide, which is self-generated soon after the hydration of cement has commenced, is strongly adhered to the steel and gives it a full protection from the reaction with water and oxygen. However, chloride ions are capable to destroy the layer and in the presence of water and oxygen, corrosion occurs.

In fact, chlorides do exist in the concrete mix itself as they are among the more abundant materials on earth and are present in variable amounts in all of the components of concrete. Specify a zero chloride content for the mix is unrealistic in practice and thus, the proper approach is to limit the total chloride content in concrete to a value less than that required to promote corrosion.

Corrosion is an electrochemical process and it involves the formation of a cathode and an anode with electrical current flowing in a loop between the two. In the early stage of corrosion, rust, which is voluminous in nature, is formed and deposited in a confined space between the steel and concrete, resulting in the formation of small cracks at the surface. Later, there is more prominent cracking of the concrete in a direction parallel to the reinforcement and a delamination of the concrete at the level of the steel. In advanced cases, spalling down to the level of the reinforcement occurs. In addition, the progress of corrosion at the anode reduces the cross-sectional area of the steel and thus, reducing its load-bearing capacity. As the migration of the ions and the chloride attack require the existence of favourable environment, the moisture content, the ionic composition of the pore water and the continuity of the pore system in hardened cement mortar play a vital role in influencing corrosion.

The coefficient of chloride diffusion was determined using a standard two-compartment test cell by Limbachiya et al (2000). For the range of design strengths, a general trend was observed that the use of up to 100% RCA has no negative influence on the chloride diffusion of high-strength concrete as the differences between these concrete were less than 1.0 x 10^{-11} m^2/s. At the same time, the chloride-induced corrosion was measured on 100mm concrete cubes with the insertion of 10mm diameter hot-rolled deformed bars. Similar trends were observed as that of chloride diffusion test with marginal difference between RCA and control mixes, suggesting equivalent corrosion activity. However, the corrosion currents of steel in 100% RCA concrete was slightly higher and the corrosion initiation time was shorter than concrete containing NCA and up to 50% RCA. The chloride diffusion coefficients obtained by Goncalves et al (2004) suggested that RCA has adverse effects on the durability characteristic of concrete. At relatively lower cement content, RCA concrete was about 20% higher in the coefficient value as compared to that of control mix. With an increment in cement content, the gap was narrowed to about 7% and it showed that the influence of mortar quality was in predominance rather than the aggregate porosity. Likewise, Berndt (2009) has made close observation as the preceding researchers. After exposure to artificial seawater for 1 year, RCA concrete exhibited relatively higher chloride diffusion coefficient than that of control mix by about 15%. Gomes & Brito (2009) has performed a laboratory test to determine the chloride migration coefficient from non-steady state migration experiments, according to the Nordtest NT Build 492 (1999). An increase of the chloride penetration depth in the concrete mixes with RCA was detected when compared with that of control mix. Nonetheless, the loss of resistance to the penetration of chlorides was limited to about 6%.

2.5.5 Carbonation

Carbonation of concrete is a process by which carbon dioxide from the air penetrates into concrete and reacts with calcium hydroxide to form calcium carbonates, resulting in a small shrinkage. With respect to durability, carbonation reduces the pH value of the pore water in hardened cement mortar.

Though carbon dioxide by itself is not reactive when it is in gaseous state, in the presence of water, carbon dioxide changes into dilute carbonic acid which reduces the alkalinity of concrete from about 13 to a value of about 8 when all the calcium hydroxide has became fully carbonated. In low alkalinity environment, the existing passivity layer of oxide surrounding the steel will be destroyed and there is a tendency for corrosion to take place, provided oxygen and moisture are available. It is interesting to note that the carbonation process is inactive if the relative humidity is out of the range of 40% to 90%. If the concrete is too dry (relative humidity <40%), carbon dioxide cannot be dissolved whereas if the concrete is too wet (relative humidity >90%), carbon dioxide diffuses slowly as it travels much faster in air. Hence, in order to retard the rate of carbonation, the following key parameters are to be taken note of.

- Strength of concrete
- Diffusivity of concrete
- Moisture content of concrete, i.e. relative humidity
- Cover depth for reinforcement bars
- Good construction practices, i.e. properly compacted and cured

Sagoe-Crentsil et al (2001) reported that when the concrete specimens were subjected to accelerated exposure condition of 4% carbon dioxide atmosphere, there was a nominal 10% increment in the carbonation depth for RCA concrete as compared to that of control mix. On the other hand, the carbonation depths correlated well with the cement content whereby the carbonation depth of RCA concrete was greatly reduced by more than 30% with a 5% increment in cement content. Similarly, as compared to that of control mix, there was a reduction of about 10% to 20% in the accelerated carbonation depths for the concrete produced with 100% RCA content (Levy & Helene, 2004; Limbachiya et al, 2004). This observation made was mainly attributed to the higher cement content used in the preparation of RCA concrete so as to achieve the same strength as control mix. However, the RCA concrete was carbonated 2 times faster than the control mix when the concrete samples were placed in a cell filled with 50% carbon dioxide at 20°C and 65% relative humidity (Buyle-Bodin & Hadjieva-Zaharieva, 2002).

2.5.6 Sulphate resistance

Naturally occurring sulphates of sodium, potassium, calcium or magnesium are sometimes found in soil or dissolved in groundwater adjacent to concrete structures and they are responsible for the damage on concrete. At the same time, excessive sulphur trioxide (SO_3) content in the cement or in the aggregate used can have deleterious action on concrete too. As the dissolved sulphate ions permeate into concrete, they react to form ettringite. Its formation is accompanied by volume expansions which cause internal stresses and cracking. Moreover, the consequences of sulphate attack also include the loss of strength of concrete due to the loss in cohesion in the hydrated cement mortar and of adhesion between it and the aggregate particles (Neville, 1995). As a result, different approaches have been recommended to mitigate the attack by sulphate. Both the use of sulphate-resisting cement and the incorporation of mineral additives (for examples, pozzolanas and GGBS) can enhance the sulphate resistance of concrete. Alternatively, the mix proportions for dense and high quality concrete with low W/C ratio are also crucial for the production of low permeable concrete.

Besides, the sulphate attack may also occur internally where a source of sulphate is incorporated into the concrete mix. The entire volume of the concrete is affected quite uniformly by internal sulphate attack. The extent of damage depends on the composition of the mixture, the curing conditions and the environmental exposure conditions. The internal sources of sulphate contents may exist in the gypsum-rich cement or less often, in the aggregate used. In the case of RCA, the likelihoods of contamination with residual sulphates are prominent as discussed in Section 2.1.8 of this chapter. However, proper screening and testing procedures should generally avoid internal sulphate attack.

Limbachiya et al (2004) conducted a study to evaluate the resistance of RCA concrete from the ingression of sulphate ions. After exposure to 0.3g/L sodium sulphate solution for 180 days, the test results revealed that 30% RCA content has not much of an effect in the linear expansion of concrete. Thereafter, the specimens expanded more with increase in RCA content. However, the differences remained within a relatively narrow band, ranging from 0.0038% to 0.0056% for concrete made with NCA and 100% RCA respectively. Berndt (2009) evaluated the sulphate resistance of RCA concrete by determining its dynamic elastic modulus periodically. The specimens were immersed in 5% sodium sulphate solution for one year and the results indicated that the initial increase in dynamic elastic modulus occurred within the first three months of the test. After which, the dynamic elastic modulus for all the specimens became stable and no distinct variation was observed. The dynamic elastic modulus of 100% RCA concrete remained lower than the control mix by 14%, but both types of concrete showed the same visual features.

2.6 Summary

The use of RCA in concrete is nothing new in modern concrete technology. Extensive research works have been carried out to investigate the effects of RCA in concrete, ranging from high to low W/C ratio. However, majority of the studies are restricted to laboratory scale and hardly any have been applied in the practical works, especially in structural buildings. Based on the literature review, a summary of the test results for properties of RCA, the concrete mix design methods and test results for properties of hardened RCA concrete were presented in Table 2.4, 2.5 and 2.6 respectively.

The major differences between RCA and NCA are in the adhered mortar on the aggregate surfaces and the contaminants due to the nature of the sources. Well-designed equipment and proper processing procedures are to be adopted to minimize the volume of attached mortar and improve the purity of RCA. In general, as compared to the corresponding properties of NCA, RCA tends to have a much lower particle density and suffers marginal reductions in mechanical strengths. The grading of RCA can fit into the envelopes of the gradation limits specified for NCA. Its particle shape is more angular while its particle texture is likely to be rougher and more porous. Though RCA may contain certain amount of residual chemical contents, in almost all cases, these values are kept well below the stipulated amount for the reactions to take place. Nonetheless, due to the presence of attached mortar, the foremost challenging matter lies in the high water absorption capacity of RCA, ranging from 5% to 10%, while the NCA usually registers 0.5% to 1% for the similar property.

Indisputably, as RCA concrete demands for relatively higher water content to achieve the equivalent fresh concrete properties of control mix, the typical concrete mix design methods can no longer be used without adjustments to accommodate the incorporation of RCA. Several mix designs have been proposed to tackle the potential deficiencies in the properties of RCA concrete. The common modifications adopted by most of the studies are listed in Table 2.7.

Table 2.4 Summary table for the comparisons made between NCA and RCA properties by other researchers

Author (Year)	Gradation	Density	Water Absorption Capacity	Los Angeles Abrasion	Crushing Value	Impact Value	10% Fine Value	Particle Shape	Surface Texture	Chemical Contents	Purity	Amount of Attached Mortar
Hansen & Narud (1983)		Lower	Higher	Higher	Higher							√
Ravindrarajah & Tam (1985)	Comparable	Lower	Higher	Higher	Higher	Higher		Angular	Rough			√
Ravindrarajah (1987)		Lower	Higher	Higher	Higher	Higher		Angular				
Bairagi et al (1993)	Coarser	Lower	Higher	Higher	Higher	Higher		Angular	Rough			
Topcu (1997)		Lower	Higher					Angular	Rough			
Limbachiya et al (2000)	Within limits	Lower	Higher		Higher	Higher	Lower	Equidimensional	Rough			
Sagoe-Crentsil et al (2001)	Within limits	Lower	Higher		Higher				Grainy			
Buyle-Bodin & Hadjieva-Zaharieva (2002)	Within limits	Lower	Higher	Comparable							√	√
Shayan & Xu (2003)	Within limits	Lower	Higher	Higher	Higher							
Gutiérrez & Juan (2004)	Within limits	Within range	Higher	Within range				Within range		Chloride, Sulphate	√	
Juan & Gutiérrez (2004)		Lower	Higher	Comparable				Cubical				
Levy & Helene (2004)		Lower	Higher									
Limbachiya et al (2004)		Lower	Higher		Higher	Higher	Lower		Rough			√
Topcu & Sengel (2004)	Within limits		Higher									√
Dhir et al (2005)		Lower	Higher							Chloride, Sulphate, Alkali		
Exteberria et al (2006)		Lower	Higher							Chloride, Sulphate	√	
Poon et al (2007)		Lower	Higher				Lower					
Casuccio et al (2008)	Comparable	Lower	Higher	Higher								
Kou & Poon (2008)		Lower	Higher				Lower				√	
Yang et al (2008)	Within limits	Lower	Higher									√
Corinaldesi & Moriconi (2009)	Coarser	Lower	Higher								√	
Gomes & Brito (2009)		Lower	Higher	Higher								
Juan & Gutiérrez (2009)		Lower	Higher	Higher						Sulphate		√
Padmini et al (2009)		Lower	Higher	Higher	Higher	Higher						
Tabsh & Abdelfatah (2009)	Comparable			Higher								

Table 2.5 Summary table for concrete mix designs determined by other researchers

Author (Year)	Type of Recycled Materials			Additional Cement	Additional Water	Adjustment of W/C ratio	Addition of Chemical Admixtures	Addition of Minerals	Workability	Slump Loss (SL) / Setting Time (ST)	Air Content
	RCA Coarse Aggregate	RCA Fine Aggregate	RA								
Hansen & Narud (1983)	√	√		√	√				√		√
Hansen & Hedegård (1984)	√			√	√				√	ST	√
Ravindrarajah & Tam (1985)	√			√	√				√		
Hansen & Bøegh (1985)	√			√	√				√		
Bairagi et al (1993)	√								√		
Topcu & Guncan (1995)	√								√		
Topcu (1997)	√								√		
Limbachiya et al (2000)	√			√	√	√	√		√		
Park (2001)	√						√		√		√
Sagoe-Crentsil et al (2001)	√			√	√		√	√	√		√
Buyle-Bodin & Hadjieva-Zaharieva (2002)	√	√		√	√	√	√		√		
Levy & Helene (2004)	√	√	√	√	√	√		√	√		
Limbachiya et al (2004)	√						√		√		√
Juan & Gutiérrez (2004)	√						√		√		√
Ting (2004)	√	√						√			
Topcu & Sengel (2004)	√								√		
Fong et al (2004)	√	√		√	√		√		√		
Dhir et al (2005)	√			√	√	√	√		√		
Kheder & Al-Windawi (2005)	√						√		√		
Exteberria et al (2006)	√			√		√	√		√		√
Poon et al (2007)	√							√	√	SL	
Casuccio et al (2008)	√				√	√	√		√		√
Kou & Poon (2008)	√										
Yang et al (2008)	√	√					√		√	SL	
Li et al (2009)	√						√	√	√	SL	
Gomes & Brito (2009)	√		√		√	√					
Tabsh & Abdelfatah (2009)	√				√	√			√		

Table 2.6 Summary table for the comparisons made between conventional and RCA concrete properties by other researchers

Author (Year)	Engineering Properties						Durability Characteristic					
	Compressive Strength	Elastic Modulus	Flexural Strength	Indirect Tensile Strength	Drying Shrinkage	Creep	Initial Surface Absorption Test	Water Permeability	Air Permeability	Chloride Ingress	Carbonation	Sulphate Resistance
Hansen & Narud (1983)	Comparable											
Hansen & Hedegård (1984)	Comparable											
Ravindrarajah & Tam (1985)	Lower	Lower	Comparable	Comparable	Higher	Higher						
Hansen & Bøegh (1985)	Comparable	Lower			Higher							
Ravindrarajah (1987)	Lower	Lower	Lower	Lower	Higher							
Bairagi et al (1993)	Lower	Lower	Lower	Lower								
Topcu & Guncan (1995)	Lower	Lower										
Topcu (1997)	Lower											
Limbachiya et al (2000)	Comparable	Comparable	Comparable		Higher	Higher	Higher		Higher	Comparable		
Park (2001)	Lower		Lower	Lower	Comparable							
Sagoe-Crentsil et al (2001)	Comparable			Comparable	Higher			Higher			Higher	
Buyle-Bodin & Hadjieva-Zaharieva (2002)	Lower							Higher	Higher		Higher	
Gomez-Soberón (2002a)						Higher						
Goncalves et al (2004)	Lower							Higher	Higher	Higher		
Levy & Helene (2004)	Lower							Higher			Lower	
Limbachiya et al (2004)	Lower	Comparable	Comparable		Higher	Higher	Higher				Comparable	Higher
Juan & Gutiérrez (2004)	Lower	Lower		Comparable	Higher							
Ting (2004)	Lower		Lower	Lower				Higher				
Kheder & Al-Windawi (2005)	Lower	Lower	Lower	Lower								
Exteberria et al (2006)	Comparable	Lower		Comparable								
Casuccio et al (2008)	Lower	Lower	Lower	Lower								
Kou & Poon (2008)	Lower	Lower		Lower								
Yang et al (2008)	Lower	Lower	Lower	Lower	Higher							
Domingo-Cabo et al (2009)	Higher	Lower			Higher	Higher						
Berndt (2009)	Lower	Lower		Lower				Higher		Higher		Higher
Li et al (2009)	Comparable		Comparable									
Gomes & Brito (2009)	Comparable	Lower						Higher		Higher	Higher	
Tabsh & Abdelfatah (2009)	Lower			Lower								

Table 2.7 Benefits and drawbacks in the alterations of mix designs

Alteration	Benefit	Drawback
Increase in water content	Maintain workability	Poor in engineering properties and durability characteristics
Increase in both water and cement content	Maintain workability and mechanical strengths	Increase in drying shrinkage and creep strains
Addition of mineral additives (Egs. FA, GGBS, SF)	Improve workability and better durability characteristics	Suffer in short-term mechanical strengths
Addition of chemical admixtures	Preserve majority of the fresh and hardened concrete properties	-

Summing up, the quality of RCA and concrete mix design method are the key contributory factors on the performance of RCA concrete. In order to safeguard the integrity of concrete, it is crucial to ensure that RCA is able to conform to the standards common to NCA; only RCA with satisfactory quality is accepted for utilization in the production of concrete. On the other hand, the adoption of concrete mix design method will also have great impacts on the properties of concrete. The fundamental principle for preserving the properties of RCA concrete is to maintain the W/C ratio for the same design strength and the alterations to the cement as well as water content should be avoided. With the aids of chemical admixtures, the fresh state of RCA concrete can be designed to match that of the control mix without affecting much of its properties in the hardened state.

Indeed, numerous works have been carried out to investigate the mechanical strengths of RCA concrete and the study findings have shown that it is viable to achieve the designed strengths which are comparable to the corresponding properties of control mix. Despite the growing interests in the use of this valuable resource material routinely, relatively lesser attentions are paid to its long-term performance such as drying shrinkage, creep and durability characteristics. Thereby, it is a pressing issue to address the concerns over these properties.

Undoubtedly, with the efforts from various bodies and organizations, it is hopeful that RCA will become a common concrete constituent in the near future.

Chapter 3 Methodology

After going through the in-depth literature review, the experimental outline of this study was established. First of all, the source of RCA was identified and sample was collected to verify its compliance with the specifications/standards. Next, based on the proposed rational mix design method, trial mixes were carried out to formulate the optimal concrete design mixes with different RCA replacement levels for usage in the next stage of study. Upon selecting the desired design mixes, different types and sizes of specimens were prepared for the full-scale evaluation of the engineering properties of hardened concrete. The respective scopes of works are stated in details in the following sections and summarized in Figure 3.1.

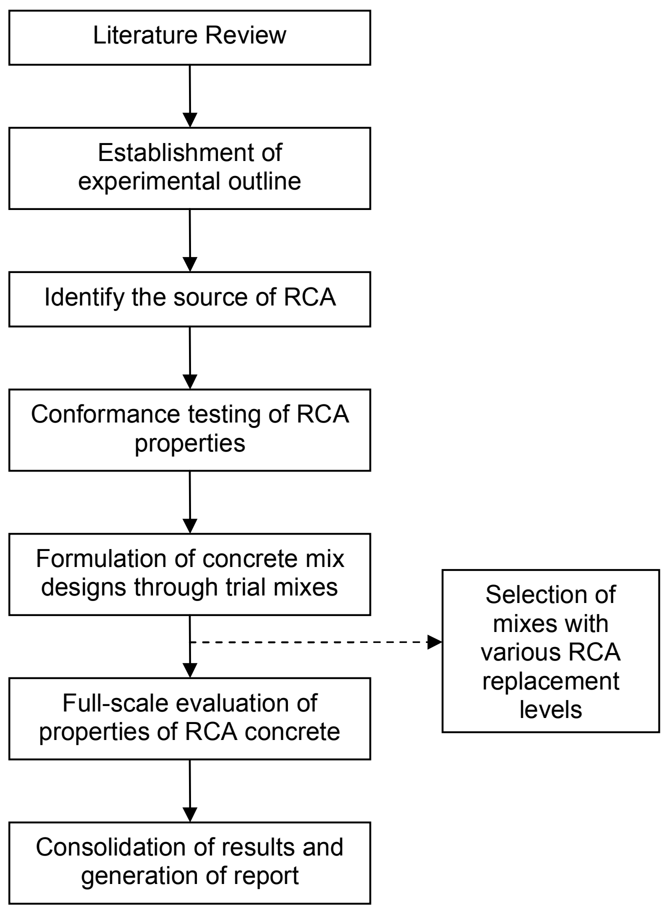


Figure 3.1 Summary of experimental programme for the study

3.1 Production of RCA from C&D waste used in this study

As the sources of C&D waste are difficult to track, paying a visit to site where C&D waste is being produced is essential. Furthermore, acceptance should be based on visual inspection on each delivery to the processing plant, supplemented by a quality control system whereby the material properties are tested according to relevant specification requirements to ensure that they conform to the standards where appropriate (The Highway Agency et al, 2007). In Singapore, the common concrete waste sources are as follows:

- Breakage of precast element during production and transportation in precast concrete industry
- Contamination free crushed concrete specimens obtained from control test specimens (from central and site testing laboratories)
- Demolition of rejected construction due to poor quality workmanship or unauthorized work
- Redevelopment of existing and old buildings or structures

Processing of C&D waste is important as it will influence the properties of the output and ultimately, the engineering properties and durability characteristics of RCA concrete. State-of-the-art equipments are used to process the C&D waste into RCA which involved crushing, screening as well as removal of ferrous metals and foreign materials. In order to get rid as much of the attached mortar as possible, a 2-stage crushing process is introduced in the production plant. Figure 3.2 and 3.3 illustrated the processes adopted by Samwoh for the production of RCA from C&D waste.

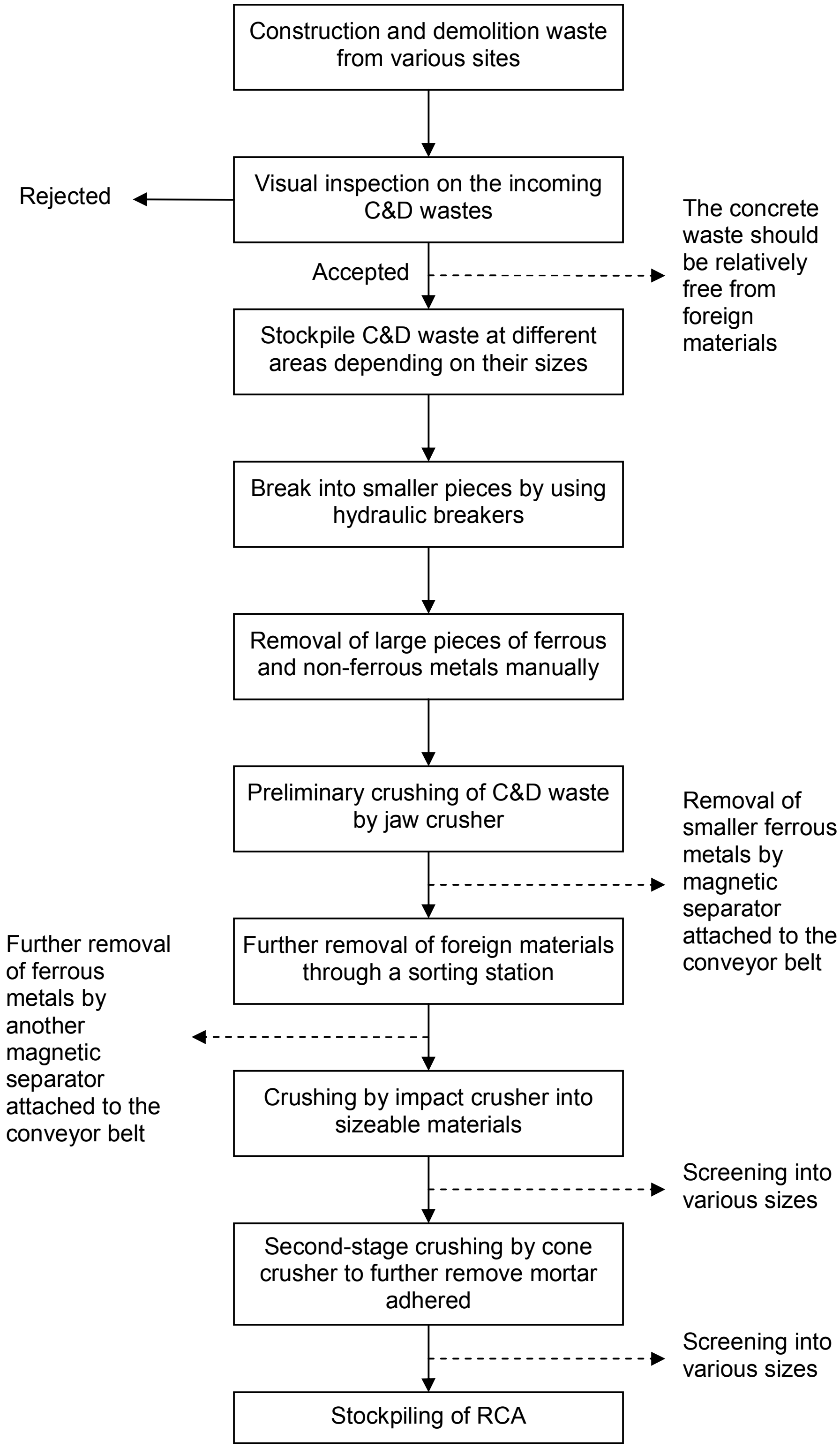


Figure 3.3 Processing of C&D waste into RCA

Figure 3.2 Flowchart for production of RCA from C&D waste

3.2 Testing on RCA properties

In order to safeguard the quality of the fresh concrete as well as the hardened concrete, it was essential that RCA used should comply with the requirements specified. Relevant tests on the aggregate properties must be performed as the results could be the contributory factors for the quality of the resultant RCA concrete. A detailed proposal for testing of aggregate properties was shown in Table 3.1.

3.3 Concrete mix designs

The “DOE method” (Teychenne et al, 1997) for concrete mix design is the most readily accepted technique used by commercial ready-mixed concrete plants in Singapore. This typical mix design was adopted and applied in the research works. However, the use of RCA in concrete for structural application is not widely practiced in Singapore and thus, there was a need to modify the existing mix designs in order to accommodate the use of RCA.

Table 3.1 List of tests on aggregate properties

Property	Test Method
➢ Constituent materials	BS 8500-2 (2006)
➢ Gradation	BS EN 933-1 (1997)
➢ Silt content	BS EN 933-1 (1997)
➢ Particle density (SSD)	BS EN 1097-6 (2000)
➢ Water absorption capacity	BS EN 1097-6 (2000)
➢ Flakiness index	BS EN 933-3 (1997)
➢ Aggregate impact value	SS 73 (1974) Clause 34
➢ Ten percent fine value	SS 73 (1974) Clause 36
➢ Los Angeles abrasion	BS EN 1097-2 (1998)
➢ Drying shrinkage	BS EN 1367-4 (1998)
➢ Alkali-silica reactivity (Chemical test)	ASTM C289-03
➢ Alkali-silica reactivity (Mortar bar method)	ASTM C1260-07
➢ Water-soluble chloride content	BS EN 1744-1 (1998)
➢ Acid soluble sulphate content	BS EN 1744-1 (1998)

Many studies have shown that RCA tends to absorb more water compared to NCA; this resulted in a drastic drop in workability (Hansen & Narud, 1983; Hansen & Boegh, 1985; Ravindrarajah & Tam, 1985; Rashwan & AbouRizk, 1997). To address this issue, the water and cement contents are normally increased in order to achieve the required workability at a constant W/C ratio. However, the increase in cement contents can affect the properties of the concrete such as shrinkage (Hansen & Boegh, 1985; Sagoe-Crentsil et al, 2001). In view of this concern, a rational method was used to replace the NCA with RCA (by mass of coarse aggregate) while keeping the W/C ratio constant. Retarder and water-reducing agent were added to obtain a desired initial slump of 125mm (i.e. immediately after mixing) in all cases before they were being accepted for the making of test specimens. The correct dosages of water-reducing agent, for a particular RCA replacement level and W/C ratio, were found by trial and error method. The process was repeated for every RCA replacement level at each W/C ratio which is described in details in Section 3.4. By doing so, the method offered a better understanding about the effects of RCA on the properties of hardened concrete by eliminating other factors such as changes in cement and water content. W/C ratios ranging from 0.67 to 0.35 were designed to cover the low- to high-strength concrete. RCA replacement levels of 0%, 20%, 40%, 60%, 80% and 100% were examined for the designed W/C ratios. The range of W/C ratios and concrete mix proportions investigated in the research works are shown in Table 3.2.

Table 3.2 Range of W/C ratios for research study

Mix Identification	**M1**	**M2**	**M3**	**M4**	**M5**
Grade	G20	G30	G40	G50	G60
Water Content (kg/m^3)	180	175	175	180	180
Cement Content (kg/m^3)	270	330	390	450	510
W/C Ratio	0.67	0.53	0.45	0.40	0.35
NCA (kg/m^3)	1050	1020	1025	1060	1110
RCA (kg/m^3)	Varies				
Fine aggregate (kg/m^3)	825	800	745	650	545
Retarder (L/m^3)	1.08	1.49	1.80	2.03	2.30
#Water-Reducing Agent (L/m^3)	0~2.70	0.50~3.96	0.98~5.46	1.58~5.40	2.81~7.65

#Note: The dosage of water-reducing agent varies with the RCA replacement level.

It is commonly understood that the use of retarder and water-reducing agent will not influence shrinkage, creep, modulus of elasticity, resistance to freezing and thawing. They have no effect *per se* on the durability of concrete. Specifically, durability on exposure to sulfate is unaffected (Neville, 1995). Hansen & Hedegård (1984) had also proven that the various types of admixtures had little or no effect on the compressive strengths of hardened RCA concrete when added to original concrete at, or above, the maximum dosages recommended by the manufactures.

The introduction of additional admixtures may incur extra cost for the production of RCA concrete. However, with the development in technology for admixtures, it has become more economical and feasible to use these admixtures to improve the performance of fresh concrete as compared to a decade ago. Furthermore, in Singapore's context, the supply of admixtures is readily available and it is widely used by the commercial ready-mixed concrete plants. As such, the overall cost of the RCA concrete, which is designed using the proposed method, can be relatively cheaper compared to conventional concrete.'

Apart from the mix design, the condition of aggregate is also an important factor which governs the properties of concrete. In fact, the batch quantity determined in the mix design process is based on saturated surface-dried aggregate. However, it is a challenge to obtain and maintain the condition of SSD state in aggregates, especially in tropical countries like Singapore where the weather is humid and warm most of the time. With this constraint in mind, an in-house method was developed for the material preparation in the laboratory.

The wet aggregates were being air-dried at a sheltered outdoor condition for minimum 3 days. By then, the condition of aggregate was likely to be somewhere near oven-dry state (Figure 3.4). On the 3rd day of air-drying, the condition of aggregate was monitored. If the particles did not agglomerate, the air-dried materials were collected and stored in the air-tight containers for future use. If not, that particular batch of aggregate would require further observation and it shall be collected once it satisfies the requirement. The material preparations were illustrated in Figure 3.5. Through this method, the state of aggregate was controlled and the consistency in preparation of specimens was assured. To obtain the SSD state for sand and NCA before mixing with other materials, the water absorption capacities of both aggregates and the moisture content of the air-dried aggregates were determined in accordance to BS EN 1097-6 (2000). On the other hand, similar treatment was implied on RCA while the shortfall in the moisture content was restored by the additional water-reducing agent. Principally, the designed W/C ratios (the amount of cement and water that placed into the mixer) must remain constant for respective concrete grades, disregard of the RCA replacement levels.

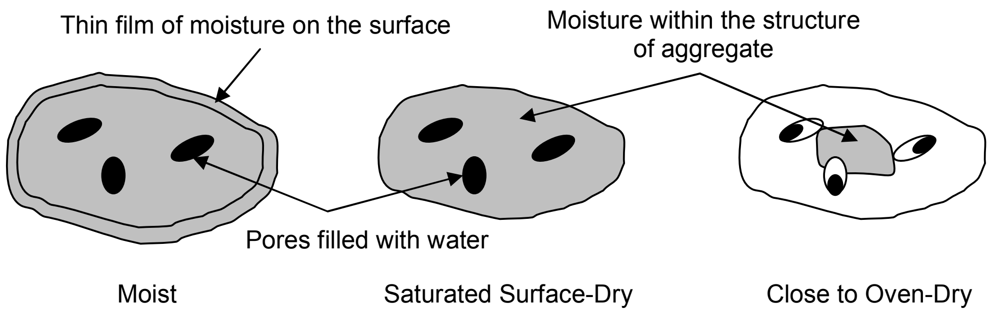


Figure 3.4 Schematic representations of conditions of aggregate

Figure 3.5 Material preparations for batching of concrete

3.4 Preparation of concrete specimens

In addition to the conformity of RCA, other constituent materials for concrete should be complied with the following requirements.

3.4.1 Cement

CEM I 42.5N Portland cement, complying with SS EN 197-1 (2008), was used in the research works.

3.4.2 Natural aggregate

Concreting sand and granite were employed for the batching of concrete in the research works. Sampling and testing were performed on both types of aggregate in accordance to SS 31 (1998) and SS EN 12620 (2008).

3.4.3 Water

Water supplied by Public Utilities Board was utilized in the preparation of concrete specimens as it is clean and free from deleterious materials to concrete.

3.4.4 Admixtures

The retarder and water-reducing agent used in concrete were in compliance with SS 320 (1987).

An in-house batching procedure was used for the production of concrete and making of test specimens which is commonly practiced by the commercial ready-mixed concrete plants in Singapore. In all cases, the specimens were prepared under the following conditions:

a) Before mixing, all aggregates used were air-dried for at least 3 days (under shelter).
b) Determine the water absorption capacity of sand and granite and if required, add additional water to achieve the SSD state for the sand and granite.
c) Mixing of the aggregate, cement, water and admixture were carried out using a pan mixer.
d) A trial mix must be carried out to determine the required retarder and water-reducing agent to achieve the designed slump. The slump was determined using the slump cone method according to BS EN 12350-2 (2009). The trial mix was carried out as shown in Figure 3.6 while Figure 3.7 illustrates the procedures for specimen preparation in details.

e) After the dosages for admixtures were determined, sufficient amount of test specimens were prepared for the tests. The initial slump value must be determined for confirmation.

f) Once the initial slump value met the requirement, the test specimens were prepared in the moulds according to BS EN 12390-2 (2009).

g) The test specimens must be cured in a water tank immediately after removal from the moulds (refer to Figure 3.8).

h) The test specimens were removed from the curing tank for the respective tests according to the required curing regimes as shown in Table 3.3.

3.5 Testing on hardened concrete properties

In Stage 2, nine 100mm cubes were cast for each W/C ratio and replacement level of RCA. These cubes were demoulded after setting in the following day and they were stored according to the curing regime C1, C2 and C7 described in Table 3.3. The testing on 3, 7 and 28-day compressive strength were carried out to determine the effect of RCA on the strength property of hardened concrete. Based on the test results, appropriate mix designs with a specific W/C ratio and various RCA replacement levels were selected as the designed concrete mixes to be used for the next phase of the study.

Stage 3 of the research works involved the in-depth study of the concrete mix designs determined in the trial mix phase. Various tests were carried out to assess the engineering properties and durability characteristics of hardened concrete. Table 3.4 gives a summary of the properties investigated conforming to the standard test methods, types and quantities of specimens prepared as well as the different curing regimes adopted.

As shown in Table 3.4, though majority of the concrete properties were tested in compliance with the standard test methods, the drying shrinkage and creep were evaluated using the in-house methods developed by the author. The justifications for the in-house test methods and the detailed testing procedures were given as below.

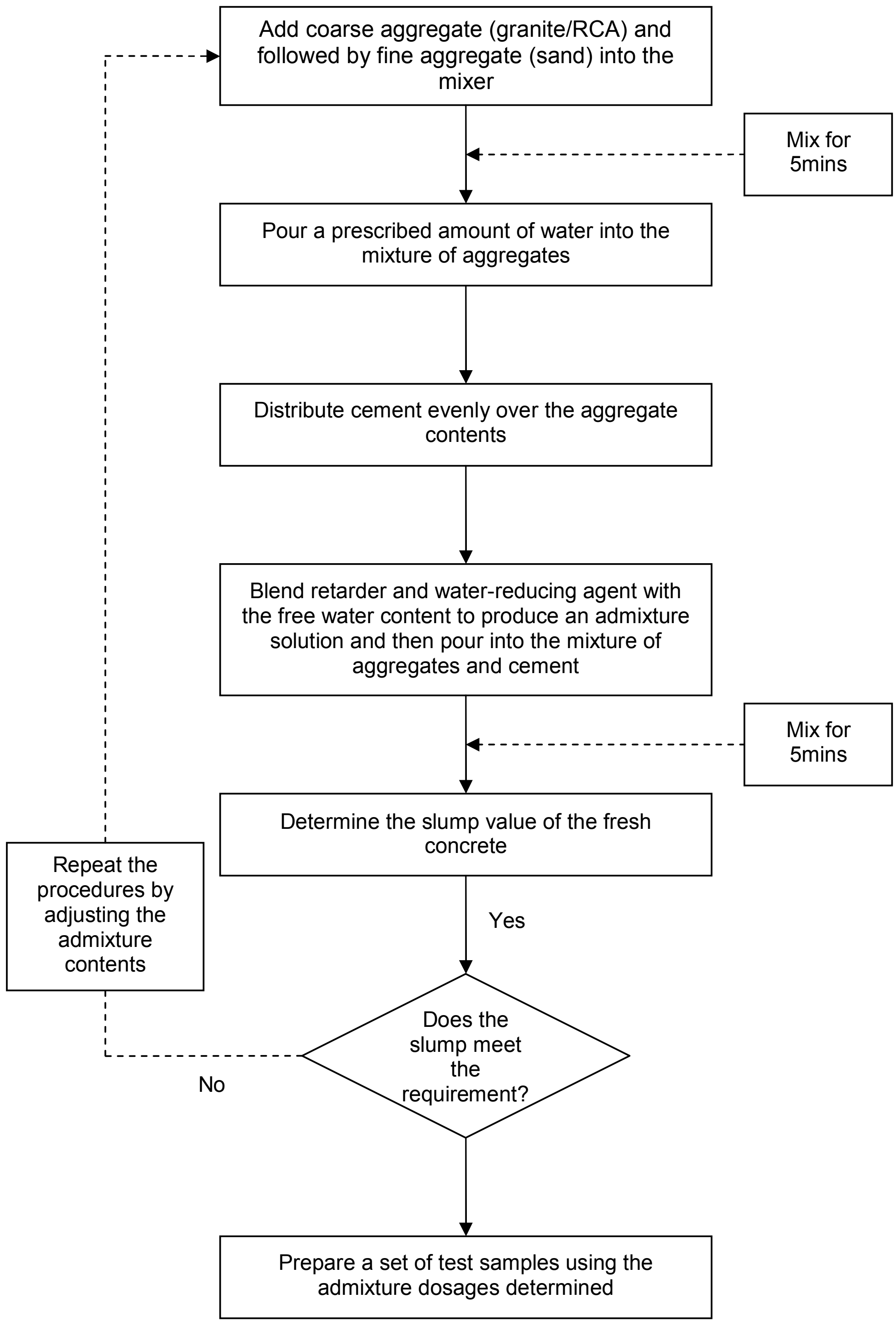


Figure 3.6 Flowchart for specimen preparation

Figure 3.7 Procedures for specimen preparation

Figure 3.8 Curing condition of hardened concrete specimens

Table 3.3 Description of different curing regimes

Notation	Curing Regime
C1	Curing in water at 25 ± 3°C for 3 days + Air-drying for 1 hour before testing
C2	Curing in water at 25 ± 3°C for 7 days + Air-drying for 1 hour before testing
C3	Curing in water at 25 ± 3°C for 7 days + Air-curing at outdoor condition after recording the initial measurement
C4	Curing in water at 25 ± 3°C for 7 days + Curing conditions according to BS 1881-5 (1970)
C5	Curing in water at 25 ± 3°C for 28 days + Oven-drying for 24 hours before testing
C6	Curing in water at 25 ± 3°C for 28 days + Oven-drying for 72 hours before testing
C7	Curing in water at 25 ± 3°C for 28 days + Air-drying for 1 hour before testing
C8	Curing in water at 25 ± 3°C for 90 days + Air-drying for 1 hour before testing
C9	Curing in water at 25 ± 3°C for 180 days + Air-drying for 1 hour before testing
C10	Curing in water at 25 ± 3°C for 365 days + Air-drying for 1 hour before testing

Table 3.4 List of tests on hardened concrete properties

Property	Type & Size of Specimen	Quantity of Specimen	Curing Regime	Test Method
▪ Compressive strength	100mm cube	18	C1, C2, C7, C8, C9 & C10	BS EN 12390-3 (2009)
▪ Elastic modulus	Ø150 x 300mm cylinder	3	C7	ASTM C469-02
▪ Flexural strength	100 x 100 x 500mm beam	3	C7	BS EN 12390-5 (2009)
▪ Indirect tensile strength	Ø150 x 300mm cylinder	3	C7	BS EN 12390-6 (2000)
▪ Creep	Ø100 x 200mm cylinder	2	C7	In-house method
▪ Drying shrinkage	50 x 50 x 200mm prism	3	C3	In-house method
▪ Drying & wetting test	50 x 50 x 200mm prism	3	C4	BS 1881-5 (1970)
▪ Initial surface absorption test	150mm cube	3	C5	BS 1881-208 (1996)
▪ Water absorbability	150mm cube	3	C6	BS 1881-122 (1983)
▪ Water permeability	150mm cube	3	C5	BS EN 12390-8 (2009)
▪ Chloride ingress	Ø100 x 50mm cylinder	2	C7	ASTM C1202-09
▪ Sulphate resistance	50 x 50 x 200mm prism	3	C7	ASTM C1012-09

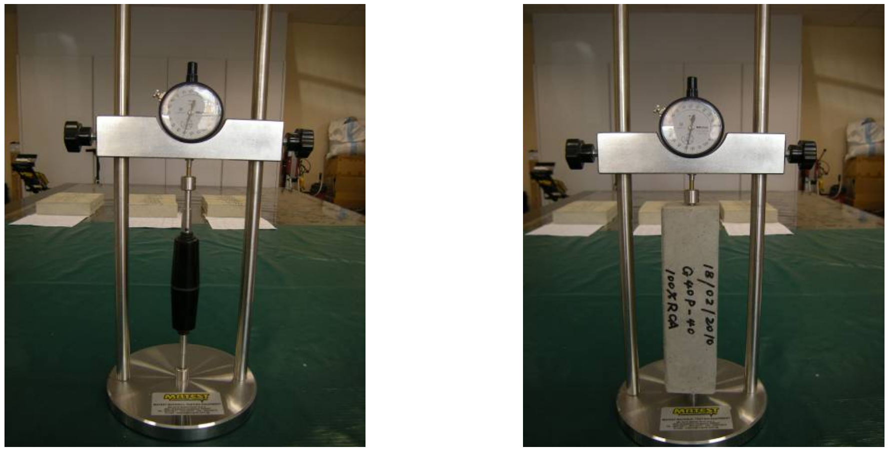
Note: (a) Refer to Table 3.3 for curing regime notations

(b) For M3 with 30% RCA replacement level, creep, drying shrinkage, chloride ingress and sulphate expansion were not determined

i) Drying shrinkage testing

The purpose of this test is to assess the drying shrinkage strain due to the withdrawal of water from concrete stored in unsaturated air without any influence from loading. However, there is no available standard test method which associate with the common practices on site and thus, a proposed testing method has been established to determine the drying shrinkage strain in laboratory. The objective of the proposed testing method is to simulate the actual site practices whereby after casting, the concrete structural elements are properly cured for about seven days before proceeding with the next construction stage. After which, these elements are left exposed to the atmosphere and the loss of water by evaporation will depend on the relative humidity as well as temperature.

- Testing procedures:
  - Remove the specimen from curing tank on the 7th day (i.e. 1 day in the mould and 6 days in the water) after casting.
  - Wipe the surfaces of specimen to get rid of free excess water with a piece of dry and clean cloth.
  - Use another wet and clean cloth to wrap around the specimen for 1 hour before carrying the measurements at room temperature (about 25 ºC).
  - Ensure that the specimen is still in moist condition but no free surface water while carrying out the initial measurement by using a length comparator equipped with a dial gauge (Figure 3.9).
  - After taking the initial readings, place the specimen in a shaded area for air-curing at outdoor condition.
  - Conditions for air-curing: 28 ± 2°C and relative humidity of 80 ± 10%.
  - Carry out daily measurements for the first two months, weekly measurements for the third month and monthly measurements for the subsequent three months.



Taking measurement of reference bar | Taking measurement of concrete specimen

Figure 3.9 Measurement of length change in specimen due to drying shrinkage

ii) Creep testing

At present, there is no commercially available laboratory equipment for creep testing as described in the ASTM C512-02. Hence, a specially-designed creep testing equipment was developed and fabricated to assess the influence of RCA on creep of concrete specimens (Figure 3.10). The equipment consists of a solid metal frame and the hydraulic cylinder pumps driven by the air compressor. The main feature of this equipment is the capability to automatically detect and adjust to any drop in the sustained stress when the concrete specimens deformed. In turn, the required stress can be sustained throughout the measuring duration. Moreover, the equipment is able to provide simultaneous axial loading on six cylindrical specimens under a constant stress.

Hydraulic system driven by air compressor

Metal frame for specimen loadings

Attachment of locating discs on specimen

Taking measurement after loading

Figure 3.10 Components and functioning of in-house creep testing equipment

- Testing procedures:
  - ➢ After demoulding, the concrete specimen is cleaned to achieve a dirt-free and smooth surface.
  - ➢ Fix two locating discs along the longitudinal axis of the specimen at a 150mm gap.
  - ➢ Place the specimen into the curing tank after the locating discs had bonded tightly to the concrete surface.
  - ➢ Remove the specimen from curing tank on the 28th day after casting.
  - ➢ Wipe the surfaces of specimen to get rid of free excess water with a piece of dry and clean cloth.

- Ensure that the specimen is still in moist condition but no free surface water while carrying out the initial measurement by using a 150mm mechanical Demec gauge.
- After taking the initial readings, place the specimen in the testing apparatus before setting the required loading (40% of the ultimate compressive cube strength).
- Conditions for measurement: 25 ± 2°C and relative humidity of 70 ± 5%.
- Carry out daily measurements for at least three to six months, depending on the deformation experienced by the concrete specimen.

Chapter 4 Test Results and Discussion

RCA was assessed in terms of four main groups of properties, namely, geometrical, mechanical & physical, thermal & weathering and chemical. Together with RCA, NCA was also tested and the results were severed as a basis for comparison. After which, the compressive strength test results for trial mixes were presented and thereby, the appropriate concrete design mixes were selected for the next stage of study. Last but not least, the results for respective tests on hardened concrete properties were consolidated and analyzed in the following sub-sections.

4.1 Properties of RCA

The two batches of RCA were collected from Samwoh Recycling Plant at the frequency of one per annum for testing. Table 4.1 presents an overview of the contaminant levels of RCA used in this research. With reference to BS 8500-2 (2006), the total allowable contaminant for RCA is specified at a rather indulgent capacity of 16.5% and thus, RCA is relatively clean as it has less than 6% of total contaminants. Additionally, RCA also shows minimal potential risk in internal sulphate attack of concrete since its acid-soluble sulphate content is only 0.03% which is much lower than the limit of 1%.

Table 4.1 Constituent materials of RCA produced by Samwoh

Composition	Result of Batch 1	Result of Batch 2	Average Result	BS 8500-2 (2006) Requirements
	% by mass fraction			
Max. masonry content	3.7	4.4	3.7	< 5
Max. fines	1.0	0.7	0.9	< 5
Max. lightweight material < 1000kg/m^3	0	0	0	< 0.5
Max. asphalt	1.3	0	0.7	< 5
Max. other foreign materials	0	0	0	< 1
Max. acid-soluble sulphate (SO_3)	0.03	0.03	0.03	< 1

Apart from the test on the constituent materials, other laboratory tests were conducted to evaluate the properties of RCA obtained from the processing plant. As shown in Figure 4.1, sieve analysis was performed to determine the particle size distribution of RCA and the results are compliant with the requirements of SS EN 12620 (2008).

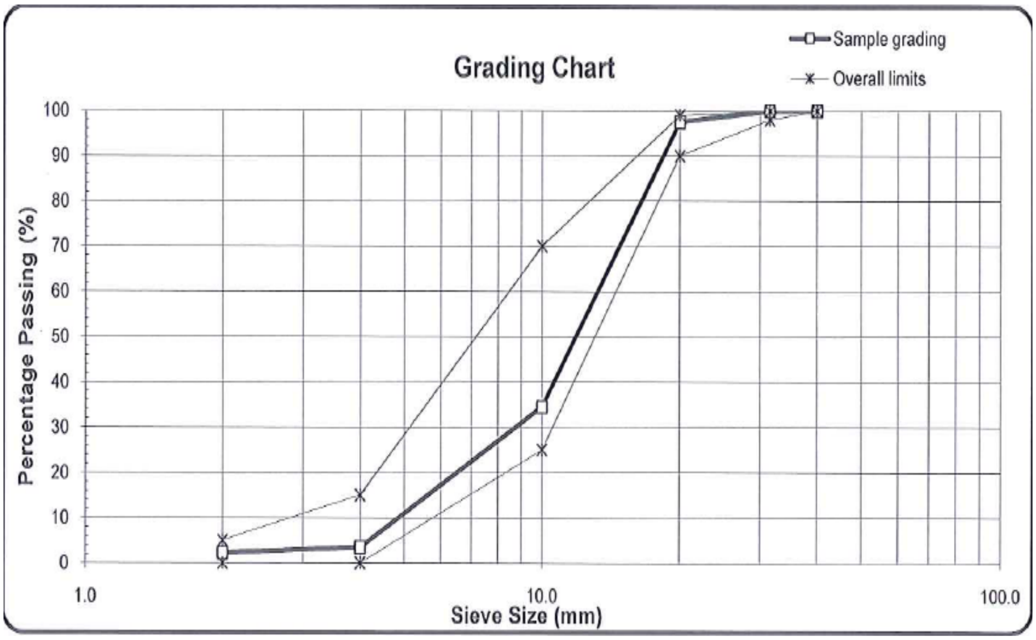


Figure 4.1 Particle size distribution of RCA produced by Samwoh

Furthermore, Table 4.2 shows the typical test results of RCA as compared to NCA. Through visual inspection, the particle shape of RCA seemed to be more angular than NCA and the flakiness index also agreed with the observation whereby RCA has obtained fairly low value as compared to that of NCA. The acquiring of more desirable particle shape might be attributed to the different stages of crushing process. Conversely, excessive crushing might result in the development of microcracking within the attached mortar and in turn, resulting in deterioration in its bonding strength with the aggregate particles. Therefore, RCA exhibited lower mechanical strengths as compared to those of NCA. However, the differences were marginal and the mechanical strengths of RCA sample were all within the respective limits as stated in SS 31 (1998) and SS EN

12620 (2008). Moreover, the presence of adhered mortar had also caused RCA to be more porous and resulted in lower particle density and very much higher water absorption capacity as compared to the corresponding properties of NCA.

As for the thermal and weathering property, RCA did not really disrupt the volume stability of the concrete, which was prepared in accordance with the mix design stated in BS EN 1367-4 (2008). The average change in length of the specimens was about 0.020% while the drying shrinkage of aggregates used in structural concrete is normally capped at 0.075% (BS EN 12620: 2002+A1, 2008).

Inert aggregate is always the preferred material in concrete production so as to safeguard the durability of finished products. For RCA, its chemical compounds are the prime concern to the users as there are scores of uncertainties in the waste streams. The alkali-silica reactivity, acid-soluble sulphate content and total chloride content of RCA were investigated and the test results were presented in Table 4.1 and 4.2. Both the chemical and mortar bar test results suggested that RCA was innocuous to alkali-silica reaction whereas RCA contained negligible amount of chloride content of less than 0.01% by mass of aggregate, which was unlikely to cause corrosion to reinforcement bars.

In conclusions, with the fulfilment of the constituent material requirements stipulated by the standard, the recovered aggregate can be readily classified as RCA and used in construction industry. Though RCA may emerge with some deficiencies in certain aspects of the aggregate properties when compared with NCA, the differences are normally insignificant and RCA can still be able to serve its intended purpose as aggregates in concrete production; RCA produced by Samwoh is volumetrically and chemically stable too. Again, the absorptive nature of RCA is still the most challenging issue faced by the researchers and concrete suppliers. Figure 4.2 illustrates some of the performance tests carried out to evaluate the properties of RCA.

Table 4.2 Other aggregate properties of RCA produced by Samwoh

Property	Test Method	RCA	NCA	Requirement/ Category
Particle Density (SSD)	BS EN 1097-6 (2000)	2.45 Mg/m^3	2.62 Mg/m^3	Declaration
Water Absorption Capacity	BS EN 1097-6 (2000)	5.4%	0.6%	Declaration
Flakiness Index	BS EN 933-3 (1997)	6%	10%	FI_{15} for both NCA and RCA
Aggregate Impact Value	SS 73 (1974)	17%	12%	Table 2, SS 31 (1998)
Ten Percent Fine Value	SS 73 (1974)	113kN	135kN	Table 2, SS 31 (1998)
Los Angles Abrasion	BS EN 1097-2 (1998)	40%	32%	LA_{35} for NCA; LA_{40} for RCA
Drying Shrinkage	BS EN 1367-4 (1998)	0.020%	-	0.075% BS EN 12620: 2002+A1 (2008)
Total Chloride Content	BS EN 1744-1 (1998)	< 0.01%	< 0.01%	< 0.15%, CISRO (2000)
Alkali-Silica Reactivity (Chemical Method)	ASTM C289-03	Innocuous	Innocuous	Fig. X1.1, ASTM C289-03
Alkali-Silica Reactivity (Mortar Bar Method)	ASTM C1260-07	0.055%	0.045%	< 0.10%, ASTM C1260-07

Reduction of bulk sample into the required portion

Determination of water absorption capacity of coarse aggregate

Sieving analysis of aggregate by mechanical shaker

Determination of flakiness index of coarse aggregate

Testing on aggregate impact value of coarse aggregate

Testing on Los Angeles abrasion value of coarse aggregate

Figure 4.2 Performance tests on the properties of NCA and RCA

4.2 Engineering properties of hardened concrete

4.2.1 Compressive strength

4.2.1.1 Trial mixes

A full-scale study which encompassed low- to high-strength concrete with various W/C ratios, ranging from 0.67 to 0.35, was investigated. At the same time different RCA replacement levels, ranging from 0% to 100%, were examined for every designed W/C ratios. A target slump of 125mm was used as the acceptance criteria for specimen preparation. The compressive strength tests were carried out to evaluate the effect of RCA on the 3-, 7- and 28-day strength property of hardened concrete (refer to Figure 4.3). Based on the test results, the appropriate mix design with a specific W/C ratio and various RCA replacement levels were selected for the next phase of the study. The compressive strength test results for M1 to M5 (refer to Table 3.2) with different RCA replacement levels are presented in Figure 4.4 to 4.8.

Figure 4.3 Compressive strength test of hardened concrete

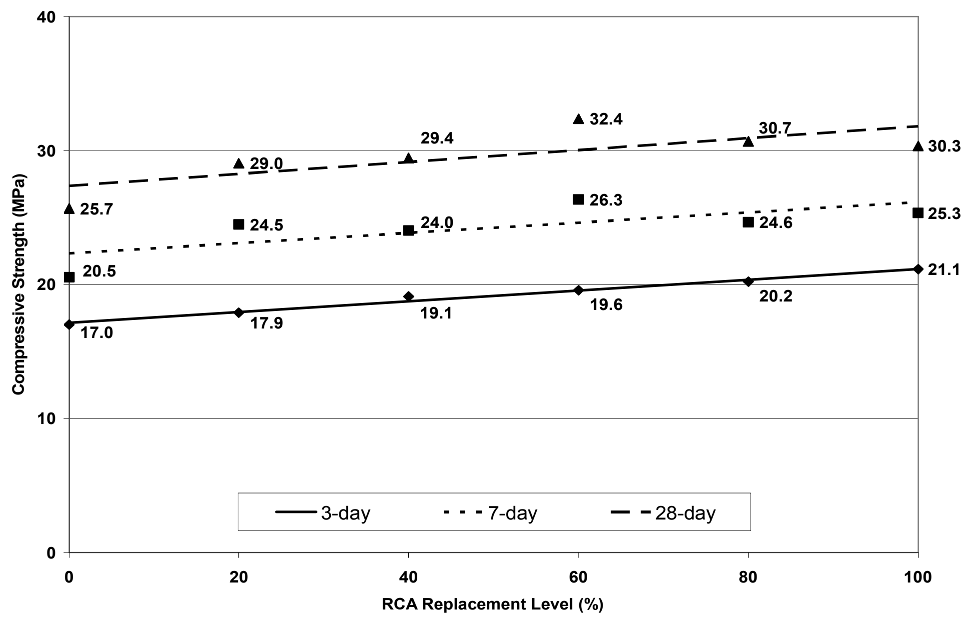


Figure 4.4 Compressive strength test results for M1

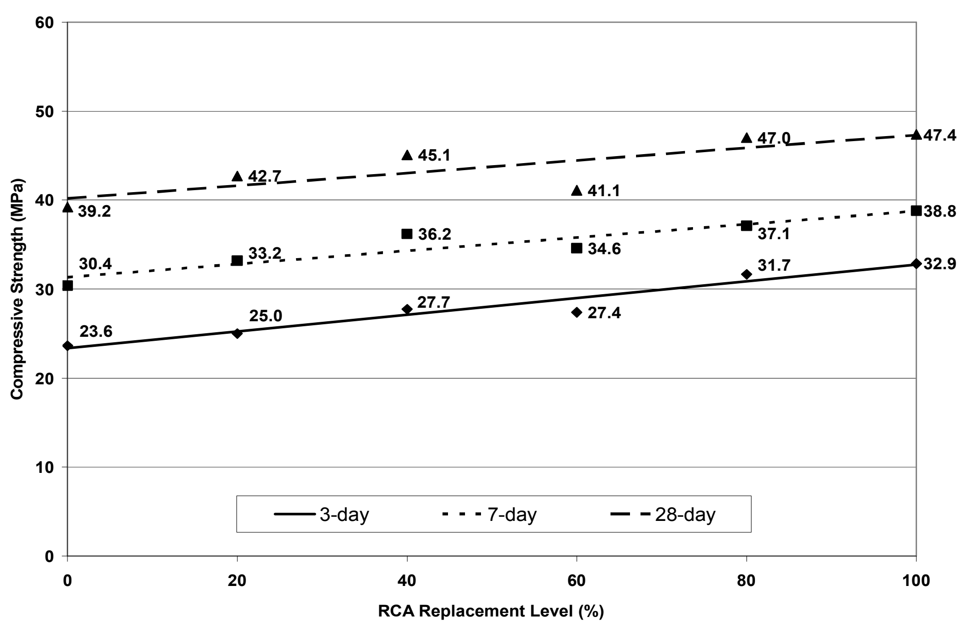


Figure 4.5 Compressive strength test results for M2

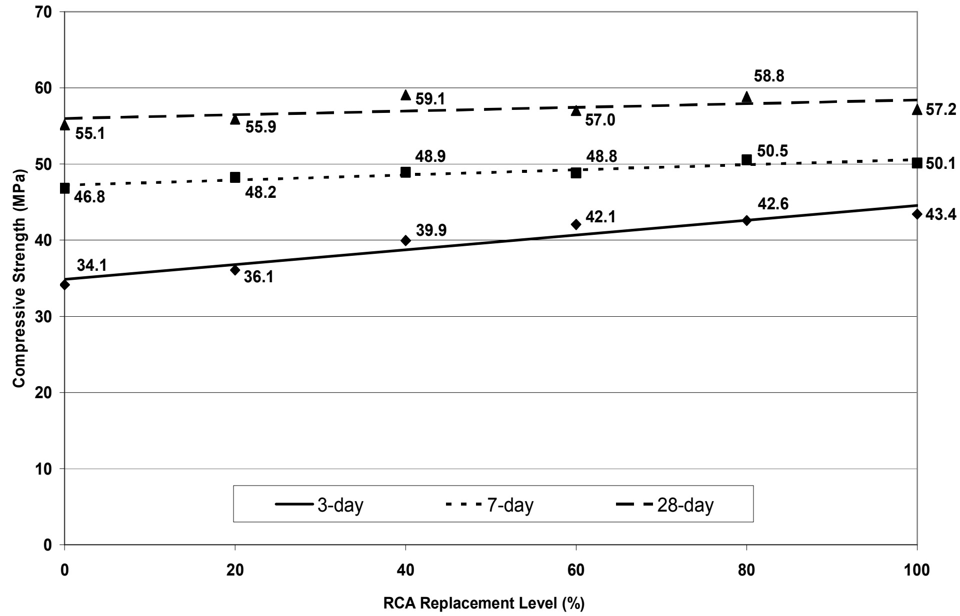


Figure 4.6 Compressive strength test results for M3

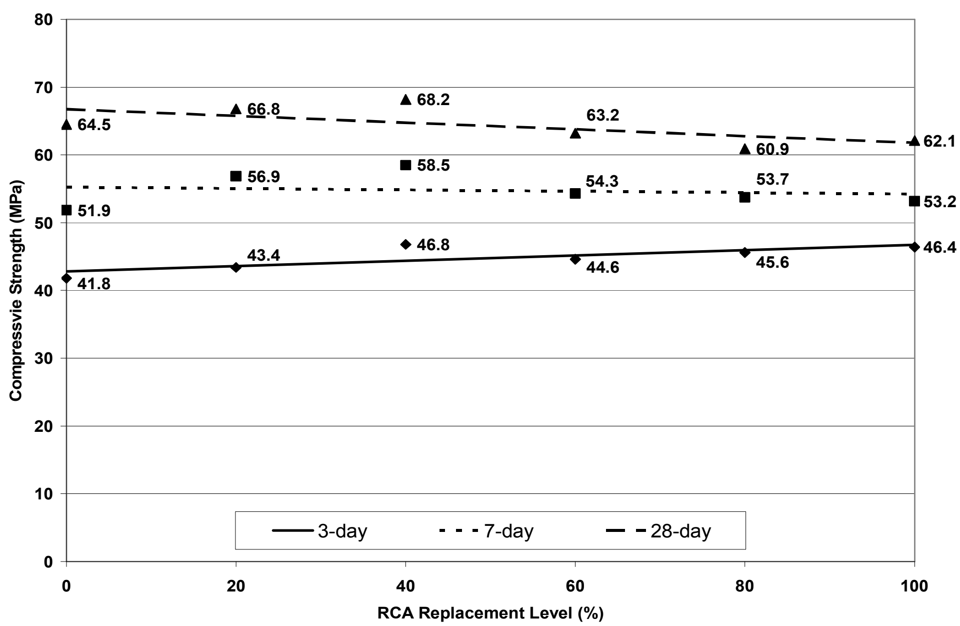


Figure 4.7 Compressive strength test results for M4

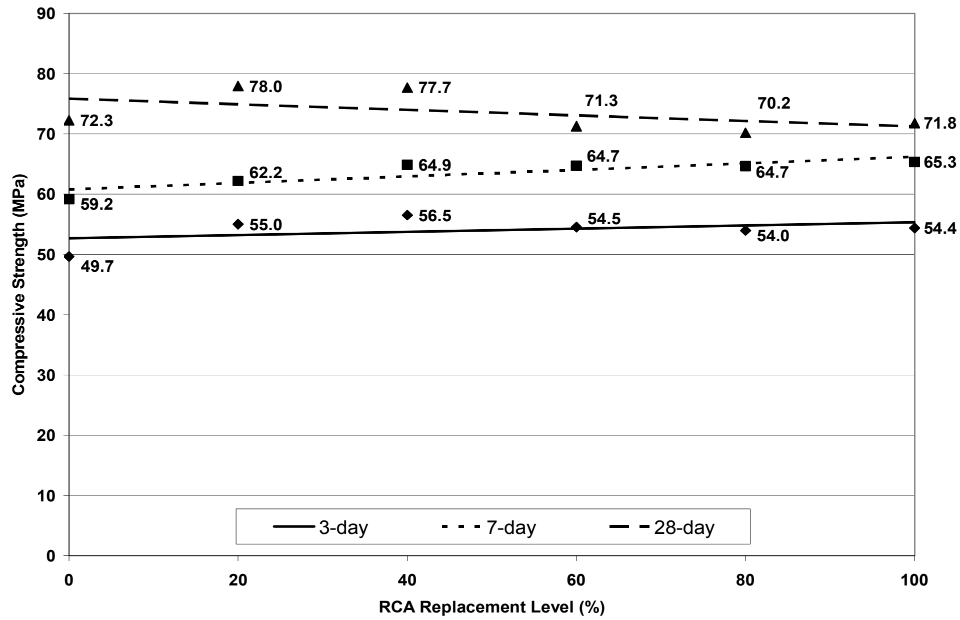


Figure 4.8 Compressive strength test results for M5

The trends shown in the results can be attributed to two factors, namely effective W/C ratio and interfacial transition zone (ITZ) between the mortar matrix and coarse aggregate. Generally, there is only one ITZ for concrete containing original aggregate whereas RCA concrete has two ITZs, i.e. the interface between the original aggregate and adhesion mortar (old ITZ) and the interface between the adhesion mortar and new mortar (new ITZ). Hence, it is believed that the adhered mortar played an important role in determining the performance of RCA concrete, particularly with respect to strength and permeability (Ryu, 2002; Otsuki et al, 2003).

In the case of low- and medium-strength concrete (M1 to M3), it is interesting to note that the trends for 28-day strength were either increasing or at least comparable to control mixes for various replacement levels. Otsuki et al (2003) had made similar observations for the influence of 'old ITZ' and 'new ITZ' on the strength property of concrete at different W/C ratios. At relatively higher W/C ratios, the compressive strengths of RCA concrete were equal to those of control mix. It could be explained that the 'new ITZ' governed the strength performance of concrete, noting that the 'new ITZ' was weaker than the old one. Furthermore, it could be seen that the cracks occurred not only around the 'old ITZ' of RCA concrete but also throughout the new mortar matrix. A similar distribution of cracks could be observed in the control mix. Therefore, the 'old ITZ' did not have much influence on the strength performance of concrete. The observations made may be due to the reduction in effective W/C ratio whereby the free water content was absorbed by RCA. As a result, the higher the replacement level, the lower the effective W/C ratio. Hence, a higher compressive strength was obtained but at the expense of lower workability. However, in this study, the required workability was not affected due to the introduction of higher dosage of water-reducing agent.

On the other hand, for high strength concrete (M4 and M5), a downward trend was observed albeit the drop was not significant based on the Student's Test at 5% significance level. The strengths for concrete with 100% RCA content were still comparable to control mixes. These downward trends for M4 and M5 as the replacement level increased may be attributed to the development of ITZ between the mortar matrix and coarse aggregate. According to Otsuki et al (2003), at low W/C ratio, the compressive strength of RCA concrete was much lower than that of the control mix. It was largely due to the stronger 'new ITZ' over the old one and because of the dominance of 'new ITZ', the 'old ITZ' became the contributory factor for the strength performance of concrete. This explanation was reinforced with the different crack patterns developed in normal and RCA concrete. It could be observed that the cracks were notably concentrated around the 'old ITZ' in RCA concrete while in the case of normal concrete, the cracks were widely dispersed. Moreover, based on the design mixes used in this study, the coarse to fine aggregate ratio increases as the W/C ratio decreases. Consequently, there is more development of 'new ITZ' within M4 and M5. Furthermore, there is an increase in ‘new ITZ’ as the RCA content increases. As such, all these factors may account for the downward trend in the strengths of M4 and M5 even though the effective W/C ratio decreases as the RCA replacement level increases.

Neville (1995) also mentioned that the influence of the type of coarse aggregate on the strength of concrete varies in magnitude and depends on the W/C ratio of the mix. For W/C ratio below 0.4, the strength of concrete is manipulated by the properties of aggregate rather than the strength of the mortar matrix. With an increase in the W/C ratio, the influence of aggregate falls off, presumably because the strength of mortar itself becomes paramount. As the strength of mortar depends greatly on the mortar class, lower effective W/C ratio will produce higher mortar strength. For the low- and medium-strength concrete, the effects of aggregate are likely to diminish and the concrete mix with better mortar class tends to attain greater strength level.

4.2.1.2 In-depth study

Based on the findings obtained from Stage 2, the concrete design mix M3 was selected for the Stage 3 of the research works which involved the in-depth study of the engineering properties and durability characteristics of hardened concrete. The selection criteria was based on the reinforce concrete with moderate exposure condition during its service life whereby the mix design limits for maximum free W/C ratio, minimum cement content and minimum concrete grade are usually fixed at 0.60, 300 kilograms per cubic meter of concrete and 35MPa respectively. The M3 mix has a W/C ratio of 0.45 with a cement content of 390 kilograms per cubic meter of concrete and a characteristic strength of 40MPa at 28-day, which fulfilled all the mix design requirements for a structural building. More importantly, the compressive strength of medium-strength concrete is not affected significantly by the incorporation of RCA, even up to 100% replacement level. To demonstrate the utmost feasibility of using RCA in structural concrete, M3 containing 30% RCA (M3-30), 50% RCA (M3-50) and 100% RCA (M3-100) were chosen for the in-depth study and comparisons were made among the control mix (M3-0), which was prepared without any RCA content, and the respective RCA concrete.

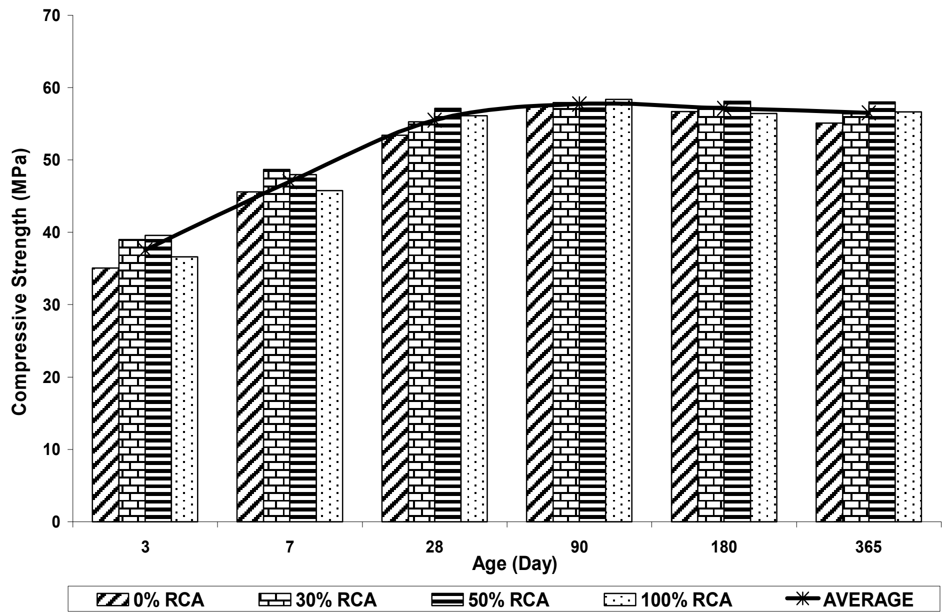


Figure 4.9 Long-term compressive strength test results for different RCA replacement levels

The long-term compressive strengths of control mix and RCA concrete containing different replacement levels were monitored over a period of 365 days and plotted as shown in Figure 4.9. As observed in the trial mix phase, similar trend was noted whereby comparable or even better compressive strengths were obtained for M3-30, M3-50 and M3-100 when compared with those of M3-0. For the entire series of concrete mixes, the strength development basically ceased after 90 days and the test results also revealed that RCA does not have any detrimental effect on the strength development of concrete as the compressive strengths remained constant after 1 year of water curing.

4.2.2 Elastic modulus

In the study, the elastic moduli of the concrete mixes were tested in accordance to ASTM C469 (2000) whereby 40% of the ultimate concrete strength was loaded and unloaded automatically by a computerized testing machine. Figure 4.10 illustrates the setting up of machine for the evaluation of elastic modulus of hardened concrete.

Figure 4.10 Evaluation of elastic modulus of hardened concrete

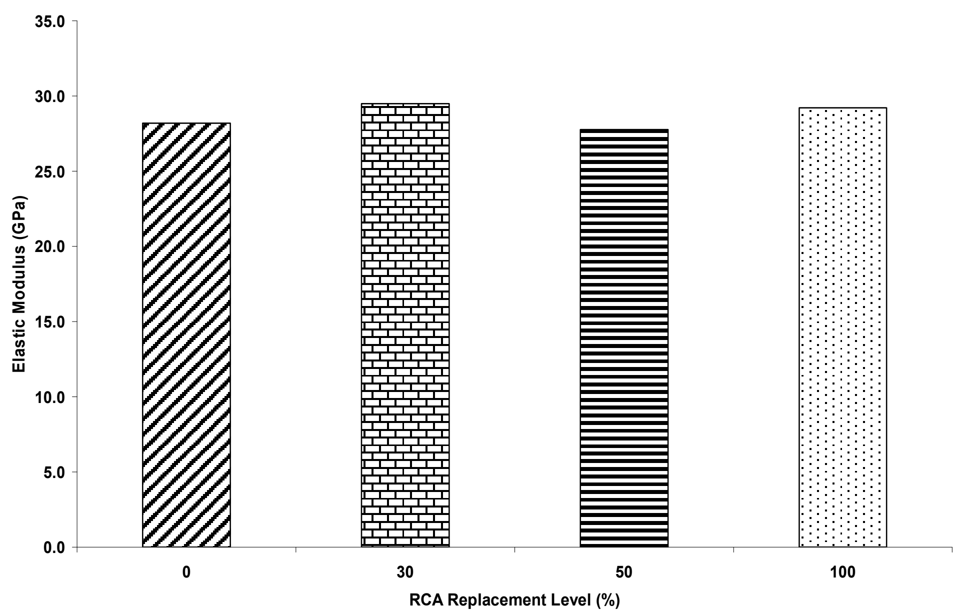


Figure 4.11 Elastic modulus – Average test results for different RCA replacement levels

With reference to Figure 4.11, the average elastic modulus of M3-0, M3-30, M3-50 and M3-100 were 28.2GPa, 29.5GPa, 27.8GPa and 29.2GPa respectively. The results did not show any significant difference at 95% confidence level between the control mix and the respective RCA mixes.

The volumetric proportions of aggregate and cement paste will affect the value of modulus of elasticity at a given strength of concrete. Generally, it is a known fact that the elastic modulus of aggregate is of a few magnitudes higher than the elastic modulus of hydrated cement paste and thus, the volume of aggregate is preferably of higher proportion than the volume of paste matrix. Due to the direct replacement method adopted, the volume of coarse aggregate was relatively higher as replacement level of RCA increased since the density of RCA is lower than that of NCA by about 6%. Moreover, without the adjustments to the cement content and water content of the mixes, theoretically, the amount of hydrated cement paste should remain constant for the concrete specimens containing different levels of RCA content.

On the other hand, a large incompatibility between the moduli of elasticity of the aggregate and hydrated cement paste adversely affects the development of bond cracks at ITZ. Furthermore, aggregate of moderate or low strength and modulus of elasticity can be valuable in preserving the integrity of concrete. The compressibility of aggregate will reduce distress in concrete while a strong and rigid aggregate may lead to cracking of surrounding mortar matrix (Neville, 1995). Owing to the porous nature of RCA, its modulus of elasticity is lower than the corresponding property of NCA and as a result, the difference in elastic modulus of RCA and hardened cement paste will be smaller than that of NCA and hardened cement paste. This will lead to lower stress concentrations at the ITZ due to the reductions in differential strain, which tends to develop between the two components (Young et al, 1998). Certainly, it will enhance the elastic modulus of RCA concrete.

Nonetheless, the inferiorities of RCA may possibly cancel out the aforementioned advantages and result in equivalent elastic moduli at a given W/C ratio. This can be attributed to the amount of weak bond zones in RCA concrete which is significantly more than in the control mix and the existence of large amount of defects within RCA, which may include microcracking created during the crushing processes.

4.2.3 Strengths in tension

As shown in Figure 4.12, the flexural and splitting tensile strength tests of concrete were performed by different sets of testing machines. The specimens for both of the tests are subjected to a constant rate of loading at 0.05N/mm^2 as specified in BS EN 12390-5 (2009) & BS EN 12390-6 (2000).

Testing frame for determination of flexural strength of concrete beam

Fractured specimen after testing

Testing machine with jig for determination of splitting tensile strength

Split specimen after testing

Figure 4.12 Testing of strength in tension of different concrete mixes

A comparison of flexural strength test results of M3-0, M3-30, M3-50 and M3-100 was presented in Figure 4.13. It was interesting to note that the flexural strengths showed an increasing trend with the increase in RCA replacement level and M3-100 achieved the highest flexural strength among the different concrete mixes. The flexural strengths of the various concrete mixes were confined to a narrow range of 3.39MPa to 3.88MPa. Based on the statistical analysis, at a 95 % level of confidence, there were no significance difference in flexural strengths for M3-30 and M3-50 as compared to M3-0 but M3-100 showed significant increase in flexural strength as compared to M3-0.

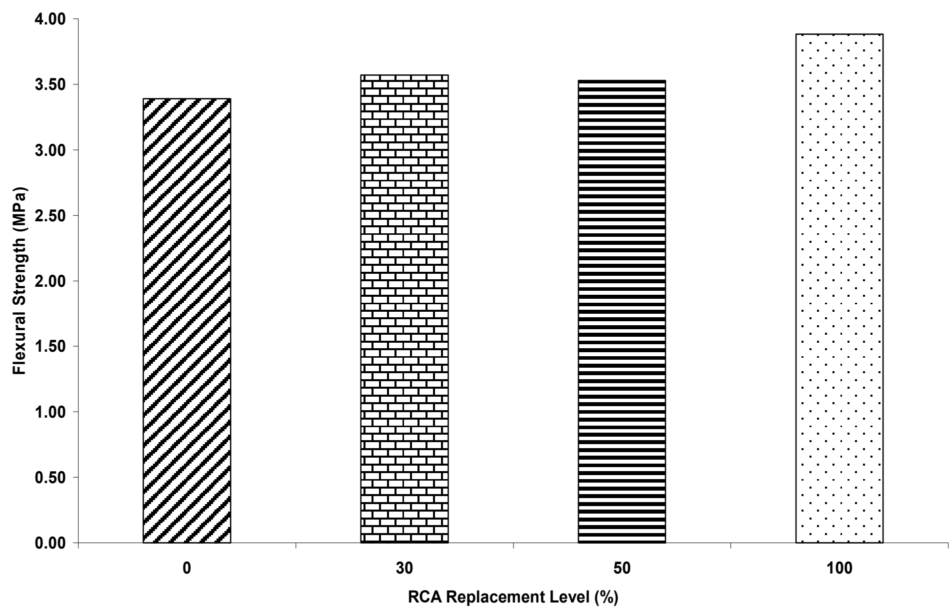


Figure 4.13 Flexural strength – Average test results for concrete with different RCA replacement levels

The splitting tensile strength test results as shown in Figure 4.14. Similarly, the splitting tensile strengths of the concrete mixes produced with different RCA replacement levels were contained within a small scale of 3.11MPa and 3.33MPa. At 95% confidence level, the splitting tensile strengths of M3-30, M3-50 and M3-100 did not show any significance difference when compared with that of M3-0.

Based on the test results, RCA may produce higher or statistically comparable strengths in tension with respect to those obtained using NCA. Though these findings appear to differ from those of the past research works, it is possible to provide some explanations for the contradictions.

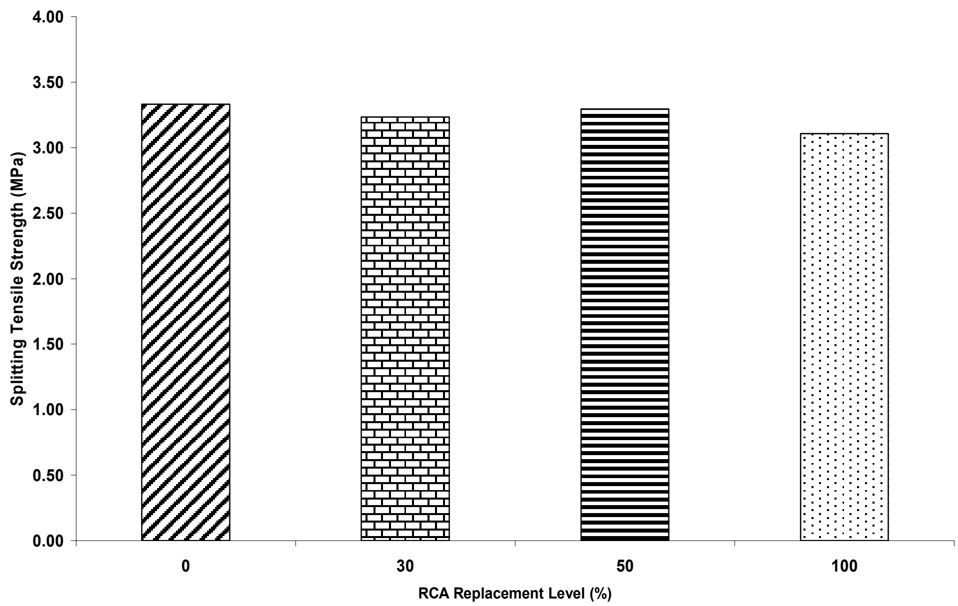


Figure 4.14 Splitting tensile strength – Average test results for concrete with different RCA replacement levels

Firstly, the flexural strength is a direct function of the aggregate characteristics and this property is sensitive to the presence of coarse aggregate. Inclusion of coarse aggregate in concrete may not change the essential microstructure of the cement gel, but generally tend to convert a homogeneous matrix into a heterogeneous system. For low- and medium-strength concrete, the properties of aggregate, especially its shape and surface texture, have greater impact in the strength in tension than the ultimate strength in compression (Neville, 1995). In the case of RCA, since it is more angular in shape and possesses a rougher surface texture as compared to the similar properties of NCA, RCA has a tendency to provide a better mechanical interlocking between itself and the mortar matrix.

Secondly, Kheder & Al-Windawi (2005) had proven that the tensile strength of RCA concrete is lower than that of its mortar matrix as the tensile bond strength of the ITZ is much lower than the tensile strength of the mortar matrix itself, particularly at high strength levels. Thereby, the tensile strength of mortar matrix sets the upper limit for tensile strength of RCA concrete. In fact, the existence of coarse aggregate in the concrete frame will prevent the concrete from reaching the ultimate tensile strength of the mortar matrix. This is probably due to the existence of very fine cracks at the ITZ, even prior to application of the load on concrete (Hsu et al, 1963). This scenario may attribute to the inevitable differences in the mechanical properties between the coarse aggregate and the mortar matrix, coupled with different coefficients of thermal expansion and different responses to changes in moisture content (Neville, 1995; Mindess, 2003). Consequently, the effect will be intensified in the case of RCA concrete as RCA normally consists of considerable amounts of porous old mortars and acquires certain extent of microcracking due to the crushing process. Hence, extra zones of weakness will be developed in the concrete composite as compared to that of control mix. Nevertheless, as discussed in Section 4.2.1, for W/C ratio greater than 0.4, the influence of mortar will be dominant over that of aggregate used with an increase in the W/C ratio. Due to the reduction in the effective W/C ratio, the mortar class of RCA concrete improves with an increase in RCA replacement level. Thereby, the effect of poor interactions between RCA and mortar matrix become less significant.

Last but not least, taking the porous nature of RCA into account, it seems that RCA concrete shows almost the same tendency as for lightweight aggregate concrete. Zhang & GjØrv (1990) pointed out that if lightweight aggregate has a dense outer layer, then the situation at the interface is the same as with normal weight aggregate. Lightweight aggregate with a more porous outer layer encourages the migration of mobile ions towards it (Maso et al, 1982). This leads to the formation of a denser interface zone and also improves the mechanical interlocking of the aggregate particles and the mortar matrix (Zhang & GjØrv, 1990). As such, these factors will contribute to the enhancement of the strengths in tension of RCA concrete.

The evidence for the enhancement in the mechanical interlocking of RCA particles and the mortar matrix can be reinforced by the observed fracture surfaces (Figure 4.15). Based on the visual inspections, apart from the failure at the matrix itself, the fracture planes were primarily at the ITZ and through NCA for the control mix. As for RCA concrete, the fracture planes noted were primarily through old mortar particles and coarse aggregates. The interfaces between old mortar and new matrix generally remained intact and this observation implies good interfacial bonding.

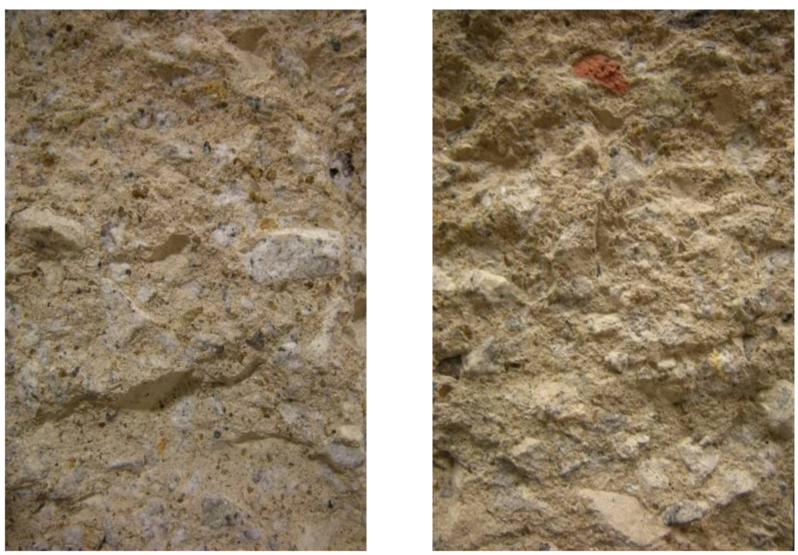


Figure 4.15 Spilt specimens for M3-0 (Left) and M3-100 (Right)

4.2.4 Drying shrinkage

The results for M3-0, M3-50 and M3-100 are summarized in Table 4.3 and Figure 4.16. The drying shrinkage strains of test specimens increased with time and stabilized after air-drying for 56 days. After which, all the concrete mixes displayed similar trend with regard to the rate of shrinkage but after monitoring for 7 months, the maximum strains registered by M3-50 and M3-100 were about 16% and 40% higher than that of M3-0 respectively. The increase in drying shrinkage of RCA concrete is expected as RCA itself has a lower modulus of elasticity as compared to NCA and offers less restraint to the potential shrinkage of mortar matrix. Higher cement content in the old mortar component of RCA may also contribute to the higher drying shrinkage. Mindess et al (2003) explained that the main reasons for the drying shrinkage of hydrated cement paste are believed to be due to the capillary stress, disjoining pressure and changes in surface free energy. These phenomena are the result from its high porosity with a network of small capillary pores, the extensive van der Waal's bonding in calcium silicate hydrate (C-S-H) as well as the high surface area and intrinsic mircoporosity of C-S-H. Moreover, the porosity of RCA can be another contributory factor to the increase in drying shrinkage. For concrete to dry out, moisture must move from the interior to the surface, where it can evaporate. Moisture transport through a porous body is a diffusion-controlled process since water must move through narrow pores (Young et al, 1998). Thereby, the more porous the concrete mix is, the ease of withdrawal of water from concrete will occur and as a result, a porous concrete will experience a higher shrinkage strain.

As compared to other studies, the increase in drying shrinkage for 100% RCA concrete registered by several researchers could be more than 60% when compared with the corresponding property of the conventional concrete (Juan & Gutiérrez, 2004; Domingo-Cabo et al, 2009; Gómez-Soberón, 2009b). The higher shrinkage found in their studies might be caused by the higher cement content used in the RCA concrete in order to achieve the equivalent strength as the conventional concrete or the additional water content to achieve the designed slump. However, in this study, the proposed mix design method does not require any increase in cement content for RCA concrete. Hence, this would result in a comparatively lower shrinkage for RCA concrete. On the other hand, the shrinkage of concrete is a fraction of that of neat cement paste because the aggregate particles not only dilute but reinforce it against contraction. Hansen & Alumdaiheem (1987) had confirmed that the volume of aggregate content contributed significantly to the restraint of the amount of concrete shrinkage. The proposed model predicted that that a change in aggregate content from 65% to 70% would result in a decrease of about 18% in ultimate shrinkage, independent of W/C ratio. As discussed in Section 4.2.2, the lower particle density of RCA tends to give more aggregate particles volumetrically by using the proposed direct replacement method and this may help to reduce the drying shrinkage of RCA concrete.

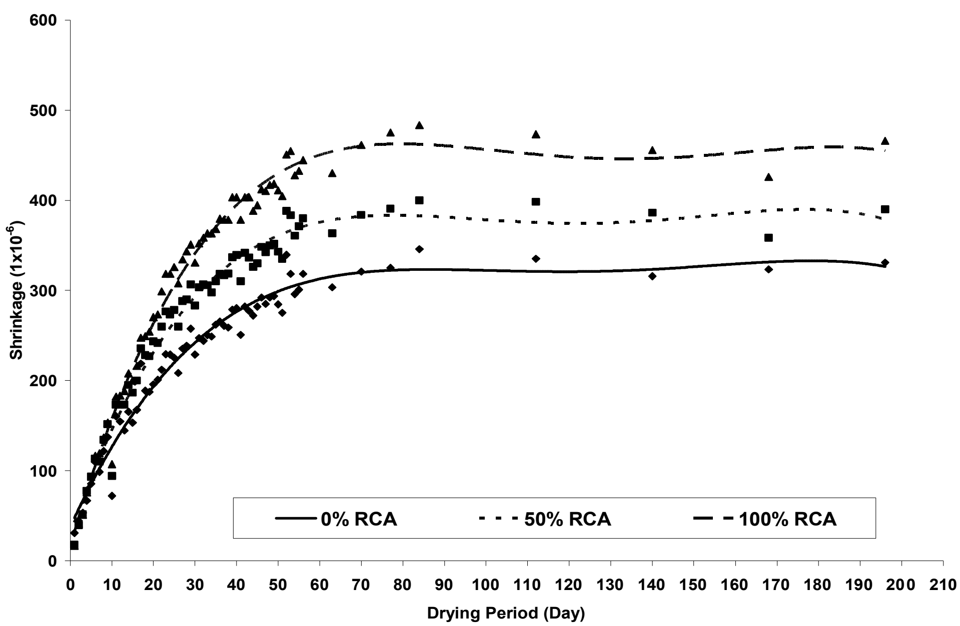


Figure 4.16 Drying shrinkage test results for M3-0, M3-50 and M3-100

In addition, the shrinkage of aggregates may be of considerable importance in determining the shrinkage of concrete. In accordance to BS EN 12620: 2002+A1 (2008), it specifies that any drying shrinkage of aggregate in excess of 0.075% is classified as an undesirable aggregate. With reference to Table 4.2, the RCA used in this study is considered stable volumetrically as it has a drying shrinkage value of 0.020%, which is well below the stipulated limit. This value is useful to provide an indication that the RCA is unlikely to cause any abnormal shrinkage of concrete.

Table 4.3 Drying shrinkage of control mix and RCA concrete

Mix	Drying Shrinkage (x 10^{-6})				Relative Shrinkage (x 10^{-6})			
	7 days	28 days	56 days	Maximum	7 days	28 days	56 days	Maximum
M3-0	99	239	318	346	1.00	1.00	1.00	1.00
M3-50	110	290	380	400	1.11	1.21	1.19	1.16
M3-100	120	343	445	483	1.21	1.44	1.40	1.40

Most importantly, based on the test results, the maximum shrinkage strains recorded by M3-50 and M3-100 were 400 and 483 microstrain respectively. Thus, the shrinkage results for all the concrete mixes are within the recommended basic shrinkage strain of normal-class concrete with a value of 850 mircostrain (AS 3600, 2001).

4.2.5 Creep

Creep strain (at any time) does not include any immediate elastic strain caused by loading and any shrinkage or swelling caused by changes in moisture. Basically, it is made up of a basic creep and a drying creep component. The sum of basic and drying creep is referred as total creep. To calculate the strain for total creep, the principle of superposition of effects is applied.

$\varepsilon_c(t) = \varepsilon(t) - \varepsilon_i(t_i) - \varepsilon_{sh}(t)$ ------------------------------ (1)

where ε_c = Total creep strain at any time t

ε = Total measured creep strain at any time t

ε_i = Instantaneous elastic strain recorded immediately after loading

ε_{sh} = Drying shrinkage strain at any time t (deterimned on unloaded specimen)

The results as presented in Figure 4.17 shown that M3-0, M3-50 and M3-100 exhibited similar trend in the development of total deformations. However, M3-50 and M3-100 were relatively higher than the control mix by about 8% and 15% respectively. After subjecting to the sustained stresses for about 7 months, the average total deformations of two specimens for each concrete mixes remained in a narrow band (2100–2600 microstrain).

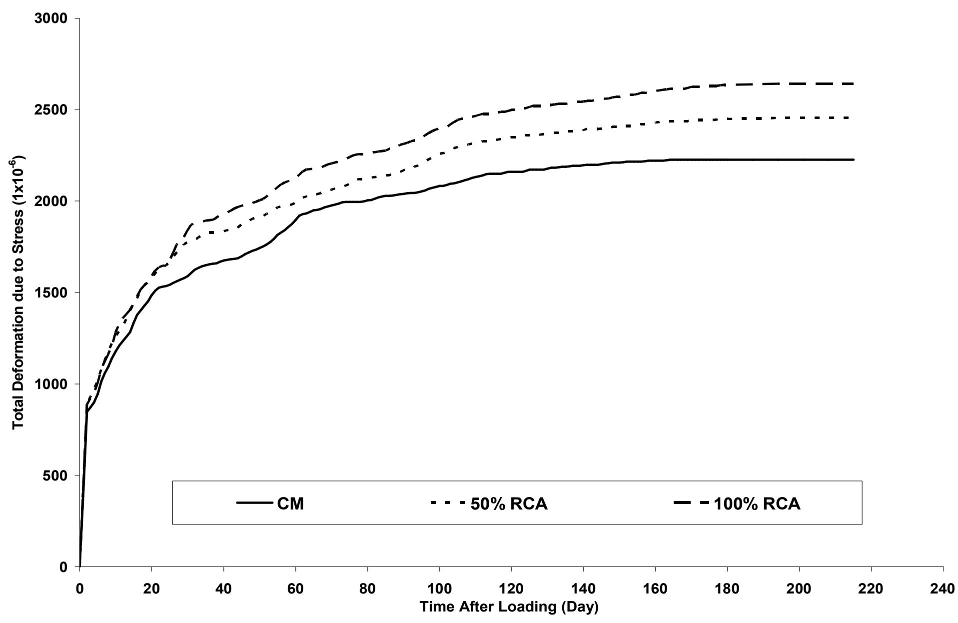


Figure 4.17 Average total deformations for M3-0, M3-50 and M3-100

Though the measured results made it possible to determine the total deformation, they did not provide a profound comparison of the creep potential in different concrete mixes since the control mix and RCA concrete showed different deformation values under compressive loading. In addition, the applied stresses to each group of specimens were different in order to maintain the 40% ultimate compressive strengths. As such, it is more judicious to carry out the comparison with the specific creep whereby it can be calculated by taking the total creep strain divided by the applied stress. As shown in Figure 4.18, the specific creep increased with the increase in RCA replacement level. With respect to M3-0, the increase in specific creep was about 18% and 25% for M3-50 and M3-100 respectively. The increase in specific creep for M3-100 was close to the observation made by Domingo-Cabo (2009). The parameters influencing creep are similar to that in the case of shrinkage. The increment in creep strains may be attributed to the lower restraining capacity of RCA particles and the presence of creeping mortar component in RCA (Ravindrarajah & Tam, 1985). Moreover, the porosity of RCA plays a direct role in the transfer of moisture within concrete and this transfer produces conditions conducive to the development of drying creep (Neville, 1995).

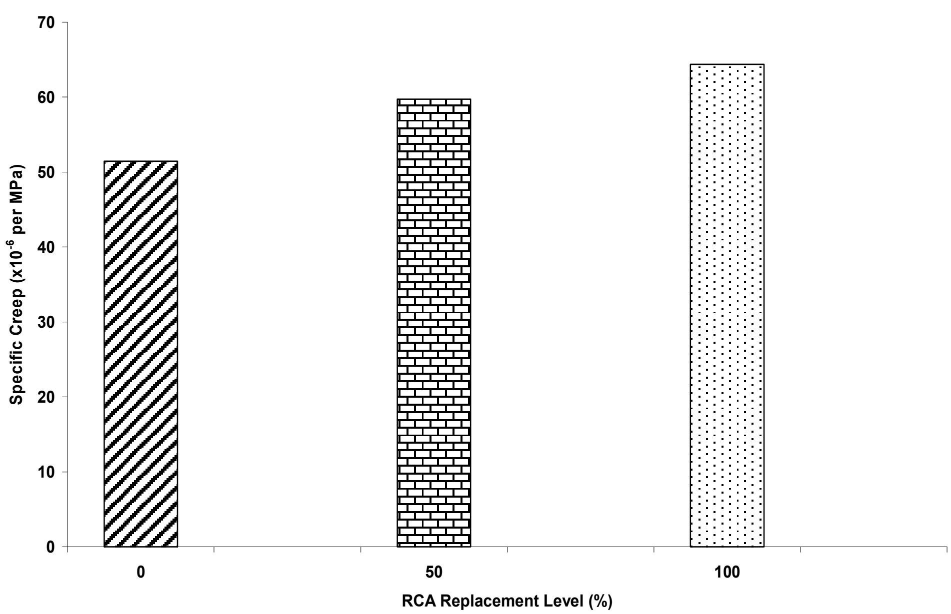


Figure 4.18 Specific creep for M3-0, M3-50 and M3-100 after 180 days of loading

In contrast, studies by Limbachiya et al (2000), Limbachiya et al (2004) and Gómez-Soberón (2009a) reported more than 50% higher specific creep when compared with control mix for up to 100% RCA replacement level. The variation in the results as compared to this study can be attributed to a number of factors. First of all, it has been found that the creep-stress relationship is non-linear for all values of stress, but approximately linear in the stress range generally used (between 0.4 and 0.6 of the ultimate compressive strength) (Young et al, 1995; Mindess et al, 2003). In accordance with Figure 4.9, the compressive strength developments of M3-0, M3-50 and M3-100 were fairly consistent and comparable. As a result, the variations in the creep of control mix and RCA concrete should be within a confined range.

Besides, it is really the hydrated cement paste which undergoes creep while the aggregate is usually not liable to creep under the stresses existing in the concrete. Creep is, therefore, a non-linear function of the volumetric content of cement paste in concrete (Neville, 1995). As the role of the aggregate in concrete being primarily that of restraint, the increase in the aggregate content by volume of 65% to 75% can decrease creep by 10% (Neville, 1964). Again, the direct replacement method adopted in this study tends to offer relatively higher volume of aggregate content in RCA concrete than that of control mix and this may help to mitigate the adverse effect of low elastic modulus of RCA in the creep of concrete to a certain extent. Furthermore, at a constant W/C ratio, the variations in the volume of hydrated cement paste for the different concrete mixes should be fluctuating within a narrow range and hence, the effect of hydrated cement paste in creep is consistent among the various concrete mixes. Unlike many other studies, in order to achieve equivalent strengths for both conventional and RCA concrete, additional cement content was added to RCA concrete and as a result, the formation of larger volume of hydrated cement paste may cause higher creep deformation.

It is also of great importance to determine the creep coefficients as they are very useful when estimating the extended deflections in concrete structures. The merit of this approach is that it takes into account the elastic properties of aggregate, which influence creep and the elastic deformation of concrete in a similar manner. The creep coefficient is defined as relation between the stress caused by creep at particular age and the elastic stress. It can be obtained in simplified form from the total creep strain, i.e. by considering it as proportional to the instantaneous strain at the constant level of stress. The equation used is: (Gomez et al, 2001)

$\varepsilon_c(t) = \varnothing(t) \times \varepsilon_i(t_0) = \varnothing(t) \times [\sigma_c / E(t_0)]$ ------------------------------ (2)

where ε_c = Total creep strain at any time t

ε_i = Instantaneous elastic strain recorded immediately after loading

$\varnothing$ = Creep coefficient at any time t

σ_c = Constant stress applied to the concrete

E = Elastic modulus of concrete for the instant t_0

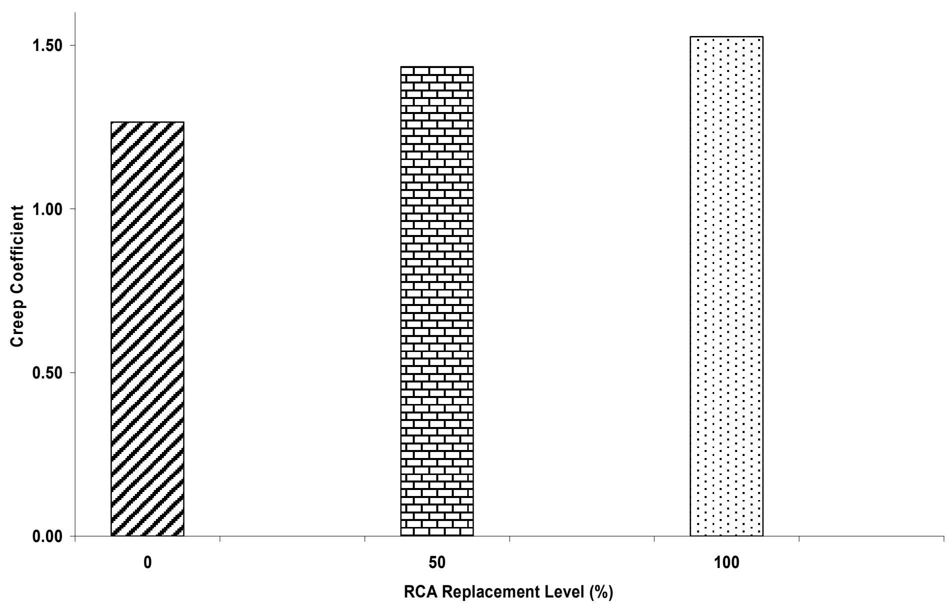


Figure 4.19 Creep coefficient for M3-0, M3-50 and M3-100 after 180 days of loading

After loading for 180 days, the creep coefficient values of M3-0, M3-50 and M3-100 were 1.27, 1.43 and 1.53 respectively (Figure 4.19). By comparison with that of the control mix, the creep coefficients of M3-50 and M3-100 were about 13% and 21% higher respectively. These coefficient values were marginally larger than those reported by Domingo-Cabo (2009), ranging from 1.1 to 1.3, and Gómez-Soberón (2009a), ranging from 0.8 to 1.2, for up to 100% RCA replacement level.

4.3 Durability characteristics of hardened concrete

4.3.1 Drying and wetting test

The durability of concrete structures is always a general concern for engineering design, maintenance and management. Moisture can either be a transport medium of external aggressive agents, namely, carbon dioxide, sulphate and chloride ions, or be itself a reactant of a degradation process, such as alkali-silica reaction. As concrete surface is exposed to drying and wetting cycles by environmental actions, there is a tendency that the capillary absorption mechanism is being activated. As a result, the actions will facilitate the moisture transport and ingression of external agents. Therefore, it is of great interest to examine the influence of RCA in concrete when the specimens are subjected to drying and wetting.

In this study, the initial drying shrinkage and wetting expansion of concrete were determined by the changes in length of the 50 x 50 x 200mm prisms after curing in accordance to Section 5 of BS 1881-5 (1970). Figure 4.20 gives a brief description of the conditioning of specimens in order to determine the following properties:

a) Initial drying shrinkage - The difference between the length of prism moulded and cured and its length when subsequently dried; and
b) Wetting expansion – The difference between the length of prism of matured concrete after immersion in water and its length when subsequently dried.

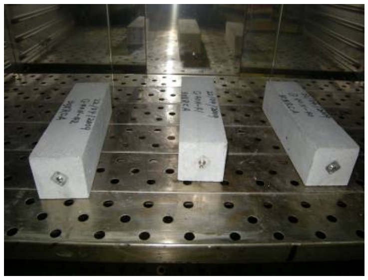


Figure 4.20 Drying with a conditioned oven and wetting in a water bath

The observed changes in length for M3-0, M3-50 and M3-100, expressed as a percentage of the length of prism, are presented in Figure 4.21. Regardless of the concrete mixes, two general trends are observed and highlighted as follows:-

a) The wetting expansion increases with the increase in the initial drying shrinkage; and
b) The initial drying shrinkage has a higher magnitude than the corresponding wetting expansion.

These observations can be explained by the mechanism of irreversible shrinkage of concrete. Mindess et al (2003) revealed that one important aspect concerning the drying shrinkage of cement paste, as well as that of concrete, is the fact that part of the total shrinkage that occurs on the first drying is irreversible, even after prolonged storage in water. Thus, the subsequent volume expansions that occur on rewetting and the volume of contractions that occur on subsequent drying are smaller. The absence of fully reversible behaviour is probably due to the introduction of additional bonds within the gel during the period of drying when closer contact between the gel particles is established (Neville, 1995). In addition, Feldman (1969) pointed out that for usual range, the irreversible part of shrinkage in concrete is about 30% to 60% of total drying shrinkage, the lower value being more common. In other words, the reversible part of shrinkage is between 40% and 70% of the total drying shrinkage. As shown in Figure 4.21, most of the results agreed with the suggested range and the shrinkage recoveries, in all cases, were nearer to the upper limit of the range.

Apart from the aspect of drying shrinkage, the swelling of concrete is also a concern as it provides a good indication on the extent of the moisture content being transported back into the structures of concrete, which may be accompanied with the ingress of aggressive ions. This swelling is identified by the increase in the length of specimen as the water molecules tends to act against the cohesive forces and tend to force the gel particles further apart with a resultant swelling pressure when the water is absorbed by the cement gel. On top of that, the ingress of water decreases the surface tension of the gel and a further small expansion take places (Powers, 1959).

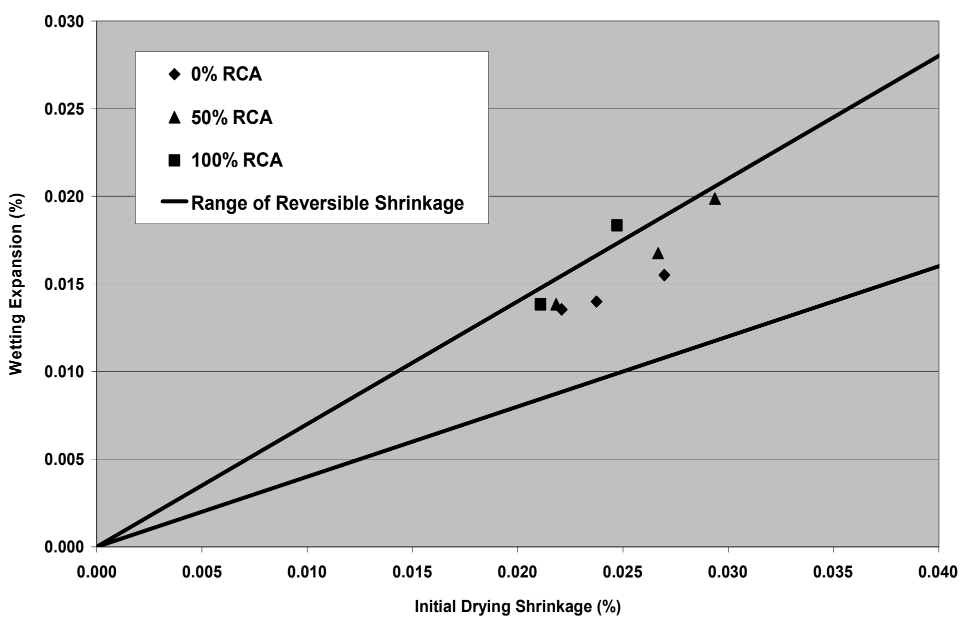


Figure 4.21 Relationship between initial drying shrinkage and wetting expansion of M3-0, M3-50 and M3-100

Overall, it seemed that both the initial drying shrinkage and wetting expansion increased with the increase in RCA replacement level. This scenario could be due to the attached mortar of RCA as it is the nature of hydrated cement paste that contributes to the bulk shrinkage and swelling of concrete. At the same W/C ratio and degree of hydration, the deviations in the porosity of hydrated cement paste of concrete should be negligible for the different concrete mixes. However, in the case of RCA concrete, the mortar component adhered on the surface of RCA may further add on to the volume of existing hydrated cement paste and in turn, the increment in the porosity of the concrete specimen. This circumstance may present a favourable path for the moisture to ingress and egress from concrete. Furthermore, the lower restraining effect of RCA may also further enhance the volume instability of concrete. Thereby, the RCA concrete tend to experience higher initial drying shrinkage and wetting expansion than those of control mix. Even so, the higher wetting expansion of RCA concrete may not be solely due to the negative impacts of RCA as there is a possibility that if the water storage periods are of sufficient duration, the continued hydration of cement results in some additional swelling so that there is a net increase in dimensions superimposed on the reversible movement due to drying and wetting (Lea, 1970). Although RCA concrete exhibited higher initial drying shrinkage and wetting expansion, the differences were minor and thus, it exemplified that the properties of RCA do not have significant influence over the moisture movement in concrete. Most importantly, the results have illustrated that the detrimental effects of RCA on the integrity of concrete is minimal.

4.3.2 Water absorbability

In accordance to BS 1881-122 (1983), the specimens are oven-dried in a well ventilated drying oven at 105 ± 5°C for 72 ± 2 hours and followed by immersion in water for 30 ± 0.5 minutes. The measured absorption of each specimen shall be calculated as the increase in mass resulting from immersion expressed as a percentage of the mass of the dry specimen. Figure 4.22 shows the testing procedures for carrying out the water absorbability test of hardened concrete.

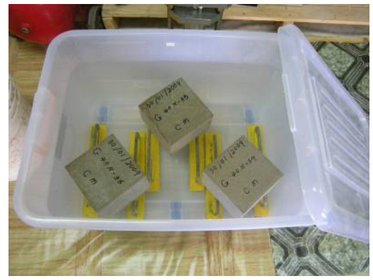


Figure 4.22 Conditioning of specimens for the determination of water absorbability

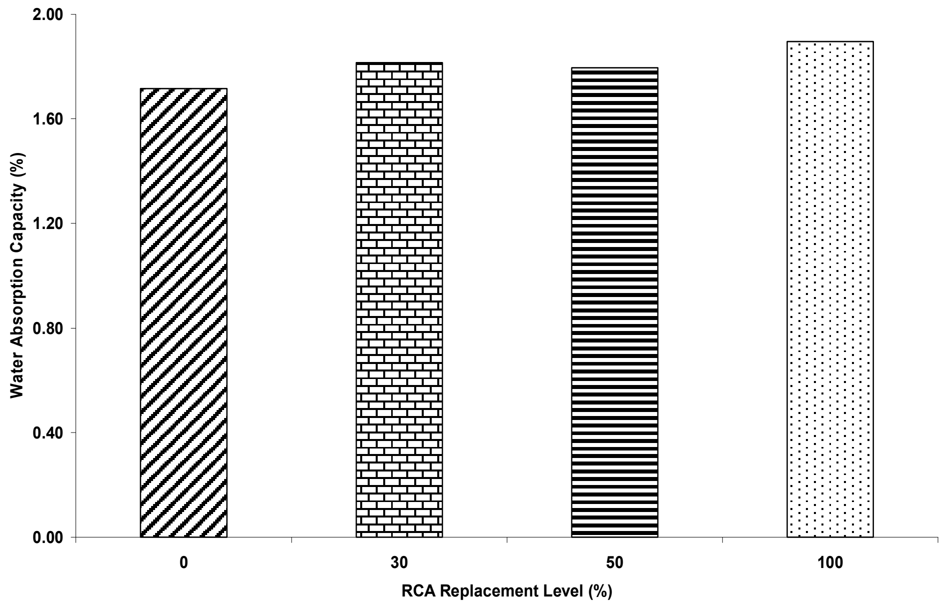


Figure 4.23 Water absorbability of control mix and RCA concrete with various replacement levels

As it could be seen in Figure 4.23, the water absorbability of concrete increased with the increase in RCA content, but the increments were limited between around 6% and 17% for concrete with different RCA replacement levels in relation to control mix. Due to the drying process before immersion in water, there is a creation of moisture gradient and it involves the capillary movement in the concrete pores. Owing to the adhered mortar on the aggregate particles, its porosity will contribute further to the overall volume of pores within a concrete specimen. Consequently, the moisture movement tends to be more intense with the increase in RCA replacement level. Despite the comparatively higher water absorbability of RCA concrete, they are graded as considerably good concrete since Neville (1995) indicated that most of the good concrete should have absorption well below 10% by mass. Based on the test results, the increase in masses were between 1.63% and 1.90% for M3-30, M3-50 and M3-100.

4.3.3 Initial surface absorption test

Figure 4.24 shows the setting up of testing apparatus for the evaluation of the resistance of hardened concrete to the ingress of water under the influence of atmospheric pressure. On the other hand, one of the specimens was spilt into halves to identify the waterfront mark at the end of the testing period. Less than 20mm of the depth of water penetration from the tested surface was identified for the various concrete mixes. Thus, the reinforcement bars embedded in the concrete can be protected from corrosion by external water source since the reinforced concrete has a typical cover of 40mm to 75mm for reinforcement bars.

Figure 4.24 Typical setup of initial surface absorption test, showing the waterfront mark after testing

The initial surface absorption values for control mix and different RCA concrete mixes exhibited similar trend as that of water absorbability. With reference to Figure 4.25, M3-0 had the lowest ISAT values at all the stipulated intervals of 10 minutes (ISAT-10), 30 minutes (ISAT-30) and 60 minutes (ISAT-60) from the start of the test. On the other hand, ISAT values for RCA concrete with different replacement levels at all the intervals were relatively higher than that of control mix and the differences were restricted to less than 20%. Moreover, ISAT values of M3-30 and M3-50 were almost identical except at the initial point whereas there is an obvious increase in ISAT values when the RCA content was increased up to 100%. This phenomenon can be due to the residual mortar attached to the aggregate particles which serve as a potential conduit for moisture transport. The high ISAT-10 of M3-100 can be associated with the reduction in effective path length for moisture movement and increase in the effective areas over which flow can take place as the porosity of RCA concrete is relatively higher than that of control mix. As time goes by, the capillary pores near the surface of the tested specimen are saturated with water for the control mix and RCA concrete. Furthermore, since the different concrete mixes had undergone similar pre-conditioned treatment, the dilution of moisture gradient should be consistent with the increase in distance away from the surface. Thereby, the absorption rate should decay and the ISAT-60 for different concrete mixes should fall within a closer range than that of the ISAT-10.

The test results are more readily to be related to the quality of surface finishing and the durability of the surface under the effects of natural weathering. The Concrete Society Working Party (1987) had established a classification table, as presented in Table 4.4, for a straightforward interpretation of the ISAT values on the quality of the concrete mixes. With reference to the results shown in Figure 4.25, the concrete qualities of all the mixes were summarized and presented in Table 4.5.

Table 4.4 Classification for quality of concrete based on ISAT (Concrete Society Working Party, 1987)

Concrete Quality	Absorption Rate (ml/m^2/s)		
	10 minutes	30 minutes	1 hour
Good	< 0.25	< 0.17	< 0.10
Average	0.25 – 0.50	0.17 – 0.35	0.10 – 0.20
Poor	> 0.5	> 0.35	> 0.20

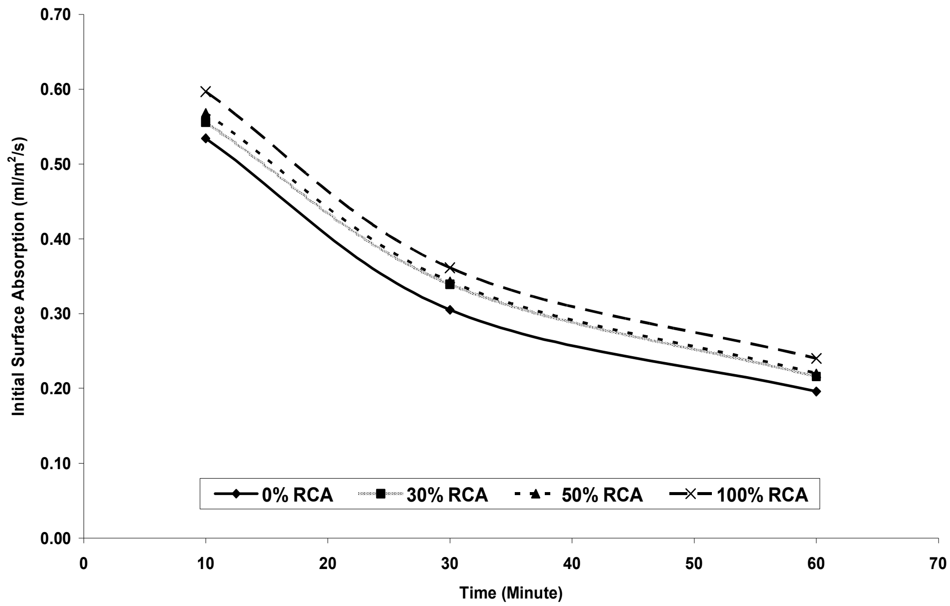


Figure 4.25 Initial surface absorption over time for concrete with different RCA replacement levels

Table 4.5 Concrete quality of control mix and RCA concrete based on ISAT results

Mix	Test Duration from the Start of Test (minutes)		
	10	30	60
M3-0	Poor	Average	Average
M3-30	Poor	Average	Average
M3-50	Poor	Average	Poor
M3-100	Poor	Poor	Poor

Nonetheless, these results can be misleading as the mass of water absorbed by concrete during the test is largely dependent on the pre-existing moisture content in the concrete. In view of this concern, all the specimens were conditioned at 105 ± 5°C and the surfaces were cooled down to room temperature prior to testing. Thus, this procedure might establish a higher moisture gradient between the internal structure of concrete and surface to be tested as compared to that of the non-over dried specimen. As a result, the ISAT values might increase accordingly and result in wrong categorization of the concrete qualities.

Although an ISAT-10 value of 0.5 ml/m^2/s is usually considered to be high, Price & Bamforth (1993) reported that a value of 1.0 ml/m^2/s is approximately the upper limit of performance for typical natural aggregate concrete. The results obtained in this study would suggest that the all the concrete mixes have a reasonably average quality surface and will provide satisfactory resistance to ingress of harmful substances.

4.3.4 Water permeability

The permeability of concrete plays a vital role in durability because it controls the rate of entry of moisture that may contain aggressive chemicals as well as the movement of water during drying and wetting. Thus, it is of great interest to determine the permeability of RCA concrete since the porosity of adhered mortar has long been confronted for the adverse effects on the durability characteristics of concrete. In this study, the permeability of various concrete mixes was measured by determining the depth of penetration of waterfront within the split specimen after subjecting to a highly pressurized water source (5 bars) for three days. Figure 4.26 describes the procedures to determine the depth of water penetration through the splitting of specimen in half (perpendicularly to its face on which water pressure was applied) after the testing period and the measurement of the maximum depth of penetration under the test area.

As shown in Figure 4.27, the depth of water penetration increased with the increase in RCA content. But once again, the range of the water penetration was confined within a narrow scope of 85mm to 90mm for the control mix and concrete with different RCA replacement levels. In addition, the difference between M3-0 and M3-100 was merely about 6%. Bonzel (1966) suggested that it was possible to use the depth of penetration of water as a qualitative assessment of concrete.

- ‘Impermeable’ – Depth of less than 50mm

- ‘Impermeable under aggressive conditions’ – Depth of less than 30mm

Arrangement of specimens on the testing bays

Before splitting of specimen

After splitting of specimen

Identification of waterfront mark for penetration depth

Figure 4.26 Testing procedures to determine the depth of water penetration under pressure

Despite the fact that the depths of water penetration acquired were extremely high with respect to the classification suggested by Bonzel (1966), this test method was primarily used as a comparative approach whereby the control mix was used as a basis for comparison. This situation was arisen due to the pre-conditioning of specimens at 105 ± 5°C. The internal structure of concrete was usually saturated with moisture content after water cured for 28 days. The author had difficulties in identifying the waterfront at the end of the testing period and thus, it was necessary to oven-dried the specimens prior to testing so that more distinctive difference between the water penetration mark and the internal structure of concrete could be achieved. Thereby, this deemed necessary procedure might have resulted in very much higher permeability characteristics of all the concrete mixes.

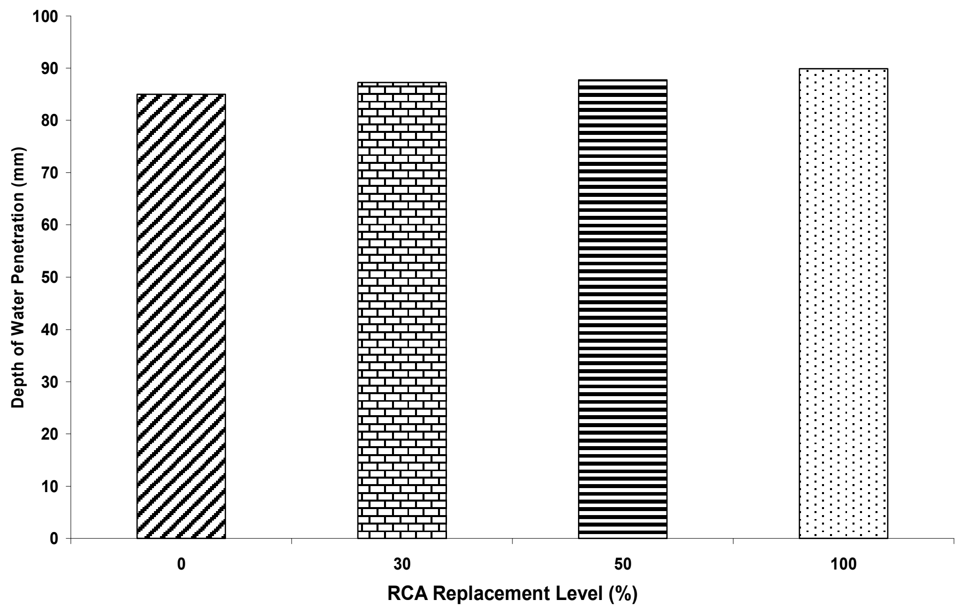


Figure 4.27 Depth of water penetration under pressure for concrete with different RCA replacement levels

With an effort to correlate the direct measurement of water flowing through a concrete specimen to the coefficient of permeability, an expression has been developed by Valenta (1969) to convert the depth of water penetration into the coefficient of permeability, which is equivalent to that used in Darcy's law:

$$K = (e^2 \times v) / (2 \times h \times t_p) \text{ ------------------------------ } (3)$$

where K = Coefficient of permeability in metres per second
e = Depth of water penetration of concrete in metres
h = Hydraulic head in metres
t_p = Time under pressure in seconds
v = Fraction of the volume of concrete occupied by pores, ranging from 0.02 to 0.06 (Vuorinen, 1985)

Based on the proposed equation, the relevant parameters were translated to the specified units and the volume of pores that might exist within the concrete was assumed to be 2% since this value was alleged to be the expected air content during the formulation of mix designs. Together with the pertinent parameters for the computation of coefficient of permeability, the values of ***K*** for the entire range of concrete mixes were presented in Table 4.6. The results illustrated that the coefficient of permeability increased with the increase in RCA content but the differences were capped at about 18%. Though the coefficient of permeability of M3-100 seemed to be the highest among all the concrete mixes, the difference could be regarded as insignificant since they were having the same order of magnitude (Neville, 1995).

Table 4.6 Coefficients of permeability of control mix and various RCA concrete

Parameters for computation of permeability coefficient	
Fraction of concrete volume occupied by pores, v	0.02 (Assumed)
Hydraulic head, h (m)	50.97
Test duration, t (s)	259200
Mix	**Coefficient of Permeability, K (m/s)**
M3-0	5.21×10^{-12}
M3-30	5.47×10^{-12}
M3-50	5.34×10^{-12}
M3-100	6.13×10^{-12}

The higher in depth of water penetration and coefficient of permeability were expected for RCA concrete as compared to the corresponding properties of conventional concrete. These were largely attributed to the porosity of attached mortar which offered a relatively shorter flow path for moisture to circumvent the aggregate particles. Nevertheless, the incorporation of RCA does not seem to have immense detrimental effects on the permeability characteristics of concrete as the differences were minor. This phenomenon can be explained by the fact that the W/C ratio is the single parameter which has the largest influence on durability. The W/C ratio has a dual role to play in concrete durability since a lower W/C ratio also increases the strength of concrete and hence, improves its resistance to cracking from the internal stresses that may be generated by adverse reactions (Mindess et al, 2003). In general terms, it is possible to say that the higher the strength of the concrete, the lower its permeability – a state of affairs to be expected because strength is a function of the relative volume of gel in the space available to it. Besides, permeability of concrete is not a simple function of its porosity but depends also on the size, distribution, shape, tortuosity and continuity of the pores. In other words, it is not appropriate to conclude that the concrete is permeable if its porosity is high (Neville, 1995).

Apart from the permeability of cement paste, permeability of aggregate particles and ITZ will also affect the behaviour of concrete. The microstructural features of ITZ in concrete are considerably different from those of the bulk matrix and ITZ is always being considered as a "weakest link" in concrete, which provides a favourable path for the transportation of water. This is even more likely since ITZ is also the locus of early microcracking. Moreover, on the average, ITZ occupies about 30% to 50% of the total volume of cement paste in concrete. For these reasons, ITZ can be expected significantly to contribute to the permeability of concrete (Young, 1988). However, Larbi (1993) found that, despite the higher porosity of ITZ, the permeability of concrete is controlled by the bulk of the hardened cement paste which is the only continuous phase in concrete. On the other hand, though the old mortar of RCA may contain additional pores, RCA are enveloped by the new cement paste so that its pores do not contribute directly and significantly to the permeability of concrete. On the whole, both the aggregate particles and ITZ do not seem to contribute much to the flow.

According to the comparable strength test results obtained in this study, the control mix and RCA concrete should achieve more or less similar permeability for the new cement paste. But the major difference still lied in the attached mortar whereby it reduced the path length for moisture movement and resulted in relatively higher permeability of concrete. Nonetheless, this effect was minimized because the permeability of hydrated cement paste was still the major contributory factor that governed the permeability of concrete. By and large, the permeability of control mix and RCA concrete should not differ much from each others.

4.3.5 Chloride ingress

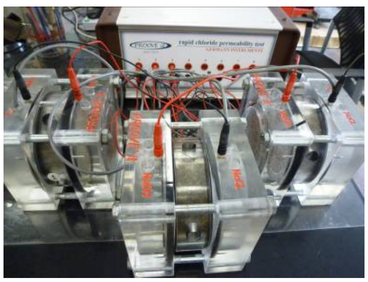
The penetration of the concrete by chloride ions is a slow process and it cannot be evaluated directly in a time frame that would be useful as a quality control measure. Therefore, in order to assess chloride penetration, a test method that accelerates the process is needed to allow the determination of diffusion values in a reasonable time. In this study, the water saturated concrete specimens (Ø100mm x 50mm) were subjected to a 60V applied DC voltage for 6 hours. In one reservoir was a 3% sodium chloride (NaCl) solution and in the other reservoir was a 0.3M sodium hydroxide (NaOH) solution. Figure 4.28 illustrates the conditioning of specimens and testing procedures to obtain the required data. The total charged passed was determined by a data logger and the values were used to classify the concrete according to the criteria included as Table 4.7.

Based on the coulombs obtained at the end of the testing duration, the concrete specimens produced with various RCA content were categorized into different permeability classes. As shown in Figure 4.29, majority of the concrete mixes were encompassed within the permeability class of ‘Moderate’ while only minorities were classified as ‘Low’. Unlike the specimens of the control mix whereby all of them had registered the charge passed within the range of 2000 to 3300As, some of the specimens made of RCA concrete had recorded with values which were less than 2000As. On average, the coulombs for M3-0, M3-30, M3-50 and M3-100 were ranged between 2100 and 2530As and interestingly, a downward trend were observed as RCA replacement level increased.

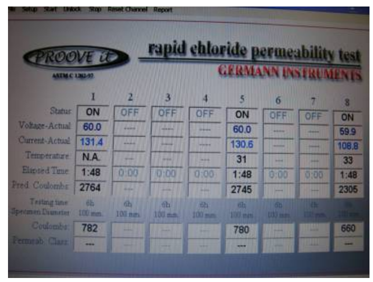
This phenomenon can be attributed to the latent chloride binding capacity of cement hydrates contained in RCA.

Conditioning of specimens before testing

Filling of cells with sodium chloride and sodium hydroxide solution



Setting up of testing apparatus for rapid chloride permeability test



Computer interface for interpretation of results

Figure 4.28 Testing procedures for rapid chloride permeability test

Table 4.7 Chloride ion penetrability based on charge passed (ASTM C1202, 2009)

Charge passed (Coulombs)	Chloride Ion Permeability
> 4000	High
2000 – 4000	Moderate
1000 – 2000	Low
100 – 1000	Very low
< 100	Negligible

In actual fact, it is not the total chloride content that is relevant to corrosion. The rate of chloride penetration into concrete is affected by the chloride binding capacity of the concrete. Concrete is not inert relative to the chlorides in the pore solution. A part of the chlorides is physically bound by the adsorption on the surface of C-S-H while another part of the chlorides is chemically bound by the products of hydration of cement, namely AFm phase (a group of minerals). It is only the third part of the chlorides, known as free chlorides, that is available for the aggressive reaction with steel. Nevertheless, the distribution of the chloride ions among the three forms is not permanent as there is an equilibrium situation that some free chloride ions are always present in the pore solution. It follows that only the chloride ions in excess of those needed for this equilibrium can become bound (Neville, 1995; Marinescu & Brouwers, 2009). As the binding removes chlorides from the pore solution, it affects the time for corrosion initiation in two ways: (1) The chloride ingress rate is decreased and (2) the chloride threshold value, expressed as total chloride content (free and bound), is increased (Nilsson et al, 1996). Other than the composition of cement, the resistance to chloride diffusion also depends on concentrations of free chlorides in the pore solution, the W/C ratio, temperature, degree of carbonation and pH of the pore solution (Marinescu & Brouwers, 2009).

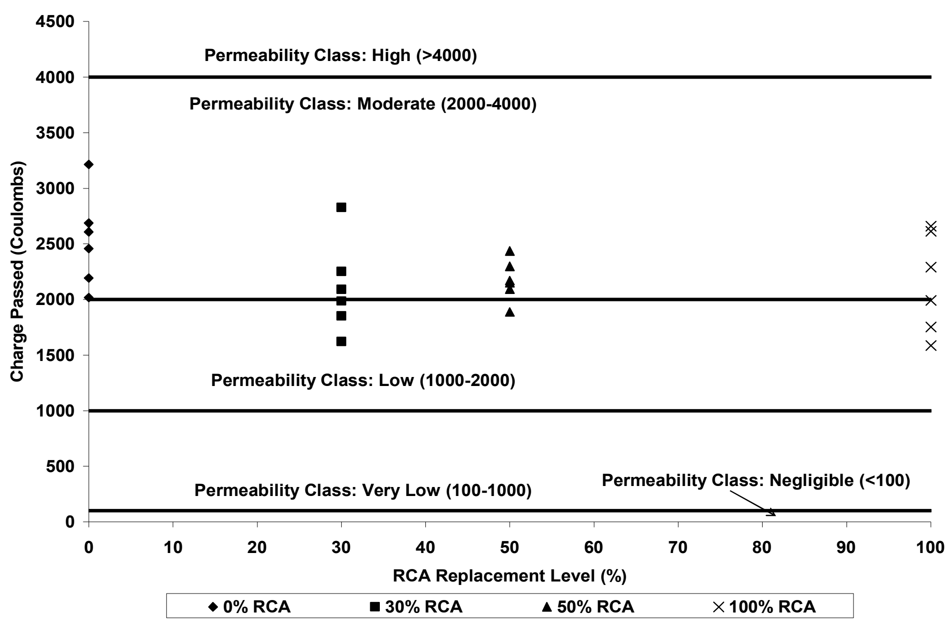


Figure 4.29 Classification of chloride permeability class of M3-0, M3-30, M3-50 and M3-100

With the presence of adhered mortar on the surfaces of RCA, it may provide additional source of CSH and AFm phase to enhance the chloride binding capacity of RCA concrete. Conversely, the influence of porosity on the chloride ingress rate must be taken into account and these two effects may contrast each other or one prevails over the other. Hence, the chloride binding capacity of RCA concrete has to be demonstrated and quantified. Villagrán-Zaccardi et al (2008) confirmed that the influence of W/C ratio on the chloride ingress rate was more important than that of aggregate porosity. This report also revealed that due to the introduction of supplementary binding capacity to the cement matrix, RCA concrete possessed a larger binding capacity than conventional concrete. However, the key factors contributing to the binding capacity of the matrix were the type and the amount of cementitious materials, with the former exerting a greater influence than the latter. Last but not least, although the chloride binding capacity of RCA series studied was greater than that of the equivalent conventional series, it was compensated by the higher chloride ingress rate.

The findings by Villagrán-Zaccardi et al (2008) will possibly be applied in this study for the better performance of RCA concrete over the control mix in the aspect of chloride diffusion. Based on the same type of cement used and constant W/C ratio, all the concrete mixes should demonstrate some level of consistencies in the chloride binding capacities and exhibit the chloride ingress rates in a close range. Owing to the presence of old mortar, RCA concrete may acquire more products of the hydration of cement than that of the control mix. As a result, the excess free chloride ions that exist in pore solution will be more readily bound by the hydration products of cement, both chemically and physically. Hence, the concentrations of free chlorides in the pore solution due to the ionic movement decreased with the increase in RCA content. In turn, the total charge passed registered by the data logger will be reduced accordingly. Since the porosity tends to modify the structure of the ITZ and develop into a preferential path for ion transport, the advantage of the higher chloride binding capacity of RCA concrete may be overridden partially by their porous nature.

4.3.6 Sulphate resistance

Concrete is rarely, if ever, attacked by solid and dry chemicals. In order to produce significant attack on concrete, corrosive chemicals must be in solution form and above some minimum concentration. Thus, the standard exposure solution used in this study contains 50g/L of sodium sulphate and the concrete prisms were immersed in the solution for up to 15 weeks (refer to Figure 4.30). The length change was measured using a length comparator at the stipulated intervals.

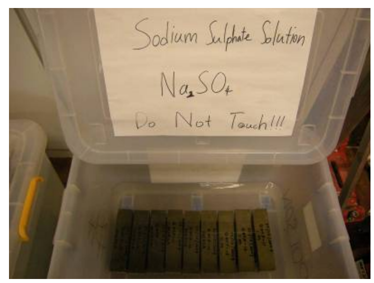


Figure 4.30 Immersion of specimens in sulphate solution

The linear expansions (on longitudinal surface) of M3-0, M3-50 and M3-100 were presented in Figure 4.31 and they were found to increase with the increase in RCA content. However, the expansions remained within a relatively narrow band (0.0103% – 0.0120%) and did not show any significant difference at 95% confidence level. In addition, RCA concrete has a rapid expansion at the early stage and stayed put after 10 weeks of monitoring whereas the control mix expanded at a fairly constant rate throughout the testing period. The trend observed can be due to the readily available pathways for the ion diffusion within the internal structure of RCA concrete and an equilibrium state will be realized sooner as the concentration gradient tends to diminish at a faster pace.

Again, the expansion experienced by the prisms can be partially owing to the continued hydration of cement results in some additional swelling so that there is a net increase in dimensions. This scenario is made possible since the specimens were stored in the solution, which is mainly made up of water, for a sufficient duration. Moreover, the tested prisms did not seem to behave in unusual ways such as warping by placing on a plane surface or the development of visible cracks. But only some whitish deposits on the surfaces of the specimens were being noticed.

These observations implied that the performance of RCA concrete, in terms of resistance to external sulphate attack, is comparable to that of control mix and the influence of RCA in the migration of sulphate ions from an outside source is minimal. As long as the sulphate content of RCA is maintained at the acceptable level, RCA concrete should not be susceptible to the damage from sulphate attack.

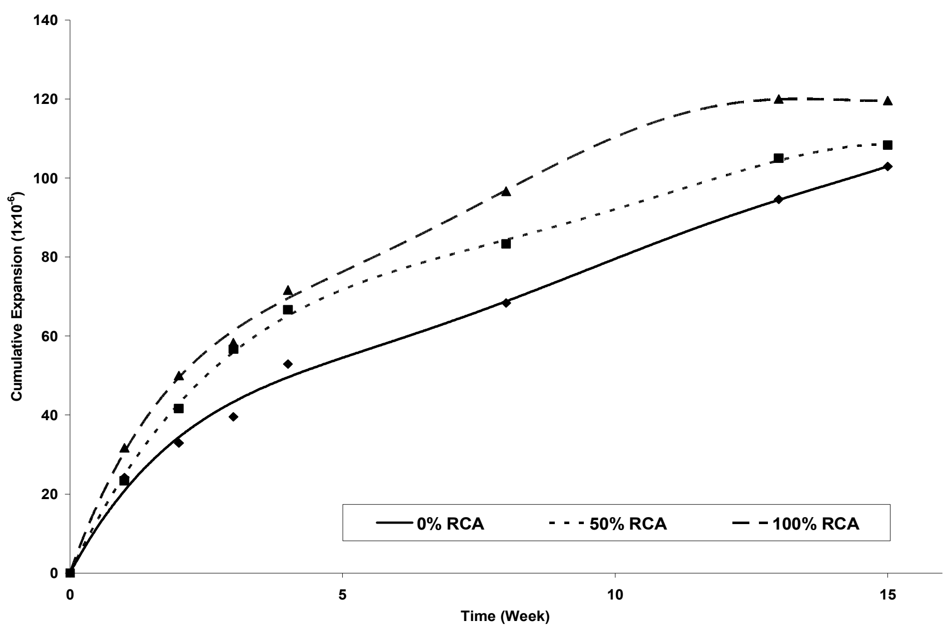


Figure 4.31 Effect of sulphate solution on the concrete mixes with various RCA replacement levels

Generally speaking, the movement of the various fluids through concrete takes place not only by flow through the porous system but also by diffusion and sorption. Thus, penetrability is the main concern for the durability characteristics of concrete. Among the many variables that control the penetrability of concrete, porosity is the key factor as fluids do not easily move through very small gel pores but controlled by an interconnecting network of capillary pores. However, the penetrability of concrete and its porosity do not have a direct relationship because of the dependency on the shape, size and concentration of the gel particles as well as the network of capillary system. In turn, the development of the capillary porosity of hydrated cement pate is largely dependent on the W/C ratio and degree of hydration. As such, these two parameters will have dominant influence in the penetrability of concrete.

Other than the aspect of the structure of hardened cement paste, the inclusion of aggregate will affect the behaviour of concrete too since concrete is a heterogeneous system. But the effects of aggregate on the penetrability of concrete can be quite complex as it exerts two opposite possessions in concrete. The aggregate particles tend to dilute and imply the tortuosity effects to reduce the flow coefficient while on the other hand, the establishment of ITZ and percolation effects increase the flow coefficient of concrete. Nonetheless, the influence of the aggregate fraction is still predominant over the effect of ITZ. In addition, the permeability of the aggregate itself does not contribute significantly to the permeability of concrete as the pores in aggregate are usually discontinuous and the aggregate particles are enveloped by the cement paste.

In conclusion, though both the hydrated cement paste and aggregate have their own impacts in the durability of concrete, it seems that the structure of the former is of paramount importance over the effect of the latter. In the case of concrete with the incorporation of RCA, the test results obtained for the evaluation of its durability characteristics in this study agree with the above-mentioned mechanisms. The expected deterioration of RCA concrete in term of penetrability is not realized. Furthermore, the increase in the diffusivity, permeability and sorptivity of RCA concrete were commonly capped at 20% for the different test methods being carried out. This phenomenon can be explained by the fact that the major drawback of RCA for its application in structural concrete lies in the attached mortar and it is due to the presence of old mortar that the porosity of RCA concrete is relatively higher than the corresponding property of control mix. Albeit the additional porous ITZ formed in RCA concrete, its function as a potential conduit for moisture transport will be restricted by the integrity of hydrated cement paste. Based on the proposed rational concrete mix design method, comparable strength properties among RCA concrete and the control mix are achievable with constant W/C ratio and same degree of hydration. It will ensure the consistency in the structure of hydrated cement paste and consequently, penetrability of the concrete produced with different RCA content.

Chapter 5 Conclusion and Recommendations

5.1 Conclusion

This study demonstrated the use of RCA in concrete with a proposed rational method of replacement of NCA with RCA. Laboratory tests were performed to investigate the aggregate properties of NCA and RCA, which was obtained from a proper processing plant. The key properties of RCA are summarized as below.

- The contaminant level of RCA collected from the recycling plant is less than 6% of total contaminants and its acid-soluble sulphate content is 0.4%. Thus, these results indicated that the constituent materials of RCA were in compliant with BS 8500-2 (2006).
- For the geometrical properties, the particle size distribution of RCA is able to fit nicely into the upper and lower limits as specified in SS 31 (1998). On the other hand, due to the crushing process, RCA generally tends to be more angular in shape and possesses a relatively lower flakiness index as compared to that of NCA.
- The presence of adhered mortar at the surface of aggregate particles has caused RCA to be more porous and resulted in lower particle density but very much higher water absorption capacity as compared to the corresponding properties of NCA.
- The development of microcracking within the attached mortar during the crushing process can be the contributory factor for lower mechanical strengths of RCA in terms of Los Angeles abrasion test, aggregate crushing test, aggregate impact test and ten percent fine value. However, the values are all within the respective limits as stated in SS 31(1998).
- RCA is considered stable volumetrically since the average change in length of the specimens is much lower than the maximum value for the drying shrinkage of aggregates used in structural concrete as specified in BS EN 12620: 2002+A1 (2008).

- The alkali-silica reactivity and total chloride content of RCA were investigated for its chemical stability. The test result revealed that the chloride content of RCA is well below the threshold value while the chemical and mortar bar test results have proven that RCA is innocuous to alkali-silica reaction.
- As a whole, RCA utilized in this study has fulfilled the fundamental requirements for the properties of aggregate to be used in the production of concrete, especially for structural applications.

Next, the concrete mix designs were formulated for various W/C ratios, containing the full range of RCA contents. The rule of thumb is to keep the W/C ratio constant for the same concrete grade at all time. At the same time, the correct dosages of water-reducing agent to be used were determined so that the losses in consistence due to the incorporation of RCA were taken into account. The compressive strengths of the RCA concrete and control mix were assessed at the appropriate curing ages. For low- and medium-strength concrete (M1 to M3), the trends for 28-day strength were either increasing or at least comparable to control mixes with the increase in RCA content; whereas the compressive strengths of M4 and M5 exhibited downward trends for 28-day strength as the replacement levels increased but the reductions were not significant based on Student's Test at 5% significance level. This scenario can be attributed to two factors: effective W/C ratio and mechanism of ITZ. For the hardened concrete with W/C ratios more than 0.45 (M1 to M3), the influence of lower effective W/C ratio surpasses the adverse effect of 'new ITZ' (between RCA and new mortar matrix) while for the case of W/C ratios less than 0.45 (M4 and M5), the contradictory effect occurs. This situation is largely due to the significant increase in the coarse/fine aggregate ratio for achieving the designed strength. Consequently, there is an increase in the 'new ITZ' fraction and the effect of reduction in effective W/C ratio is compensated by the effect of weak bonding strength between the new mortar matrix and RCA.

Based on the findings, the concrete design mix M3 was selected for the subsequent research works. Since M3 mix has a W/C ratio of 0.45 with a cement content of 390 kilograms per cubic meter of concrete and a characteristic strength of 40MPa at 28-day, it can fulfill the mix design requirements for the reinforce concrete with moderate exposure condition during its service life whereby the mix design limits for maximum free W/C ratio, minimum cement content and minimum concrete grade are usually fixed at 0.60, 300 kilograms per cubic meter of concrete and 35MPa respectively. Furthermore, the compressive strength of medium-strength concrete is not affected significantly by the incorporation of RCA, even up to 100% replacement level. Thus, M3 with the inclusion of 0%, 30%, 50% and 100% RCA content were produced for specimen preparation for the investigation of other hardened concrete properties. Further tests on other engineering properties, including durability, had been carried out to evaluate the performance of RCA concrete for structural applications, namely long-term compressive strength, elastic modulus, flexural strength, indirect tensile strength, drying shrinkage, creep, drying and wetting, water absorption capacity, initial surface absorption, water permeability, chloride ingress and sulphate resistance. The observations made from these tests were highlighted as follows.

- Compressive strength – Comparable compressive strengths were obtained for concrete with and without RCA content, even up to 100% replacement level. For the long-term effect of RCA on the strength property of concrete, specimens were prepared and crushed after water-curing for 90, 180 and 365 days. For the entire series of concrete mixes, the strength development basically ceased after 90 days and the test results also revealed that RCA does not have any detrimental effect on the strength development of concrete.
- Elastic modulus - The elastic moduli of all the concrete mixes were within the range of 27GPa to 30GPa and the results did not show any significant difference at 95% confidence level between the control mix and the respective RCA concrete.

- Strengths in tension – The tensile strength of hardened concrete were determined by the splitting tensile test and flexure test (third-point loading). The flexural strengths of the control mix and RCA concrete increased with the increase in RCA content whereas the splitting tensile strengths experienced an opposing outcome. At 95% confidence level, the splitting tensile strengths of M3-30, M3-50 and M3-100 did not show any significance difference when compared with that of M3-0. Conversely, there was no significance difference in flexural strengths among M3-0, M3-30 and M3-50 but the flexural strength of M3-100 was significantly higher than that of M3-0.
- Drying shrinkage – After subjecting to air-curing for about 7 months, the length changes of the control mix and RCA concrete exhibited similar trend with regard to the rate of shrinkage. The drying shrinkage strains of test specimens increased with time and stabilized after monitoring for 56 days. The maximum shrinkage strains registered by M3-50 and M3-100 were about 16% and 40% higher than that of M3-0 respectively. Nonetheless, the drying shrinkage results for all the concrete mixes are within the allowable limit for basic shrinkage strain of normal-class concrete as specified in AS 3600 (2001).
- Creep – The concrete specimens, produced with 0%, 50% and 100% RCA content, were subjected to sustained loading at a stress/strength ratio of 0.40 for about 7 months. The total deformations for the various concrete mixes remained in a narrow range of 2100 to 2600 microstrain. However, the creep strains of M3-50 and M3-100 were relatively higher than that of the control mix by about 8% and 15% respectively. On the other hand, the specific creep also increased with the increase in RCA replacement level. With reference to M3-0, the increase in specific creep was about 18% and 25% for M3-50 and M3-100 respectively. Based on the total creep strains recorded after loading for 180 days, the creep coefficient values were computed. The respective calculated result for M3-0, M3-50 and M3-100 was

1.27, 1.43 and 1.53. By comparison with that of the control mix, the creep coefficients of M3-50 and M3-100 were about 13% and 21% higher respectively.

- Drying and wetting – Two general trends were observed for the length changes in specimens, regardless of RCA replacement levels, under the drying and wetting conditions.
    - The wetting expansion increased with the increase in the initial drying shrinkage; and
    - The initial drying shrinkage had a higher magnitude than the corresponding wetting expansion.

  Based on the absolute figures, the reversible part of shrinkages for M3-0, M3-50 and M3-100 were ranged between 58% and 70% of the total drying shrinkage. This observation agreed with the range suggested by Feldman (1969) whereby the irreversible part of shrinkage is about 30% to 60% of total drying shrinkage, the lower value being more common. Overall, both the initial drying shrinkage and wetting expansion increased with the increase in RCA replacement level. Although RCA concrete exhibited higher initial drying shrinkages and wetting expansions than the control mix, the differences were minor.
- Water absorbability – The increase in masses of M3-0, M3-30, M3-50 and M3-100 were between 1.72% and 1.90%. Though the water absorbability increased with the increase in RCA content, there were only marginal increments of 5% to 10% for concrete with different RCA replacement levels in relation to the control mix.
- Initial surface absorption – As expected, M3-100 displayed the highest ISAT values at all the intervals among the different concrete mixes whereas M3-0 experienced the lowest ISAT values throughout the testing period. On other hand, ISAT values of M3-30 and M3-50 coincided with each other at the stipulated intervals. Comparing to that of control mix, RCA concrete recorded higher ISAT values, ranging from 6% to 20%, for the different intervals.

- Water permeability – The range of the water penetration registered in this study was between 85mm and 90mm for the control mix and concrete with different RCA replacement levels. Even though the depth of water penetration increased with the increase in RCA content, the variation between M3-0 and M3-100 was simply about 6%. In addition, the expression developed by Valenta (1969) to convert the depth of water penetration into the coefficient of permeability (***K***), which is equivalent to that used in Darcy's law, illustrated that the ***K*** values increased with the increase in RCA content, ranging from 5.47×10^{-12} to 6.13×10^{-12} m/s, but the differences were restricted to about 12%. Though the coefficient of permeability of M3-100 seemed to be the highest among all the concrete mixes, the difference could be regarded as insignificant since they were having the same order of magnitude (Neville, 1995).
- Chloride ingress – On average, all the concrete mixes were classified as 'Moderate' for the chloride ion permeability class. Surprisingly, the coulombs acquired by all the specimens of the control mix were greater than 2000As and capped at 3300As. In opposition, the coulombs recorded by the specimens of RCA concrete were between 1600As and 2900As. This unexpected phenomenon can be attributed to the unique characteristic of RCA whereby the attached mortar contributes to the latent chloride binding capacity of cement hydrates.
- Sulphate resistance – Both M3-50 and M3-100 exhibited a relatively rapid increase in lengths at the early stage and stabilized after monitoring for 10 weeks whereas the control mix expanded at a fairly constant rate throughout the testing period. The linear expansions of M3-0, M3-50 and M3-100 were found to increase with the increase in RCA content. However, the differences remain within a relatively narrow band (0.0103% – 0.0120%) and did not show any significant difference at 95% confidence level.

In summary, it is crucial to formalize a quality control protocol for the production of aggregates recovered from the inert waste in order to safeguard the integrity of the concrete with the incorporation of RCA. The establishment of acceptance criteria to provide adequate assurance in the quality of RCA is deemed necessary. Despite the differences in properties between RCA and NCA, concrete containing RCA can be formulated and proportioned so there is minimal, if any, impact on either the fresh or the hardened properties of new concrete. In fact, it is the effect of a given RCA on the performance of concrete that is most important and a more logical approach will be to relate limits on allowable RCA on performance-based properties. In a nutshell, RCA concrete can be designed at its own right. The test data obtained in this research study have clearly demonstrated that equivalent mechanical properties of RCA concrete, comparing with those of control mix, are achievable without adjustment to W/C ratio. In contrast, though the other engineering properties and durability characteristics of RCA concrete maybe more inferior than the corresponding properties of conventional concrete, the differences are either within the tolerable levels or inconsequential. Nevertheless, these differences are unsatisfactory to preclude the use of RCA, which offers economic, environmental and potential performance benefits. As a whole, it is feasible to produce RCA concrete with the proposed rational method by replacement of the NCA with RCA and addition of different dosages of water-reducing agent.

5.1.1 Research limitations

Due to the timeline of the study, the evaluation of creep deformation was ceased after loading for about 7 months. Thus, the creep effects of RCA on concrete may not be fully realized and long period of testing is preferred. Moreover, only the total creep strains were captured in the study while the elastic recovery, creep recovery and residual strain of the specimens, which are being observed upon the removal of sustained load, were unable to be determined in time.

5.1.2 Research contributions

Based on the favourable laboratory test results, the study has demonstrated the utmost feasibility of using RCA in the production of concrete and grants a groundbreaking technique to upgrade the use of C&D waste for more beneficial applications. With these findings, the concrete design mixes determined in the study were applied in the construction of a 3-storey office building known as Samwoh Eco-Green Building. The accomplishment of the office building marks a significant milestone in the modern concrete technology and has opened a new chapter in Singapore's sustainable development. In addition, it also provides the following key advantages:

a) Significant contributions to the existing literature since little works had been reported on the actual usage of RCA in structural building;
b) The pioneering country in this region to use high dosage of RCA in structural building;
c) The real-time data collected from the structural health monitoring system can be used to evaluate the effect of RCA concrete on the structure;
d) Both the laboratory and field data collected are valuable to BCA for the formulation of specifications and expansion of existing building codes to permit the use of RCA in structural concrete; and
e) A great leap of faith in the use of recycled materials by the stakeholders and the confidence to future users of RCA in structural concrete can be enhanced.

As such, it is hopeful that RCA can be used in future structural buildings which will reduce the dependence on natural aggregate and thereby, contribute towards the nation's goal to achieve sustainable development.

5.2 Recommendations

5.2.1 Enhancements to current research

By comparison of the individual mixture results, its accuracy and reliability are always doubtful. With the integration of three basic and classical concepts of mix design: 'Abrams' law' for hardened concrete, 'Lyse's law' for fresh concrete and 'Molinari's law' for cement content, a graphical form is able to be established once the trial mixes are performed. The use of this graphical presentation, which is termed as mix design nomogram (MDN), introduced by Monteiro et al (1993) provides complete and fast prediction of fresh and hardened concrete properties. It allows the researchers to select the most adequate mix parameters for experimental and scientific purposes as well as the determination of a mix proportion for a specified property. It is noted that when studying the effect of mix parameters on the properties of concrete, certain constraints should be imposed.

Based on the results obtained in this study, it seems that the W/C ratio is the most important variable in the concrete mix proportion and it can significantly change the properties of hardened concrete. To avoid its influence when studying the effect of other variable, such as the replacement level of RCA, the W/C ratio should be kept constant. Upon keeping the W/C ratio constant, any changes observed can be attributed to the influence of RCA in the concrete mix proportion. Thus, the compressive strength MDN was developed for the control mixes coupled with concrete containing up to 100% RCA content at various W/C ratios, as indicated in Figure 5.1. The main purpose of the MDN is to provide some form of guidelines to the future users of RCA concrete as it sets the boundaries for strength property of concrete with and without RCA content at different W/C ratios, encompassing from low- to high-strength concrete. Thereby, the intermediate class of concrete can be interpolated from the MDN. Likewise, though the optimal percentage of RCA for usage in structural concrete may not be 100% for majority of the cases, the other dosages of RCA content can also be determined with the aids of this graph.

Furthermore, the principles of the proposed rational method coincides with the rules of thumb for MDN whereby the effects of the mix parameters on the properties of concrete are easy to determine once the analysis is performed under two conditions: constant consistence and constant W/C ratio. However, since there will be a loss in consistence due to the absorptive nature of RCA, this study has adopted the approach of utilizing additional water-reducing agent for the increase in RCA replacement level while fixing the other variables. In this way, the constant consistence is maintained without adjusting the water and cement content. As a result, higher dosage of water-reducing agent is expected with the increase in RCA content. The recommended range of dosages of water-reducing agent for the control mixes and 100% RCA concrete at various W/C ratios are presented in Figure 5.2.

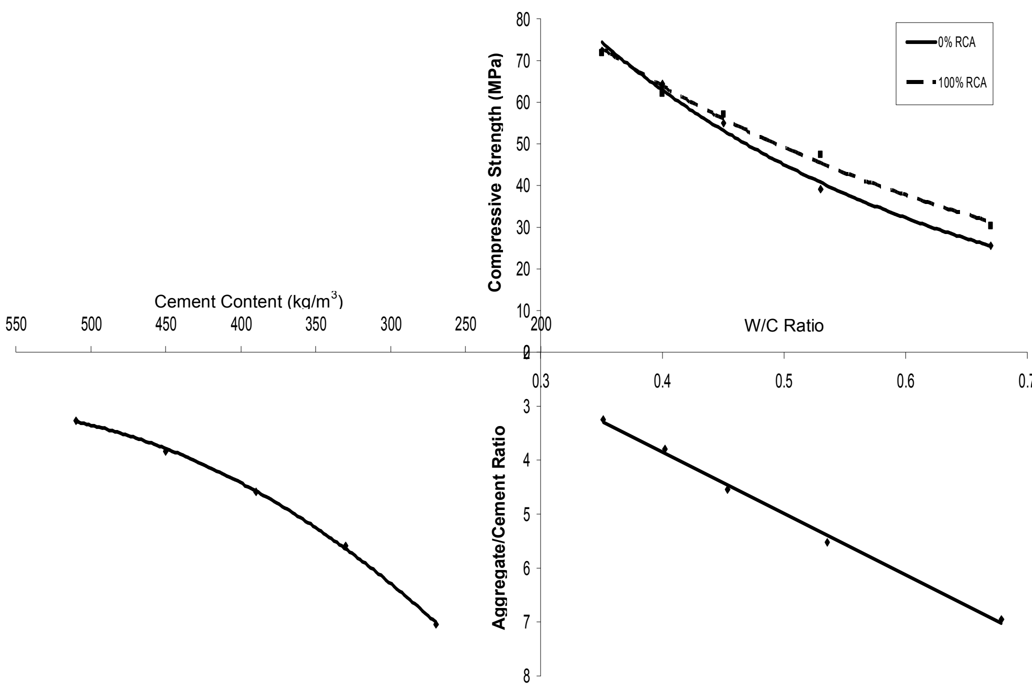


Figure 5.1 Compressive strength MDN for control mixes and concrete produced with 100% RCA content

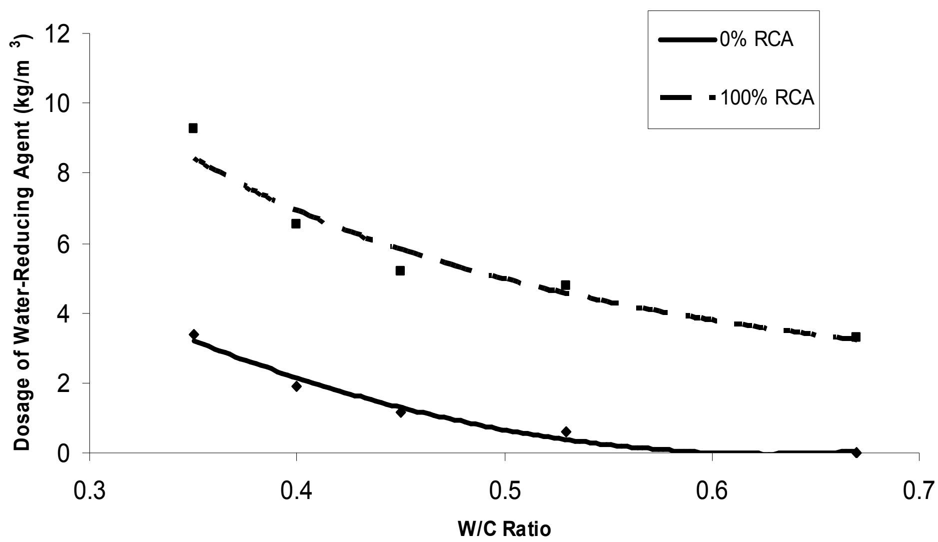


Figure 5.2 Dosage of water-reducing agent for various W/C ratios, with and without RCA content, to achieve initial slump of 125mm

Apart from the extensive laboratory works, the practicability of using RCA concrete in actual site casting is also essential. With the experiences gained in the construction of Samwoh Eco-Green Building, some areas of improvement on the pre-conditioning of RCA and application of RCA concrete have been identified.

Firstly, at the ready-mixed concrete plant, it is advisable to pre-wet the RCA at the stockpile area so as to minimize the absorptive effect of the adhered mortar, especially during hot weather. Moreover, sufficient time has to be allowed for the water to penetrate into the pores of RCA in order for it to get close to the SSD condition. However, extra care has to be taken as over spraying of water may result in the introduction of additional free water content in the concrete matrix. Thereby, it is critical for the industry or concrete supplier to establish an operational protocol to regulate the procedures for pre-treatment of RCA prior using it for the concrete production at plant. The protocol may include the methods for spraying water at the RCA stockpile and the ways to evaluate the instant moisture content.

Secondly, proper curing of concrete on site is definitely a vital procedure in order to safeguard the integrity of the structural building, especially for mass casting such as floor slabs. This good construction practice is imperative, in particular, for the case of RCA concrete. In fact, the initial setting time of RCA concrete tends to be shortened with the increase in replacement level and it is important to highlight to the users that the application of curing compound or water should be carried out earlier, instead of the typical interval of about 2 hours after casting. If not, plastic shrinkage will take place rapidly and numerous hairline cracks will be formed. This scenario is attributed to the slump loss issue due to the incorporation of RCA in the concrete mix proportion. Thereby, more leeway should be given to the initial slump of RCA concrete with respect to that of conventional concrete. By doing so, the slump loss due to the travelling distance and nature of casting works can be accounted for. Therefore, it is more appropriate to achieve the target slump on site rather than using the initial slump as the acceptance criteria for a concrete mix. Consequently, further works should emphasize on the monitoring of changes in slump values over time and even the utilization of admixtures with better performance range. Moreover, similar to the compressive strength MDN approach to analyze the effects of RCA in concrete, the MDN can also be established for the other engineering properties of hardened concrete, namely, elastic modulus, flexural strength, permeability, carbonation and etc.

Besides, the structural health monitoring system implemented in Samwoh Eco-Green Building is able to capture the real-time deformations of columns, which are constructed with RCA concrete, under the influence of the dead and live load of the building. The data obtained will be useful for the analysis of the long-term performance of the building, especially the drying shrinkage and creep effect of concrete, and it will contribute significantly to the existing building codes. Till date, the monitoring of the displacements in columns is still in progress and the results acquired from the analysis will be published shortly.

5.2.2 Extensions to future research

There are growing interests in the adoption of RCA in concrete applications. However, this recycled material has long been confronted with questions on the economic feasibility and environmental impacts due to lacks of unbiased, accurate and verified information. Thus, suggestions are made for future research in order to address these concerns.

a) To evaluate the cost for the production of one tonne of RCA and NCA;
b) To assess the carbon footprints for the production of one tonne of RCA and NCA. The carbon footprint shall be quantified as the sum of carbon emissions from consumptions of all materials, energy and waste across the activities in the productions for the two different aggregates;
c) Cost benefits analysis of replacing NCA with RCA in concrete; and
d) To carry out comparative environmental assessment of concrete with and without RCA.

For both (a) and (b), the analysis shall be performed on the processes for raw material extraction, pre-production (transportation of raw material to the production line), production (processing of raw material), post-production (transportation of processed material to stockpile) and distribution to customers. On the other hand, it is necessary that the different concrete types fulfil similar functional requirements in order to compare the economical and environmental aspects of all the concrete mixes. This means that the concrete mixes have approximately the same mechanical strength and durability characteristics. With this constraint in mind, the concrete mix designs of control mix and RCA are determined so that both types of concrete have the same compressive strength and workability. Besides, the control mix shall always be used a basis for comparison in all cases. The functional unit of one cubic metre of control mix and RCA concrete is proposed to be used in the future works.

The overall goal of cost and environmental declarations is to encourage the demand for and supply of recycled material that cause less stress on the economic and environment. Thereby, this goal will simulate the potential market-driven continuous environmental improvement. Using data produced by the studies will also enhance the confidence to stakeholders, designers and end users who wish to ensure that they have taken the full account of the economic issues and life cycle environmental impacts of the recycled material that they are using.

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Appendix A

Data for Compressive Strength of Concrete with Various W/C Ratios and RCA Replacement Levels

Compressive Strength of M1 (MPa)						
Curing Age (Day)	**RCA Replacement Level (%)**					
	0	**20**	**40**	**60**	**80**	**100**
3	15.4	17.2	19.8	19.4	20.9	20.6
	18.4	16.9	19.0	18.9	19.6	21.5
	17.2	19.1	18.5	20.4	20.1	21.3
7	20.4	23.0	23.6	25.5	25.3	25.1
	20.5	26.1	24.1	26.0	25.1	26.1
	20.7	24.3	24.4	27.5	23.5	24.8
28	25.8	30.6	30.2	31.5	30.1	30.2
	25.1	28.2	28.9	32.0	30.3	31.0
	26.1	28.3	29.2	33.6	31.6	29.8

Compressive Strength of M2 (MPa)						
Curing Age (Day)	**RCA Replacement Level (%)**					
	0	**20**	**40**	**60**	**80**	**100**
3	23.9	25.2	28.2	27.5	31.1	32.3
	22.9	24.9	27.6	27.3	32.2	33.5
	24.1	24.9	27.4	27.4	31.7	32.8
7	29.4	32.9	36.5	35.7	38.2	40.0
	30.5	33.0	36.5	34.8	38.0	38.7
	31.3	33.8	35.6	33.4	35.2	37.6
28	39.5	42.9	44.3	41.0	45.6	47.2
	38.8	41.7	44.9	40.6	47.3	47.3
	39.3	43..5	46.0	41.7	48.1	47.6

Compressive Strength of M3 (MPa)						
Curing Age (Day)	**RCA Replacement Level (%)**					
	0	**20**	**40**	**60**	**80**	**100**
3	33.9	36.0	38.7	40.9	43.0	42.6
	34.4	35.5	39.3	43.5	42.0	44.0
	34.1	36.7	41.8	41.8	42.7	43.6
7	47.5	49.5	51.0	47.7	51.3	50.5
	46.4	48.2	48.1	49.1	49.4	50.5
	46.5	46.9	47.6	49.6	50.9	49.4
28	55.5	55.8	58.8	58.2	59.6	58.1
	55.9	54.9	59.7	55.7	56.9	56.2
	54.0	57.0	58.7	57.1	60.0	57.2

Compressive Strength of M4 (MPa)						
Curing Age (Day)	**RCA Replacement Level (%)**					
	0	**20**	**40**	**60**	**80**	**100**
3	43.4	44.1	47.1	45.1	45.3	47.1
	41.4	44.0	47.2	42.9	45.9	45.0
	40.7	42.1	46.1	45.7	45.6	47.1
7	51.4	57.4	58.8	54.0	53.1	54.9
	53.0	55.3	58.7	53.9	53.2	52.9
	51.2	57.9	58.0	55.0	54.9	51.7
28	62.9	66.0	68.3	62.0	59.9	60.8
	65.0	67.4	68.5	63.2	62.1	63.6
	65.6	67.1	67.9	64.5	60.7	61.9

Compressive Strength of M5 (MPa)						
Curing Age (Day)	**RCA Replacement Level (%)**					
	0	**20**	**40**	**60**	**80**	**100**
3	50.2	55.5	56.5	55.1	53.0	55.4
	50.0	54.6	57.4	54.4	54.1	54.3
	48.8	55.0	55.7	54.1	54.8	53.4
7	61.4	62.4	64.3	63.5	66.6	65.2
	58.8	62.2	64.8	64.6	64.3	64.3
	57.4	62.0	65.5	66.1	63.1	66.5
28	72.7	77.7	75.4	68.2	72.4	72.4
	72.0	79.6	79.8	71.0	69.9	71.1
	72.2	76.6	77.9	74.7	68.4	71.9

Appendix B

Data for Long-term Strength Development of M3-0, -30, -50 and -100

Long-term Strength Development of M3 (MPa)				
Curing Age (Day)	RCA Replacement Level (%)			
	0	30	50	100
3	35.0	41.8	40.5	38.1
	35.5	38.6	39.6	37.3
	35.5	39.7	40.7	35.4
	38.4	40.2	41.0	37.1
	40.3	41.9	41.5	35.8
	38.6	40.9	40.3	35.8
	31.4	37.3	36.5	36.3
	30.5	35.3	37.8	37.6
	30.3	35.3	38.0	36.1
7	44.9	46.9	48.4	49.2
	45.4	49.0	48.1	45.4
	46.8	48.7	47.3	45.4
	45.2	49.3	50.1	44.6
	45.4	46.8	49.2	43.1
	46.4	49.3	49.4	41.0
	48.4	49.2	46.1	46.8
	43.9	49.4	46.3	47.8
	43.7	49.6	46.9	48.5
28	49.3	53.9	56.9	56.9
	54.4	53.8	58.8	57.9
	55.6	53.0	56.7	58.2
	52.0	53.8	54.7	53.8
	51.5	57.8	56.1	55.9
	51.0	53.7	57.0	53.8
	57.1	56.8	59.7	58.4
	55.5	58.3	56.5	56.2
	54.4	56.3	57.8	53.8

Long-term Strength Development of M3 (MPa)				
Curing Age (Day)	**RCA Replacement Level (%)**			
	0	**30**	**50**	**100**
90	58.1	59.2	59.1	56.7
	53.7	57.2	57.4	54.6
	55.4	57.1	56.9	57.3
	55.1	55.3	56.3	57.5
	55.1	56.3	54.8	59.4
	54.5	56.1	57.2	61.4
	63.8	61.2	59.2	57.5
	60.7	58.8	56.7	59.4
	60.8	59.9	56.9	61.4
180	54.6	55.6	56.8	53.5
	55.8	59.3	56.4	61.3
	55.9	56.9	58.8	54.7
	51.9	54.4	54.9	55.3
	55.6	58.4	55.1	56.7
	52.7	57.5	57.9	56.0
	61.0	59.0	59.5	56.9
	61.0	57.1	62.3	56.7
	61.6	58.3	61.1	56.7
365	53.3	55.1	58.0	57.5
	52.5	52.7	60.7	55.4
	52.2	54.3	56.9	55.1
	53.0	58.2	55.1	56.1
	52.0	55.2	56.9	55.9
	52.0	53.6	54.0	56.0
	60.1	60.0	60.1	57.4
	59.4	58.4	62.1	58.6
	61.5	59.5	58.4	57.8

Appendix C

Data for Other Mechanical Strengths of M3-0, -30, -50 and -100

Elastic Modulus of M3 (GPa)				
Curing Age (Day)	RCA Replacement Level (%)			
	0	30	50	100
28	24.9	30.1	26.9	29.2
	22.4	29.5	28.7	28.9
	27.5	31.0	25.0	28.8
	27.5	27.6	28.3	28.4
	28.0	30.2	29.8	27.7
	28.8	31.4	28.4	28.9
	32.3	29.2	27.8	30.1
	30.0	28.8	26.7	30.8
	32.3	27.4	28.4	30.3

Flexural Strength of M3 (MPa)				
Curing Age (Day)	RCA Replacement Level (%)			
	0	30	50	100
28	3.02	3.52	3.12	4.02
	3.04	2.84	3.34	4.44
	2.85	3.25	3.15	3.85
	3.32	3.52	3.92	3.48
	3.54	3.44	3.24	3.68
	3.85	3.45	3.55	4.43
	3.96	4.23	3.96	3.90
	3.31	3.94	3.94	3.65
	3.61	3.96	3.55	3.53

Splitting Tensile Strength of M3 (MPa)				
Curing Age (Day)	RCA Replacement Level (%)			
	0	30	50	100
28	3.22	3.16	3.56	3.26
	2.99	3.26	3.20	3.07
	3.31	2.98	3.45	3.50
	3.43	3.37	3.34	2.99
	3.24	3.26	3.30	2.48
	3.03	3.04	3.18	3.45
	3.50	3.57	3.16	2.84
	3.44	3.13	3.42	3.14
	3.84	3.34	3.05	3.22

Appendix D

Data for Drying Shrinkage of M3-0, -50 and -100

Appendix D

Observing Period (Day)	Measurements for M3-0							Ave. ΔL (Division)	Ave. Shrinkage Strain (1×10^{-6})	Ave. Cum. Shrinkage Strain (1×10^{-6})
	Reference Rod (Division)	Prism 1		Prism 2		Prism 3				
		Length (Division)	ΔL (Division)	Length (Division)	ΔL (Division)	Length (Division)	ΔL (Division)			
0	1539	3000	0	2811	5	2807	0	0	0	0
1	1531	2985	7	2804	3	2790	9	6	31	31
2	1532	2983	3	2821	-13	2789	2	3	13	44
3	1533	2981	3	2801	17	2789	1	2	9	53
4	1532	2976	4	2803	-2	2786	3	3	13	67
5	1535	2976	3	2805	-1	2784	5	4	19	85
6	1535	2970	6	2802	5	2780	4	5	24	110
7	1535	2973	-3	2803	-3	2782	-2	-2	-11	99
8	1535	2969	4	2800	3	2777	5	5	23	122
9	1531	2963	2	2785	11	2769	4	3	15	137
10	1535	2979	-12	2796	-7	2787	-14	-13	-65	72
11	1532	2957	19	2796	0	2767	17	18	88	160
12	1532	2959	-2	2794	2	2767	0	-1	-6	155
13	1533	2963	-3	2793	1	2770	-2	-2	-10	145
14	1535	2959	6	2791	2	2770	2	4	21	165
15	1533	2961	-4	2788	3	2768	0	-2	-12	153
16	1532	2956	4	2811	5	2765	2	3	14	168
17	1528	2942	11	2804	3	2752	9	10	51	219
18	1532	2951	-6	2821	-13	2761	-5	-6	-30	189
19	1532	2952	-1	2801	17	2761	0	0	-1	188
20	1532	2951	1	2803	-2	2759	2	2	8	196
21	1532	2950	1	2805	-1	2758	1	1	5	201
22	1532	2946	4	2802	5	2757	1	2	11	212
23	1532	2943	3	2803	-3	2753	4	3	17	229

Observing Period (Day)	Measurements for M3-0							Ave. ΔL (Division)	Ave. Shrinkage Strain (1×10^{-6})	Ave. Cum. Shrinkage Strain (1×10^{-6})
	Reference Rod (Division)	Prism 1		Prism 2		Prism 3				
		Length (Division)	ΔL (Division)	Length (Division)	ΔL (Division)	Length (Division)	ΔL (Division)			
24	1535	2946	0	2791	0	2756	0	0	0	229
25	1534	2946	-1	2791	-1	2755	0	-1	-3	225
26	1536	2951	-3	2795	-2	2762	-5	-3	-17	208
27	1534	2944	5	2788	5	2754	6	5	27	235
28	1533	2942	1	2786	1	2753	0	1	3	239
29	1532	2938	3	2781	4	2748	4	4	19	258
30	1533	2945	-6	2788	-6	2754	-5	-6	-29	229
31	1533	2941	4	2784	4	2751	3	4	18	247
32	1533	2941	0	2785	-1	2752	-1	-1	-3	244
33	1533	2940	1	2784	1	2750	2	1	6	250
34	1533	2940	0	2784	0	2751	-1	0	-2	249
35	1529	2932	4	2778	2	2745	2	3	13	262
36	1531	2934	0	2780	0	2745	2	1	3	265
37	1531	2936	-2	2780	0	2746	-1	-1	-5	261
38	1531	2936	0	2780	0	2747	-1	0	-2	259
39	1530	2931	4	2776	3	2741	5	4	20	279
40	1532	2933	0	2777	1	2743	0	0	2	280
41	1533	2940	-6	2784	-6	2750	-6	-6	-30	251
42	1531	2931	7	2776	6	2742	6	6	31	282
43	1531	2933	-2	2776	0	2743	-1	-1	-5	277
44	1532	2934	0	2778	-1	2746	-2	-1	-5	272
45	1532	2932	2	2776	2	2744	2	2	10	282
46	1534	2932	2	2776	2	2744	2	2	10	292
47	1533	2934	-3	2776	-1	2743	0	-1	-7	285

Observing Period (Day)	Measurements for M3-0							Ave. ΔL (Division)	Ave. Shrinkage Strain (1×10^{-6})	Ave. Cum. Shrinkage Strain (1×10^{-6})
	Reference Rod (Division)	Prism 1		Prism 2		Prism 3				
		Length (Division)	ΔL (Division)	Length (Division)	ΔL (Division)	Length (Division)	ΔL (Division)			
48	1532	2931	2	2774	1	2741	1	1	7	292
49	1531	2929	1	2773	0	2740	0	0	2	294
50	1533	2933	-2	2777	-2	2744	-2	-2	-9	285
51	1533	2935	-2	2779	-2	2746	-2	-2	-9	275
52	1528	2917	13	2761	13	2730	11	13	64	340
53	1530	2923	-4	2767	-5	2734	-2	-4	-21	318
54	1531	2929	-5	2772	-4	2739	-4	-5	-23	296
55	1531	2928	1	2771	1	2739	0	1	5	301
56	1531	2925	3	2767	4	2736	3	4	18	318
63	1531	2928	-3	2770	-3	2739	-3	-3	-15	303
70	1532	2924	5	2769	2	2736	4	4	18	321
77	1533	2925	0	2768	2	2737	0	1	4	325
84	1528	2916	4	2759	4	2729	3	4	21	346
112	1532	2922	-2	2765	-2	2734	-1	-2	-11	335
140	1528	2923	-5	2764	-3	2735	-5	-4	-19	316
168	1518	2912	1	2752	2	2724	1	2	8	323
196	1220	2612	2	2453	1	2423	3	2	8	331

Note: Positive in value – Shrinkage; Negative in value – Expansion

Observing Period (Day)	Measurements for M3-50							Ave. ΔL (Division)	Ave. Shrinkage Strain (1 x 10^{-6})	Ave. Cum. Shrinkage Strain (1 x 10^{-6})
	Reference Rod (Division)	Prism 1		Prism 2		Prism 3				
		Length (Division)	ΔL (Division)	Length (Division)	ΔL (Division)	Length (Division)	ΔL (Division)			
0	1539	2873	0	2737	0	2711	0	0	0	0
1	1531	2859	7	2726	3	2702	1	4	18	18
2	1532	2857	3	2725	3	2695	8	4	22	40
3	1533	2854	4	2724	2	2695	2	2	12	52
4	1532	2848	5	2717	5	2688	5	5	26	78
5	1535	2848	3	2717	3	2688	3	3	16	93
6	1534	2843	4	2713	3	2682	5	4	20	113
7	1535	2845	-1	2715	-1	2683	0	-1	-3	110
8	1534	2840	5	2709	5	2677	5	5	24	134
9	1531	2832	5	2703	3	2671	3	4	18	152
10	1535	2849	-13	2718	-11	2686	-11	-12	-58	94
11	1531	2828	17	2699	15	2666	16	16	79	173
12	1531	2828	0	2699	0	2666	0	0	0	173
13	1532	2829	0	2700	0	2667	0	0	0	173
14	1535	2828	4	2698	5	2666	4	4	22	195
15	1532	2827	-2	2696	-1	2665	-2	-2	-8	187
16	1531	2823	3	2693	2	2661	3	3	13	200
17	1528	2813	7	2682	8	2652	7	7	36	236
18	1532	2819	-2	2688	-2	2656	-1	-2	-8	228
19	1532	2819	0	2689	-1	2656	0	0	-1	227
20	1532	2816	3	2685	4	2653	3	3	16	243
21	1532	2816	0	2685	0	2654	-1	0	-2	242
22	1533	2814	3	2683	3	2650	5	4	18	260
23	1532	2809	4	2679	3	2646	3	3	17	277

Appendix D

Observing Period (Day)	Measurements for M3-50							Ave. ΔL (Division)	Ave. Shrinkage Strain (1×10^{-6})	Ave. Cum. Shrinkage Strain (1×10^{-6})
	Reference Rod (Division)	Prism 1		Prism 2		Prism 3				
		Length (Division)	ΔL (Division)	Length (Division)	ΔL (Division)	Length (Division)	ΔL (Division)			
24	1535	2812	0	2682	0	2651	-2	-1	-3	273
25	1533	2810	0	2679	1	2647	2	1	5	278
26	1535	2816	-4	2684	-3	2653	-4	-4	-18	260
27	1533	2808	6	2677	5	2645	6	6	28	288
28	1533	2808	0	2676	1	2645	0	0	2	290
29	1531	2802	4	2670	4	2641	2	3	17	307
30	1532	2808	-5	2676	-5	2646	-4	-5	-23	283
31	1533	2805	4	2673	4	2643	4	4	20	303
32	1532	2804	0	2671	1	2641	1	1	3	307
33	1532	2804	0	2671	0	2642	-1	0	-1	306
34	1532	2805	-1	2673	-2	2643	-2	-2	-8	298
35	1529	2800	2	2667	3	2638	2	2	12	310
36	1529	2798	2	2666	1	2636	2	2	8	318
37	1530	2800	-1	2666	1	2638	-1	0	-2	317
38	1530	2799	1	2666	0	2638	0	0	2	319
39	1530	2796	3	2663	3	2633	5	4	18	337
40	1530	2795	1	2663	0	2633	1	1	3	339
41	1532	2803	-6	2670	-5	2641	-7	-6	-29	310
42	1531	2796	6	2662	7	2634	6	6	32	342
43	1530	2796	-1	2663	-2	2633	0	-1	-5	336
44	1532	2800	-2	2666	-1	2638	-3	-2	-10	326
45	1530	2797	1	2664	0	2635	1	1	4	330
46	1532	2795	4	2662	4	2634	3	4	18	348
47	1531	2796	-2	2662	-1	2634	-1	-1	-6	343

Observing Period (Day)	Measurements for M3-50							Ave. ΔL (Division)	Ave. Shrinkage Strain (1 x 10^{-6})	Ave. Cum. Shrinkage Strain (1 x 10^{-6})
	Reference Rod (Division)	Prism 1		Prism 2		Prism 3				
		Length (Division)	ΔL (Division)	Length (Division)	ΔL (Division)	Length (Division)	ΔL (Division)			
48	1531	2794	2	2661	1	2632	2	2	8	350
49	1530	2793	0	2659	1	2631	0	0	2	352
50	1532	2796	-1	2664	-3	2635	-2	-2	-9	343
51	1532	2798	-2	2666	-3	2635	0	-2	-8	335
52	1528	2783	11	2650	12	2622	9	11	53	388
53	1530	2787	-2	2653	-1	2624	0	-1	-5	383
54	1530	2792	-5	2657	-4	2629	-5	-5	-23	361
55	1531	2791	2	2656	2	2627	3	2	10	371
56	1531	2789	2	2653	3	2627	0	2	9	380
63	1531	2792	-3	2658	-5	2629	-2	-3	-17	363
70	1532	2788	5	2655	4	2627	3	4	20	384
77	1533	2789	0	2655	1	2625	4	1	7	391
84	1528	2781	3	2648	2	2619	1	2	9	400
112	1532	2787	-2	2652	0	2622	1	0	-2	398
140	1528	2782	1	2650	-2	2624	-6	-2	-12	386
168	1518	2780	-8	2644	-4	2619	-5	-6	-28	358
196	1220	2475	7	2340	6	2315	6	6	32	390

Note: Positive in value – Shrinkage; Negative in value – Expansion

Appendix D

Observing Period (Day)	Measurements for M3-100							Ave. ΔL (Division)	Ave. Shrinkage Strain (1×10^{-6})	Ave. Cum. Shrinkage Strain (1×10^{-6})
	Reference Rod (Division)	Prism 1		Prism 2		Prism 3				
		Length (Division)	ΔL (Division)	Length (Division)	ΔL (Division)	Length (Division)	ΔL (Division)			
0	1539	2873	0	2906	0	2965	0	0	0	0
1	1531	2860	5	2897	1	2953	4	3	17	17
2	1532	2857	5	2894	4	2948	6	5	25	41
3	1532	2855	2	2893	1	2945	3	2	10	51
4	1532	2850	6	2888	5	2940	5	5	25	76
5	1534	2848	4	2887	3	2938	4	4	18	93
6	1533	2842	5	2882	4	2932	5	5	23	117
7	1535	2843	1	2884	0	2933	1	1	3	120
8	1534	2839	3	2879	4	2929	3	3	17	137
9	1531	2833	3	2873	3	2922	4	3	17	153
10	1535	2847	-10	2887	-10	2934	-8	-9	-46	107
11	1531	2826	17	2868	15	2917	13	15	75	182
12	1531	2826	0	2868	0	2916	1	0	1	183
13	1532	2826	1	2868	1	2916	1	1	5	188
14	1535	2825	4	2866	5	2916	3	4	20	208
15	1532	2824	-2	2865	-2	2914	-1	-2	-8	200
16	1531	2819	4	2861	3	2910	3	3	16	216
17	1528	2810	6	2852	7	2901	6	6	31	248
18	1532	2813	1	2857	-1	2904	1	0	2	249
19	1532	2813	0	2855	2	2903	1	1	5	254
20	1532	2809	4	2852	3	2900	3	3	16	270
21	1531	2808	0	2850	1	2898	1	1	3	273
22	1533	-	-	2846	6	2896	4	5	26	299
23	1532	-	-	2841	4	2891	4	4	19	318

Appendix D

Observing Period (Day)	Measurements for M3-100							Ave. ΔL (Division)	Ave. Shrinkage Strain (1 x 10^{-6})	Ave. Cum. Shrinkage Strain (1 x 10^{-6})
	Reference Rod (Division)	Prism 1		Prism 2		Prism 3				
		Length (Division)	ΔL (Division)	Length (Division)	ΔL (Division)	Length (Division)	ΔL (Division)			
24	1534	-	-	2843	0	2893	0	0	0	318
25	1532	-	-	2839	2	2890	1	2	8	326
26	1534	-	-	2845	-4	2895	-3	-4	-18	308
27	1533	-	-	2838	6	2890	5	5	27	335
28	1533	-	-	2836	2	2888	2	2	9	343
29	1531	-	-	2832	2	2885	1	2	8	351
30	1532	-	-	2837	-4	2890	-4	-4	-20	331
31	1533	-	-	2834	5	2887	4	4	21	352
32	1532	-	-	2832	1	2884	2	1	6	358
33	1532	-	-	2830	2	2884	0	1	5	363
34	1532	-	-	2831	-1	2883	1	0	0	363
35	1529	-	-	2826	2	2880	0	1	5	368
36	1529	-	-	2824	2	2878	3	2	11	380
37	1530	-	-	2825	0	2879	0	0	-1	378
38	1529	-	-	2824	0	2878	0	0	0	378
39	1530	-	-	2820	5	2874	5	5	25	403
40	1530	-	-	2820	0	2874	0	0	0	403
41	1532	-	-	2827	-5	2881	-5	-5	-25	378
42	1529	-	-	2819	5	2873	5	5	25	403
43	1529	-	-	2819	0	2873	0	0	0	403
44	1532	-	-	2825	-3	2879	-3	-3	-15	388
45	1529	-	-	2821	2	2875	1	1	6	395
46	1530	-	-	2817	4	2873	3	4	18	412
47	1531	-	-	2819	-1	2874	0	0	-2	410

Observing Period (Day)	Measurements for M3-100							Ave. ΔL (Division)	Ave. Shrinkage Strain (1×10^{-6})	Ave. Cum. Shrinkage Strain (1×10^{-6})
	Reference Rod (Division)	Prism 1		Prism 2		Prism 3				
		Length (Division)	ΔL (Division)	Length (Division)	ΔL (Division)	Length (Division)	ΔL (Division)			
48	1531	-	-	2818	1	2873	2	1	7	417
49	1530	-	-	2816	1	2872	-1	0	1	418
50	1531	-	-	2819	-2	2874	-1	-2	-8	411
51	1532	-	-	2822	-2	2875	0	-1	-6	405
52	1527	-	-	2808	9	2861	9	9	46	451
53	1530	-	-	2811	1	2863	1	1	4	455
54	1529	-	-	2814	-5	2868	-6	-5	-27	428
55	1530	-	-	2814	1	2868	1	1	5	433
56	1530	-	-	2812	2	2865	3	2	12	445
63	1530	-	-	2814	-2	2869	-4	-3	-14	430
70	1531	-	-	2809	6	2864	6	6	31	461
77	1533	-	-	2807	4	2864	2	3	14	475
84	1528	-	-	2801	1	2857	2	2	8	483
112	1532	-	-	2807	-2	2863	-2	-2	-10	473
140	1528	-	-	2804	-1	2865	-6	-4	-18	456
168	1518	-	-	2802	-8	2859	-4	-6	-30	426
196	1220	-	-	2495	9	2554	7	8	40	466

Note: a) Positive in value – Shrinkage; Negative in value – Expansion

b) Prism 1 was damaged during the observing period and thus, the average cumulative shrinkage strain was based on the results of specimens

Appendix E

Data for Total Creep Deformation of M3-0, -50 and -100

Observing Period (Day)	Measurements for M3-0							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
0	814	839	0	0	807	0	0	0
1	813	756	835	835	724	845	845	840
2	813	754	21	856	723	10	856	856
3	812	749	42	898	720	21	877	888
4	811	746	21	919	717	21	898	909
5	811	740	63	982	709	84	982	982
6	811	738	21	1003	699	105	1087	1045
7	811	735	31	1035	697	21	1108	1072
8	811	731	42	1077	692	52	1161	1119
9	811	727	42	1119	688	42	1203	1161
10	811	724	31	1151	685	31	1235	1193
11	810	720	31	1183	681	31	1267	1225
12	809	717	21	1204	678	21	1288	1246
13	809	715	21	1225	676	21	1309	1267
14	809	712	31	1257	673	31	1341	1299
15	809	706	63	1320	666	73	1415	1368
16	810	705	21	1341	665	21	1436	1389
17	810	703	21	1362	661	42	1478	1420
18	809	701	10	1373	658	21	1499	1436
19	809	698	31	1405	655	31	1531	1468
20	809	695	31	1437	652	31	1563	1500
21	810	695	10	1448	650	31	1595	1522
22	810	695	0	1448	648	21	1616	1532
23	809	694	0	1448	647	0	1616	1532

Observing Period (Day)	Measurements for M3-0							Ave. Cum. Total Deformation (1 x 10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1 x 10^{-6})	Cum. Total Deformation (1 x 10^{-6})	Length (Division)	Total Deformation (1 x 10^{-6})	Cum. Total Deformation (1 x 10^{-6})	
24	809	694	0	1448	646	10	1627	1538
25	809	692	21	1469	646	0	1627	1548
26	809	691	10	1480	645	10	1638	1559
27	809	690	10	1491	645	0	1638	1565
28	809	689	10	1502	644	10	1649	1576
29	809	689	0	1502	643	10	1660	1581
30	809	687	21	1523	642	10	1671	1597
31	809	686	10	1534	639	31	1703	1619
32	809	685	10	1545	638	10	1714	1630
33	809	684	10	1556	637	10	1725	1641
34	809	683	10	1567	637	0	1725	1646
35	809	682	10	1578	637	0	1725	1652
36	809	682	0	1578	636	10	1736	1657
37	809	682	0	1578	636	0	1736	1657
38	809	681	10	1589	636	0	1736	1663
39	809	681	0	1589	634	21	1757	1673
40	809	681	0	1589	633	10	1768	1679
41	809	681	0	1589	633	0	1768	1679
42	809	681	0	1589	632	10	1779	1684
43	809	681	0	1589	632	0	1779	1684
44	809	680	10	1600	632	0	1779	1690
45	809	679	10	1611	630	21	1800	1706
46	809	678	10	1622	629	10	1811	1717
47	809	678	0	1622	628	10	1822	1722

Appendix E

Observing Period (Day)	Measurements for M3-0							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
48	809	678	0	1622	626	21	1843	1733
49	809	677	10	1633	626	0	1843	1738
50	808	675	10	1644	624	10	1854	1749
51	808	674	10	1655	624	0	1854	1755
52	809	674	10	1666	623	21	1875	1771
53	809	674	0	1666	621	21	1896	1781
54	810	673	21	1687	619	31	1928	1808
55	810	672	10	1698	617	21	1949	1824
56	810	670	21	1719	617	0	1949	1834
57	810	669	10	1730	615	21	1970	1850
58	810	667	21	1751	614	10	1981	1866
59	810	665	21	1772	613	10	1992	1882
60	810	662	31	1804	610	31	2024	1914
61	810	661	10	1815	609	10	2035	1925
62	809	659	10	1826	608	0	2035	1931
63	809	658	10	1837	608	0	2035	1936
64	809	658	0	1837	606	21	2056	1947
65	809	658	0	1837	605	10	2067	1952
66	809	658	0	1837	605	0	2067	1952
67	809	657	10	1848	604	10	2078	1963
68	809	656	10	1859	604	0	2078	1969
69	809	656	0	1859	603	10	2089	1974
70	809	655	10	1870	603	0	2089	1980
71	809	655	0	1870	602	10	2100	1985

Observing Period (Day)	Measurements for M3-0							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
72	809	654	10	1881	602	0	2100	1991
73	809	654	0	1881	601	10	2111	1996
74	809	654	0	1881	601	0	2111	1996
75	809	654	0	1881	601	0	2111	1996
76	809	654	0	1881	601	0	2111	1996
77	809	654	0	1881	601	0	2111	1996
78	809	654	0	1881	601	0	2111	1996
79	809	653	10	1892	601	0	2111	2002
80	809	653	0	1892	600	10	2122	2007
81	809	653	0	1892	600	0	2122	2007
82	809	651	21	1913	600	0	2122	2018
83	809	651	0	1913	599	10	2133	2023
84	809	650	10	1924	599	0	2133	2029
85	809	650	0	1924	599	0	2133	2029
86	809	650	0	1924	599	0	2133	2029
87	809	650	0	1924	598	10	2144	2034
88	809	650	0	1924	598	0	2144	2034
89	809	649	10	1935	598	0	2144	2040
90	809	649	0	1935	598	0	2144	2040
91	809	648	10	1946	598	0	2144	2045
92	809	648	0	1946	598	0	2144	2045
93	809	648	0	1946	598	0	2144	2045
94	809	648	0	1946	597	10	2155	2051
95	809	647	10	1957	597	0	2155	2056

Appendix E

Observing Period (Day)	Measurements for M3-0							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
96	809	647	0	1957	596	10	2166	2062
97	809	646	10	1968	595	10	2177	2073
98	809	646	0	1968	595	0	2177	2073
99	809	645	10	1979	594	10	2188	2084
100	809	645	0	1979	594	0	2188	2084
101	809	645	0	1979	594	0	2188	2084
102	809	644	10	1990	593	10	2199	2095
103	809	644	0	1990	593	0	2199	2095
104	809	643	10	2001	593	0	2199	2100
105	809	643	0	2001	592	10	2210	2106
106	809	642	10	2012	592	0	2210	2111
107	809	642	0	2012	591	10	2221	2117
108	809	641	10	2023	591	0	2221	2122
109	809	640	10	2034	590	10	2232	2133
110	809	640	0	2034	590	0	2232	2133
111	809	639	10	2045	590	0	2232	2139
112	809	638	10	2056	590	0	2232	2144
113	809	638	0	2056	589	10	2243	2150
114	809	638	0	2056	589	0	2243	2150
115	809	638	0	2056	589	0	2243	2150
116	809	638	0	2056	589	0	2243	2150
117	809	638	0	2056	589	0	2243	2150
118	809	637	10	2067	588	10	2254	2161
119	809	637	0	2067	588	0	2254	2161

Appendix E

Observing Period (Day)	Measurements for M3-0							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
120	809	637	0	2067	588	0	2254	2161
121	809	637	0	2067	588	0	2254	2161
122	809	637	0	2067	588	0	2254	2161
123	809	637	0	2067	588	0	2254	2161
124	809	636	10	2078	587	10	2265	2172
125	809	636	0	2078	587	0	2265	2172
126	808	635	0	2078	586	0	2265	2172
127	808	635	0	2078	586	0	2265	2172
128	809	636	0	2078	587	0	2265	2172
129	809	636	0	2078	587	0	2265	2172
130	809	635	10	2089	586	10	2276	2183
131	809	635	0	2089	586	0	2276	2183
132	809	635	0	2089	586	0	2276	2183
133	809	634	10	2100	586	0	2276	2188
134	809	634	0	2100	586	0	2276	2188
135	809	634	0	2100	586	0	2276	2188
136	809	634	0	2100	585	10	2287	2194
137	809	634	0	2100	585	0	2287	2194
138	809	634	0	2100	585	0	2287	2194
139	809	634	0	2100	585	0	2287	2194
140	809	634	0	2100	584	10	2298	2199
141	809	634	0	2100	584	0	2298	2199
142	809	634	0	2100	584	0	2298	2199
143	809	634	0	2100	584	0	2298	2199

Observing Period (Day)	Measurements for M3-0							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
144	809	634	0	2100	584	0	2298	2199
145	809	634	0	2100	583	10	2309	2205
146	809	634	0	2100	583	0	2309	2205
147	810	634	10	2111	584	0	2309	2210
148	810	634	0	2111	584	0	2309	2210
149	810	634	0	2111	584	0	2309	2210
150	810	634	0	2111	584	0	2309	2210
151	809	633	0	2111	582	10	2320	2216
152	809	633	0	2111	582	0	2320	2216
153	809	633	0	2111	582	0	2320	2216
154	810	634	0	2111	583	0	2320	2216
155	810	634	0	2111	583	0	2320	2216
156	810	634	0	2111	583	0	2320	2216
157	810	634	0	2111	582	10	2331	2221
158	810	634	0	2111	582	0	2331	2221
159	810	634	0	2111	582	0	2331	2221
160	810	634	0	2111	582	0	2331	2221
161	810	634	0	2111	582	0	2331	2221
162	810	634	0	2111	582	0	2331	2221
163	810	634	0	2111	581	10	2342	2227
164	810	634	0	2111	581	0	2342	2227
165	810	634	0	2111	581	0	2342	2227
166	810	634	0	2111	581	0	2342	2227
167	810	634	0	2111	581	0	2342	2227

Observing Period (Day)	Measurements for M3-0							Ave. Cum. Total Deformation (1 x 10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1 x 10^{-6})	Cum. Total Deformation (1 x 10^{-6})	Length (Division)	Total Deformation (1 x 10^{-6})	Cum. Total Deformation (1 x 10^{-6})	
168	810	634	0	2111	581	0	2342	2227
169	810	634	0	2111	581	0	2342	2227
170	810	634	0	2111	581	0	2342	2227
171	810	634	0	2111	581	0	2342	2227
172	810	634	0	2111	581	0	2342	2227
173	810	634	0	2111	581	0	2342	2227
174	810	634	0	2111	581	0	2342	2227
175	810	634	0	2111	581	0	2342	2227
176	810	634	0	2111	581	0	2342	2227
177	810	634	0	2111	581	0	2342	2227
178	810	634	0	2111	581	0	2342	2227
179	810	634	0	2111	581	0	2342	2227
180	810	634	0	2111	581	0	2342	2227
187	810	634	0	2111	581	0	2342	2227
194	810	634	0	2111	581	0	2342	2227
201	810	634	0	2111	581	0	2342	2227
208	810	634	0	2111	581	0	2342	2227
215	810	634	0	2111	581	0	2342	2227

Note: Loading at 3000 psi for both specimens.

Appendix E

Observing Period (Day)	Measurements for M3-50							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
0	814	778	0	0	785	0	0	0
1	813	695	857	857	702	861	861	859
2	813	689	63	920	699	31	893	907
3	812	686	21	941	691	73	967	954
4	811	681	42	983	688	21	988	986
5	811	672	94	1078	685	31	1020	1049
6	811	667	52	1131	679	63	1083	1107
7	811	662	52	1184	675	42	1125	1155
8	811	658	42	1226	670	52	1178	1202
9	811	656	21	1247	663	73	1252	1250
10	811	654	21	1268	661	21	1273	1271
11	810	650	31	1300	656	42	1315	1308
12	809	648	10	1311	653	21	1336	1324
13	809	640	84	1395	649	42	1378	1387
14	809	637	31	1427	646	31	1410	1419
15	809	633	42	1469	643	31	1442	1456
16	810	631	31	1501	641	31	1474	1488
17	810	627	42	1543	637	42	1516	1530
18	809	624	21	1564	634	21	1537	1551
19	809	624	0	1564	632	21	1558	1561
20	809	621	31	1596	629	31	1590	1593
21	810	619	31	1628	628	21	1611	1620
22	810	618	10	1639	628	0	1611	1625
23	809	617	0	1639	623	42	1653	1646

Appendix E

Observing Period (Day)	Measurements for M3-50							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
24	809	617	0	1639	622	10	1664	1652
25	809	611	63	1702	622	0	1664	1683
26	809	607	42	1744	621	10	1675	1710
27	809	605	21	1765	619	21	1696	1731
28	809	604	10	1776	615	42	1738	1757
29	809	604	0	1776	612	31	1770	1773
30	809	603	10	1787	612	0	1770	1779
31	809	602	10	1798	612	0	1770	1784
32	809	601	10	1809	611	10	1781	1795
33	809	600	10	1820	610	10	1792	1806
34	809	597	31	1852	610	0	1792	1822
35	809	597	0	1852	609	10	1803	1828
36	809	597	0	1852	609	0	1803	1828
37	809	597	0	1852	609	0	1803	1828
38	809	597	0	1852	609	0	1803	1828
39	809	597	0	1852	608	10	1814	1833
40	809	596	10	1863	608	0	1814	1839
41	809	596	0	1863	607	10	1825	1844
42	809	596	0	1863	607	0	1825	1844
43	809	596	0	1863	607	0	1825	1844
44	809	594	21	1884	604	31	1857	1871
45	809	593	10	1895	604	0	1857	1876
46	809	592	10	1906	603	10	1868	1887
47	809	590	21	1927	603	0	1868	1898

Observing Period (Day)	Measurements for M3-50							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
48	809	590	0	1927	601	21	1889	1908
49	809	589	10	1938	601	0	1889	1914
50	808	589	-10	1928	599	10	1900	1914
51	808	588	10	1939	598	10	1911	1925
52	809	588	10	1950	598	10	1922	1936
53	809	588	0	1950	597	10	1933	1942
54	810	587	21	1971	596	21	1954	1963
55	810	587	0	1971	595	10	1965	1968
56	810	587	0	1971	594	10	1976	1974
57	810	587	0	1971	594	0	1976	1974
58	810	586	10	1982	594	0	1976	1979
59	810	585	10	1993	594	0	1976	1985
60	810	584	10	2004	592	21	1997	2001
61	810	583	10	2015	590	21	2018	2017
62	809	582	0	2015	588	10	2029	2022
63	809	581	10	2026	588	0	2029	2028
64	809	580	10	2037	588	0	2029	2033
65	809	580	0	2037	587	10	2040	2039
66	809	580	0	2037	587	0	2040	2039
67	809	578	21	2058	587	0	2040	2049
68	809	578	0	2058	586	10	2051	2055
69	809	578	0	2058	585	10	2062	2060
70	809	577	10	2069	585	0	2062	2066
71	809	576	10	2080	585	0	2062	2071

Observing Period (Day)	Measurements for M3-50							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
72	809	575	10	2091	585	0	2062	2077
73	809	575	0	2091	584	10	2073	2082
74	809	575	0	2091	583	10	2084	2088
75	809	573	21	2112	582	10	2095	2104
76	809	572	10	2123	581	10	2106	2115
77	809	572	0	2123	580	10	2117	2120
78	809	572	0	2123	580	0	2117	2120
79	809	572	0	2123	580	0	2117	2120
80	809	571	10	2134	580	0	2117	2126
81	809	571	0	2134	579	10	2128	2131
82	809	571	0	2134	579	0	2128	2131
83	809	571	0	2134	578	10	2139	2137
84	809	571	0	2134	577	10	2150	2142
85	809	571	0	2134	577	0	2150	2142
86	809	571	0	2134	577	0	2150	2142
87	809	571	0	2134	575	21	2171	2153
88	809	571	0	2134	574	10	2182	2158
89	809	570	10	2145	574	0	2182	2164
90	809	569	10	2156	573	10	2193	2175
91	809	569	0	2156	573	0	2193	2175
92	809	568	10	2167	572	10	2204	2186
93	809	568	0	2167	571	10	2215	2191
94	809	567	10	2178	571	0	2215	2197
95	809	567	0	2178	568	31	2247	2213

Appendix E

Observing Period (Day)	Measurements for M3-50							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
96	809	566	10	2189	567	10	2258	2224
97	809	565	10	2200	566	10	2269	2235
98	809	564	10	2211	565	10	2280	2246
99	809	563	10	2222	564	10	2291	2257
100	809	563	0	2222	563	10	2302	2262
101	809	563	0	2222	563	0	2302	2262
102	809	562	10	2233	562	10	2313	2273
103	809	561	10	2244	561	10	2324	2284
104	809	560	10	2255	560	10	2335	2295
105	809	560	0	2255	560	0	2335	2295
106	809	559	10	2266	559	10	2346	2306
107	809	559	0	2266	559	0	2346	2306
108	809	558	10	2277	559	0	2346	2312
109	809	558	0	2277	558	10	2357	2317
110	809	558	0	2277	558	0	2357	2317
111	809	557	10	2288	557	10	2368	2328
112	809	557	0	2288	557	0	2368	2328
113	809	557	0	2288	557	0	2368	2328
114	809	556	10	2299	557	0	2368	2334
115	809	556	0	2299	557	0	2368	2334
116	809	556	0	2299	556	10	2379	2339
117	809	556	0	2299	556	0	2379	2339
118	809	555	10	2310	556	0	2379	2345
119	809	555	0	2310	555	10	2390	2350

Appendix E

Observing Period (Day)	Measurements for M3-50							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
120	809	555	0	2310	555	0	2390	2350
121	809	555	0	2310	555	0	2390	2350
122	809	554	10	2321	555	0	2390	2356
123	809	554	0	2321	554	10	2401	2361
124	809	554	0	2321	554	0	2401	2361
125	809	554	0	2321	554	0	2401	2361
126	808	553	0	2321	553	0	2401	2361
127	808	553	0	2321	553	0	2401	2361
128	809	553	10	2332	554	0	2401	2367
129	809	553	0	2332	554	0	2401	2367
130	809	553	0	2332	553	10	2412	2372
131	809	553	0	2332	553	0	2412	2372
132	809	553	0	2332	553	0	2412	2372
133	809	553	0	2332	553	0	2412	2372
134	809	552	10	2343	552	10	2423	2383
135	809	552	0	2343	552	0	2423	2383
136	809	552	0	2343	552	0	2423	2383
137	809	552	0	2343	552	0	2423	2383
138	809	552	0	2343	552	0	2423	2383
139	809	552	0	2343	552	0	2423	2383
140	809	551	10	2354	551	10	2434	2394
141	809	551	0	2354	551	0	2434	2394
142	809	551	0	2354	551	0	2434	2394
143	809	551	0	2354	551	0	2434	2394

Observing Period (Day)	Measurements for M3-50							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
144	809	551	0	2354	551	0	2434	2394
145	809	550	10	2365	551	0	2434	2400
146	809	550	0	2365	551	0	2434	2400
147	810	551	0	2365	551	10	2445	2405
148	810	551	0	2365	551	0	2445	2405
149	810	551	0	2365	551	0	2445	2405
150	810	550	10	2376	551	0	2445	2411
151	809	549	0	2376	550	0	2445	2411
152	809	549	0	2376	550	0	2445	2411
153	809	549	0	2376	550	0	2445	2411
154	810	550	0	2376	550	10	2456	2416
155	810	549	10	2387	550	0	2456	2422
156	810	549	0	2387	550	0	2456	2422
157	810	549	0	2387	550	0	2456	2422
158	810	548	10	2398	550	0	2456	2427
159	810	548	0	2398	550	0	2456	2427
160	810	548	0	2398	549	10	2467	2433
161	810	548	0	2398	549	0	2467	2433
162	810	547	10	2409	549	0	2467	2438
163	810	547	0	2409	549	0	2467	2438
164	810	547	0	2409	549	0	2467	2438
165	810	547	0	2409	549	0	2467	2438
166	810	547	0	2409	549	0	2467	2438
167	810	547	0	2409	549	0	2467	2438

Observing Period (Day)	Measurements for M3-50							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
168	810	547	0	2409	549	0	2467	2438
169	810	547	0	2409	549	0	2467	2438
170	810	546	10	2420	549	0	2467	2444
171	810	546	0	2420	549	0	2467	2444
172	810	546	0	2420	549	0	2467	2444
173	810	546	0	2420	549	0	2467	2444
174	810	546	0	2420	549	0	2467	2444
175	810	546	0	2420	549	0	2467	2444
176	810	546	0	2420	549	0	2467	2444
177	810	545	10	2431	549	0	2467	2449
178	810	545	0	2431	549	0	2467	2449
179	810	545	0	2431	549	0	2467	2449
180	810	545	0	2431	549	0	2467	2449
187	810	545	0	2431	548	10	2478	2455
194	810	545	0	2431	548	0	2478	2455
201	810	545	0	2431	548	0	2478	2455
208	810	545	0	2431	548	0	2478	2455
215	810	545	0	2431	548	0	2478	2455

Note: Loading at 3000 psi for both specimens.

Appendix E

Observing Period (Day)	Measurements for M3-100							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
0	814	790	0	0	820	0	0	0
1	813	707	876	876	737	874	874	875
2	813	704	31	908	735	21	895	902
3	812	700	31	940	729	52	948	944
4	811	698	10	951	727	10	959	955
5	811	689	94	1046	717	105	1064	1055
6	811	684	52	1099	713	42	1106	1103
7	811	680	42	1141	708	52	1159	1150
8	811	677	31	1173	706	21	1180	1177
9	811	670	73	1247	699	73	1254	1251
10	811	664	63	1310	693	63	1317	1314
11	810	660	31	1342	688	42	1359	1351
12	809	657	21	1363	685	21	1380	1372
13	809	655	21	1384	683	21	1401	1393
14	809	652	31	1416	681	21	1422	1419
15	809	648	42	1458	677	42	1464	1461
16	810	645	42	1500	674	42	1506	1503
17	810	642	31	1532	671	31	1538	1535
18	809	639	21	1553	670	0	1538	1546
19	809	637	21	1574	667	31	1570	1572
20	809	635	21	1595	662	52	1623	1609
21	810	634	21	1616	661	21	1644	1630
22	810	632	21	1637	660	10	1655	1646
23	809	631	0	1637	659	0	1655	1646

Observing Period (Day)	Measurements for M3-100							Ave. Cum. Total Deformation (1 x 10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1 x 10^{-6})	Cum. Total Deformation (1 x 10^{-6})	Length (Division)	Total Deformation (1 x 10^{-6})	Cum. Total Deformation (1 x 10^{-6})	
24	809	630	10	1648	659	0	1655	1652
25	809	626	42	1690	654	52	1708	1699
26	809	622	42	1732	651	31	1740	1736
27	809	618	42	1774	645	63	1803	1789
28	809	618	0	1774	644	10	1814	1794
29	809	615	31	1806	643	10	1825	1816
30	809	611	42	1848	637	63	1888	1868
31	809	610	10	1859	637	0	1888	1874
32	809	609	10	1870	637	0	1888	1879
33	809	608	10	1881	637	0	1888	1885
34	809	607	10	1892	637	0	1888	1890
35	809	607	0	1892	636	10	1899	1896
36	809	607	0	1892	636	0	1899	1896
37	809	607	0	1892	635	10	1910	1901
38	809	606	10	1903	633	21	1931	1917
39	809	604	21	1924	633	0	1931	1928
40	809	604	0	1924	632	10	1942	1933
41	809	604	0	1924	630	21	1963	1944
42	809	603	10	1935	629	10	1974	1955
43	809	602	10	1946	628	10	1985	1966
44	809	602	0	1946	627	10	1996	1971
45	809	601	10	1957	627	0	1996	1977
46	809	600	10	1968	627	0	1996	1982
47	809	600	0	1968	626	10	2007	1988

Appendix E

Observing Period (Day)	Measurements for M3-100							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
48	809	599	10	1979	626	0	2007	1993
49	809	599	0	1979	625	10	2018	1999
50	808	597	10	1990	623	10	2029	2010
51	808	596	10	2001	623	0	2029	2015
52	809	596	10	2012	621	31	2061	2037
53	809	595	10	2023	619	21	2082	2053
54	810	594	21	2044	619	10	2093	2069
55	810	592	21	2065	618	10	2104	2085
56	810	592	0	2065	615	31	2136	2101
57	810	591	10	2076	615	0	2136	2106
58	810	590	10	2087	615	0	2136	2112
59	810	590	0	2087	614	10	2147	2117
60	810	589	10	2098	612	21	2168	2133
61	810	587	21	2119	610	21	2189	2154
62	809	585	10	2130	607	21	2210	2170
63	809	585	0	2130	606	10	2221	2176
64	809	585	0	2130	606	0	2221	2176
65	809	585	0	2130	605	10	2232	2181
66	809	585	0	2130	605	0	2232	2181
67	809	584	10	2141	605	0	2232	2187
68	809	583	10	2152	604	10	2243	2198
69	809	582	10	2163	604	0	2243	2203
70	809	582	0	2163	603	10	2254	2209
71	809	581	10	2174	603	0	2254	2214

Observing Period (Day)	Measurements for M3-100							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
72	809	581	0	2174	602	10	2265	2220
73	809	581	0	2174	601	10	2276	2225
74	809	579	21	2195	601	0	2276	2236
75	809	578	10	2206	600	10	2287	2247
76	809	577	10	2217	600	0	2287	2252
77	809	577	0	2217	599	10	2298	2258
78	809	577	0	2217	599	0	2298	2258
79	809	577	0	2217	599	0	2298	2258
80	809	576	10	2228	599	0	2298	2263
81	809	575	10	2239	599	0	2298	2269
82	809	575	0	2239	599	0	2298	2269
83	809	574	10	2250	599	0	2298	2274
84	809	574	0	2250	599	0	2298	2274
85	809	574	0	2250	598	10	2309	2280
86	809	573	10	2261	597	10	2320	2291
87	809	572	10	2272	597	0	2320	2296
88	809	572	0	2272	596	10	2331	2302
89	809	571	10	2283	595	10	2342	2313
90	809	571	0	2283	595	0	2342	2313
91	809	570	10	2294	594	10	2353	2324
92	809	569	10	2305	594	0	2353	2329
93	809	568	10	2316	594	0	2353	2335
94	809	567	10	2327	593	10	2364	2346
95	809	567	0	2327	592	10	2375	2351

Observing Period (Day)	Measurements for M3-100							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
96	809	566	10	2338	590	21	2396	2367
97	809	565	10	2349	589	10	2407	2378
98	809	564	10	2360	588	10	2418	2389
99	809	564	0	2360	587	10	2429	2395
100	809	563	10	2371	587	0	2429	2400
101	809	563	0	2371	587	0	2429	2400
102	809	562	10	2382	586	10	2440	2411
103	809	561	10	2393	584	21	2461	2427
104	809	560	10	2404	583	10	2472	2438
105	809	559	10	2415	582	10	2483	2449
106	809	559	0	2415	582	0	2483	2449
107	809	559	0	2415	582	0	2483	2449
108	809	558	10	2426	581	10	2494	2460
109	809	558	0	2426	580	10	2505	2466
110	809	558	0	2426	580	0	2505	2466
111	809	557	10	2437	579	10	2516	2477
112	809	557	0	2437	579	0	2516	2477
113	809	557	0	2437	579	0	2516	2477
114	809	557	0	2437	579	0	2516	2477
115	809	556	10	2448	579	0	2516	2482
116	809	556	0	2448	578	10	2527	2488
117	809	556	0	2448	578	0	2527	2488
118	809	556	0	2448	578	0	2527	2488
119	809	555	10	2459	577	10	2538	2499

Observing Period (Day)	Measurements for M3-100							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
120	809	555	0	2459	577	0	2538	2499
121	809	555	0	2459	577	0	2538	2499
122	809	554	10	2470	576	10	2549	2510
123	809	554	0	2470	576	0	2549	2510
124	809	554	0	2470	576	0	2549	2510
125	809	553	10	2481	575	10	2560	2521
126	808	552	0	2481	574	0	2560	2521
127	808	552	0	2481	574	0	2560	2521
128	809	553	0	2481	575	0	2560	2521
129	809	553	0	2481	575	0	2560	2521
130	809	553	0	2481	574	10	2571	2526
131	809	553	0	2481	574	0	2571	2526
132	809	552	10	2492	574	0	2571	2532
133	809	552	0	2492	574	0	2571	2532
134	809	552	0	2492	574	0	2571	2532
135	809	552	0	2492	574	0	2571	2532
136	809	552	0	2492	573	10	2582	2537
137	809	551	10	2503	573	0	2582	2543
138	809	551	0	2503	573	0	2582	2543
139	809	551	0	2503	573	0	2582	2543
140	809	551	0	2503	572	10	2593	2548
141	809	551	0	2503	572	0	2593	2548
142	809	551	0	2503	572	0	2593	2548
143	809	550	10	2514	572	0	2593	2554

Observing Period (Day)	Measurements for M3-100							Ave. Cum. Total Deformation (1×10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	Length (Division)	Total Deformation (1×10^{-6})	Cum. Total Deformation (1×10^{-6})	
144	809	550	0	2514	571	10	2604	2559
145	809	550	0	2514	571	0	2604	2559
146	809	550	0	2514	571	0	2604	2559
147	810	550	10	2525	572	0	2604	2565
148	810	550	0	2525	571	10	2615	2570
149	810	550	0	2525	571	0	2615	2570
150	810	550	0	2525	571	0	2615	2570
151	809	548	10	2536	570	0	2615	2576
152	809	548	0	2536	569	10	2626	2581
153	809	548	0	2536	569	0	2626	2581
154	810	549	0	2536	570	0	2626	2581
155	810	548	10	2547	569	10	2637	2592
156	810	548	0	2547	569	0	2637	2592
157	810	548	0	2547	569	0	2637	2592
158	810	548	0	2547	568	10	2648	2598
159	810	547	10	2558	568	0	2648	2603
160	810	547	0	2558	568	0	2648	2603
161	810	547	0	2558	567	10	2659	2609
162	810	547	0	2558	567	0	2659	2609
163	810	546	10	2569	567	0	2659	2614
164	810	546	0	2569	567	0	2659	2614
165	810	546	0	2569	567	0	2659	2614
166	810	546	0	2569	567	0	2659	2614
167	810	546	0	2569	567	0	2659	2614

Appendix E

Observing Period (Day)	Measurements for M3-100							Ave. Cum. Total Deformation (1 x 10^{-6})
	Reference Rod (Division)	Cylinder 1			Cylinder 2			
		Length (Division)	Total Deformation (1 x 10^{-6})	Cum. Total Deformation (1 x 10^{-6})	Length (Division)	Total Deformation (1 x 10^{-6})	Cum. Total Deformation (1 x 10^{-6})	
168	810	546	0	2569	567	0	2659	2614
169	810	545	10	2580	566	10	2670	2625
170	810	545	0	2580	566	0	2670	2625
171	810	545	0	2580	566	0	2670	2625
172	810	545	0	2580	566	0	2670	2625
173	810	545	0	2580	566	0	2670	2625
174	810	544	10	2591	566	0	2670	2631
175	810	544	0	2591	566	0	2670	2631
176	810	544	0	2591	566	0	2670	2631
177	810	544	0	2591	566	0	2670	2631
178	810	544	0	2591	565	10	2681	2636
179	810	544	0	2591	565	0	2681	2636
180	810	544	0	2591	565	0	2681	2636
187	810	543	10	2602	565	0	2681	2642
194	810	543	0	2602	565	0	2681	2642
201	810	543	0	2602	565	0	2681	2642
208	810	543	0	2602	565	0	2681	2642
215	810	543	0	2602	565	0	2681	2642

Note: Loading at 2900 psi for both specimens.

Appendix F

Data for
Drying and Wetting Test on
M3-0, -50 and -100

Appendix F

RCA Replacement Level (%)	Batch No.	Initial Drying Shrinkage (%)	Wetting Expansion (%)
0	1	0.0283	0.0162
		0.0252	0.0148
		0.0274	0.0155
	2	0.0255	0.0169
		0.0210	0.0138
		0.0225	0.0131
		0.0210	0.0114
		0.0205	0.0123
		0.0220	0.0138
	3	0.0230	0.0140
		0.0195	0.0110
		0.0230	0.0140
		0.0250	0.0150
		0.0250	0.0145
		0.0270	0.0155

RCA Replacement Level (%)	Batch No.	Initial Drying Shrinkage (%)	Wetting Expansion (%)
50	1	0.0303	0.0208
		0.0287	0.0197
		0.0291	0.0191
	2	0.0270	0.0175
		0.0260	0.0175
		0.0270	0.0165
		0.0265	0.0160
		0.0260	0.0160
		0.0275	0.0170
	3	0.0265	0.0140
		0.0220	0.0140
		0.0225	0.0135
		0.0190	0.0135
		0.0190	0.0135
		0.0220	0.0145

RCA Replacement Level (%)	Batch No.	Initial Drying Shrinkage (%)	Wetting Expansion (%)
100	1	0.0325	0.0220
		0.0310	0.0215
		0.0305	0.0220
	2	0.0223	0.0190
		0.0240	0.0180
		0.0260	0.0195
		0.0245	0.0165
		0.0235	0.0180
		0.0280	0.0190
	3	0.0225	0.0150
		0.0205	0.0145
		0.0200	0.0150
		0.0200	0.0135
		0.0210	0.0120
		0.0225	0.0130

Note: For all concrete mixes, test results of batch no. 1 are the average of 3 specimens whereas test results of batch no. 2 and 3 are the average of 6 specimens.

Appendix G

Data for Water Absorbability Test on M3-0, -30, -50 and -100

RCA Replacement Level (%)	Water Absorbability (%)
0	1.680
	1.741
	1.681
	1.936
	1.939
	2.013
	1.637
	1.490
	1.563
30	1.989
	2.130
	1.989
	1.718
	1.596
	1.779
	1.315
	1.575
	1.519
50	1.967
	1.967
	2.045
	2.035
	1.901
	2.034
	1.752
	1.531
	1.593
100	1.945
	2.196
	2.069
	1.849
	1.627
	1.697
	1.771
	1.985
	1.922

Appendix H

Data for
Initial Surface Absorption Test
on M3-0, -30, -50 and -100

Interval from Start of Test (min)	RCA Replacement Level (%)			
	0	30	50	100
10	0.64	0.62	0.68	0.73
	0.65	0.64	0.67	0.78
	0.56	0.69	0.53	0.72
	0.54	0.47	0.51	0.52
	0.44	0.45	0.50	0.59
	0.42	0.50	0.53	0.52
	0.49	0.44	0.50	0.51
	0.54	0.45	0.48	0.50
	0.50	0.48	0.51	0.50
30	0.34	0.35	0.42	0.45
	0.35	0.42	0.41	0.44
	0.33	0.39	0.32	0.43
	0.22	0.32	0.34	0.27
	0.26	0.31	0.29	0.29
	0.23	0.32	0.32	0.30
	0.29	0.27	0.29	0.40
	0.30	0.31	0.27	0.34
	0.29	0.28	0.29	0.33
60	0.23	0.25	0.27	0.31
	0.23	0.23	0.26	0.29
	0.23	0.29	0.23	0.31
	0.14	0.21	0.18	0.18
	0.16	0.19	0.16	0.18
	0.15	0.21	0.18	0.18
	0.19	0.17	0.19	0.24
	0.20	0.19	0.17	0.23
	0.19	0.18	0.18	0.24

Appendix I

Data for Depth of Water Penetration under Pressure on M-0, -30, -50 and -100

RCA Replacement Level (%)	Depth of Water Penetration (mm)
0	73
	74
	75
	92
	87
	93
	90
	80
	80
30	76
	77
	76
	85
	87
	88
	90
	95
	95
50	77
	78
	75
	94
	96
	91
	80
	82
	87
100	92
	100
	93
	77
	80
	79
	96
	94
	98

Appendix J

Data for
Rapid Chloride Permeability
Test on M3-0, -30, -50 and -100

RCA Replacement Level (%)	Charged Passed – Coulombs (A.s)	Permeability Class
0	2687	Moderate
	2609	Moderate
	2192	Moderate
	3215	Moderate
	2018	Moderate
	2459	Moderate
30	1987	Low
	2252	Moderate
	1853	Low
	2829	Moderate
	1622	Low
	2091	Moderate
50	2437	Moderate
	1887	Low
	2094	Moderate
	2298	Moderate
	2149	Moderate
	2171	Moderate
100	1991	Low
	2659	Moderate
	1753	Low
	2614	Moderate
	1587	Low
	2290	Moderate

Appendix K

Data for Sulphate Resistance Test on M3-0, -50 and -100

Observing Period (Day)	Measurements for M3-0 (Division)								Ave. Expansion (1×10^{-6})	Ave. Cum. Expansion (1×10^{-6})
	Reference Rod	Prism 1		Prism 2		Prism 3		Average ΔLength		
		Length	ΔLength	Length	ΔLength	Length	ΔLength			
0	1533	2777	0	2650	0	2633	0	0	0	0
1	1532	2779	3	2657	8	2636	4	5	24	24
2	1532	2780	1	2659	2	2638	2	2	9	33
3	1532	2781	1	2661	2	2639	1	1	7	40
4	1531	2782	2	2662	2	2642	4	3	13	53
5	1530	2783	2	2664	3	2645	4	3	15	68
6	1528	2786	5	2668	6	2648	5	5	26	95
7	1528	2787	1	2670	2	2650	2	2	8	103
8	1533	2777	0	2650	0	2633	0	0	0	0

Observing Period (Day)	Measurements for M3-50 (Division)								Ave. Expansion (1×10^{-6})	Ave. Cum. Expansion (1×10^{-6})
	Reference Rod	Prism 1		Prism 2		Prism 3		Average ΔLength		
		Length	ΔLength	Length	ΔLength	Length	ΔLength			
0	1533	2850	0	2780	0	2745	0	0	0	0
1	1532	2853	4	2785	6	2748	4	5	23	23
2	1531	2855	3	2788	4	2751	4	4	18	42
3	1531	2858	3	2792	4	2753	2	3	15	57
4	1531	2859	1	2795	3	2755	2	2	10	67
5	1530	2860	2	2800	6	2756	2	3	17	83
6	1528	2863	5	2802	4	2758	4	4	22	105
7	1528	2864	1	2803	1	2758	0	1	3	108
8	1533	2850	0	2780	0	2745	0	0	0	0

Observing Period (Week)	Measurements for M3-100 (Division)								Ave. Expansion (1×10^{-6})	Ave. Cum. Expansion (1×10^{-6})
	Reference Rod	Prism 1		Prism 2		Prism 3		Average ΔLength		
		Length	ΔLength	Length	ΔLength	Length	ΔLength			
0	1533	2908	0	2913	0	2903	0	0	0	0
1	1532	2918	11	2915	3	2907	5	6	32	32
2	1530	2920	4	2917	4	2908	3	4	18	50
3	1530	2921	1	2919	2	2910	2	2	8	58
4	1530	2923	2	2922	3	2913	3	3	13	72
5	1530	2927	4	2928	6	2918	5	5	25	97
6	1528	2930	5	2931	5	2920	4	5	23	120
7	1528	2930	0	2931	0	2920	0	0	0	120
8	1533	2908	0	2913	0	2903	0	0	0	0