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Systematic Approach of Characterisation and Behaviour of Recycled Aggregate Concrete

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The recycling of resource by the aggregate behavior of a diverse array of agents is much more than the sum of the individual actions.

—John Henry Holland

Dedicated to the Recyclers!

Foreword

The recycling of construction and demolition waste (C&DW) as an aggregate in concrete which is an alternative to the natural aggregate is one of the recent developments in the field of concrete. This book is a distinct collation of the spectrum of information related to the use of construction and demolition waste (C&DW) as an aggregate in the concrete field published by the researchers worldwide from 1980 to 2016. This book of eight chapters comprises five distinct groups of contents, namely the production methodology, specifications and characterization of recycled aggregate (RA), short and long-term and durability aspects of recycled aggregate concrete (RAC), microstructural characterization of RAC, structural behavior of RAC under different loads, and various treatment techniques for the improvement of the quality of RA and RAC.

Recycled aggregates are the natural aggregates (NA) adhered to old cement mortar. The basic difference lies between the two aggregates in the presence of attached old mortar in the recycled aggregate. The authors nicely compiled and elaborated the factors such as the quantity and quality of adhered mortar, source of old concrete, strength of parent concrete from which the RA derived, moisture condition of aggregates, method of crushing, age of crushing related to the characterization of RA. As the recycled aggregates are relatively poorer than NA, the properties of RAC prepared with RA are obviously worse than normal concrete (NC). This book clearly demonstrated the difference between RAC and NC with respect to the mechanical properties and their interrelationships, long-term properties like shrinkage, creep and the durability aspects such as permeability, carbonation, chloride penetration with a number of illustrations along with sound reasons systematically. Another difference lying between NC and RAC is the number of ITZs. RAC has two ITZs, i.e., the interfacial transition zone (ITZ) between the original aggregate and the adhered mortar (old ITZ), and another interface between the adhered mortar and new mortar matrix (new ITZ). In contrast, the normal concrete has only one ITZ i.e. interface between the aggregate and mortar. This book rigorously explained the characteristics of these ITZs with a number of illustrations precisely and methodically. The structural behavior of reinforced concrete beams made with recycled aggregate concrete under various types of loads such as axial, flexural, shear, and impact are emphasized with appropriate figures.

To mitigate the shortfalls of RA and RAC to certain extent, various quality improvement techniques such as acid treatment, thermal treatment and mechanical treatment for RA and addition of mineral admixtures, impregnation of recycled aggregate in mineral admixtures or cement slurry and modified mixing approaches for RAC suggested by few investigators are well demonstrated with neat schematic diagrams and illustrations in this book.

I congratulate the authors to succeed in writing this book in a comprehensive dealing of the state of knowledge of RAC made with RA produced from the C&DW published by the researchers worldwide systematically. I wish that this book not only will be helpful for the graduate students, researchers and concrete technologist but also serves as a reference for the practice engineers who face numerous problems while using these materials in the field of concrete.

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Preface

Concrete is the most widely used manmade material in the globe since its invention. Concrete mainly comprises of the binding material, fine aggregate, coarse aggregate and water. Among these, aggregates alone contribute around 75% of the total volume of concrete and therefore, aggregates play a vital role on the behavior of concrete. The rapid growth in industrial and infrastructural development and depletion of natural resources create a huge crisis for the natural aggregates in the concrete field. To meet the demand and supply of these aggregates and preserve the environment from the viewpoint of sustainability, the recycling of construction and demolition waste (C&DW) as an aggregate in concrete which is an alternative to the natural aggregate is one of the recent developments in the field of concrete.

In general, the C&DW contains concrete, bricks, cement mortar, plastics, glass, reinforcement, wood, etc. Therefore, the separation of different kinds of these wastes is very much essential before using them in various applications. Though there are certain benefits of recycled aggregate concrete in terms of economic, environmental and saving natural resources, there are some constraints too during implementation in the aspects of both management and technology. The management problems include lack of suitable regulations, lack of codes, specifications, standards and guidelines, lack of experience, etc., and the technical problems such as poor grading, high porosity, weak interfacial transition zone (ITZ), transverse cracks generated, variations in quality, etc. The countries like Japan, Germany, UK, Hong Kong, Australia, China, India, etc., and RILEM have prepared and published the specifications and guidelines on the use of recycled aggregates in the production of concrete. The major difference lies between the natural aggregate and the recycled aggregate obtained from C&DW is the quality and quantity of old cement mortar adhered on the surface of it. This parameter significantly affects the physical and mechanical properties of aggregates and hence the properties of concrete at both fresh and hardened states particularly the strength, long-term and durability. Many other important parameters such as w/c ratio, the characteristics of the constituent materials of concrete, especially the aggregate, i.e., type of aggregate, maximum size of aggregate, water absorption of aggregate, moisture state of recycled aggregate, strength of original concrete from which the recycled aggregate is derived, the amount of substitution of RA in place of natural aggregate, microstructure, method of curing, etc., also greatly affect the properties of recycled aggregate concrete. The interfacial transition zone (ITZ) between the aggregate and the cement mortar matrix is the most important interface in concrete. A fundamental study of this ITZ gives a more insight into the understanding of the concrete characteristics. In general, the ITZ is the weakest link of the chain and is treated as the strength-limiting phase in concrete. The study of the interface is more important in recycled aggregate concrete, as the recycled aggregate concrete has more interfaces than normal concrete, i.e., the interfacial transition zone between the original aggregate and the adhered mortar (old ITZ), and another interface between the adhered mortar and new mortar matrix (new ITZ) in recycled aggregate concrete. The characteristics of these interfacial transition zones mainly depend on the quality and quantity of adhered mortar, the strength of source concrete, etc. Further, understanding the behavior of the recycled aggregate concrete under various types of loading is very much essential before its acceptance in the concrete field. In the recent past, numerous investigations have been carried out in the field of recycled aggregate obtained from the C&DW and their applications in concrete. The information disseminated in various journals and conference proceedings up to January 2017 has been considered and compiled precisely and systematically by the authors in this book.

This book mainly focuses on the utilization of construction waste material as coarse aggregate in making concrete. The whole book mainly comprises of eight chapters. The first chapter discusses the importance of recycling of C&DW, applications of recycled aggregate, and various benefits and constraints while implementation of these recycled aggregates in the field of concrete. Further, the current scenario of the recycling of C&DW on the globe, the codes and practices developed in different countries on the specifications of recycled aggregate obtained from the construction and demolition waste and their use in the concrete applications are also discussed. The second chapter exclusively focuses on the demolition techniques and production technology of recycled aggregate from the C&DW. The third chapter emphasizes on the physical and mechanical characteristics of recycled aggregate and various influencing parameters of it. The fourth chapter discusses the engineering properties of recycled aggregate concrete. Further, the interrelationships among the mechanical properties of recycled aggregate concrete are discussed. The fifth chapter discusses elaborately the long-term properties like shrinkage and creep and the durability properties of recycled aggregate concrete. The sixth chapter describes the microstructural characterization. The behavior of recycled aggregate under impact load is discussed in detail along with other structural applications. At the end, the various quality improvement techniques of recycled aggregate and recycled aggregate concrete are explained. It is hoped that this book will be definitely helpful for the graduate students, concrete technologist and particularly researchers who carry out further research in this field. Also, it serves as a reference for the practice engineers who face numerous problems while using these materials in the field of concrete.

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Kharagpur, India — Sriman Kumar Bhattacharyya
Kharagpur, India — Sudhirkumar V. Barai

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Achieving sustainability in construction materials is one of the prime concerns in construction sector. Recycling of construction and demolition waste as an alternative source for the aggregate in producing concrete is one such attempt. Works of any kind can move step by step, finally to give the shape of a book as the present one. This could be made possible today just because of the support of many well-wishers either directly or indirectly, who helped to stand determined, drove to march fast, gave strength to overcome the impediments in achieving the final objective.

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Contents

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Symbols and Abbreviations

Symbols

a/c	Aggregate−cement ratio
c	Cement
E_c	Static modulus of elasticity (MPa)
f_f	Flexural strength (MPa)
f_{ck}	Cube compressive strength (MPa)
f_{sp}	Split tensile strength (MPa)
HV	Vickers microhardness
L	Length (m)
N	Rebound number
p	Porosity (%)
T	Time (s)
UH	Anhydrous cement
V	Pulse velocity (m/s or km/s)
w/c	Water–cement ratio
w/b	Water–binder ratio
γ_a	Apparent density (kg/m^3)
γ_b	Bulk density after immersion and boiling in water (kg/m^3)
γ_d	Bulk density dry (kg/m^3)
γ_s	Bulk density after immersion in water (kg/m^3)
ε	Strain
ρ	Density (kg/m^3)
σ	Stress (MPa)
Ω	Resistance

Abbreviations

3R	Reduce, reuse, recycle
AAR	Alkali–aggregate reaction
AC	Air cured

ACI	American Concrete Institute
ACV	Aggregate crushing value
AD	Air dry
AM	Adhered mortar
ASTM	American Standard Testing of Materials
BCSJ	Building Contractors Society of Japan
BIS	Bureau of Indian Standard
BMC	Brihanmumbai Municipal Corporation
BRE	Building Research Establishment
BFS	Blast furnace slag
BS	British Standards
BSEN	British European Standards Specifications
BSE	Backscattered scanning electron
C&DW	Construction and demolition waste
CA	Coarse aggregate
CB	Crushed brick
CC	Crushed concrete
CFST	Normal concrete filled steel tubular
CH	Calcium hydroxide
CMRA	Construction Materials Recycling Association
CPCB	Central Pollution Control Board
CRCA	Coarse recycled concrete aggregate
CRMA	Coarse recycled masonry aggregate
CS	Construction Standard
C-S-H	Calcium silicate hydrate
CSIRO	Commonwealth Scientific and Industrial Research Organisation
DAFSTB	German Committee for Reinforced Concrete
DAQ	Data acquisition system
DIN	Deutsches Institut für Normung eV (German Institute for Standardization)
DMDA	Densified mixture design algorithm
DMM	Double mixing method
EDS	Energy-dispersive X-ray spectrometer
Eqn.	Equation
ERDAS	Earth Resources Data Analysis System
EU	European Union
FA	Fly ash
FFT	Fast Fourier transform
Fig.	Figure
FRA	Fine recycled aggregate
FRC	Fiber-reinforced concrete
FRCA	Fine recycled concrete aggregates
FRMA	Fine recycled masonry aggregates
FRP	Fiber-reinforced plastic
GBA	Ground bagasse ash

GDP	Gross domestic product
GFA	Ground fly ash
GGBS	Ground granulated blast furnace slag
GOI	Government of India
GRNN	General regression neural network
GS	General specifications
HPC	High-performance concrete
ISAT	Initial surface absorption test
ITZ	Interfacial transition zone
JIS	Japanese Industrial Standards
JSCE	Japan Society of Civil Engineers
KTS	Kai Tak airport site
LNMO	Liquid nitrogen–microwave
Max	Maximum
MCD	Municipal Corporation of Delhi
Min	Minimum
MK	Metakaolin
MOC	Ministry of Construction
MRA	Mixed recycled aggregate
NCA	Natural coarse aggregate
NDT	Nondestructive test
NI	National instruments
NMA	Normal mixing approach
OD	Oven dry
OPC	Ordinary Portland cement
OPI	Oxygen permeability index
PC	Parent concrete
PCFA	Portland cement with fly ash
PFA	Pulverized fly ash
PP	Pozzolanic powder
RA	Recycled aggregate
RAC	Recycled aggregate concrete
RACFST	Recycled concrete-filled steel tubular
RAWA	Real-time assessment of water absorption
RCA	Recycled coarse aggregate
RCC	Reinforced cement concrete
RG	Recycled gravel
RH	Relative humidity
RILEM	International Union of Testing and Research Laboratories for Materials
RMA	Recycled masonry aggregates
RS	Recycled sand
SEM	Scanning electron microscope
SEPC	Stone enveloped with Portland cement
SEPP	Stone enveloped with pozzolanic powder

SF	Silica fume
SMA	Single-stage mixing approach
SP	Superplasticizer
SSD	Saturated surface dry
ST	Soundness test
TFV	Ten percent fines value
TIFAC	Technology Information Forecasting and Assessment Council
TKOL	Tseung Kwan O site further crushed in the laboratory
TKOS	Tseung Kwan O site
TR	Technical report
TSMA	Two-stage mixing approach
$TSMA_{(p1)}$	Modified two-stage mixing $approach_{(proportional\text{-}1)}$
$TSMA_{(p2)}$	Modified two-stage mixing $approach_{(proportional\text{-}2)}$
$TSMA_{s}$	Two-stage mixing $approach_{(silica\ fume)}$
$TSMA_{sc}$	Two-stage mixing $approach_{(silica\ fume\ and\ cement)}$
UK	United Kingdom
UPV	Ultrasonic pulse velocity
WBTC	Work Bureau Technical Circular
WC	Water cured
WPC	White Portland cement

List of Figures

List of Tables

Chapter 1
Introduction

1.1 Introduction

Solid waste has become one of the major environmental concerns of the today's world. The rapid increase in the construction activities and large amount of construction and demolition waste result a huge contribution to the total solid waste. The composition of typical construction and demolition waste is presented in Fig. 1.1 (Oikanomou 2005). The construction industry contributes substantially to the generation of solid waste in almost all the countries. In North America, the construction and demolition waste contributes around 25–40% of the total waste generated depending upon the region (Tabsh and Abdelfatah 2009). The Construction Materials Recycling Association (CMRA) has conducted a study on construction and demolition waste, related to the buildings and it was estimated to be around 136 million tonnes of waste material. Also, it was reported that apart from the building waste, a millions of tonnes of waste is coming from roads, bridges and airports construction and renovation. In developed countries, the annual per capita building and construction waste generation was 500–1000 kg and in European countries, the building and construction waste was estimated to be around 175 million tonnes per year (Nitivattananon and Borongan 2007). As per the European Commission (DG ENV 2011), the European Union construction industry generates 531 million tonnes of construction and demolition waste annually which is approximately 25% of the total waste materials exists in the world and the construction waste and recycling rate in each of the European countries is presented in Table 1.1 (Ozalp et al. 2016). Approximately 46% of construction and demolition waste was recycled in the 27 member countries of the European Union (Ozalp et al. 2016). The construction and demolition waste generation scenario in Asian countries is also in the same trend. It was reported that Asia alone generates about 760 million tonnes of construction and demolition waste every year (World Bank 1999). In China, around 15.5 million tonnes of construction waste generated annually. Recent natural disasters such as Wenchuan earthquake in 2008, Yushu

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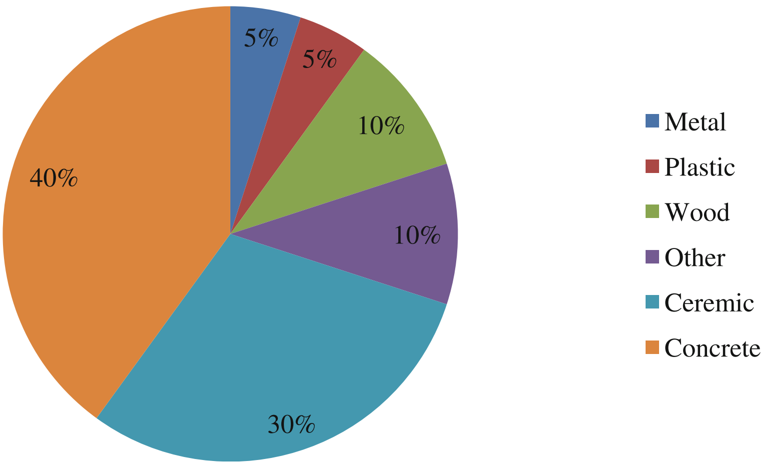

Fig. 1.1 Composition of construction and demolition waste (Oikonomou 2005)

Table 1.1 As per European Commission (DG ENV 2011), EU construction and demolition waste (C&DW) quantity and recycling rates

Country	C&DW (Million tonnes)	Recycling (%)	Country	C&DW (Million tonnes)	Recycling (%)
Denmark	5.27	94	Malta	0.8	0
Estonia	1.51	92	Holland	23.9	98
Finland	5.21	26	Poland	38.19	28
France	85.65	45	Portuguese	11.42	5
Germany	72.4	86	Romania	21.71	0
Greece	11.04	5	Slovakia	5.38	0
Hungary	10.12	16	Slovenia	2	53
Ireland	2.54	80	Spain	31.34	14
Italy	46.31	0	Sweden	10.23	0
Latvia	2.32	46	England	99.1	75
Lithuania	3.45	60	EU-27	531.38	46
Luxemburg	0.67	46	–	–	–

earthquake in 2010, Yunnan earthquake in 2011 in China have resulted a large quantity of waste concrete (Xiao and Li 2013). According to the annual report of Dubai Municipality's Waste Management Department, there was about 27.7 million tonnes of construction waste generated from various construction sites in the city in 2007 (Shrivastava and Chini 2009). This was recording growth in construction waste generation of 163% in comparison to the waste generated in 2006.

Like other developing countries, India too is generating a huge quantity of construction and demolition waste due to rapid growth in construction industry. According to eleventh five-year plan, the construction industry was second to agriculture in terms of magnitude (Government of India 2007). It is one of the largest employers in the country. The employment figures have shown steady increase from 14.6 million in 1995 to 31.46 million in 2005. The construction industry in India significantly affects the economic growth of the country. During 2004–2005, over US$ 100 billion has been invested in this sector. Due to the Government of India's (GOI) recent initiative to allow 100% foreign direct investment in real estate development projects, the construction sector likely to continue to record higher growth in the coming years (Market Research 2006). The contribution of the construction industry in total gross domestic product (GDP) has risen from 6.4% in 2000–2001 to 7.2% in 2004–2005 (TIFAC Ed 2005). Technology Information, Forecasting and Assessment Council (TIFAC) indicates that the total construction work was equivalent to $847 billion during the period 2006–2011 (TIFAC Ed 2005). According to the tenth five-year plan, the materials' cost was around 40–60% of the total project cost. The construction and demolition waste in India was estimated to be approximately 14.5 million tonnes per year (Pappu et al. 2007). The Central Pollution Control Board (CPCB) had estimated the total solid waste generation as 48 million tonnes per year for the year 2001 and out of which 12–14.7 million tonnes from the construction industry alone and by 2010, this was expected to be around 24 million tonnes (TIFAC Ed 2005). In addition, the new zoning bye-laws, legitimization of squatter settlements and increase in the urban population due to industrial development have led to the demolition of structures in the larger cities. Insufficient capacity of old road bridges for present and future growing traffic and modernization of highway bridges needs the demolition of old bridges too. Also, structures are destroyed due to either natural disasters like earthquakes, cyclones, etc. or man-made disasters. Hence, the entire world is facing the problem of handling the waste material generated from the demolition. On the other side, there is a huge requirement of raw materials in the construction sector in India. Projections for building material requirement of the housing sector indicate a shortage of aggregates to the extent of about 55,000 million cubic meters. For achieving the target for road development up to 2010, an estimated 750 million cubic meters of coarse aggregate as subbase material shall be required (TIFAC Ed 2005). Recycling of aggregate material from the construction and demolition waste may reduce the demand–supply gap in both these sectors.

The use of old construction materials in new constructions is not a new technique. Many civilizations have used and reused the construction materials of earlier civilizations or their own destroyed architectures either due to war or due to natural disaster to construct new structures. The best example is that the construction of Vatican Basilica with the stones of ruined Romanesque. The waste management hierarchy which consists of four strategies is shown in Fig. 1.2 including reducing, reusing, recycling and disposing waste (Peng et al. 1997).The main principle of this hierarchy is to minimize the usage of resources and elimination of environmental pollution, which happens to be the two main aspects of sustainable construction

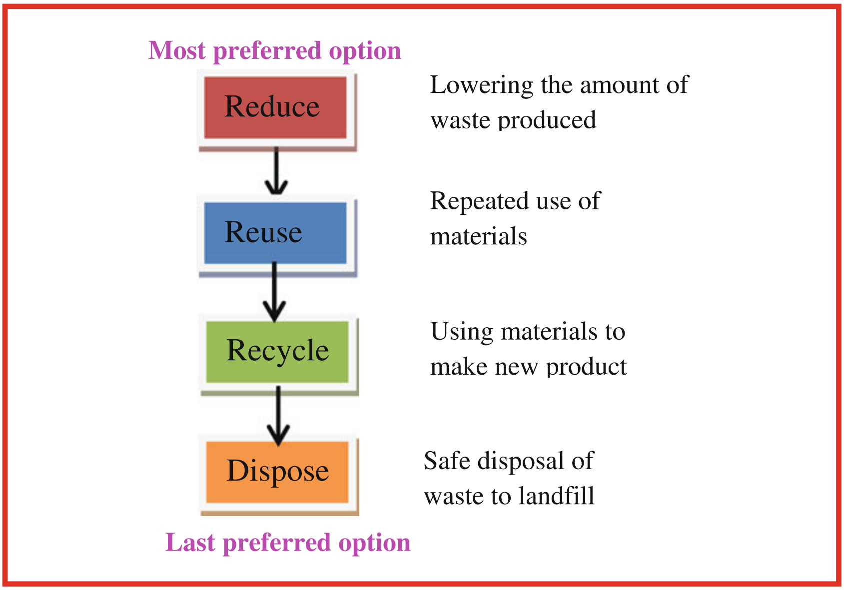

Fig. 1.2 Waste management hierarchy (Ghafourian et al. 2016)

sector (Peng et al. 1997; Zare et al. 2016). The first three strategies of waste management hierarchy are often called 3R's in the management of C&DW (Ghafourian et al. 2016). Though the "3R" formula, i.e., reduce, reuse, recycle is one of the best policies to achieve the sustainable construction, due to partial implementation of this technique in most of the countries still lots of quantities of construction and demolition waste is lying in the site and deposited on landfills. The European Demolition Association estimates that about 200 million tonnes of waste generated annually, out of which 30% of this quantity being recycled. However, there was large difference in the quantities of recycling in the region wise. For example, Netherlands and Belgium achieve recycling rates of about 90%, whereas, other European countries like Italy and Spain, the recycling rate was below 10% (Collepardi 2002). The Japan and Germany have also reached the recycling rates of around 96% and 86%, respectively. The Construction Materials Recycling Association (CMRA) estimate that 25% of the construction and demolition waste was recycled and most of these recycled materials were used as base materials for road construction. In India, the recycling of the construction and demolition waste is being started recently.

Although there is a considerable potential for using the construction and demolition waste as aggregates in concrete, considerable amount is either remain in site or landfilled, the "last resort" in the waste management hierarchy (Rao 2005). Most of the developed/developing countries, the construction waste treated as inert waste, harmless, and bulky, which does not give rise to problems. However, this waste consists of mixture of various materials of different characteristics that are often deposited (dumped on land) without any considerations, causing many problems and encouraging the illegal dumping of other kinds of waste. This puts an additional burden to the solid waste management. Also, there is a shortage of dumping sites in the developed countries. Further, there is increase in the cost of transport to dispose waste to the dumping sites. Additionally, there is a need to preserve the depletion of natural resources from the environmental pollution point of view and also, it is essential for the sustainable development. Therefore, there is no wonder that the recycling is one of the best solutions sought. The recycling technology not only solves the problem of waste disposal, but reduces the cost and preserves environment also. In addition, the recycling and proper management of construction and demolition waste gives better opportunities to handle the other kinds of waste, as less land is used for dumping of construction and demolition waste.

Recycling of different materials has been tried in the past in different forms. Recycling of coarse aggregates, generated from the demolished or disaster driven waste concrete, has attracted the researchers in the recent past. Coarse aggregates, one of the important ingredients of concrete, are becoming dearer in terms of the increasing cost of the materials and its availability. Hence, recycling of coarse aggregates is of utmost importance to overcome these difficulties. Thus, this book mainly focuses on using the construction and demolition waste as recycled coarse aggregate (RCA) in the production of concrete. The Building Contractors Society of Japan (BCSJ 1978) had proposed the following terminology on recycled aggregate and recycled aggregate concrete (RAC).

Waste concrete: It is the concrete debris from demolished structures as well as the fresh and hardened concrete refused by ready-mix plants or site mix concrete producers or concrete product manufacturers.

Original concrete: It is the concrete from plain and reinforced concrete structures or precast concrete elements which can be used as raw material for the production of recycled aggregates.

Recycled concrete aggregates: These are the aggregates produced by the crushing of original concrete. These aggregates may be either fine or coarse recycled aggregates.

Recycled aggregate concrete (RAC): It is the concrete produced by using the recycled aggregates or the combination of recycled and natural aggregates.

1.2 Applications of Recycled Aggregates

Application of recycled aggregates are very important to attain the sustainability in the construction sector as it reduces the construction and demolition waste and preserves the natural resources thereby reduces the environmental pollution. In general, the applications without any processing include (Yong and Teo 2009):

a. many types of general bulk fills;
b. bank protection;
c. base or fill for drainage structures;
d. road construction;
e. noise barriers and embankments.

After removal of contaminants through selective demolition, screening, air separation and size reduction in a crusher to aggregate sizes, crushed concrete can be used as (Yong and Teo 2009):

a. new concrete for pavements, shoulders, median barriers, sidewalks, curbs and gutters, building and bridge foundations;
b. structural grade concrete;
c. soil-cement pavement bases;
d. lean-concrete bases;
e. bituminous concrete.

Growth in the use of recycled concrete for retaining wall backfill, Portland cement concrete mix, landscaping rock, drainage aggregates and erosion control is also happening (Nuruzzaman and Salauddin 2016). The some of the applications of RAC are shown in Figs. 1.3, 1.4, 1.5, 1.6, 1.7, and 1.8.

Fig. 1.3 RAC pavement in Shanghai, China (Li 2009). This pavement was designed for a width of 7 m and 240 mm height. 50% RCA was used. The RCA was derived from the highway of an airport

Fig. 1.4 Shanghai ecological building, China (Li 2008). This was a demonstrative ecological green building constructed in Xinzhuang, Shanghai, in 2004 with a total area of 1900 m^2. Large amount of RAC (388 m^3) was used mainly in foundations and walls. The RCA was produced by crushing old concrete pavements and structural elements

Fig. 1.5 The BRE office building in Watford, UK, 1995/96 (BRE 1998). For the foundations, a C25 mix (75 mm slump) was used with a minimum OPC-based cement content of 350 kg/m^3 and a maximum free water–cement ratio of 0.50 was required. For floor slabs, a C35 mix, also with 75 mm slump was specified. Over 1500 m^3 of RAC supplied for foundations, floor slabs, structural columns, and waffle floors

Fig. 1.6 Waldspirale residential building in Darmstadt, Germany, 1998 (Marinkovc and Ignjatovic 2010) Total 12 000 m^3 of RAC was built in (Source: http://www.b-i-m.de/projekte/projframe.htm)

Component	Type	Quantity (kg/m^3)	
		C30/37	C25/30
Recycled aggregate	0/2 mm	616	615
	2/8 mm	530	290
	8/16 mm	569	334
	16/32 mm		554
Portland cement	CEM I 42,5 R	300	
Portland cement	CEM I 32,5 R		290
Additives	Pulverised fuelash	50	40
Superplasticiser		1.5 kg/m^3	
Workability		Normal (According to DIN 1045)	
Compressive strength (Avg. 28 days)		42.9 MPa	36.4 MPa

1.3 Benefits of Recycled Aggregates

The benefits of recycled aggregate concrete are advantageous to the environment. The major benefits are (i) economic aspects, (ii) reducing environmental aspects and (iii) saving natural resources.

Fig. 1.7 Vilbeler Weg office building, Darmstadt, Germany, 1997/98 (Marinkovc and Ignjatovic Marinkovic and Ignjatovic 2010) Total 480 m^3 of RAC was built in (Source: http://www.b-i-m.de/projekte/projframe.htm)

Component	Type	Quantity (kg/m^3)
		C30/37
Recycled aggregate	0/2 mm 2/8 mm 8/16 mm	585 545 568
Portland cement	CEM I 42,5 R	300
Free water		170
Additives	Pulverised fuelash	40
Superplasticiser		5-18 ml/kg of cement
Workability (flow table value)		550 mm
Avg. Compressive strength (28 days)		45 MPa

1.3.1 Economic Aspects

Using construction and demolished concrete as aggregate is an effective and economically viable option to recycle waste materials. This option reduces solid waste, thus saving landfill spaces and minimizing consumption of natural sources (TDS 1998).

Fig. 1.8 New high school building in Norway (Mehus and Hauck 2002), in which 35% recycled coarse aggregates were used in foundations, basement walls, and columns

1.3.2 Reducing Environmental Impacts

Recycling of waste can greatly reduce the environmental damages caused by incorrect disposal, extend the useful life of landfills and preserve finite natural resources (Carneiro 2000). The main advantage of recycling of construction and demolition waste concrete is that substances are reused which would otherwise be classified as waste. Recycled aggregate can have lower embodied energy in addition to abridged transport emissions especially where recycled materials were reused in close juxtaposition to the site of processing.

1.3.3 Saving Resources

Recycling of concrete demolition wastes can provide opportunities for saving resources, energy, time and money. Furthermore, recycling and controlled management of concrete demolition wastes will save use of land and create better opportunities for handling other kinds of wastes.

1.4 Constraints of Recycled Aggregate Concrete

Even though there are benefits to the industry and environment by the use of recycled aggregate concrete, there are some constraints during implementation in the aspects of both technology and management.

1.4.1 Management Problems (Tam and Gao 2003)

1.4.1.1 Lack of Suitable Regulations

Suitable regulations are insufficient to managing the use of recycled materials (Kawano 2000). As a result, the industry is loath to adopt recycled products which need investment in research, production and use to the non-mandatory nature in adoption.

1.4.1.2 Lack of Codes, Specifications, Standards and Guidelines

There are sufficient codes, standards, specifications and guidelines for normal concrete. In the recent times, few countries have published the specifications and norms for the use of recycled aggregate in structural and non-structural concrete applications. But, these are very limited and many countries are yet to move in this direction.

1.4.1.3 Lack of Experience

Experience needs to be accumulated to ensure safety in the use of any new materials; the lack of which forms a barrier in the use of recycled aggregate concrete (Chan et al. 2000)

1.4.2 Technology Problems

Even though a few researchers suggested new techniques for quality improvement of recycled aggregate and recycled aggregate concrete, still there are some technical problems assorted to RAC mainly comes from the poor performance of recycled aggregates (Tam and Gao 2003). These includes

1.4.2.1 Cement Mortar Attached to Aggregate

Recycled aggregates are mainly consists of considerable amount of light porous old cement paste, thus effecting the physical and mechanical properties and performance of recycled aggregate. Further, it affects the properties of concrete particularly the long-term and durability performance of recycled aggregate concrete.

1.4.2.2 Poor Grading

During the crushing process, large amount of finer particles produced due to the adhered old cement mortar on the surface of the recycled aggregate. Poor grading, such as too harsh or too many fines, is one of the problems in the use of recycled aggregate concrete. Fine particles of recycled aggregate will severely affect the water demand and water-to-cement ratio in RAC.

1.4.2.3 High Porosity of Recycled Aggregates

As the recycled aggregates are obtained by crushing the old concrete, light porous nature of old cement mortar attached to recycled aggregates. Therefore, the recycled aggregates are more porous and thus less resistance against mechanical actions compared to natural aggregates.

1.4.2.4 Weak Interfacial Transition Zone

In general, the interfacial transition zone (ITZ) between cement mortar and aggregate plays a major role in determining the mechanical properties of concrete. In recycled aggregate concrete, there are more ITZs: ITZ between RA and new cement paste and ITZ between old cement paste and new cement paste. Therefore, these weak transition zones thus exist between old and new mortars and aggregates exhibit a different microstructure in recycled aggregate concrete and this will directly influence the performance of concrete.

1.4.2.5 Transverse Cracks Generated

Recycled aggregate seems to have direct relationship with transverse cracks within the concrete (Buch et al. 2000). In general, the poor performance of RAC is related with the cracks and fissures, which were formed in the recycled aggregate during crushing process, thereby rendering the aggregate susceptible to permeation, diffusion and absorption of fluids (Olorunsogo and Padayachee 2002). Larbi et al. (2000) reported the classification of the extent on microcracking is presented in Table 1.2.

Table 1.2 Criteria used for classifying the extent of micro-cracking of the treated concrete aggregate (Larbi et al. 2000)

Classification of the extent of microcracking	Description of classification
Very low	$\leq$ 20% of the aggregates in a specimen contains more than 5 microcracks
Low	20–40% or less of the aggregates in a specimen contains extra than 5 microcracks
Moderate	40–60% or less of the aggregates in a specimen contains more than 5 microcracks
High	60–80% or less of the aggregates in a specimen contains extra than 5 microcracks
Very high	$\geq$ 80% of the aggregates in a specimen contains more than 5 microcracks

1.4.2.6 Variations in Quality

The quality of demolished concrete depends on type of structures and the quality of parent concrete that was used and these are vary from site to site and structure to structure. Therefore, this brings a wide variation in the quality of recycled aggregate (Kawano 2000).

1.4.2.7 High Impurity

Even though the standards limits the levels of chloride and sulfate compositions for the use of aggregate in concrete, other impurities in recycled aggregate (RA) required to be monitored and controlled to confirm the finished concrete has consistent strength and durability (Coventry 1999).

1.4.2.8 Low Quality

The main problem in usage of recycled aggregate concrete is that the quality of recycled aggregate is poorer than the natural aggregate due to the light and porous nature of old cement mortar attached with the surface of RA (Tomosawa and Noguchi 2000). That is why most of the users are not having assurance in adopting recycled aggregate.

1.5 Classification of Recycled Aggregates

In general, the C&DW contains the concrete rubble/ceramic, brick, glass, wood, etc. Based on the composition of C&DW, the recycled aggregates were classified in different National Standards and are presented in Table 1.3 (Martin-Morales 2013).

1.6 Current Global Scenario

This section describes the recycling status of the construction and demolition waste, the specifications and guidelines existing on the use of recycled aggregates in the production of concrete in countries like Japan, Germany, UK, Hong Kong, Australia, China.

1.6.1 Japan

The Japanese government has launched the Recycling Law in the year 1991. The Ministry of Construction (MOC) has established the "Recycle 21" in 1992, which specifies the targets for recycling of different kinds of construction by-products. By the year 2000, 96% of the demolished concrete was recycled against the target of 90%. But, all most all the recycled aggregates were used as sub-base materials for road pavements. The current Japanese Industrial Standards (JIS) JIS A 5308 for ready mixed concrete does not allow using the recycled aggregate in the concrete. To encourage the recycled materials in the construction industry, the JIS Civil Engineering Committee has made a recommendation in 1998. In response to this, the Japanese Concrete Institute established a committee to draft a new JIS for recycled materials in construction. One of the drafts released by JIS in 2000 was Technical Report TR A 0006 "Recycled Concrete using Recycled Aggregates." It allows recycled concrete to be used independently from JIS A 5308. According to this, the quality of recycled aggregates should satisfy the requirements presented in Table 1.4 (Hirotaka and Kawano 2002). In addition, it was specified that the grading limits for recycled coarse aggregates are same as natural aggregate. However, the grading specifications for fine recycled aggregate were changed from that of natural aggregate: The limit of upper percentage of fine particles under 0.15 mm is raised from 10% to 15% and the ranges of 1.2–2.5 mm and 2.5–5.0 mm were widened considering the state of actual products.

The recycled concrete was classified into three categories, and their requirements are presented in Table 1.5.

Normal recycled concrete denotes filling and leveling concrete that is for non-structural purpose where high strength and high durability are not prime concern. Chloride controlled concrete is same as normal concrete, but for members

Table 1.3 Recycled aggregate classifications on the basis of composition (%) (Martin-Morales et al. 2013)

Scope	Standard/ guidelines	Standard class	Unified class	Concrete	Masonry	Natural aggregate	Organic material	Contaminants/ impurities	Lightweight materials	Fines
Australia	CSIRO	Class 1A	RCA	<100			n.a.	1	n.a.	n.a.
		Class 1B	MRA	<70	<30		n.a.	2	n.a.	n.a.
Belgium	PTV 406	Crushed concrete debris	RCA	>90	<10		0.5	0.5(a)	n.a.	n.a.
		Crushed mixed debris	MRA	>40	>10		0.5	1(a)	n.a.	n.a.
		Crushed brickwork debris	RMA	<40	>60		0.5	1(a)	n.a.	n.a.
Brazil	NBR 15116	ARC	RCA	>90		(b)	n.a.	3	n.a.	7
		ARM	MRA	<90		(b)	n.a.	3	n.a.	10
China (c)	DG/TJ07/ 008	Type I	RCA	>95	<5		0.5	1	n.a.	n.a.
		Type II	MRA	<90	>10		n.a.	n.a	n.a.	n.a.
Denmark	DS 2426	GP1	RCA	>95			n.a.	n.a	n.a.	n.a.
		GP2	MRA	>95			n.a.	n.a	n.a.	n.a.
Germany	DIN 4226-100	Type 1	RCA	>90	<10		n.a.	1(e)	n.a.	1
		Type 2	RCA	>70	<30		n.a.	1(e)	n.a.	1.5
		Type 3	RMA	<20	>80	<20	n.a.	1(e)	n.a.	3
		Type 4	MRA		>80(d)		n.a.	1(e)	n.a.	4
Hong Kong	WBTC 12	Type II	RCA	<100			n.a.	1	0.5	4
Japan (c)	JIS A 5021	ARH	RCA				n.a.	3	0.5	4
Netherlands	CUR	ARH	RCA	>95	<5		n.a.	0.1	n.a.	
	NEN 5905	ARH	RCA	<80		<20	n.a.	n.a	0.1	3
Norway	NB 26	Type 1	RCA	>94	<5	(b)	n.a.	1(e)	0.1	n.a.
		Type 2	MRA	>90		(b)	n.a.	1(e)	0.1	n.a.

(continued)

Table 1.3 (continued)

Scope	Standard/ guidelines	Standard class	Unified class	Concrete	Masonry	Natural aggregate	Organic material	Contaminants/ impurities	Lightweight materials	Fines
Portugal	LNECE 471	ARB 1	RCA	>90	<10	(b)	n.a.	0.2(f)	1	n.a.
		ARB 2	RCA	>70	<30	(b)	n.a.	0.5 (f)	1	n.a.
		ARC	MRA	>90		>10	n.a.	1 (f)	1	n.a.
Spain	EHE-08	RCA	RCA		<5		0.5	(g)	1	2
Switzerland	SIA 2030	BC	RCA		<3		n.a.	1	n.a.	n.a.
		BNC	MRA				n.a.	2	n.a.	n.a.
UK	BS 8500-2	RCA	RCA	>95	<5		n.a.	1 (h)	0.5	5
		RA	MRA		<100		n.a.	1 (h)	1	3
	BRE Digest 433	RCA I	RMA		<20	>80	n.a.	5	1	n.a.
		RCA II	RCA	<20		>80	n.a.	1	0.5	n.a.
		RCA III	MRA	<10	<10	>80	n.a.	5	2.5	n.a.
	RILEM	Type I	RMA		<100		1	5	1	3
		Type II	RCA	<100			0.5	1	0.5	2
		Type III	RCA	<20	<10	>80	0.5	1	0.5	2

n.a no limit available in the standard or guideline
(a) Less than 5% of bituminous material in all types
(b) Included in the percentage of recycled concrete aggregate
(c) This standard classifies recycled aggregate according to its properties
(d) 20% bituminous materials and others
(e) For bituminous materials 1% in all types.
(f) Contaminants of bituminous materials ARB 1 < 5%; ARB 2 < 5%; ARC < 10%
(g) Bituminous materials < 1%; glass, metals, plastics, etc. < 1%
(h) Bituminous materials, RCA < 5%; RA < 10%

Table 1.4 Quality of recycled aggregates in Japan (Hirotaka and Kawano 2002)

	Water absorption (%)	Fine particle content (%)
Coarse aggregate	<7	<2
Fine aggregate	<10	<10

Table 1.5 Requirements for recycled concrete for different usage (Hirotaka and Kawano 2002)

Description	Class		
	Normal	Chloride controlled	Flexible use
Nominal strength (MPa)	12	12	18
Max. grain size (mm)	20 or 25	20 or 25	As required
Slump (mm)	150	15	As required
Chloride content (kg/m^3)	–	0.6	As required

with steel reinforcement, whereas flexible use recycled concrete is basically for wide range of members, may be sometimes for structural use, but under the guidance of engineer who has expert knowledge on recycled concrete. It is important to mention that the recycled aggregates should be presoaked for controlling the workability and also, the blast furnace slag cement (Class B) or fly-ash cement (Class B) should be used to reduce the alkali–aggregate reaction (AAR).

Quality The quality of different classes, i.e., normal and chloride controlled recycled concrete are established by experiments. The individual experiment results of any strength must be greater than 10 N/mm^2 and the mean of three such test results must be greater than 12 N/mm^2 for strength. Similarly, the results of chloride content and slump tests must lower than 0.3 kg/m^3 and 15 cm, respectively. Further, the test result of air content must lies between 3.0% and 7.0%. According to JIS A 5308, any strength of individual test result should be larger than the nominal strength and the mean of three such test results should be larger than 85% of the nominal strength.

Mix Proportion the Technical Report (TR) outline specifies that by using one of the following methods the mix proportion shall be fixed: test mixing (but w/c < 65%); standard mixing w/c < 60%, cement > 280 kg/m^3. Based on the composition and physical properties, presently three types of recycled aggregates such as high-quality RA (type H), medium-quality RA (type M) and low-quality RA (type L) are considered in Japan. The JIS A 5021 was established in 2005 for the use of high-quality recycled aggregate in concrete (Pellegrino and Faleschini 2016). Recently, this has been replaced by JIS A 5021-2011. Type H recycled aggregates are produced from the demolition of concrete structures by advanced processing (crushing, grinding). With respect to the type M and type L recycled aggregates, the RA of type H should strictly follow the limits of contaminants, composition and physical properties. Type H recycled aggregates can be used in structures with nominal strength lower than 45 MPa provided if they have less than 3% of other

Table 1.6 Acceptance criteria of recycled aggregate type according to Japanese Standards (Pellegrino and Faleschini 2016)

	Absorption ratio of aggregate (%)	Oven-dry density (kg/m^3)
Type H coarse RA	≤ 3.0	≥ 2500
Type H fine RA	≤ 3.5	≥ 2500
Type M coarse RA	≤ 5.0	≥ 2300
Type M fine RA	≤ 7.0	≥ 2200
Type L coarse RA	≤ 7.0	No requirement
Type L fine RA	≤ 13.0	No requirement

non-concrete and non-virgin aggregate materials. The recycled aggregate of Type M can be used where the members not imperiled to frost action (concrete filled in steel tubes, piles, and underground beams), and the RA of type L can be used in the applications of filling and leveling and backfilling. As an assessment of the degree of extent of alkali–aggregate reactivity, the recycled aggregates of type L can be used only with type B blended cements. The criteria for acceptance of type H, type M and type L recycled aggregates in terms of absorption ratio and oven-dry density are listed in Table 1.6 (Pellegrino and Faleschini 2016).

1.6.2 Germany

According to Federal Statistical Office (Destatis 2005a, b, 2006), Germany, the construction and demolition waste generated in 2002 and 2003 was 241 and 233 million tonnes, respectively. Out of which, 85.6% and 86.2% were recycled in 2002 and 2003, respectively, and the rest of the construction and demolition waste was disposed on land. High material, energy, labor and waste disposal costs of Germany favors the economics of recovering, reusing, and recycling as much construction and demolition waste as possible. Additionally, strong waste management systems have long been required bylaws and regulations at all levels of government in order to minimize the impact of construction and demolition waste in the waste stream.

"Guideline for Recycled Concrete Aggregate" of the German Standardization Association for Reinforced Concrete has introduced the possibility to recycle the construction and demolition waste as aggregate for structural concrete since 1998. According to this guideline, up to 30% recycled aggregate may be used as concrete material. However, there is a limitation on grade of concrete and exposure conditions. Attempts were also made to develop the guidelines for masonry rubble in higher concentrations for concrete aggregate in low-grade concrete or for non-structural applications provided a sufficient durability can be assured. The guidelines for the quality requirements for aggregates from mixed waste are given in newly published Standard DIN 4226-100 (2002). According to this standard, the recycled aggregates are classified into four types (Type 1–4) based on the content of

Table 1.7 Composition and use of recycled aggregates according to DIN 4226-100 (2002) (Tam and Gao 2003)

Constituents	Maximum or minimum content as percentage by mass			
	Type 1: Concrete Aggregate	Type 2: Building Aggregate	Type 3: Masonry Aggregate	Type 4: Mixed Aggregate
Concrete and natural aggregate as in DIN 4226-1	≥ 90	≥ 70	≥ 20	≥ 80
Clinker, solid bricks	≤ 10	≤ 30	≥ 80	
Calcareous sandstone			≤ 5	
Other mineral constituents such as porous bricks, aerated concrete, lightweight concrete, plaster, mortar, porous slag, pumice	≤ 2	≤ 3	≤ 5	≤ 20
Asphalt	≤ 1	≤ 1	≤ 1	
Foreign matter such as glass, non-ferrous metal slag, lump, gypsum, rubber, plastic, wood, plant residue, paper and other similar materials	≤ 0.2	≤ 0.5	≤ 0.5	≤ 1
Oven-dry density (kg/m^3)	≥ 2000	≥ 2000	≥ 1800	≥ 1500
Water absorption after 10 min	≤ 10	≤ 15	≤ 20	Not specified

Table 1.8 Maximum amount of coarse recycled aggregate as percentage of total aggregate (Mc Govern 2002)

Application	Maximum or minimum content as percentage by mass			
	Type 1: Concrete Aggregate	Type 2: Building Aggregate	Type 3: Masonry Aggregate	Type 4: Mixed Aggregate
Reinforced concrete: interior elements	50	40	40	–
Exterior elements	40	–	–	–
Fill or subbase material include fine recycled aggregate	100	100	100	100

concrete, natural aggregates, clinker, non-pored bricks, sand-lime bricks, asphalt, other materials such as pored bricks, lightweight concrete, no-fines concrete, plaster, mortar, porous slag, pumice, stone and foreign substances, e.g., glass, non-ferrous metal slag, gypsum, plastic, wood, paper, etc. The guidelines given in DIN 4226-100 (2002) for the composition and use of recycled aggregate are presented in Tables 1.7 and 1.8 respectively. Further, requirements on properties of recycled aggregates are similar to the requirements on natural aggregates.

Table 1.9 Substitution ratio (in % by volume) according to EN 206-1 and DIN 1045-2 (Pellegreno and Faleschini 2016)

Field of application		Type 1	Type 2
Dry	Exposure Class XC 1	≤ 45	≤ 35
Humid	Exposure Class X 0	≤ 45	≤ 35
	Exposure Class XC 1 to XC 4	≤ 45	≤ 35
	Exposure Class XF 1 and XF 3	≤ 35	≤ 25
	Exposure Class XA 1	≤ 25	≤ 35

Pellegreno and Faleschini (2016) were reported the other two types, i.e., Type 3 and Type 4 recycled aggregates are barred in making the structural concrete. Further, the usage of crusher sand also barred from the recycled aggregate. The minimum size of recycled aggregate permitted is 2 mm. It was also reported that the limits are specified in code: Concrete with recycled aggregate, in which the substitution ratios and strength class (C30/C37) are specified in relation with the field of application and exposure class (Table 1.9).

1.6.3 United Kingdom

The construction and demolition waste generation has been consistent at 90 million tonnes from the year 2001 to 2005 (Capita Symonds Ltd. 2007). This was an increase of about 21 million tonnes from the year 1999. Recycling of the construction and demolition waste using crushers and screeners has increased from 49% in 2001 to 52% in 2005. However, the proportion of construction and demolition waste sent to landfill has increased from 26 to 31% and the amount of waste going to exempt sites has dropped from around 25 to 17%. To bridge the gap between the current United Kingdom (UK) practice and specifications, The Building Research Establishment (BRE) has published the guidelines on the use of recycled aggregates. According to BRE Digest 433 (1998), the recycled aggregates are classified into three types based on relative composition of concrete to brick masonry and are presented in Table 1.10.

Table 1.10 Classification of recycled aggregate (BRE Digest 433 1998)

Class	Origin	Brick content by weight	Strength by ten percent fines test	Relative quality
RCA (I)	Brickwork	0%–100%	70 kN	Lowest
RCA (II)	Concrete	0%–10%	>100 kN	Highest
RCA (III)	Concrete and Brick	0%–50%	70 kN	Moderate

Table 1.11 Maximum recommended limits of impurities (by weight) (BRE Digest 433 1998)

Type of impurity	Type of Application		
	Use in concrete as coarse aggregate	Use in road construction	Hardcore, fill or granular material
Asphalt and Tar	Included in limit for other foreign material	10% in RCA (I) and (III) 5% in RCA (II)	10%
Glass		Contents above 5% to be documented	
Wood	1% in RCA (I) 0.5% in RCA (II) 2.5% in RCA (III)	Subbase: 1% Capping layer: 2%	2%
Sulfates	Concrete and CBM: 1% acid-soluble SO_3		
Other foreign material such as metals, plastics, clay, etc.	5% in RCA (I) and (III) 1% in RCA (II)		

The BRE Digest 433 (1998) has also specified the maximum limits on impurities and is presented in Table 1.11. If recycled aggregates of classes RCA (I), RCA (II), or RCA (III) satisfy the quality and grading requirements of BS 882, "Specifications for aggregates from natural sources for concrete" may be used in concrete production.Pellegreno and Faleschini (2016) stated that in the UK, the difference between RCA and RA is clearly distinguished. The former is derived from concrete-based material, in which the maximum content of masonry allowed is 5% (in weight) and the latter is derived from a mixed waste. Vazquez (2013) reported that the use of RA is limited to the applications of underpinning works and road surfaces specified in the complementary UK Standard to EN 206-1; BS 8500-2 2015. The application of recycled aggregate is restricted to mild exposure conditions and strength class of C16/20.

The choice of use of RA, which reveals in a large inconsistency among their origin, composition and properties are ruled by the lack of exact regulation. Whereas, within the recommended exposure classes: X0, XC1, XC2, XC3, XC4, XF1, DC-1 and a concrete strength class of C40/50, up to 20% by weight of natural aggregate is allowed to replace with RCA. The severe freeze–thaw exposure conditions (XF2-XF4) and the salt (XS, XD) are excluded. If the prescriber takes his own responsibility for the experimental results, higher substitution of recycled aggregate-to-natural aggregate is accepted subject to the acceptance of experimental results.

It was reported that BS 8500-2 (BSI 2015) published the coarse recycled aggregate requirements in concrete (Pellegreno and Faleschini 2016). According to this code, the recycled aggregates are classified into recycled concrete aggregate (RCA) and recycled aggregate (RA) based on its composition and other requirements of these classes are presented in Table 1.13. Further, the coarse recycled concrete aggregates are allowed only up to a concrete strength class of C40/50 and within the exposure classes as listed in Table 1.14.

Table 1.12 Designations for RCA and RA concreting aggregates for general use recommended by BS EN 12620-2002 (Collins et al. 2004)

Properties	Category to BS EN 12620-2002 or other limit[2]
Grading Coarse aggregate	Annex C in BS PD 6682-1: 2003
Flakiness index	FI_{35} (There should be no difficulty in consistently producing RCA or RA to this limit)
Fines RCA RA	f_4 (BS 8500-2: 2002 will accept up to 5%) f_3
Resistance to fragmentation	No requirement. RCA will normally comply with LA_{40} and RA with LA_{40} or LA_{50}
Acid-soluble sulfate content	$AS_{1.0}$
Total sulfur	$\leq$ 1% by mass
Masonry[a] RCA RA	$\leq$ 5% by mass $\leq$ 100% by mass
Lightweight material[b, c] RCA RA	$\leq$ 0.5% by mass $\leq$ 1.0% by mass
Asphalt[c] RCA RA	$\leq$ 5% by mass $\leq$ 10% by mass
Other foreign material such as glass, plastics, metals[c]	$\leq$ 1.0% by mass
Constituents in RCA or RA fine aggregate which alter the rate of setting and hardening of concrete [d] -increase in mortar setting time -decrease in compressive strength of mortar	$\leq$ 120 min $\leq$ 20% at 28 days

[b]Material with a Density Less Than 1000 kg/m^3
[c]Property where currently no BS EN 12620-2002 limit—test and limits taken from BS 8500-2:2002
[d]Comparison should be with mortar made with standard clean sand; heating one sample according to the method in BS EN 1744-1:1998 is not appropriate for RCA/RA. Alternatively, if concrete is made on a regular basis from these materials, consistent strength development in the concrete should be checked for each day's production or batch of materials.

The guidelines give provisions for the use of recycled concrete aggregate in concrete with other exposure classes provided that it is demonstrated the resulting concrete is suitable for the intended applications.

Table 1.13 Requirements of coarse recycled concrete aggregate and recycled aggregate specified by BS 8500-2 (Pellegreno and Faleschini 2016)

Property	Recycled concrete aggregate	Recycled aggregate
Maximum masonry content (%)	5	100
Maximum fines (%)	5	3
Max. Lightweight material (density < 1000 kg/m^3) (%)	0.5	1.0
Maximum asphalt (%)	5	10
Max. other foreign materials (%)	1.0	1
Max. acid-soluble sulfates, SO_3	1.0	3

(1) Where the material to be used is obtained by crushing hardened concrete of known composition that has not been contaminated by use, and the only requirements are those for grading and maximum fines. (2) The provisions for recycled concrete aggregate may be applied to mixtures of natural coarse aggregate blended with the listed constituents. (3) The appropriate limit needs to be determined on a case-by-case basis

Table 1.14 Limitations on the use of coarse recycled concrete aggregate in concrete with different exposure classes in BSI 2002 (Pellegreno and Faleschini 2016)

Description		Severity of exposure			
X0	No risk of corrosion or attack	X0	–	–	–
XC	Corrosion induced by carbonation	XC-1	XC-2	XC-3	XC-4
XD	Corrosion induced by chlorides	*	*	*	*
XS	Corrosion induced by chlorides (seawater)	*	*	*	*
XF	Freeze/thaw attack	XF-1	*	*	*
DC	Sulfate attack	DC-1	*	*	*

*The guidelines give provisions for the use of recycled concrete aggregate in concrete with other exposure classes provided that it is demonstrated the resulting concrete is suitable for the intended applications

1.6.4 Hong Kong

There was about 20 million tonnes of construction and demolition waste generated in 2004, out of which 12% was disposed off at landfills and the rest was at public filling areas (Poon 2007). The amount of construction and demolition waste generated was about four to five times of that of municipal solid waste. The annual generation of construction and demolition waste was more than double from 1999 to 2004. The management of construction and demolition waste has become a major environmental issue in Hong Kong. The General Specifications (GS) for civil engineering works banned the use of recycled inert construction and demolition

Table 1.15 Specifications for the use of recycled coarse aggregates in concrete (WBTC 12/2002)

Mandatory requirements	Limits	Testing method
Minimum dry particle density (kg/m^3)	2000	BS 812: Part 2
Maximum water absorption (%)	10	BS 812: Part 2
Maximum content of wood and other material less dense than water (%)	0.5	Manual sorting in accordance with BRE Digest 433
Maximum content of other foreign materials (e.g., metals, plastics, clay lumps, asphalt and tar, glass, etc.) (%)	1	
Maximum fines (%)	4	BS 812: Sect. 103.1
Maximum content of sand (< 4 mm) (% m/m)	5	BS 812: Sect. 103.1
Maximum content of sulfate (% m/m)	1	BS 812: Part 118
Flakiness index (%)	40	BS 812: Sect. 105.1
Ten percent fines value (kN)	100	BS 812: Part 111
Grading	Table 3 of BS 882:1992	

materials except its use as fill material in reclamation and earth filling projects until 2001. But the revision of GS in 2001 in the form of corrigendum No. 1/2001 allows the use of recycled aggregates for use in earthworks, drainage and marine works. The Work Bureau Technical Circular (WBTC) 12/2002 published the specifications for the use of recycled aggregates in concrete applications in public work projects in Hong Kong and is presented in Table 1.15.

For lower grade applications, 100% recycled coarse aggregates were allowed in the production of concrete. The recycled fine aggregates were not allowed in the production of concrete. The target strength was specified at 20 MPa, and this concrete can be used in stools, benches, concrete mass walls, planter walls and other minor concrete structures where specifically permitted in the contract. For higher grade applications (25–35 MPa concrete), the above specifications allow a maximum of 20% recycled coarse aggregates in the production of concrete and it can be used for general applications except in water retaining structures.

After reviewing the various standards and WBTC 12/2002, the recent standard published by The Government of Hong Kong special administrative region for aggregates for concrete “Construction Standard (CS3: 2013)” has been included the specifications for recycled coarse aggregates in accordance with the WBTC 12/2002. The grading requirements for recycled coarse aggregate as per CS3: 2013 is presented in Table 1.16.

Table 1.16 Grading of recycled coarse aggregate (CS3: 2013)

Sieve size (mm)	Nominal size of graded aggregates (mm)			Nominal size of single-sized aggregates (mm)				
	40 to 5	20 to 5	14 to 5	40	20	14	10	5
50	100	–	–	100	–	–	–	–
37.5	90–100	100	–	85–100	100	–	–	–
20	35–70	90–100	100	0–25	85–100	100	–	–
14	25–55	40–80	90–100	–	0–70	85–100	100	–
10	10–40	30–60	50–85	0–5	0–25	0–50	85–100	100
5	0–5	0–10	0–10	–	0–5	0–10	0–25	45–100
2.36	–	–	–	–	–	–	0–5	0–30

Note: For coarse recycled 20 and 10-mm single-sized aggregates, the percentage by mass passing 4-mm test sieve shall not exceed 5%

1.6.5 Australia

About 32.4 million tonnes of solid waste was generated annually of which the waste from construction and demolition sector is of about 13.75 million tonnes (42% of total solid waste). From this, around 57% of the construction and demolition waste was recycled (Tam 2009). Among different types of construction and demolition wastes, concrete waste constitutes about 81.8% of the total waste, from which 54% of the concrete waste was recycled.

The Commonwealth Scientific and Industrial Research Organization (CSIRO) initiated to promote the use of recycled aggregate in the production of concrete. In 1998 and 2002, CSIRO has published two set of guidelines for the use of recycled aggregates in concrete for non-structural applications "Guidance on the preparation of non-structural concrete made from recycled concrete aggregates" and "Guide to the use of recycled concrete and masonry materials," respectively (CSIRO 1998, H155-2002). Two classes of recycled aggregates, namely Class 1 and Class 2, were recommended for non-structural applications and are presented in Table 1.17.

1.6.6 China

The amount of construction and demolition waste has reached 30–40% of the total solid waste. Among all the construction and demolition wastes, the waste generated from concrete was large. In 2006, the annual waste generated from concrete was about 100 million tonnes and it accounts for about 1/3 of the total construction and demolition wastes. Based on the annual cement production, the concrete waste forecasted for the future and it will be 638 million tonnes in 2020 (Shi and Xu 2006). In recent years, due to the rapid urbanization and the requirement of sustainable

Table 1.17 Classification of recycled aggregate (CSIRO 1998 and H155-2002)

Class	Subclass	Definition
Class 1	Class 1A	Uniformly graded coarse aggregate (4–32 mm) produced by crushing waste concrete with total contaminant levels lower than 1% of the bulk mass
	Class 1B	Class 1A recycled aggregate blended with not more than 30% crushed brick
	Grade 1	Plain unreinforced and reinforced concrete made with a maximum of 30% uniform quality of Class 1A recycled aggregate with characteristic strength up to and including N40 grade, i.e., 40 MPa
	Grade 2	Plain unreinforced and reinforced concrete made with up to 100% uniform quality of Class 1(A or B) recycled aggregate having characteristic strength up to including N25 grade, i.e., 25 MPa, concrete for use in non-structural concrete applications
Class 2	Class 2A1	Suitable for use in roads with a traffic loading of greater than 1×10^6 ESA as either base or subbase course
	Class 2A2	Suitable for use in roads with a traffic loading less than or equal to 1×10^6 ESA as either base or subbase course
	Class 2B	For use as a base layer for pavers in pedestrian areas, car parking, and shopping malls
	Class 2C	General filling behind curbs and gutters, retaining walls, or beneath grassed areas
	Class 2D	Bulk filling for urban and rural development for construction of embankments.
	Class 2E	Backfilling for subsoil drains and storm water pipes

development, more and more research activities have been undertaken. So far, over 30 universities, institutes, and companies in China have been engaged in the research and applications of recycled aggregate concrete (RAC). After experiencing some successful applications of RAC in pavements and buildings, a technical code for Application of Recycled Aggregate Concrete (DG/TJ07-008) was published in 2007 at Shanghai as regional standards (SCSS 2007). The details of the requirements of RAC in DG/TJ07-008 are presented in Table 1.18. In this code, the recycled coarse aggregates were classified in two types, namely Type 1 and Type 2 based on their water absorption, saturated-surface-dry (SSD) density and masonry content. The grading of the recycled coarse aggregates must fall within the limits for natural aggregates specified in current Chinese Codes, i.e., JGJ 52-2006 "Standard for technical requirements and test method of sand and crushed stone or gravel." The recycled fine aggregates were not allowed in RAC in this code.

Further, the following standards have been published related to the recycled coarse and fine aggregates for concrete and mortar and the highlights are presented in Tables 1.19, 1.20, and 1.21 (Bodet 2014).

Table 1.18 Requirements of recycled aggregates in concrete in DG/TJ07-008 (Li 2008)

Item	Type 1	Type 2
SSD density (kg/m^3)	≥ 2400	≥ 2200
Absorption (%)	≤ 7	≤ 10
Masonry content (%)	≤ 5	≤ 10
Crushing value (%)	≤ 30	
Soundness (mass loss %)	≤ 18	
Flakiness index (%)	≤ 15	
Clay content (%)	≤ 4	
Sulfate content SO_3 (%)	≤ 1.0	
Chlorides content (%)	≤ 0.25	
Organic material (%)	≤ 0.5	
Impurity content (%) (metal, glass, plastics, asphalt, wood)	≤ 1	

Table 1.19 Particles made of concrete, mortar, stone, tile and brick from construction waste, with size larger than 4.75 mm as per GB/T 25177-2010 (Bodet 2014)

Items	Level-1	Level-2	Level-3
Content of fine powder (mass%)	<1.0	<2.0	<3.0
Content of silt lump (mass%)	<0.5	<0.7	<1.0
Water absorption (mass%)	<3.0	<5.0	<8.0
Elongated and flaky particle (mass%)	<10	<10	<10
Hazardous content: organic	Conforming	Conforming	Conforming
Hazardous content: sulfate mass %	<2.0	<2.0	<2.0
Hazardous content: chloride mass %	<0.06	<0.06	<0.06
Impurities content mass %	<1.0	<1.0	<1.0
Soundness: mass loss %	<5.0	<10.0	<15.0
Crushing index %	<12	<20	<30
Apparent density (kg/m3)	>2450	>2350	>2250
Void %	<47	<50	<53

Level 1: any concrete
Level 2: concrete below C40 (including C40)
Level 3: concrete below C25 (including C25). Not suitable for anti-freezing concrete
Note
i Recycled aggregates cannot be used in precast concrete
ii Recycled aggregates not meeting the above standard can be used in non-structural concrete component

(i) Recycled coarse aggregate for concrete (GB/T 25177-2010);
(ii) Recycled fine aggregate from concrete and mortar (GB/T 25176-2010);
(iii) Technical specification for application of recycled aggregate (JGJ/T 240-2011).

Table 1.20 Particles made of concrete, mortar, stone, tile, and brick from construction waste, smaller than 4.75 mm size as per GB/T 25176-2010 (Bodet 2014)

Items	Level-1	Level-2	Level-3
Content of fine powder (mass% @ MB < 1.40)	<5.0	<7.0	<10.0
Content of fine powder (mass% @ MB ≥ 1.40)	<1.0	<3.0	<5.0
Content of clay lump (mass%)	<1.0	<2.0	<3.0
Hazardous content: mica (mass%)	<2.0	<2.0	<2.0
Hazardous content: organic	Conforming	Conforming	Conforming
Hazardous content: sulfate mass (%)	<2.0	<2.0	<2.0
Hazardous content: chloride mass (%)	<0.06	<0.06	<0.06
Hazardous content: light materials mass (%)	<1.0	<1.0	<1.0
Soundness: mass loss % in saturated Na_2SO_4	<8.0	<10.0	<12.0
Single Grade Max. Crushing index (%)	<20	<25	<30
Apparent density (kg/m^3)	>2450	>2350	>2250
Bulk density (kg/m^3)	>1350	>1300	>1200
Void (%)	<46	<48	<52

Level 1: Concrete below C40 (including C40)
Level 2: concrete below C25 (including C25)
Level 3: Non-structural component

Table 1.21 Recycled aggregates for block and brick as per JGJ/T 240-2011 (Bodet 2014)

Item	Limit
(a) *Coarse aggregates*	
Content of fine powder (mass%)	<5.0
Water absorption (mass%)	<10.0
Impurities Content (mass%)	<2.0
Silt lump, hazardous, soundness, crushing index, alkali reaction index	Conforming to Table 1.13
(b) *Fine aggregates*	
Content of fine powder (mass% @ MB < 1.40)	<12.0
Content of fine powder (mass% @ MB > 1.40)	<6.0
Silt lump, hazardous, soundness, crushing index, alkali reaction index	Conforming to Table 1.14

1.6.7 Spain

In Spain, the recommendations on the use of concrete made with recycled aggregate obtained from waste concrete crushing was established in structural concrete instruction EHE-08 (Martinez et al. 2010). The recycled aggregate concrete can be

Table 1.22 Requirement for coarse aggregate for structural concrete in EHE-08 (Martinez et al. 2010)

Property	Test standard	Mixed aggregate	Natural aggregate	Recycled aggregate
Type of recycled aggregate	prEN 933-11	–	–	Concrete recycled aggregates
Ceramic content (%)	prEN 933–11			≤ 5
Asphalt content (%)	prEN 933–11			≤ 1
Contents of other materials (%) (glass, plastics, metals, etc.)	prEN 933–11			≤ 1
Fine content (%) (< 0.063 mm)	UNE–EN 933–1	≤ 1.5	≤ 1.5	≤ 1.5
Flakiness index	UNE–EN 933–3	<35	<35	<40
Absorption (%)	UNE–EN 1097-6	≤ 5	≤ 4.5	≤ 7
Los Angeles coefficient	UNE–EN 1097-2	≤ 40	≤ 40	≤ 40
Cl- water soluble (%)	UNE–EN 1744–1	≤ 0.05	≤ 0.05	≤ 0.05
Acid soluble sulfates SO_3 = (%)	UNE–EN 1744–1	≤ 0.8	≤ 0.8	≤ 0.8
Total sulfur compounds SO_3 = (%)		≤ 1	≤ 1	≤ 1
Light particles (%)	UNE–EN 1744–1	≤ 1	≤ 1	≤ 1
Clay particles (%)	UNE 7133	≤ 0.25	≤ 0.15	≤ 0.60
Weight loss in magnesium sulfate (%)	UNE–EN 1367-2	≤ 18	≤ 18	≤ 18
Lower declassified (%)	UNE–EN 933–1	≤ 10	≤ 10	≤ 10
Content of particles < 4 mm (%)	UNE–EN 933–1			≤ 5

used in structural concrete and mass concrete up to strength of 40 MPa. The maximum substitution of recycled coarse aggregate was limited to 20% of the total coarse aggregate in structural concrete applications. The requirement of recycled aggregate for structural concrete as per Annex 15 of EHE-08 is presented in Table 1.22.

Annex 18 of EHE-08 included the specifications of RA for non-structural applications which admit 100% recycled coarse aggregate provided that it satisfy the specifications mentioned in Table 1.23. If recycled coarse aggregate does not satisfy the limits specified in Table 1.22, it can be mixed with natural aggregate in order to report these limitations.

1.6.8 RILEM Specifications

The International Union of Testing and Research Laboratories for Materials and Structures (RILEM) has been actively involved in preparing the specifications for the use of recycled aggregates and its use in concrete. In 1994, RILEM had published the specifications for the use of recycled aggregates in concrete. In these specifications, the classification of recycled coarse aggregates and applications in the field of concrete containing these recycled aggregate classes in terms of acceptable environmental exposure classes and strength limits are also specified. The recycled aggregates were classified into three types such as Type I, Type II and Type III and are presented in Table 1.24.

The maximum water absorption limits for Type I, Type II and Type III were 20, 10, and 3% respectively. The requirements for the use of recycled aggregates in concrete are given in Table 1.25.

The limits for maximum strength of concrete with different types of recycled aggregates are presented in Table 1.26.

The above-specified recycled aggregates can be used in plain and reinforced concrete provided they satisfy all other durability requirements specified in RILEM and CEN Codes. The recycled fine aggregates are not allowed.

1.6.9 India

The Indian construction industry is one of the major sources for employment and the successive five-year plans of India accounts around 50% of the capital outlay from the construction sector. The projected outlay of the construction in the industrial sector continues to show a rising trend. At the same time, it contributes a large quantity of solid waste which include concrete, gravel, sand, bricks, metal, plastic, etc. A key concern for the town planners is to manage the construction and demolition waste, mainly due to swelling in quantity of demolition rubble, persistent dearth of dumping sites, intensification of transportation and disposal cost and above all budding alarm about pollution and environmental deterioration.

The Central Pollution Control Board (CPCB) has reported that out of 48 million tonnes/annum of solid waste generation, one-fourth of the waste was from construction sector only. This sets a huge burden on solid waste management. The demolition of buildings contributes a major portion to this waste generation. Asnani

Table 1.23 Specifications for aggregate used for base course, subbase course and subgrade in EHE-08 (Martinez et al. 2010)

Property	Standard	Carriageway Traffic T2	Carriageway Traffic T3	Carriageway Traffic T4	Shoulders Traffic T2	Shoulders Traffic T31	Shoulders Traffic T32 and T4
Type of recycled aggregate	prEN 933-11	Concrete and mixed	Concrete and mixed	Concrete and mixed	Concrete and mixed	Concrete and mixed	Concrete and mixed
Asphalt content	prEN 933-11	≤ 5%	≤ 5%	≤ 5%	≤ 5%	≤ 5%	≤ 5%
Content of light particles and other elements: wood, glass, gypsum, etc.	prEN 933-11	≤ 2%	≤ 2%	≤ 2%	≤ 2%	≤ 2%	≤ 2%
Sand equivalent	UNE-EN 933-8	>35	>35	>35	>35	>30	>30
Plasticity index	UNE 103103 & 103104	Non-plastic material	Non-plastic material	LL < 25 IP < 6	Non-plastic material	Non-plastic material	LL < 30 IP < 10
Los Angeles coefficient	UNE-EN 1097-2	<35	<35	<35	<35	<35	<35
Flakiness index	UNE-EN 933-3	<35	<35	<35	<35	<35	<35
Crushed particles	UNE-EN 933-5	≥ 75%	≥ 50%	≥ 50%	≥ 50%	≥ 50%	≥ 50%
Quotient of material passing through sieve 0.063 mm and material passing sieve 0.25 mm	UNE-EN 933-2	<2/3	<2/3	<2/3	<2/3	<2/3	<2/3
Organic matter content	UNE 103204	<1%	<1%	<1%	<1%	<1%	<1%
Settling in collapse test	NLT 254	<1%	<1%	<1%	<1%	<1%	<1%
Free swelling	UNE 103601	<3%	<3%	<3%	<3%	<3%	<3%

Table 1.24 Classification of recycled aggregates

Type	Origin
Type I	Implicitly understood to originate primarily from masonry rubble
Type II	Implicitly understood to originate primarily from concrete rubble
Type III	Implicitly understood to consist of a blend of recycled aggregates (maximum 20%) and natural aggregates (mandatory minimum 80%). The maximum content of Type I aggregate is 10%

Table 1.25 Mandatory requirements for recycled aggregates

Requirements	Type I	Type II	Type III	Test method
Maximum dry particle density (kg/m^3)	1500	2000	2400	Pr EN 1097-6
Maximum weight % with SSD < 2200 kg/m^3	–	10	10	Pr EN 1744-1
Maximum weight % with SSD < 1800 kg/m^3	10	1	1	
Maximum weight % with SSD < 1000 kg/m^3	1	0.5	0.5	
Maximum weight % of foreign materials such as metals, glass, soft material, tar, crushed asphalt.	5	1	1	Test by visual separation as in pr EN 933-7
Maximum content of metals (% m/m)	1	1	1	Visual
Maximum content of organic material (% m/m)	1	0.5	0.5	NEN 5933
Maximum content of filler less than 0.063 mm (% m/m)	3	2	2	Pr EN 933-1
Maximum content of sand less than 4 mm (% m/m)	5	5	5	Pr EN 933-1
Maximum content of sulfate (% m/m)	1	1	1	BS 812: part 118

Table 1.26 Maximum allowable strength for concrete with recycled aggregates

	Type I	Type II	Type III
Grade of concrete	C16/20[a]	C 50/60	No limit

[a]The strength may be increased to C 30/37 subjected to the condition that the SSD density of the RCA exceeds 2000 kg/m^3

(1996) reported that around 300 kg/m^2 from semi-pucca buildings and 500 kg/m^2 from pucca buildings waste was generated. The total amount of waste from construction sector was estimated to be 12–14.7 million tonnes per annum. The amount of different types of wastes that were arising from the construction industry was estimated and is presented in Table 1.27. Predictions for building material requirement of the housing sector show a dearth of aggregates to the extent of about 55,000 million cubic meters. For achieving the target for road development up to

Table 1.27 Waste constituents in million tonnes in India (Asnani 1996)

Constituent	Quantity generated in million tonnes per annum
Soil, sand, and gravel	4.20–5.14
Bricks and Masonry	3.60–4.40
Concrete	2.40–3.67
Metals	0.60–0.73
Bitumen	0.25–0.30
Wood	0.25–0.30
Others	0.10–0.15

2010, an estimated 750 million cubic meters of coarse aggregate as sub-base material shall be required. Production of recycled aggregate by recycling the construction and demolition waste may shrink the demand–supply gap in both these sectors. The waste from concrete and masonry is more than 50% of the total construction and demolition waste and is not currently recycled in India.

In view of the importance of recycling in the construction industry, Technology, Information, Forecasting and Assessment Council (TIFAC) appointed a techno-market survey on utilization of waste from construction industry (www.tifac.com). The main objective of this study was to gauge the knowledge of Indian construction industry on the probability of recycling of construction and demolition waste. According to the outcomes of survey, the main reason for not implementing recycling of construction and demolition waste was "the unawareness of the recycling techniques." While 70% of the respondents have cited the lack of awareness as one of the reasons, 30% of the respondents have indicated that they were not even aware of recycling possibilities. The response of industries indicates that presently, the existing specifications do not allow the use of recycled product in the construction activity. Sixty-seven percent of the respondents from user industry have quoted unavailability of recycled product as one of the reasons for not adopting it. It is a very good move in India that recently the Municipal Corporation of Delhi (MCD) has established the construction and demolition waste processing plant at Burari in North Delhi, by way of which the processed waste would be used in roadworks.

Existing Regulations on the use of Fine Recycled Aggregate (FRA) Past research shows that the concrete production with fine recycled aggregate (FRA) is very limited. The specifications regulated by different National Standards are presented in Table 1.28. Nevertheless, the recent researchers have reported that it is feasible to use the fine recycled aggregate in structural concrete and preserve its characteristics within the tolerable limits (Evange lista and de Brito 2014).

Table 1.28 General summary of the use of FRA in concrete in National Specifications (Evangelista and de Brito 2014)

Country	FRA type	Maximum % of FRA	$f_{c,max}$ (MPa) FRAC	Notes
Germany		0		Only CRA
Belgium	FRCA	100	37	With similar characteristics to FNA
Brazil	FRCA/ CDW FRA	100	15	Non-structural
China		0		Only CRA
Denmark	FRCA/ CDW FRA	20	40/20	Non-aggressive environments
Spain		0		Only CRA
USA	FRCA	100	No limit	Any type of concrete
Holland	FRCA/ CDW FRA	100	50/25	Only if used with CAN
Hong Kong		0		Only CRA
Japan	CDW FRA	100	18	Less demanding foundations
Portugal		0		Only CRA
UK		0		Only CRA
Russia	CDW FRA	50/100	15/20	Non-prestressed concrete
Switzerland	FRCA/ CDW FRA	20/100	37/-	Limited for prestressed concrete/plain concrete

1.7 Summary

The importance of recycling of construction and demolition waste from the viewpoint of solid waste management, preserving the depletion of natural resources from the environmental pollution and also from the sustainable construction is discussed. Terminology on recycled aggregate and recycled aggregate concrete (RAC) proposed by the Building Contractors Society of Japan (BCSJ 1978) is also described. Further, the current scenario of recycling of construction and demolition waste in various countries is discussed. The specifications laid down by different countries and RILEM on recycled aggregates are also highlighted. Based on these discussions, the following observations are highlighted.

- Though the “3R” formula, i.e., reduce, reuse, recycle, is one of the best policies to achieve the sustainable construction, due to partial implementation of this technique in most of the countries still lots of quantities of construction and demolition waste is lying in the site and deposited on landfills.
- The recycled aggregates could be used as structural grade concrete for many concrete applications provided if the demolished waste screened, separated various impurities and graded properly.

- Recycled aggregates have potential benefits such as economic, reduce environmental impacts and saving resources. But at the same time, there are certain constraints during the implementation in terms of both technological and management. Hence, researchers have to pay a serious attention toward the minimizing these problems so that the advantages of recycled aggregate could be availed in the industry.
- Few countries like Japan, Germany, China, Hong Kong, Spain, UK, have been established the specifications, guidelines on the use of RA in various applications, but still most of the countries are yet to make their move in this direction.

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Chapter 2
Demolition Techniques and Production of Recycled Aggregate

2.1 Introduction

At the time of construction in the past, no one was thought of re-use of building materials after completion of their service life. Even present-day, profoundly not investigated the demolition of structures/buildings as one favourable scheme at the end of their service life. While buildings from the second half of the previous century reach end of their service life, and rebuilding the structures at the same places becomes always more important and further, demolition techniques become progressively significant. Any uncontrolled method of demolition is no longer suitable due to the situations of high traffic rates and compact buildings (Kamrath and Hechler 2011).

Another problem concern was the waste management. Rise in urban population due to industrial development and new zoning bye-laws, legitimization of squatter settlements have led to the demolition of structures in the greater cities. Insufficient capacity of old road bridges for present and future growing traffic and rejuvenation of highway bridges desires the demolition of old bridges too. Further, many structures are destroyed due to either natural disasters like earthquakes, cyclones etc. or man-made disasters. Hence, today the whole world is facing the problem of handling the waste material generated from the demolition of structures. Especially in Europe where the areas of largely populated, the demolition of structures generated the recycled concrete or masonry is almost twofold as much as the necessity for recycled material, for example as a substitute to natural aggregate (Kamrath 2013). Normally the availability of natural resources mainly decides the requirements of recycled aggregates from the crushed concrete or masonry. Thus, if the availability of natural resources (aggregates) is less abundant locally, inclination towards the use of recycled products is higher and recycling becomes more imperative as an alternative resource (Kamrath 2013).

Demolition may be defined as the dismantling, destroying, ruining or wrecking any building or structure or any part thereof by pre-planned and precise way.

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Demolition may be a complete or partial dismantling of a building or structure. The method of demolition may depend on many factors such as the area and its location, type of building/structure, its condition, purpose of demolition and the possible ways of disposal, construction materials present, building height, building base plant area, surrounding available area, weather conditions and C&DW management (Fueyo 2003; AEDED 2008). The selection of demolition method is a function of cost and the equipment availability with the demolition contractor (da Costa 2009). Duration of demolition is also one of the factors which affect the cost of demolition. Normally time saving methods are more costly than the slower ones. Further, if dust, sound and vibrations are to be controlled during demolition the cost of demolition increases. Demolition is one of the most precarious activities in the construction sector. The demolition of a building or structure can be partial or full. In general, the full demolition is meant for the recovery of the area for subsequent re-use, while partial demolition is targeted at the retrieval of the building for refurbishing or rebuilding. Patel (2011) stated that the total demolition generally related to the buildings for which their service life is completed. It is normally performed with machineries furnished with demolition hammers and pneumatic cutters or more simply with normal excavators.

2.2 Methods of Demolition

A careful study shall be made about the building or structure which is to be demolished and all of its surrounding environments before the actual execution of demolition. This includes particularly the study of the way in which the demolition of different parts of the structure are supported and how far the safety of the contiguous structure affected by the stage by stage demolition. The manner in which the loads of various structural parts are supported decides the appropriate plan of the procedure of the demolition work. The concerned engineer in-charge shall prepare and approve the plan of demolition and it shall be followed strictly as nearly as possible during the execution of the actual demolition project. The demolition work shall be carried out in such a way that (i) it produces the least damage and annoyance to the surrounding structures and the public and (ii) it satisfies all the safety requirements to elude any kind of accidents (IS 4130). The steps involved in demolition process mainly (i) surveying (ii) removal of hazardous material (iii) preparation of plan and structural stability and (iv) safety measures. Two types of survey shall be carried out: Building survey and structural survey. In building survey the following information should be collected.

- Types of construction material used
- Usage of building before and present during demolition
- The existence of wastewater, matters arising from toxic chemicals, hazardous materials flammable or explosive and radioactive materials, etc.

- Drainage conditions and possible problems on water pollution, flooding and erosion
- Common facilities with adjoining building, including staircases, partition walls
- Pedestrian and vehicular traffic conditions adjacent to the demolished building/ structure
- The sensitivity of neighborhood with respect to noise, dust, vibration and traffic impact.

The structural survey mainly involves the structural materials used, method of construction, structural conditions of the adjoining structures, the structural system and structural conditions of basements, underground vaults and tanks, etc., and stability of the building. If any hazardous materials found during the investigation of site for demolition, it should be removed prior to the demolition of building/ structure. Based on the survey an appropriate demolition plan shall be prepared which covers the location of the structure/building to be demolished, a topography of the site and its surroundings, distance between the demolished building and adjacent structures, streets, etc., structural supporting system of the building, method of demolition to be adopted, plan of safety measures, process of handling of debris and time required for the completion of demolition process. A stability report shall also be supplemented with the demolition plan (Building Department 2004).

The choice of demolition method depends on the project conditions, site constraints and sensitivity of the neighborhood and availability of the equipment (Building Department 2004). At present there are various methods of demolition available like

(i) Non-engineering demolition
   (a) Manual demolition
(ii) Engineering demolition
   (a) Mechanical method
   (b) Implosion or Explosion
   (c) Deconstruction method
(iii) Top-down demolition

2.2.1 Non-engineering Demolition

This is a manual demolition technique. Normally this is carried out by contractors using manual tools which are portable like Sledge hammer, Jack hammers and drillers (Fig. 1.1).

Fig. 2.1 Manual tools (**a**) Jack hammer (**b**) Drill and (**c**) Sledge hammer (Adopted from Prakash 2014)

2.2.2 Engineering Demolition Techniques

2.2.2.1 Mechanical Methods

Mechanical methods are classified into

(i) Wrecking ball method
(ii) Pusher arm technique
(iii) Thermal process
(iv) Non-explosive demolition
(v) Abrasive process
(vi) Deliberate collapse method
(vii) Pressure jetting

Wrecking Ball Method Most of the structures can be demolished by using this technique, but it requires skilled practice that cannot be self-taught (NZDAA 2013). This technique is a viable and most effective for the demolition of multistory structures that have suffered from the structural damage and a hazard assessment determined in the structure. In this technique, a steel ball (weight approximately 0.5–1.0 tonne) anti spin device is suspended by a steel rope and swung by a drag rope from a crane of adequate capacity. The structure is demolished by hitting with

Fig. 2.2 Demolition by wrecking ball technique (Adopted from Prakash 2014)

a steel ball (Fig. 2.2). This method is much faster than manual method. Converted drag lines are the best machines as they are robust and stable. Cranes with hydraulic rams must not be used for balling (NZDAA 2013).

Pusher Arm Technique A hydraulically powered pushed arm machine is mounted on tracked or wheeled chassis. These machines are extremely mobile, having high output and are able to work on vertical faces and floors above standing level. The main disadvantage is that it requires adequate access, a relatively flat and firm base to work from and can only work within the reach of their booms (NZDAA 2013). For effective and efficient operation, the length of the boom when fully extended should be at least 1.5 m above the height of the building being demolished. This technique is best suitable for masonry infill structures, but not suitable for confined sites or large buildings. The resistance of a brick wall against the power of an excavator is low, so it is possible to push a wall inside the building with the help of the backacter or to put the backacter on top of a wall and to pull the wall outwards (Fig. 2.3; Kamrath 2013).

Thermal Process For practical demolition purposes generally a very high temperatures are not required (general fusion temperature of steel is 1600 °C), the applications of these techniques are somewhat specialized. Therefore these techniques are likely to be applied in cases of special structures such as nuclear power plants and at job locations which are difficult to access (Coelho and De Brito 2013a). Thermal demolition process can be classified into three categories (Manning 1991).

(i) Drilling and thermal cutting using torches, laser or plasma (Fig. 2.4)
(ii) Removal of concrete through heating of steel reinforcement
(iii) Surface concrete removal by direct heat application.

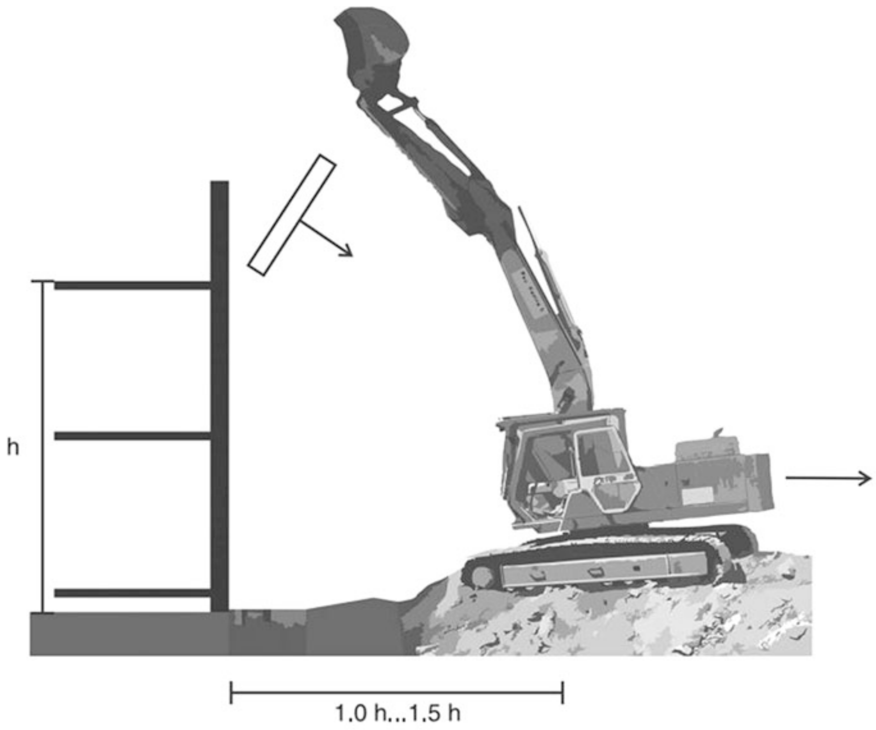


Fig. 2.3 The excavator pulls on a wall (Kamrath 2013)

Fig. 2.4 Cutting of reinforcement through high temperature (Prakash 2014)

Non-explosive Demolition A series of holes are drilled along the desired line of separation on the concrete surface and filled with slurry of special nature. After filling with slurry, the water is poured into the desired holes and after few hours the slurry expands which would results cracks in the concrete structures which will help to demolish the structure easily (Prakash 2014).

Abrasive Process This process is normally done with very hard materials such as diamond, carborundum or with high pressure water. This technique generally limited to partial demolition, as this technique could be highly expensive for complete demolition of a building/structure (Coelho and de Brito 2013a). Diamond discs can cut through concrete and steel (RCC or Pre-stressed) with a 40 m^2 area cutting surface yield. Although wireless diamond disc cutting machines available in the market, water refrigerated cutting discs are usually adopted (Da Costa 2009). The diamond string cutting is more efficient than discs cutting and the diamond strings are mostly used with granite, marble and concrete. Coelho and De Brito (2013a) stated that according to Fueyo (2003), using diamond strings the cutting produces 3 and 5 m^2/h, while the string velocity can be as high as 40 m/s.

Deliberate Collapse Method The Collapse is commonly achieved either by removing key structural elements (e.g. with explosive charges) or by wire rope pulling at a high level of overturn. It needs engineering expertise to decide which structural element should be removed to cause a collapse. This method is best suitable for silos, bridges, chimneys and isolated structures or heavily controlled and secure sites. A survey may be conducted to know the height and radius of the structure so that the fall area can be find out and proper protection can be made to prevent involuntary entry during collapse (NZDAA 2013).

Pressure Jetting Generally the jet heads are small and therefore action of jet is mainly to loosen the aggregates by washing out softer mortar (water with high pressure) (Fig. 2.5). Demolition with highly pressurized water (Hilmersson 1999) is a high yielding technique. It does not damage the overall structure, does not produce dust, vapour or slag, has no induced vibrations, has small reaction forces, and has a vast application range. However, it cannot generally be used to cut through reinforcement and cracks can slow down the progress.

Fig. 2.5 Demolition by water jet (Prakash 2014)

2.2.2.2 Demolition by Implosion or Explosion

Implosion or explosion demolition is an effective and efficient method of demolition, and it can reduce both cost and time to bring dangerous multi-storey structures to ground in comparison to conventional demolition methods (NZDAA 2013). In most of the cases, the demolition by explosion can reduce the time by about 80% with the majority of the being spent on the period of preparation and cleaning process following implosion. Before adopting this procedure, the blue print of the building should be properly studied to identify the key elements at different levels from bottom to top for explosion. There are two main types of explosives for demolition i.e. commercial or military. The commercial explosives are mostly dynamite based with detonation speeds ranging from 3000 to 7000 m/s. whereas, military explosives have high detonation period ranging from 6000 to 9000 m/s (Coelho and De Brito 2013a).

2.2.2.3 Deconstruction

Deconstruction is a slow and careful process that is almost reverse process of construction. It involves the systematic and manual disassembly of the affected sections of a structure, saving as many of the components as possible for reuse or recycling. Generally this technique is applied when recycling and re-use of construction material is significant from environmental, economic and social reasons. Coelho and De Brito (2013b) mentioned that according to ITEC (1995), the main features of the deconstruction technique is as follows.

- Official information shall be made to all entities that may be affected by or have jurisdiction over the deconstruction activity
- Deconstruction area setup
- Disconnection of all services those are still active in the building
- Erection of scaffolding
- Bracing construction elements that may collapse if their internal state of stress changes significantly
- Preparation and execution of personnel protection measures
- Routing and separate storage space for recovered materials
- Workers individual safety measures.

In all the demolition techniques, the first three features are common. To avoid unexpected structural failure due to sudden changes in stress condition in structural elements, bracing system in deconstruction technique is usually adopted. Further, to remove façade elements or if certain elements are to be sent directly to the out of building, scaffolding is essential in this technique. Job site routing is essential to have maximum material recovery for both re-use and recycle.

2.2.3 Top-Down demolition

According to Building Code Hong Kong (2004) demolition by Top-down means the one starts demolition from roof to ground in a progressive way. Depending up on the site conditions and structural elements to be demolished, the particular sequences of demolition may vary. The demolition sequence is determined according to the building layout and constraints, as well as conditions of the site. Normally, the following sequence can be applied:

- First all overhanging elements such as verandahs, balconies, cantilever projections, emergency stairs, etc. should be demolished prior to the main building demolition. Roof installations like lifts, air conditioning units, etc. should be removed to avoid them fall down during the demolition process.
- Demolition of floor slabs should begin at mid-span, and progress towards the supporting beams.
- Floor beams are demolished in the following order

  (i) cantilevered beams;
  (ii) secondary beams; and
  (iii) main beams.

- Non-loading bearing beams are first removed. Subsequently, load-bearing beams are removed from the top down.
- As soon as possible, the ground floor should be wrecked to avoid demolition waste lying on it. Due to huge load, this floor could otherwise collapse.
- Columns and load bearing walls shall be removed after removal of beams on top.

General demolition of top-down of a building demolition is presented in Fig. 2.6 (Kamrath 2013). The summary of various methods of demolition is presented in Table 2.1.

2.3 Production Technology of Recycled Aggregates

A recycling plant is quite similar to the plant producing crushed natural aggregate. The recycling plant incorporates various types of crushers, screens, transfer equipment and devices for removal of foreign materials. A number of different processes are possible for the crushing and sieving of construction and demolition waste. A typical layout of a closed system which is usually recommended for the production of recycled aggregates is shown in Fig. 2.7 (Hansen 1985). An open system (Fig. 2.8) has an advantage of having larger capacity but maximum aggregate size is less well defined and this can lead to greater variations in the size of the end product, particularly when the input flow changes (Hansen 1985). Both these plants treated as first generation plants. These plants do not have facilities to remove the contaminants and are generally used for pavement rehabilitation and recycling projects.

Fig. 2.6 Typical demolition from top to down. After demolition of all non-bearing structures, the bearing columns are left. In the next step, the columns will be demolished beginning with the top level. After that the demolition starts again with the next field (Kamrath 2013)

A clean concrete waste is not available always, based on the first generation plants, certain provisions are made to remove the foreign materials in the second generation plants and a typical layout of it is presented in Fig. 2.9.

As per Simonds Group (1999), the construction and demolition waste (C&DW) recycling plants can be classified as Level 1, Level 2 and Level 3 (Fig. 2.10). Level 1 plant is suitable for the processing of inert C&DW; Level 2 plants have metal removal and more complex sorting and sieving provisions and is therefore suitable for mixed C&DW; and Level 3 plants have additional facilities like hand sorting, washing plant for other C&DW (wood) than Level 2 and is suitable for any C&DW (mixed and contaminated).

As stated above Level 3 plants have additional facilities and higher quality control and automation apart from the methods and technologies adopted in Type 1 and Type 2 plants, yields in higher purity and variety in recovered material fluxes (Coelho and De Brito 2013b). The installation of these plants may be mobile or stationary.

Table 2.1 Summary of the general characteristics of demolition methods (Building Code Hong Kong 2004)

Method	Principle	Applicability					General conditions	Remarks
		Column	Beam	Slab	Wall	Foundation		
Top down with manual jack hammer or pneumatic	Breaking away the concrete by hand held jack hammer or pneumatic hammer	Very effective					• On a floor by floor downward sequence • Need precautionary measures for restricted site	• Broad scope of application • Effective in narrow and localized place
Top down machine/ Percussive breaker	Breaking away structure by machine mounted percussive breaker	Very effective					• On a floor by floor downward sequence • Adequate floor support for machine • Need precautionary measures for restricted site	• Wide range of application • Good mobility
Top down/machine with hydraulic crusher	Breaking away structure by machine mounted hydraulic crusher	Very effective				Moderately to slightly effective	• On a floor by floor downward sequence • Adequate floor support for machine • Need precautionary measures for restricted site	• Wide range of application • Good mobility • Ability to separate steel bars and frames
Hydraulic crusher with long boom	Breaking away structure by machine mounted hydraulic crusher with long arm extension	Very effective				Moderately to slightly effective	• Restricted entry to work area • Flat and firm working ground • Adequate clear space	• Wide range of application • Good mobility • Ability to separate steel bars and frames
Wrecking ball	Destruction by impact of steel ball suspended from a crane	Very effective	Moderately to slightly effective		Very effective	Not efficient	• Restricted entry to work area • Flat and firm working ground	• Good efficiency • Poor application for underground columns and foundations

(continued)

Table 2.1 (continued)

Method	Principle	Applicability					General conditions	Remarks
		Column	Beam	Slab	Wall	Foundation		
							• Adequate clear space	
Implosion	Use of explosives	Very effective		Not efficient		Very effective	• Protection from noise, debris and vibration • Qualified blaster • Notification and evacuation of neighbourhood • Check and cautiously handle of miss fire	• Excellent demolition strength • Could shorten the work period and reduce labour • Risk assessment required to continued
Wire saw cutting	Cutting with wire saw	Very effective				Not efficient	• Solid working platform • Arrangement for hoisting out cut section • Counter measure to prevent danger of wire breaks	• Allows precise separation • Good for cutting massive structures
Drilling	Coring, drilling and cutting by stitch drilling	Moderately to slightly effective		Very effective			• Solid working platform	• Allows precise separation • Good for cutting massive structures
Thermal lance	Use of intense heat by fusion of metal	Moderately to slightly effective				Not efficient	• Protection of person and properties from intensive heat	–
Water jet	Jetting of water at high pressure	Moderately to slightly effective				Not efficient	• Protection of person and properties from high pressure water	–

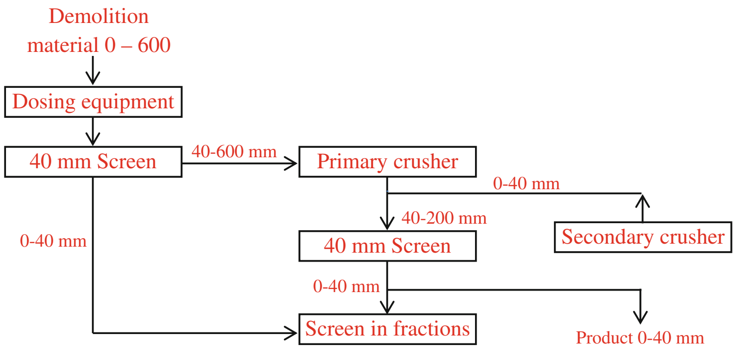


Fig. 2.7 Flow chart of typical plant for production of RA from concrete debris which is free from foreign matter, closed system (Hansen 1985)

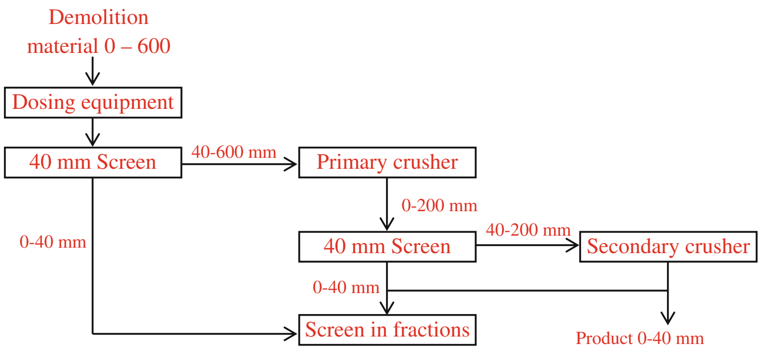


Fig. 2.8 Flow chart of typical plant for production of RA from concrete debris which is free from foreign matter, open system (Hansen 1985)

2.3.1 Mobile Plants

Mobile C&DW recycling plants have become more popular due to the need for conventionally fixed equipment, such as feeders, crushers, magnetic separators and vibrating screens used at different times and locations. Sometimes technically or economically even better to place a simplified version of mobile plant at site, instead of transporting the C&DW to a fixed plant located far away from the site (Coelho and De Brito 2013b). The technology adopted in mobile plants is same as in fixed plants, through limited to feeders, crushers, magnetic separators and vibrating screens. Mobile plants are mounted on tracks but tire mounted plants are also available commercially. The main advantages and disadvantages of mobile plants are as follows (Lindsell and Mulheron 1985). A typical mobile plant fitted with jaw crusher is shown in Fig. 2.11 (Kumbhar et al. 2013).

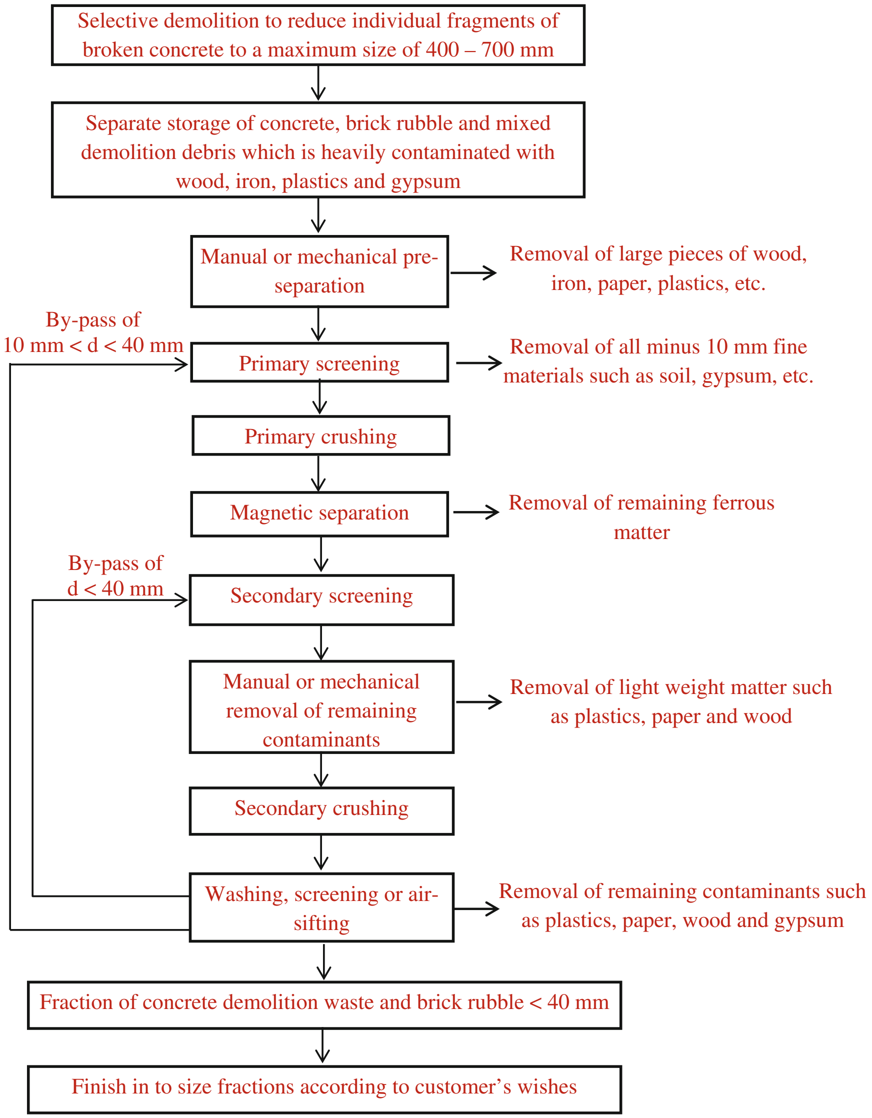


Fig. 2.9 Processing procedure for building and demolition waste (Hansen 1985)

Advantages

- Transport in the vicinity of the site is reduced, particularly if the rubble is produced, recycled and reused on the same site
- Disposal costs are reduced due to less dumping
- The local supply of aggregates are increased and therefore less quantity of aggregates required to be imported into the site area

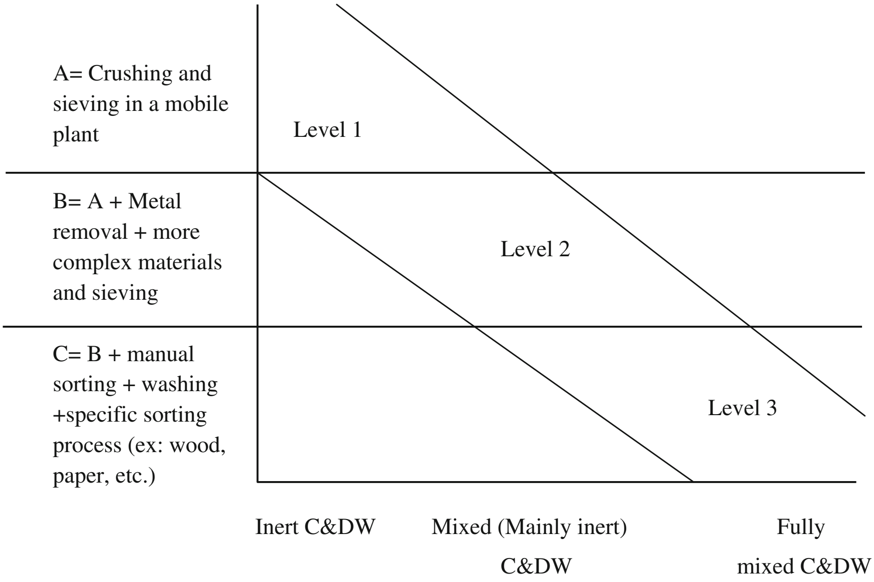
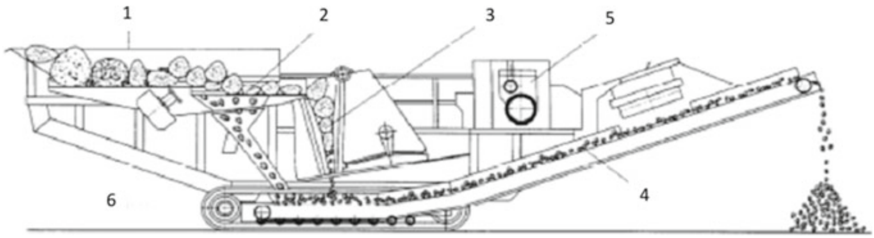


Fig. 2.10 Classification of C&DW recycling plants (Symonds 1999)



1.Feeding hopper; 2.Oscillating conveyor; 3.Jaw crusher; 4.Discharging transport belt; 5. Diesel engine as a power belt; 6. Mobile by wheels, Crawlers or skids

Fig. 2.11 Mobile crushing plant (Adopted from Kumbhar et al. 2013)

- The recycling plant can be relatively easily movable to the another site
- Economical for C&DW of 5000–6000 tonne per site (Kumbhar et al. 2013).

Disadvantages

- The recycling plants can cause high levels of noise and dust which would not be acceptable to the surrounding residential areas
- This type of plant can be used only, if there is a sufficient quantity of rubble on the site to justify the expense of setting up the recycling plant.

2.3.2 *Stationary/Fixed Plants*

Stationary recycling plant normally consists of a large primary crusher working in combination with a secondary or tertiary crusher. It also attached with various cleaning and sieving devices, to generate high quality recycled aggregate. Furthermore, magnets, air sifters and/or float separators, jiggers, spirals, etc. can be included in the stationary plant depending upon the type of C&DW and quality of recycled aggregate desired. According to Silva et al. (2016) the advantages and disadvantages of stationary plants are as follows. A typical layout of a stationary recycling plant is shown in Fig. 2.12.

Advantages

- The recycling is capable of producing high quality RA
- Better efficiency than mobile recycling plant because different recycled products of various particle sizes can be produced
- Due to less disposal the cost of dumping is reduced
- High manufacturing capacity.

Disadvantages

- The initial investment may be high for setting up of such plants
- Since the plants are installed far away from the sites, the transportation cost of C&DW increased
- The production is mainly depends on the constant input supply.

The choice of the type of recycling plant is difficult and case to case basis is to be analysed by taking many factors like technical, financial and environmental issues

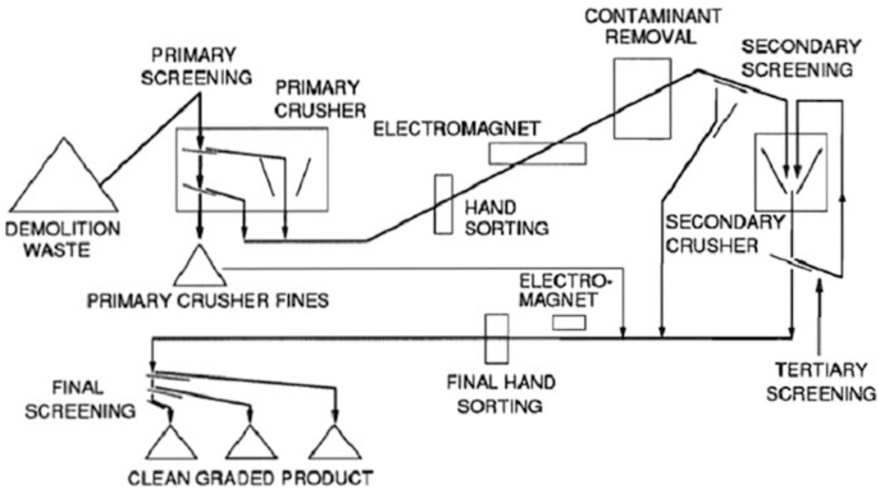


Fig. 2.12 Layout of a stationary plant (Adopted from Kumbhar et al. 2013)

(i.e. plant capacity, transportation cost, amount of C&DW, hauling distances, scale of economy, price of natural aggregates, etc.) into account (Zhao et al. 2010).

2.4 Process of Recycling Technology

A flow chart of possible combinations of recycling process that can produce a relatively of better quality of RA, with least contaminants, without spending too much energy presented in vide Fig. 2.9. In some cases like plain concrete blocks, manual or mechanical removal of contaminants can be bypassed, thus saving energy (Silva et al. 2016). After reaching of C&DW to the recycling plant, it may either directly be fed into the processing operation or required to be broken down using hydraulic breakers mounted on tracked or wheeled excavators to obtain required workable particle sizes. In both the cases to minimize the degree of contamination, separation of large pieces of iron, wood, plastic and paper by manually may be needed. Three types viz: Jaw, impact and gyratory crushers are mostly used for crushing of C&DW.

2.4.1 Jaw Crusher

A jaw crusher operates by allowing material to pass through the two jaws, one of which is in stationary and the other oscillates back and forth relative to it. The distance between the jaws reduces as the material travels downward under the effect of gravity and the motion of the movable jaw, until the material completely passes through the lower opening. The jaw crusher can endure large pieces of reinforced concrete, which would possibly cause other types of crushers to breakdown. Therefore, before going through the other types of crushers, the material is initially reduced in jaw crusher (Silva et al. 2016). The degree of size reduction of particle depends on the minimum and maximum size of the space at the plates. Molin et al. (2004) reported that the most suitable grain size distribution of RA for concrete production can be obtained by using jaw crushers.

2.4.2 Impact Crusher

An impact crusher breaks the construction and demolition waste by striking it with a high speed rotating impact, which imparts a shearing force on the rubble. An impact crusher consists of a heavy steel frame equipped with a rotor fastened with a series of hard steel blades. Material fall onto the rotor and are caught by the hard steel blades mounted on the rotor, which hurl them against the breaker plate, smashing them to smaller size particles. Normally the impact crusher has larger reduction factor and is defined as the ratio of input particle size to the output particle size (Lindsell and Mulheron 1985). Therefore, the impact crusher produces larger amount of fines. One advantage of the impact crusher is its high efficiency and less sensitive to the material which cannot be crushed i.e. reinforcement. Consequently impact crushers suffer high wear and tear and hence, high maintenance costs (O' Mahony 1990).

2.4.3 Gyratory Crushers

Gyratory crushers work on the same principle as the cone crushers, which are characterized by a gyrating mantle mounted within a deep bowl. Gyratory crushers provide continuous crushing action and are used for both primary and secondary crushing of hard, tough and abrasive materials. These crushers are relatively low energy consumption, reasonable amount of control over particle size and production of low amount of fine particles (Silva et al. 2016).

A typical jaw, impact and gyratory crushers are shown in Figs. 2.13, 2.14 and 2.15 respectively.

Silva et al. (2016) stated that to produce expectable grading curve, it is better at least to process material into two crushing stages. It may be possible to consider a tertiary crushing stage and further, which would produce better quality course recycled aggregate (i.e. less attached mortar and with a rounder shape) undoubtedly. However, concrete produced with RA subjected to a tertiary crushing stage may have somewhat better performance than that made with RA from a secondary crushing stage (Gokce et al. 2011; Nagataki et al. 2004). At the same time, more crushing stages would yield products with decreasing particle sizes, which opposes the mainstream use of coarser RA, which generally preferred. These factors should be taken into account from the view point of economic and environmental while

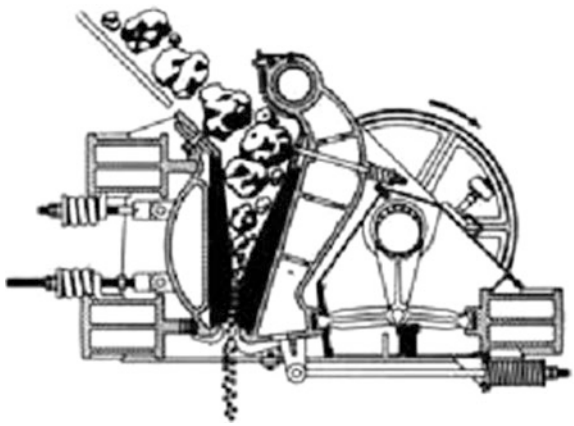


Fig. 2.13 Jaw crusher (Adopted from Silva et al. 2016)

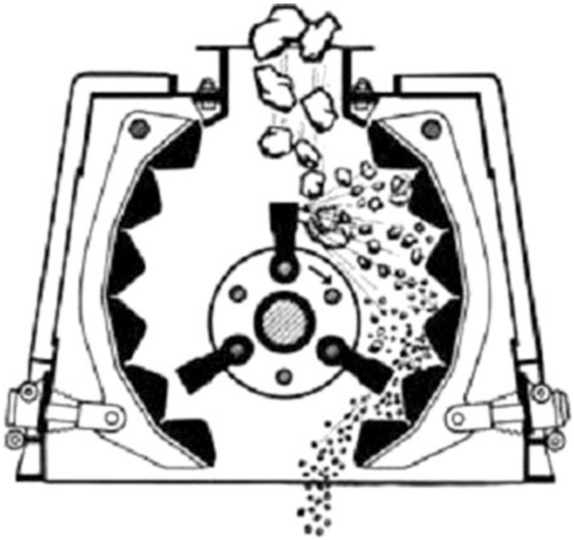


Fig. 2.14 Impact crusher (Adopted from Silva et al. 2016)

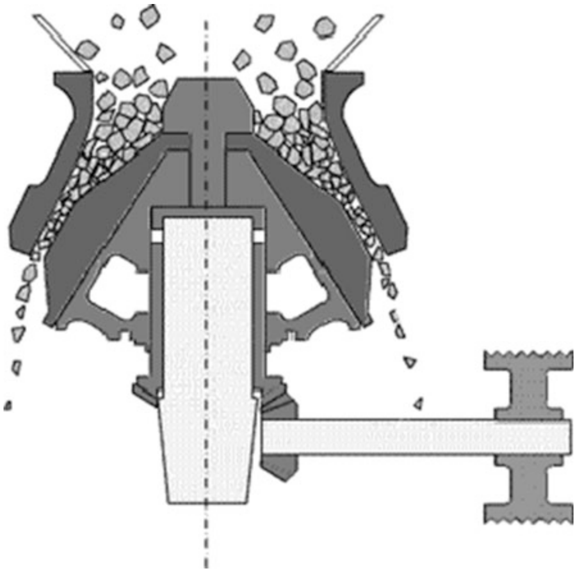


Fig. 2.15 Gyratory crusher (Adopted from Silva et al. 2016)

producing RA. That is relatively better quality of aggregates can be produced with lower energy consumption and with a higher proportion of coarse aggregates, if the number of crushing stages is sensibly reduced.

According to Mahony (1990), the contaminants from the C&DW can be removed in two stages i.e. (i) Pre-crushing separation and (ii) post crushing separation.

2.4.4 Pre-crushing Separation

In this technique, the debris can be sorted while the structure is being demolished. Even though this technique can be expensive and time consuming for the demolition contractor, later it gives large benefits in terms of financial and ecological. Maximum sorting normally takes place after reaching the C&DW to the recycling plant. Once the debris reaches, it is stockpiled according to their major constituents and presence of contaminants. Therefore, the plant operator can take the necessary steps for each case. This initial sorting process can help to optimize the crushing time and product quality e.g. When large amount of clean debris accumulated in a stockpiling, they can then be crushed in a single run, continuous. Since most of the times, primary screening is conducted before the debris arrive to the primary crusher and if the material of required size is already available, then primary crushing can be bypassed. Further, if these should be of concrete based and of low degree contamination, instead of disposing on landfill, it is possible to make use of the material finer than 10 mm in the primary screening stage itself.

On the other side, post crushing separation technique is adopted after number of crushing stages, where several techniques may be employed for removal of different contaminants. The easiest method is the hand sorting, in which the contaminants from the conveyor belt are removed by manually. The efficiency of the hand sorting system is mainly governed by the attention of the operators and the speed of the conveyor belt. Even though the human eye can recognize the contaminants which would be difficult to remove by mechanically (e.g. glass, asphalt), it is also the expensive approach (Silva et al. 2016).

After primary crushing stage, self-cleaning electromagnets normally employed at strategic locations in the conveyor belt to remove the pieces of steel reinforcement and other ferrous materials. The distance between the magnet and debris, the speed of the conveyor belt, the volume of the debris passing on conveyor belt and the angle of the magnet influence the efficiency of the magnet. It is more efficient when it is placed directly above and parallel to a slow moving conveyor belt with low concentration of material. The electromagnets may be in a fixed position above a conveyor belt or take the form of a rotating magnetic belt (Fig. 2.16) (Silva et al. 2016). The advantage of the magnetic belt is that it carries the materials to a side, instead of accumulating them on the magnet. Eddy current separator is a device which can be used to remove the non-ferrous metals like aluminium, copper, brass, etc. present in the C&DW. It works on the principle that the eddy currents are generated in the metal, when a conducting metal led through a varying magnetic field. By keeping this device at the end of the conveyor belt, metals are thrown off the belt and other materials simply fall off due to gravity. As the eddy current separator may get damaged due to ferromagnetic metals, at earlier stage itself these must be removed from the C&DW. By passing the crushed aggregates through a set of scalping screens, the dirt, gypsum, plaster and other fine impurities can be removed at a later stage. Using dry screening, the materials can be separated into different size fractions, which can later be recombined to obtain well graded

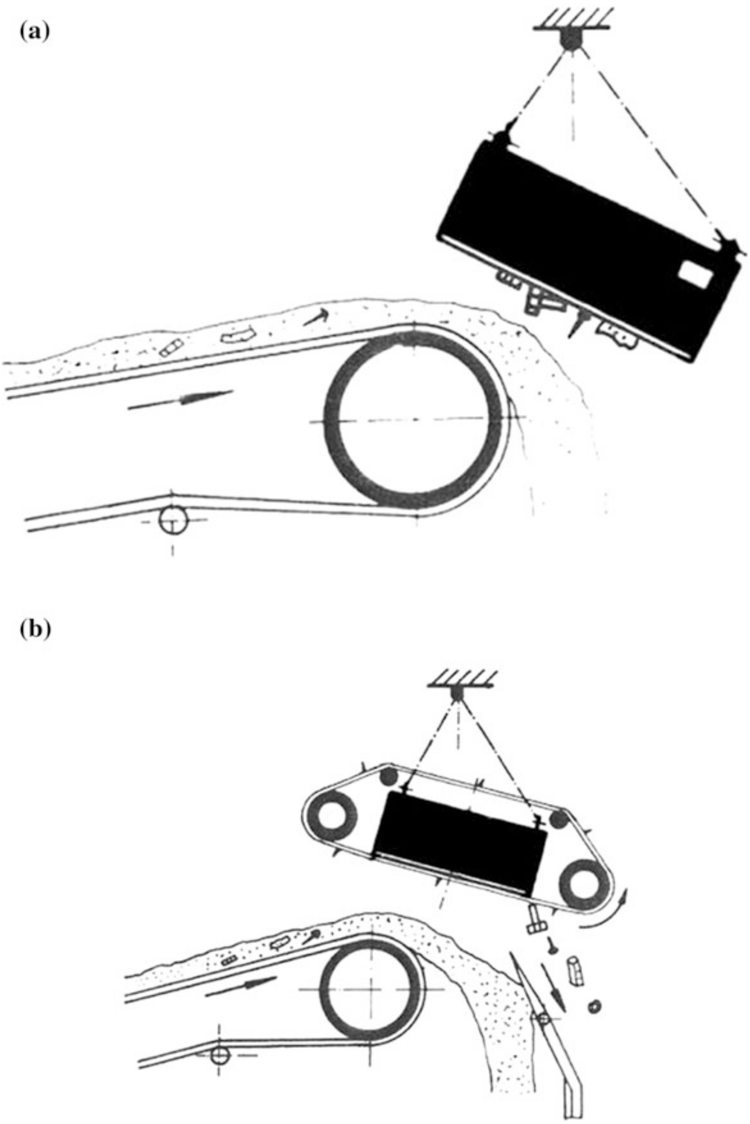


Fig. 2.16 Electromagnets (**a**) fixed and (**b**) rotated (Adopted from Silva et al. 2016)

recycled aggregates. According to Building Contractors Society of Japan (BCSJ) (1981), inclined screens vibrating at low frequencies and large amplitudes are capable to separate the coarse materials more efficiently, while the fine materials can be separated efficiently by using horizontal screens, vibrating at high frequencies and small amplitudes. This process only separates based on the size and shape. Finally the low density contaminants can be removed either by using air sifting or wet separation techniques. In wet separation technique, the material is placed in a tank with full of water and the water is rotated at a faster rate so that currents are setup by water jets. The lightweight impurities and wood which will float in water are removed by combs which move from one end of the tank to the other end. Normally this technique is limited to the materials of size more than 10 mm due to the excessive quantities of sludge which would produce if smaller size fractions are used in a tank. The other technique i.e. air sifting may also be equally effective as wet separation, in terms of removal of lightweight materials such as wood, hardboard, plastics, straw and asbestos fibres and would also avoid the use of large quantities of water, but the wet separation allows the leaching of chlorides and sulphates (Galvin et al. 2014; Weimann and Muller 2004). In unbound or bitumen bound applications the air sifting can be used instead of wet separation, as the chlorides and sulphates have little impact on these applications.

As a compliment to the aforesaid crushing procedures, there are some other methods which can remove the adhered cement mortar from the surface of the natural aggregates and are discussed in Chap. 8.

2.5 Summary

In this chapter various methods of demolition of construction and demolition waste and production technology of recycled aggregate is discussed. Based on the discussion the following observations are highlighted.

- The waste generated from the demolition of structures is one of major components of total waste; simply dumping on landfills severely affects the environment, social and economic life cycle.
- The demolition of the structure shall be carried out in such a way that (i) it causes the least damage and nuisance to the surrounding structures and the public and (ii) it satisfy all the safety requirements to avoid any kind of accidents.
- The choice of demolition method depends on the project conditions, site constraints, and sensitivity of the neighborhood and availability of the equipment. Among all the methods, deconstruction technology is the best suitable method when recycling and re-use of construction material is significant from environmental, economic and social reasons.
- Among the Level 1, 2 and 3 recycling plants, Level 3 recycling plant is more capable of producing the best quality of recycled aggregate from any mixed C&DW, which compliance with the present regulations for RA in production of new concrete.
- The choice of the type of recycling plant is difficult and case to case basis is to be analysed by taking many factors like technical, financial and environmental issues (i.e. plant capacity, transportation cost, amount of C&DW, hauling distances, scale of economy, price of natural aggregates, etc.) into account.

- Mobile plants are preferred when low quality output is acceptable for a given concrete applications and to avoid the transportation and quantity of C&DW is in the range of 5000–6000 tonnes per site. Stationary plants are suitable for recycling of large quantity of mixed C&DW with higher quality output materials.
- The contents of C&DW must be properly analysed before the delivery to the recycling facility so that it is possible to find the most appropriate recycling process to enhance the output's quality. Also, this will reduce processing time, produce higher quality RA, increase the work rate and help to avoid excessive costs incurred by unnecessary recycling stages.
- The number of crushing stages undoubtedly reduces the adhered mortar content and irregularity of the aggregates and thus produces better quality of recycled coarse aggregate. Though the tertiary crushing stage may enhance the quality of recycled aggregates slightly, but it decreases the coarse to fine aggregate ratio and increase the costs and energy. Therefore, by taking the economic and environmental aspects into account, a relatively better quality of aggregates can be produced with lower energy consumption and with a higher proportion of coarse aggregates, if the number of crushing stages is sensibly reduced.

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Chapter 3
Properties of Recycled Aggregates

3.1 Introduction

Concrete is an end product of intimate mixture of mainly three components namely cement, water and aggregates. Around 70–80% of the total volume of concrete is occupied by aggregates. Because of its large proportion in concrete, properties of aggregates are of considerable importance as they can affect the workability, strength, durability and structural performance of concrete. Aggregate was initially treated as inert and cheaper material in the concrete as it contributes to the large volume of concrete. However, the performance of concrete is influenced by the physical, chemical and thermal properties of aggregate. The inclusion of aggregate in concrete not only contributes to the economy, but also gives considerable advantage of higher volume stability, better durability and higher strength than hydrated cement paste alone (Neville and Brooks 2005). Aggregates are derived from rock, either by naturally or artificially crushed. Thus, the properties of the aggregate depend on the chemical and mineral composition, petrographic classification, specific gravity, hardness, strength, pore structure, etc., of the parent rock. In addition, the size, shape and surface texture of aggregates also influence considerably the fresh and hardened properties of concrete (Neville and Brooks 2005).

The basic difference between recycled aggregate and natural aggregate is lies in the presence of attached old mortar in the recycled aggregate (Fig. 3.1). This adhered old mortar changes the properties of recycled aggregate and consequently the properties of fresh and hardened concrete made with these recycled aggregate. In general, the density and water absorption of recycled aggregates are greatly influenced by the quantity of adhered old mortar.

M. C. Rao et al., *Systematic Approach of Characterisation and Behaviour of Recycled Aggregate Concrete*, Springer Transactions in Civil and Environmental Engineering,
https://doi.org/10.1007/978-981-10-6686-3_3

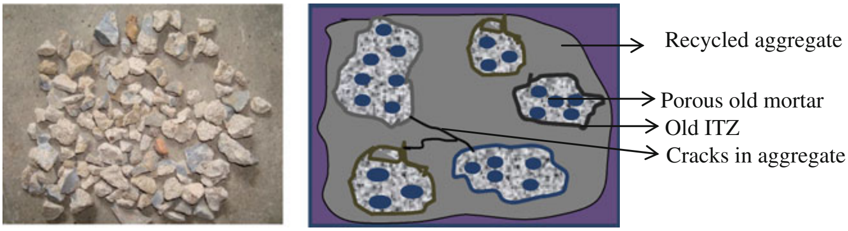


Fig. 3.1 Recycled aggregate and its pictorial representation

3.2 Physical Properties of Recycled Aggregates

3.2.1 Grading, Shape and Surface Texture

Buck (1973) reported that the recycled aggregate does not have a high amount of flaky and elongated particles. Frondistou-Yannas (1977) reported that good particle size distribution of recycled aggregate can be produced from uncontaminated concrete rubbles. Hasaba et al. (1981) ascertained that depending on the quality of the concrete from which recycled aggregate derived, the recycled coarse aggregate contains 1.3–1.7% particles finer than 88 µm size in maximum 25 mm size aggregates. Hansen and Narud (1983) reported that using jaw crusher with a single passing of 100 mm maximum size concrete rubble, well-graded recycled aggregate of good shape can be produced. Ravindrarajah (1987) observed that the recycled aggregate obtained by crushing the old concrete in a jaw crusher was more angular than natural aggregate. Bairagi et al. (1990) reported that the recycled coarse aggregates were angular with porous and rough texture and they were relatively coarser than natural aggregates due to the presence of old mortar in recycled aggregate. Prasad and Kumar (2007) had reported similar result. Buyle-Bodin (2002) and Hadjieva-Zaharieva (2002) observed that the recycled fine aggregate was relatively coarser than natural sand and its fineness modulus was little higher than acceptable limits for normal concrete sand. Zaharieva et al. (2003) reported that the recycled fine aggregate was occasionally coarser than natural fine aggregate. Salem et al. (2003) reported that well-graded recycled aggregate that abides by ASTM could be produced without much difficulty. In addition, it was reported that the recycled aggregates were more angular and the surface texture was porous and rough due to the existence of old mortar. Katz (2003) studied the influence of crushing age on the properties of aggregates and concrete and it was reported that using a jaw crusher set at a specific opening, similar grading and other properties of aggregates can be produced irrespective of the age of crushing. Zega et al. (2010) studied the influence of different types of coarse aggregates and w/c ratio on the properties of recycled aggregate. The authors ascertained that the grading of recycled aggregate does not depend on the w/c ratio of the concrete from which the recycled aggregate derived and the shape and texture of parent concrete aggregate.

Chakradhara Rao (2010) reported the particle size distribution of natural coarse aggregate and recycled coarse aggregates obtained from different sources through Figs. 3.2, 3.3 and 3.4 along with the minimum and maximum limits specified by BIS (IS: 383-1970) for natural aggregate used in concrete. Figure 3.2 shows the grading curves of natural coarse aggregate and recycled coarse aggregates obtained from Source 1 (RCC culvert Medinipur), and it was observed that the recycled coarse aggregates are relatively finer than natural coarse aggregates. Hence, the fineness modulus of recycled coarse aggregate (6.692–6.77) was less than that of natural coarse aggregate of 6.782. It was also observed that the particles of size less than 4.75 mm in a mix with maximum size of recycled coarse aggregate of 20 mm are approximately 2.3%. Similarly, the grading curves of both natural and recycled

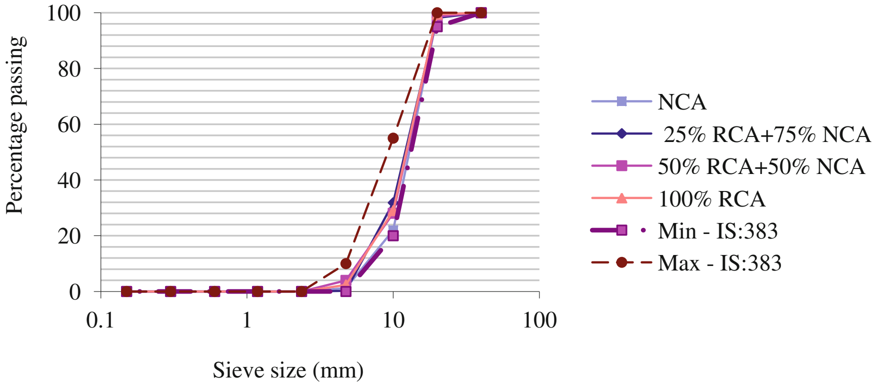


Fig. 3.2 Particle size distribution of natural coarse aggregate and recycled coarse aggregate obtained from RCC culvert near Medinipur (Source 1) (Chakradhara Rao 2010)

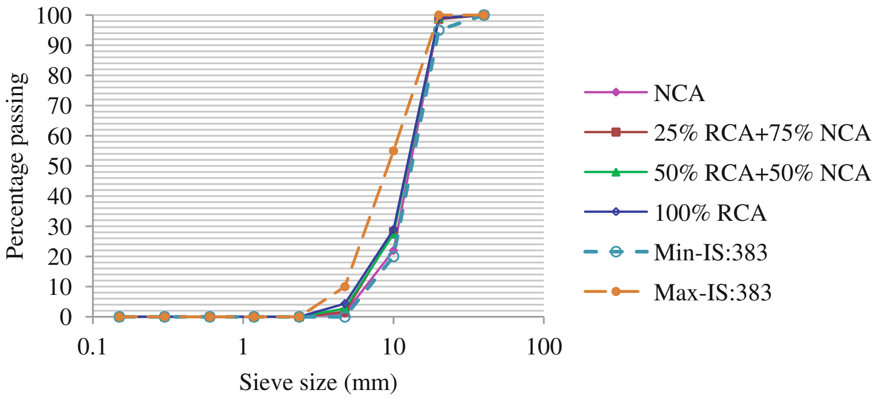


Fig. 3.3 Particle size distribution of natural coarse aggregate and recycled coarse aggregate obtained from RCC culvert near Kharagpur (Source 2) (Chakradhara Rao 2010)

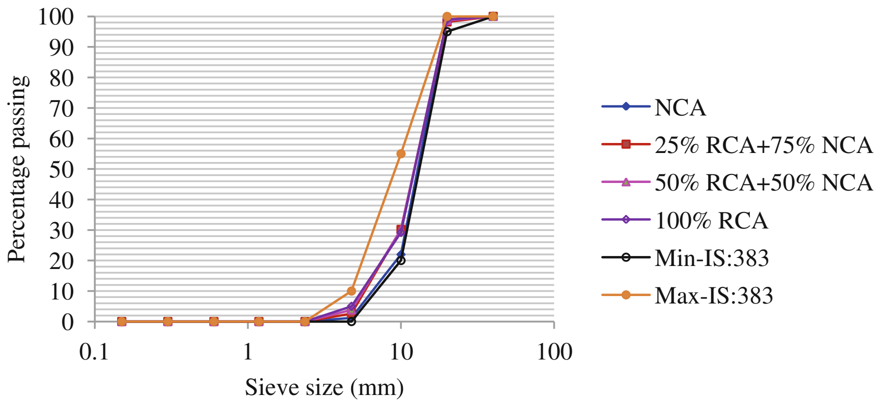


Fig. 3.4 Particle size distribution of recycled coarse aggregate obtained from RCC slab of an old residential building near Vizianagaram (Source 3) (Chakradhara Rao 2010)

coarse aggregates for all coarse aggregate replacement percentages obtained from Source 2 (RCC culvert at Kharagpur) and Source 3 (RCC slab of an old residential building at Vizianagaram) are presented in Figs. 3.3 and 3.4, respectively.

The grading curves show that the recycled coarse aggregates obtained from both Sources are relatively finer than those of natural coarse aggregates. This may be due to the combination of aggregates crushed both manually as well as by jaw crusher. The percentage of particles finer than 4.75 mm in 20 mm maximum sized recycled coarse aggregate is 4.34 and 5 in Sources 2 and 3, respectively. However, it is observed that the grading curves of recycled coarse aggregates for all the coarse aggregate replacement percentages obtained from all the three Sources are within the grading limits specified by BIS (IS: 383-1970). Hence, the recycled coarse aggregates may be used in the production of concrete from the consideration of grading of aggregates; however, other properties are to be examined before arriving at the concluding remarks.

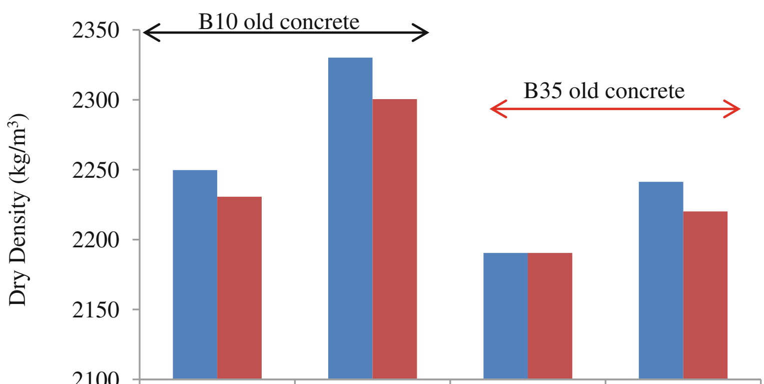
The visual observation (Fig. 3.1) reveals that the surface texture of recycled coarse aggregates is porous and rough due to the adherence of old cement paste to the recycled aggregates after the recycling process. This may demand more water for achieving the required workability. Also, this will improve the interlocking between mortar and aggregates and thereby improves the bond between them.

3.2.2 *Density and Specific Gravity*

The density is one of the important physical properties of aggregate. In short, it defines how densely the material is packed in a given volume. This parameter is useful in proportioning the concrete mixes by volume. In general, due to lower density of old mortar adhered to recycled aggregate, the saturated-surface-dry (SSD) density of recycled aggregates is always less than that of natural aggregate.

BCSJ (1978); Hasaba et al. (1981); Hansen and Narud (1983) have reported that irrespective of the quality of concrete from which the recycled aggregates derived, for same grinding machine and process, SSD density of recycled aggregate was increased with the increase in size of the particles. Most of the researchers reported that the recycled aggregates have SSD density in the range of 2290–2510 kg/m^3. In addition, it was reported that the density of recycled aggregate depends on the strength of the concrete from which the recycled aggregate derived. Bairagi et al. (1990) reported that the recycled coarse aggregate had lower SSD density compared to natural aggregate. The authors reported that this may be attributed to the adherence of old mortar to natural aggregate. Hasse and Dahm (1998) concluded that for the same particle size, higher the strength of concrete from which the recycled aggregate derived, lower is the density of recycled aggregates (Fig. 3.5). This is due to the fact that in higher strength of concrete from which the recycled aggregates derived, lower quantity of low-density old mortar adhered to aggregates.

For the same particle size, higher the strength of concrete from which the recycled aggregate derived, lower is the density of recycled aggregates (Padmini et al. 2009; Pedro et al. 2014). This was due to the fact that in higher strength of concrete from which the recycled aggregates derived, lower quantity of low-density old mortar adhered to the aggregates. When the RCA obtained from 30–100 MPa strength concrete, the density of 10 and 20 mm size aggregate does not follow any specific trend with the increase in strength of parent concrete (PC) (Kou and Poon 2015). Whereas Nagataki (2000) concluded that for the same quantity of cement



	2-8 mm	8-16 mm	2-8 mm	8-16 mm
Pycnometer method	2249.735	2330.159	2190.476	2241.27
Under-Water weighing	2230.68	2300.529	2190.476	2220.106

Size of aggregate

Fig. 3.5 Density of recycled aggregate as a function of size and strength of concrete from which recycled aggregate derived (Hasse and Dham 1998)

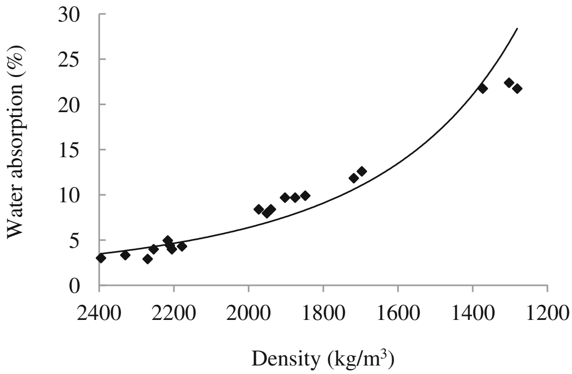


Fig. 3.6 Water absorption as a function of density of recycled concrete aggregates (Kreijger 1981)

mortar content, the recycled aggregates obtained from higher strength concrete had higher density. Buyle-Bodin and Hadjieva-Zaharieva (2002) reported that the density and porosity of recycled aggregate were lower and higher, respectively, than those of natural aggregate due to the presence of old mortar and light impurities in recycled aggregates. Zaharieva et al. (2003) had reported similar results. In a study by Katz (2003), it was reported that the bulk-specific gravity of recycled fine to coarse aggregates was ranged from 2.23 to 2.60 when compared to 2.70 for natural aggregate. This may be attributed to the presence of adhered mortar in aggregates after crushing. In addition, it was reported that the bulk density of recycled fine aggregate was higher than that of medium size recycled aggregate despite the lower specific gravity of former. This was due to the better grading of recycled fine aggregate over a large range of sizes leading to a dense packing. Tu et al. (2006) concluded that the specific gravity and density of recycled aggregates were always lower than that of natural aggregates due to the presence of loose mortar and brick content in construction & demolished waste.

In a study by Gonzalez-Fonteboa and Martinez-Abella (2008), it was reported that the density of recycled aggregate size ranging from 4–12 mm and 10–25 mm were 11.4 and 10.1% lower than that of corresponding natural aggregate. Juan and Gutierrez (2009) concluded that the density of recycled aggregate was a function of adhered cement mortar and the authors found that the density decreased with the increase in percentage of adhered mortar content. In a study by Zega et al. (2010) concluded that the specific gravity and density of recycled aggregate were lower when higher specific gravity of natural aggregate used in original concrete. This attributes to the larger amount of adhered mortar to the recycled aggregate. The density and specific gravity (SSD) of natural fine and coarse aggregates and recycled coarse aggregate obtained from different Sources were examined in accordance with BIS (IS: 2386-1963 (Part 3)) and the results are shown in Table 3.1 (Chakradhara Rao 2010). It was observed that the specific gravity (SSD) of natural fine and coarse aggregates were 2.617 and 2.75, respectively. The specific gravity of recycled coarse aggregates obtained from all the three Sources demonstrates lower values with the increase in percentage of recycled coarse aggregate. The SSD specific gravity of 100% RCA obtained from Sources 1, 2, and 3 are 2.51, 2.47, and 2.417, respectively, against 2.75 for natural coarse aggregate. It was reported in the literature that the specific gravity of recycled coarse aggregate was in the range of 2.23 to 2.6. The reduction in specific gravity of recycled coarse aggregate obtained from the Sources 1, 2, and 3 are 8.7, 10.2, and 12.1%, respectively, compared to natural coarse aggregate. Mortar is generally adhered to the recycled aggregate surface. The attached mortar is light porous in nature. Therefore, it is obvious that the specific gravity of recycled coarse aggregate is relatively less when compared to natural coarse aggregate. The specific gravity of RCA obtained from Sources 2 and 3 was little lower than those obtained from Source 1. This may be due to fact that the recycled coarse aggregates obtained from these Sources have 4–5% particles finer than 4.75 mm. This may be attributed to the adherence of relatively higher quantity of old cement mortar to the recycled aggregates. Due to lower specific gravity of recycled coarse aggregates, there is a reduction in the amount of recycled coarse aggregate to be used in the recycled aggregate concrete.

The results of both compact and loose bulk densities of natural fine and coarse aggregate and recycled coarse aggregates are presented in Table 3.1.

It shows that the natural aggregate has higher packing capacity than recycled coarse aggregate obtained from different Sources. The compact density of natural coarse aggregate is 1.581 kg/l when compared to 1.413, 1.34 and 1.35 kg/l for recycled coarse aggregates obtained from the Sources 1, 2, and 3, respectively. The compact density of RCA obtained from all the three Sources are 10.6%, 15.2% and 14.6%, respectively, lower than that of natural coarse aggregate. Similarly, the loose bulk density of RCA obtained from the Sources 1, 2, and 3 are 12.2, 17 and 12.3% lower than that of natural coarse aggregate. As the old cement mortar adhered to aggregate in recycled coarse aggregate is light and porous in nature, it is obvious that the bulk density of recycled coarse aggregates are lower than that of natural coarse aggregate. However, the recycled coarse aggregates are found to be relatively finer than natural aggregate as explained earlier, yielding lesser voids when packed, therefore, increasing the bulk density of the mix and partially compensate the lower bulk density due to adhered cement paste in recycled aggregate and therefore reduced the difference of this characteristic in comparison to natural aggregate. It is observed from Table 3.1 that there is a difference in bulk density of recycled coarse aggregates obtained from different Sources. This is probably that the bulk density of recycled coarse aggregate depends on the cement content of the adhered mortar which in turn depends on the quality of source concrete.

3.2.3 Water Absorption

The water absorption is one of the major differences between natural and recycled aggregates. The water absorption of recycled aggregate is much higher than that of

Table 3.1 Physical characteristics of natural fine and coarse aggregate and recycled coarse aggregate obtained from different Sources (Chakradhara Rao 2010)

Property	Natural Fine Aggregate	Natural CA	Source 1 Recycled coarse aggregate			Source 2 Recycled coarse aggregate			Source 3 Recycled coarse aggregate			IS: 383-1970 limits
			25% RCA + 75% NCA	50% RCA + 50% NCA	100% RCA	25% RCA + 75% NCA	50% RCA + 50% NCA	100% RCA	25% RCA + 75% NCA	50% RCA + 50% NCA	100% RCA	
Bulk density (compact) (kg/l)	1.618	1.581	1.56	1.5	1.413	1.53	1.45	1.34	1.5	1.44	1.35	–
Bulk density (Loose) (kg/l)	1.544	1.495	1.45	1.45	1.312	1.47	1.39	1.24	1.42	1.39	1.31	–
Specific gravity (SSD)	2.617	2.75	2.66	2.602	2.51	2.71	2.64	2.47	2.671	2.617	2.417	–
Apparent specific gravity	2.63	2.827	2.804	2.78	2.803	2.82	2.808	2.701	2.822	2.817	2.672	–
Water absorption (%)	0.201	1.129	1.911	2.63	3.92	1.515	1.92	3.009	2.005	2.704	3.934	–
Flakiness Index (%)	–	24.1	–	–	6.27	–	–	7.4	–	–	4.65	25
Elongation Index (%)	–	20.06	–	–	11.63	–	–	13.5	–	–	13.29	30

natural aggregate due to high absorption capacity of old mortar adhered to aggregate in recycled aggregate. Water absorption of recycled aggregate is a function of size, strength of original concrete, adhered mortar content and density.

In a study by BCSJ (1978), it was ascertained that the water absorption was 3.6–8% and 8.3–12% for recycled coarse and fine aggregates, respectively. Hasaba et al. (1981) had observed similar results and it was reported that the water absorption of recycled aggregate was around 7% for 5–25 mm size and 11% for below 5 mm size aggregates. Hansen and Narud (1983) found that irrespective of the quality of concrete from which the recycled aggregate derived the water absorption of recycled coarse aggregate size ranging from 16–32 mm and 4–8 mm were 3.7% and 8.7%, respectively. In addition, the relationship between the density and water absorption of recycled aggregates for different w/c ratios was reported and is presented in Table 3.2.

Bairagi et al. (1993) found that the water absorption of recycled aggregates in the first 30 min was 76% and the 4 h absorption was 96% of the absorption in 24 h, respectively. Katz (2003) found that the water absorption was ranging from 3.2–12% for coarse to fine recycled aggregates. Also, it was reported that the crushing age of original concrete from which the recycled aggregate produced does not influence much on water absorption of recycled aggregate. As already mentioned, the density and adhered mortar content are the function of concrete from which the recycled aggregate derived, the water absorption is also a function of the strength of parent concrete. Poon et al. (2004) reported that the water absorption was higher for

Table 3.2 Properties of natural gravel and recycled aggregates according to Hansen and Narud (1983)

Type of aggregate	Size fraction (mm)	Specific gravity SSD (kg/m^3)	Water absorption (%)	Los Angeles abrasion loss (%)	Volume percent of mortar attached to natural gravel
Original natural gravel	4–8 8–16 16–32	2500 2620 2610	3.7 1.8 0.8	25.9 22.7 18.8	0 0 0
Recycled aggregate (H) (w/c = 0.4)	4–8 8–16 16–32	2340 2450 2490	8.5 5.0 3.8	30.1 26.7 22.4	58 38 35
Recycled aggregate (M) (w/c = 0.7)	4–8 8–16 16–32	2350 2440 2480	8.7 5.4 4.0	32.6 29.2 25.4	64 39 28
Recycled aggregate (L) (w/c = 1.2)	4–8 8–16 16–32	2340 2420 2490	8.7 5.7 3.7	41.4 37 31.5	61 39 25
Recycled aggregate (M) (w/c = 0.7)	<5	2280	9.8	–	–

Note: H—High strength, M—Medium strength, L—Low strength recycled aggregates obtained from normal strength concrete than those obtained from high-performance concrete (HPC). Tu et al. (2006) in their study observed that the water absorption of recycled fine and coarse aggregates were 10 and 5%, respectively, as compared to 1 and 2% for natural fine and coarse aggregate. Padmini et al. (2009) concluded that the water absorption increased with the increase in strength of concrete from which the recycled aggregate derived. This was due to high amount of old mortar content adhered to recycled aggregates obtained from higher strength of concrete.

Tam et al. (2008) in a study concluded that the current British Standards (BS) method for water absorption measurement of natural aggregate was not suitable for recycled aggregate, as patches of cement pastes adhered to the surface of recycled aggregate may affect the water absorption. In addition, the 24 h of saturation was not enough for recycled aggregate, as it was influenced by the amount of cement paste adhered to the recycled aggregate which varies from site to site. The authors proposed a new technique called real-time assessment of water absorption (RAWA) for accurately measuring the water absorption of recycled aggregate. The proposed new technique gives the water absorption at different time intervals and it can avoid the removal of cement paste during soaking and drying process of the recycled aggregate samples. Zega et al. (2010) in their study concluded that the water absorption of recycled aggregate depends on the type of natural aggregate used in the original concrete. It was reported that higher specific gravity of natural aggregate used in original concrete, higher the adhered mortar content in recycled aggregate and hence higher was the water absorption of recycled aggregate.

Yang et al. (2008) found that the water absorption and adhered cement paste on the surface of the aggregates were directly proportional to each other. Juan and Gutierrez (2009) observed similar result and the authors found the following relationship between them.

$$\mathrm{y} = 0.18\mathrm{x} + 0.36\left(\mathrm{R}^2 = 0.5\right) \tag{3.1}$$

Where y and x are the water absorption and adhered mortar content in percentages, respectively.

The density of recycled aggregates also influences the water absorption capacity of recycled aggregates. Lower the density higher is the water absorption. The relationship between water absorption and density is presented in Figs. 3.6 and 3.7 and Eqs. 3.2 and 3.3, based on the results presented by Kreijger (1981) and three German firms, namely BAM, HOLZMANN, TEL (Kun 2005), respectively.

$$y = 7 \times 10^{11} \times x^{-3.347}\left(R^2 = 0.927\right) \tag{3.2}$$

$$y = 3 \times 10^{18} \times x^{-5.283}\left(R^2 = 0.893\right) \tag{3.3}$$

Some of the researchers suggested that it was better to use the presoaked recycled aggregates in the production of concrete to maintain the uniform quality due to high absorption capacity of recycled aggregates. Before carrying out the mix

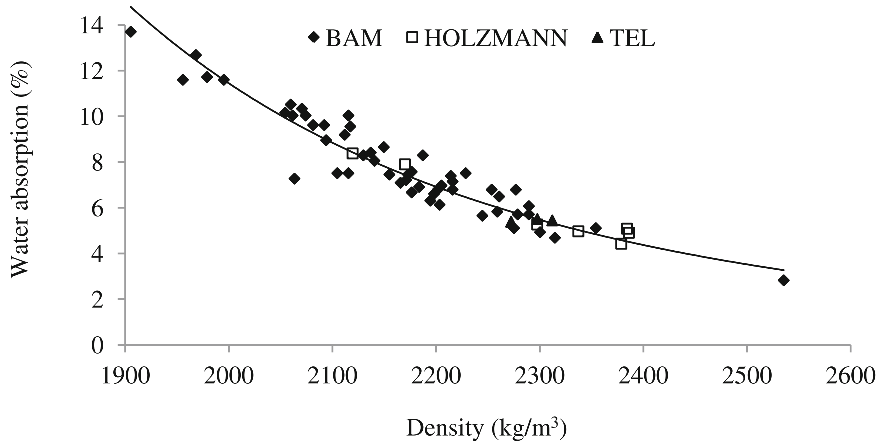


Fig. 3.7 Relationship between water absorption and density of recycled aggregate produced by three German firms (Kun 2005)

design for production of concrete, the density and water absorption of recycled coarse and fine aggregates must be studied.

It is worth noting that the water absorption of recycled coarse aggregate is higher than that of natural coarse aggregate irrespective of the source of recycled coarse aggregate (Chakradhara Rao 2010). This is expected due to porous and high absorption capacity of old mortar adhered to aggregate in recycled aggregate. The absorption capacity of recycled coarse aggregates obtained from the Sources 1, 2, and 3 are 3.92, 3.009, and 3.934%, respectively, when compared to 1.121% for natural coarse aggregate. This indicates that the recycled coarse aggregate has 2.7–3.5 times higher water absorption than that of natural aggregate. It was reported in the literature that the absorption capacity mainly depends on the strength of concrete from which the recycled coarse aggregate derived and the size of the coarse aggregate. The aggregate obtained from higher strength concrete will have higher cement paste and thereby higher water absorption.

3.2.4 Flakiness and Elongation Indices

The aggregates are flaky if their thickness is less than 0.6 times the mean sieve size of the size fraction to which the particle belongs. Similarly, an aggregate is said to be elongated, when a particle whose longest dimension is greater than 1.8 times the average sieve size of the size fraction. The test results of flakiness and elongation indices of natural coarse aggregate and recycled coarse aggregate obtained from different Sources are presented in Table 3.1. It is observed that the flakiness and elongation indices of natural aggregate are 24 and 20%, respectively. These values are just satisfying the requirements for normal concrete. On the other hand, the flakiness index of RCA obtained from the Sources 1, 2, and 3 are 6.27, 7.4, and 4.65%, respectively. Similarly, the elongation index of RCA obtained from the Sources 1, 2, and 3 are 11.13, 21.5, and 13.29%, respectively. These values are well below the requirements for aggregates for normal concrete. The results reveal that the recycled coarse aggregates demonstrate better indices when compared to natural aggregates. It is felt that these improvements may be due to the use of crushing methodology namely manual crushing along with jaw crushing.

3.3 Mechanical Properties of Recycled Aggregates

In general, the recycled aggregates are found to be weaker than the natural aggregates against mechanical resistance such as aggregate crushing value, ten percent fines value, impact value and Los Angeles abrasion loss value. This was due to the weaker old cement mortar adhered to the recycled aggregates and weaker bond between old mortar and aggregate in recycled aggregates. These values mainly depend on the strength of concrete from which the recycled aggregate derived, amount of old mortar adhered to the aggregate and the quality of aggregate employed in the original concrete. In addition, the method of crushing also influences these properties. Ravindrarajah (1987) reported the mechanical strengths of recycled aggregate were low when they were derived from low strength concrete. Bairagi et al. (1993) found the mechanical properties of recycled aggregate depend on the age of concrete from which the recycled aggregate produced. In a study by Padmini et al. (2009) reported, for a given strength of concrete from which the recycled aggregate derived, the resistance against mechanical actions decreased with the reduction in maximum size of aggregate. This can be attributed to the higher surface area of smaller size aggregates facilitating higher amount of mortar adhered.

3.3.1 Aggregate Crushing Value

Hasaba et al. (1981) reported that British Standards (BS) aggregate crushing value for 5–25 mm size recycled aggregate was 23 and 24.6% when they were derived from high strength concrete and low strength concrete, respectively. In addition, it was reported that the corresponding BS ten percent fines values were 13.3 and 11.3 tonnes, respectively. Bairagi et al. (1993) found that the crushing value increased and the ten percent fines value decreased with the age of original concrete from which the recycled aggregate produced. This may be attributed to the presence of weak adhered mortar which increased with the age of parent concrete. Shayan and Xu (2003) found that the crushing value of recycled coarse aggregate was 24%. Poon et al. (2004) observed that the ten percent fines value of recycled aggregate was lower when the recycled aggregates derived from normal strength concrete than those from high-performance concrete.

3.3.2 Los Angeles Abrasion Resistance Value

As discussed earlier, the Los Angeles abrasion loss depends on the strength of concrete from which the recycled aggregates derived, size of aggregate and the amount of adhered mortar. Hansen and Narud (1983) reported the Los Angeles abrasion loss values for recycled aggregates obtained by crushing 40 MPa strength concrete had lower abrasion than those obtained by crushing 16 MPa strength concrete and is presented in Table 3.3.

Hasaba et al. (1981), BCSJ (1978) and Yoshikane (2000) had reported similar results. Here the crushing details are unknown. For lower size recycled aggregates, the abrasion loss was more due to high amount of adhered mortar. Shayan and Xu (2003) found that the Los Angeles abrasion value of recycled aggregate was 32%. Juan and Gutierrez (2009) found that the abrasion loss is directly proportional to the adhered mortar content. Zega et al. (2010) reported that the w/c ratio of original concrete from which the recycled aggregate derived was less significant than that of the abrasion loss of natural aggregate used in original concrete on the loss of abrasion of the corresponding recycled aggregate.

Chakradhara Rao (2010) studied the mechanical characteristics, namely aggregate crushing value, ten percent fines value, aggregate impact value and Los Angeles abrasion resistance, are assessed for both natural and recycled coarse aggregates obtained from different Sources in accordance with BIS (IS: 2386-1963 (Part 4)). The test results of all coarse aggregates along with the limits specified by BIS (IS: 383-1970) are presented in Table 3.4.

From the table, it is observed that the aggregate crushing value of natural coarse aggregate is 17.52% against 33.5, 34.4, and 36.7%, respectively, for recycled coarse aggregate obtained from all the three Sources. This indicates that the crushing value of recycled coarse aggregates is almost double than the natural aggregate. The aggregate crushing value is an indication of resistance of gradually applied compression load. Higher the aggregate crushing value, lower is the

Table 3.3 Los Angeles abrasion loss of recycled aggregates obtained from 40 and 16 MPa strength concrete by different researchers

Author(s)	Los Angeles abrasion loss percentage							
	40 MPa strength concrete				16 MPa strength concrete			
	16–32 mm	8–16 mm	4–8 mm	5–25 mm	16–32 mm	8–16 mm	4–8 mm	5–25 mm
Hansen and Narud (1983)	22.4	26.7	30.1		31.5	37	41.4	
Hasaba et al. (1981)				23				24.6
BCSJ (1978)[a]	25.1–35.1							
Yoshikane (2000)				20.1				28.7

[a]Recycled coarse aggregates obtained from 15 different concrete of different strengths

Table 3.4 Mechanical characteristics of natural and recycled coarse aggregates obtained from different Sources (Chakradhara Rao 2010)

Property	Natural CA	Recycled coarse aggregate			IS: 383-1970 limits
		Source 1	Source 2	Source 3	
Crushing value (%)	17.52	33.57	34.4	36.7	Less than 30% for wearing surfaces and less than 45% for other than wearing surfaces
Ten percent fines value (kN)	231.3	115.3	120.5	118.7	
Los Angeles abrasion resistance (%)	21.56	38.8	36.7	44.16	Less than 30% for wearing surfaces and less than 50% for other concretes
Impact value (%)	17.37	35.81	35.95	38.42	Less than 30% for wearing surfaces and less than 45% for other than wearing surfaces

strength of the aggregate. Therefore, the resistance against crushing of recycled coarse aggregate obtained from all the three Sources is weaker than that of natural coarse aggregate. This attributes the formation of finer particles due to the adhered old cement mortar during the application of compressive load. However, the crushing values of recycled coarse aggregates are within the limits specified by BIS (IS: 383-1970) for various applications of normal concrete aggregates. Similarly, the minimum load required to produce ten percent fines value in case of natural coarse aggregate is 231.3 kN. Whereas, the load required in case of recycled coarse aggregates obtained from the Sources 1, 2, and 3 are 115.3, 120.5, and 118.7 kN, respectively. Unlike the crushing value, a higher value indicates higher the strength of aggregate. According to the British Standards (BS: 882-1992), the minimum load required to produce ten percent fines for heavy duty floors is 150 kN, for wearing surfaces is 100 kN and for other concrete works is 50 kN.

The results of toughness and hardness of recycled coarse aggregate revealed a similar trend. The aggregate impact value of recycled coarse aggregate obtained from Sources 1, 2, and 3 are 35.81, 35.95, and 38.42%, respectively. These values are almost double when compared to natural aggregates of 17.37%. However, these are within the limits of BIS (IS: 383-1970) for natural aggregate concrete of different applications. Similarly, the Los Angeles abrasion resistance of recycled coarse aggregates is almost twice than that of natural aggregate. As it is reported by several researchers in the literature that the mechanical properties of recycled coarse aggregates are relatively weaker compared to natural aggregates due to separation and crushing of light porous mortar adhered from recycled aggregates during testing.

3.4 Summary

The physical properties, viz: grading, size and texture, density, specific gravity, water absorption, flakiness, and elongation indices of recycled aggregates are highlighted. The various factors affecting these properties of RA are also discussed. In general, the recycled aggregates are found to be weaker than the natural aggregates against mechanical resistance such as aggregate crushing value, ten percent fines value, impact value and Los Angeles abrasion loss value. The results of mechanical properties of RA reported by various researchers are discussed in this chapter. Based on these discussions, the following are the important conclusions drawn.

- Using jaw crusher, one can produce reasonably well-graded recycled coarse aggregates of good shape from the uncontaminated demolished concrete rubble. These recycled aggregates are more angular and the surface texture is rough and more porous due to the presence of old mortar. The recycled fine aggregates obtained from the crusher are somewhat coarser and angular than desirable, for production of good quality concrete mixes. Therefore, the concrete made exclusively with fine and coarse recycled aggregates tend to be harsh and unworkable. However, by adding a certain percentage of natural fine aggregate to the recycled aggregate, it is possible to bring the fine recycled aggregates within the grading limits and at the same time, the workability is greatly improved.
- The amount of adhered cement paste on the aggregate surface depends mainly on the grinding process, aggregate size, and strength of original concrete. The quantity of adhered paste increases with the decrease in size of aggregate. The recycled aggregates obtained from the concrete having lower strength have the higher quantity of adhered cement paste for the given size of aggregate.
- The saturated-surface-dry (SSD) density of recycled aggregates were lower than that of natural aggregates due to the low density of cement mortar, that was attached to the concrete from which the recycled aggregates derived and many researchers found that the range is in between 2340–2490 kg/m^3. Due to better grading of recycled fine aggregate over a large range of sizes, the bulk density of recycled fine aggregate was higher than that of medium size recycled aggregate despite the lower specific gravity of former.
- Like the density and SSD specific gravity, the water absorption of recycled aggregate is also a function of strength of parent concrete. The water absorption of recycled aggregate was inversely proportional to the strength of parent concrete. The water absorption of coarse recycled aggregates was much higher than that of natural aggregates due to higher absorption capacity of old cement mortar adhered to the recycled aggregates. The water absorption capacity of fine recycled aggregate is still higher than the coarse recycled aggregates. Due to high water absorption, it was proposed to use presoaked aggregates for production of RAC so that uniform quality may be maintained during the preparation of the mix.

- The recycled aggregates are found to be weaker than natural aggregates against mechanical resistance such as aggregate crushing value, the ten percent fines value and Loss Angeles abrasion loss value due to the weaker old mortar adhered to the recycled aggregates and weaker bond between recycled aggregate and old mortar. These values mainly depend on the strength of parent concrete, the quality of aggregate employed in the parent concretes and the method of crushing.

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Chapter 4
Properties of Recycled Aggregate Concrete

4.1 Introduction

The fresh and hardened properties of recycled aggregate concrete depend on the strength of original concrete from which the recycled aggregate derived, the w/c ratio, the size of recycled aggregate, amount of recycled aggregate and moisture condition of recycled aggregate, etc. Many researchers have studied the influence of different qualities of recycled coarse aggregate with different proportions of natural and recycled aggregates on properties of fresh and hardened recycled aggregate concrete, and their findings are presented below.

4.2 Workability

The properties of hardened concrete are seriously affected by its degree of compaction. It is absolutely necessary that the fresh concrete should be workable such that the concrete can be mixed, transported, placed, properly compacted and finished easily and without any segregation.

4.2.1 Factors Influencing the Workability

The various factors like type of aggregate, moisture state of recycled aggregate, water absorption of aggregate, maximum size of aggregate, w/c ratio, strength of source concrete etc., severely affect the workability of recycled aggregate concrete. Further, the physical properties of aggregate viz. aggregate size and shape, surface texture affects the workability of concrete (Behra et al. 2014). The important factors are discussed in the following subsections.

M. C. Rao et al., *Systematic Approach of Characterisation and Behaviour of Recycled Aggregate Concrete*, Springer Transactions in Civil and Environmental Engineering,
https://doi.org/10.1007/978-981-10-6686-3_4

4.2.1.1 Effect of Water Requirement on Workability

It is well known that the recycled aggregates absorb more water than natural aggregate. In general, the water absorption capacity of recycled aggregate was in the range of 3–12% against less than 1% in majority times in case of natural aggregate. The high absorption capacity of recycled aggregate greatly influences the workability of fresh concrete. Therefore, it has to be taken care while using the recycled aggregate in the production of concrete either by presoaking the aggregate in water before mixing of all ingredients or by adding the additional water calculated based on their absorption at the time of mixing. It was reported that the concrete made with recycled coarse aggregate and natural sand requires 5% more free water than the concrete made with natural aggregate in order to obtain the same workability (slump) (Buck 1977; Frondistou-Yannas 1977; Mukai et al. 1978; Hansen and Narud 1983; Ravindrarajah and Tam 1985). If both recycled coarse and fine aggregates are used, it needs 15% more free water to achieve the same slump. This was mainly due to rough surface and angularity of recycled coarse aggregate as against to the round and smooth surfaces of natural aggregate. Whereas, Rasheeduzzafar and Khan (1984) did not agree, as the recycled aggregate concrete workability assessed in their study in terms of compaction factor and Vee-bee seconds was found to be higher when compared to normal concrete. Hansen and Narud (1983) also reported the RAC mixes are more cohesive than normal concrete mixes due to the adherence of old mortar and generation of finer particles during mixing. Therefore, the RAC made with lower quality of RCA increase the cohesiveness as they adhered with more amount of old cement mortar. In addition, it was reported the loss of workability with time increased in case of RAC mixes due to dry recycled coarse aggregates that continue to absorb water after mixing compared to natural aggregate in normal concrete.

Yrjanson (1981) found that the use of recycled coarse aggregate does not have any significant effect on the mix proportions and workability when compared to normal concrete. However, when recycled fine aggregate was used, the workability of the mix became less and needs more water and therefore more cement. Substitution of natural sand up to 30% in place of recycled fine aggregate improves the workability of the mix. Bairagi et al. (1990) conducted series of experiments on concrete made with laboratory crushed recycled aggregates and found the RAC mixes exhibits similar workability to that of normal concrete. The workability of recycled aggregate concrete not only depends on the absorption capacity of recycled aggregate but also on the shape and texture of recycled aggregates (Rashwan and Abourizk 1997). As the recycled aggregates are more angular due to the crushing and thereby higher surface-to-volume ratio, internal friction was increased, required more mortar to provide better workability. Buyle-Boudin and Hadjieve-Zaharieva (2002) had reported similar results in the literature. Yang et al. (2008) in their study concluded that the water absorption of recycled aggregate (both fine and coarse) had less influence on initial slump, whereas, increase in relative water absorption of recycled aggregates increases the rate of slump loss. Sagoe-Crentsil et al. (2001) suggested the recycled aggregates should be saturated or pre-wetted to prevent a rapid reduction in workability. The authors also reported the recycled aggregate obtained from plant was relatively smoother spherical shape, which leads to improved concrete workability than normal concrete with natural basalt aggregate for the same grading and fine-to-coarse aggregate ratio. The workability of RAC made with dry aggregates is more than the expected when the calculated water based on its absorption is added at the time of mixing (De Pauw et al. 1998). Knights (1998) suggested that the recycled coarse aggregates should be pre-soaked for 20 min in half of the mixing water calculated based on moisture and absorption, before mixing all the constituents to control the initial slump and slump loss.

Chakradhara Rao (2010) conducted a series of experiments on both normal and recycled aggregate concrete mixes with different percentages of RCA obtained from three different demolished old structures and the details of all the mixes are presented in Table 4.1. All the mixes were prepared with a free water–cement ratio (w/c) of 0.43 and a cement content of 401 kg/m^3.

It was observed that in all the mixes, the workability decreases with the increase in percentage of recycled coarse aggregate. The surface texture of recycled aggregate is more granular due to the old adhered mortar, which increases the requirement of water. Further due to interparticle friction, it requires higher energy for compaction. Hence, the workability decreases. It was also observed from Table 4.1 that the requirement of superplasticizer for keeping the slump between 50 and 60 mm increases with the increase in percentage of recycled coarse aggregate. This indicates that the recycled coarse aggregate had higher water absorption. As stated above, this may be ascribed to the surface texture of recycled aggregates is

Table 4.1 Test results of workability of both normal concrete and RAC made with RCA obtained from three Sources (Chakradhara Rao 2010)

Source of RCA	Mix designation	RCA (%)	Total a/c ratio[a]	Superplasticizer[b]	Slump (mm)
Normal concrete	M-RAC0	0	4.58	0.05	57
Source 1: RCC culvert near Medinipur, West Bengal	MM-RAC25	25	4.46	0.05	55
	MM-RAC50	50	4.36	0.175	50
	MM-RAC100	100	4.22	0.225	50
Source 2: RCC culvert near Kharagpur, West Bengal	M-RAC0	0	4.58	0.05	57
	MK-RAC25	25	4.53	0.05	54
	MK-RAC50	50	4.43	0.175	52
	MK-RAC100	100	4.24	0.225	49
Source 3: RCC slab of a old residential building near Vizianagaram, Andhra Pradesh	M-RAC0	0	4.57	0.05	56
	MV-RAC25	25	4.52	0.05	54
	MV-RAC50	50	4.38	0.175	49
	MV-RAC100	100	4.12	0.225	51

[a]Total aggregate/cement ratio
[b]Percentage by weight of cement

more porous and rough due to the adherence of mortar in recycled aggregates. It was also observed that there is a little difference in slump values between the mixes from different Sources. This may be due to the difference in total aggregate by cement ratio and the relative difference in the properties of cement used.

4.2.1.2 Influence of Moisture State of Recycled Aggregate

Like the water absorption of recycled aggregate, the moisture conditions of the aggregate also affect the workability of recycled aggregate concrete. Poon et al. (2004) investigated the influence of air-dry (AD), oven-dry (OD), and saturated-surface-dry (SSD) aggregates on workability of concrete mixes in terms of slump and the results are depicted in Fig. 4.1.

It was reported that the initial slump of RAC made with oven-dry and air-dry recycled coarse aggregates increased with the increase in RCA content compared to the saturated-surface-dry (SSD) aggregate. This was due to the higher absorption capacity, leading to larger amount of free water added to the mix for saturating the aggregates. In addition, it was reported that the slump loss was more in case of RAC with oven-dry (OD) and air-dry (AD) RCA compared to RAC with SSD aggregate due to the high absorption of dry aggregates. The authors also reported that the RAC mixes were less cohesive when compared to normal concrete mixes. The lack of cohesiveness affects the homogeneity of the mix and in turn affects the mechanical and durability characteristics of the concrete. Poon et al. (2007) ascertained that the RAC made with air-dried recycled aggregates resulted in higher initial slump and longer period to come to zero slump when compared to normal concrete. Further, it was noticed that with the increased amount of recycled aggregate, the initial slump of RAC was increased. The authors also found that 25% addition of fly ash in place of cement further improved the slump in both normal and recycled aggregate concrete.

Mefteh et al. (2013) also investigated three different moisture conditions viz. oven-dried, pre-wetting, and SSD recycled aggregates on the workability of RAC made with different proportions of recycled coarse aggregate. The variation of slump with respect to time for different mixes under different moisture conditions of aggregate is presented in Figs. 4.2, 4.3, and 4.4. Figure 4.2 reveals that the initial slump was reduced with the increased percentages of RCA. Further, the decrease in slump was more when the wait time increased. The initial slump and slump after 60 min of concrete mix prepared with 100% RCA in dry state were 15 mm and 10 mm, respectively. These slump values were much lower than the initial target values. This was mainly due to higher water absorption of recycled aggregate. The slump loss in first 15 to 30 min in case of concrete mixes prepared with RCA in dry state was significant. This was due to higher rate of water absorption of recycled aggregate in the early period when the recycled aggregates were in contact with water. After 30 min, the slump loss seems to be decreased. Figures 4.3 and 4.4 represent the variation of slump of concrete mixes prepared with recycled aggregate in pre-wetting and saturated-surface-dry conditions, respectively. The initial slump

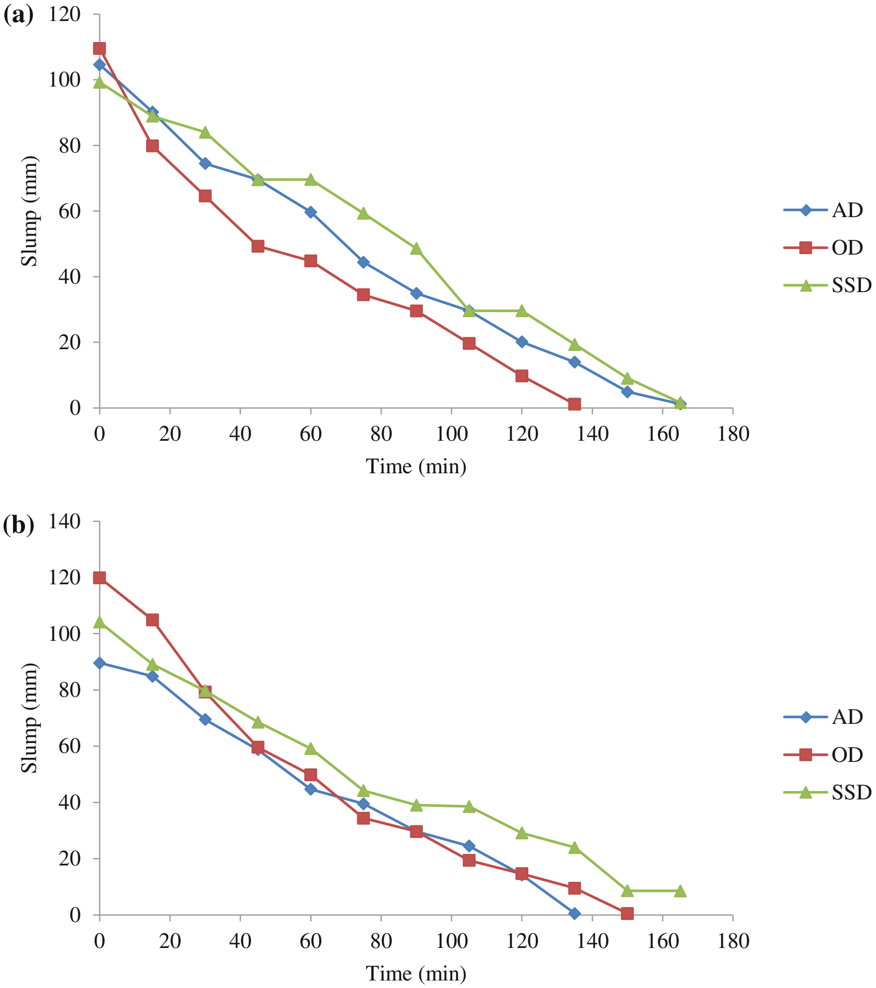

Fig. 4.1 Variation in slump of concrete mixes with different moisture conditions of aggregates. (**a**) 100% crushed granite aggregate (**b**) 80% crushed granite + 20% Recycled coarse aggregate (**c**) 50% crushed granite + 50% recycled aggregate and (**d**) 100% Recycled coarse aggregate (Poon et al. 2004)

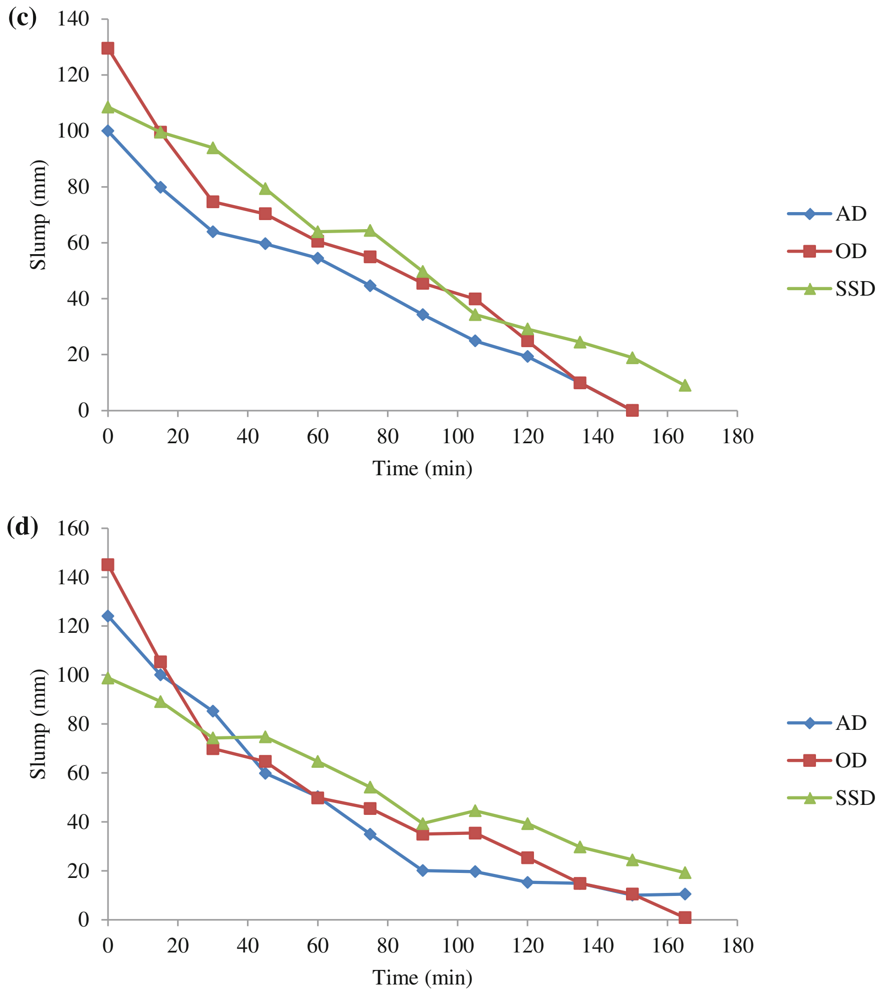

Fig. 4.1 (continued)

of concrete mixes ranges from 150 mm with RCA 100% to 90 mm with RCA 20% in pre-wetting condition and these slump values were decreased as the waiting time increased. However, the slump of all RAC mixes was more than that of natural aggregate concrete. This was because of the higher amount of water mixed with recycled aggregate compared to natural aggregate, i.e., an extra additional amount of water supplied by the pre-wetting process of RCA corresponding to their water absorption, apart from the quantity of mixing water based on w/c ratio for a concrete mix. It was found that the curves of RCA in saturated-surface-dry condition were almost similar trend as the curves of RCA in pre-wetting condition (Fig. 4.4). However, these curves were not comprehensible with the percentage replacement of

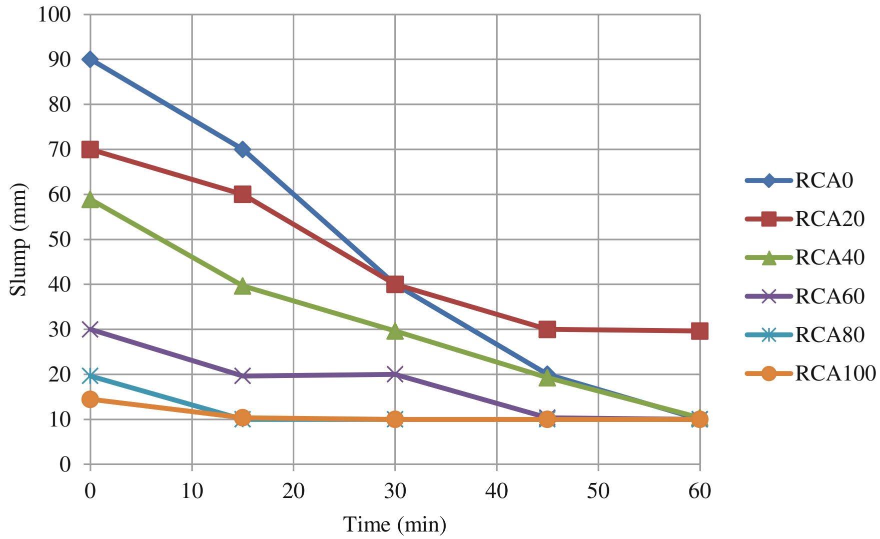

Fig. 4.2 Slump variation in concrete mixes of RCA in dry state (Mefteh et al. 2013)

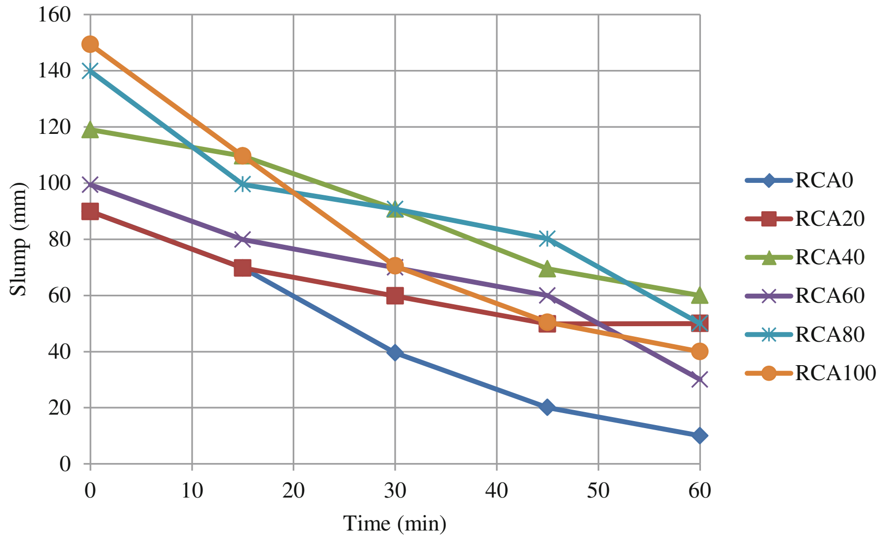

Fig. 4.3 Slump variation in concrete mixes of RCA in pre-wetting state (Mefteh et al. 2013)

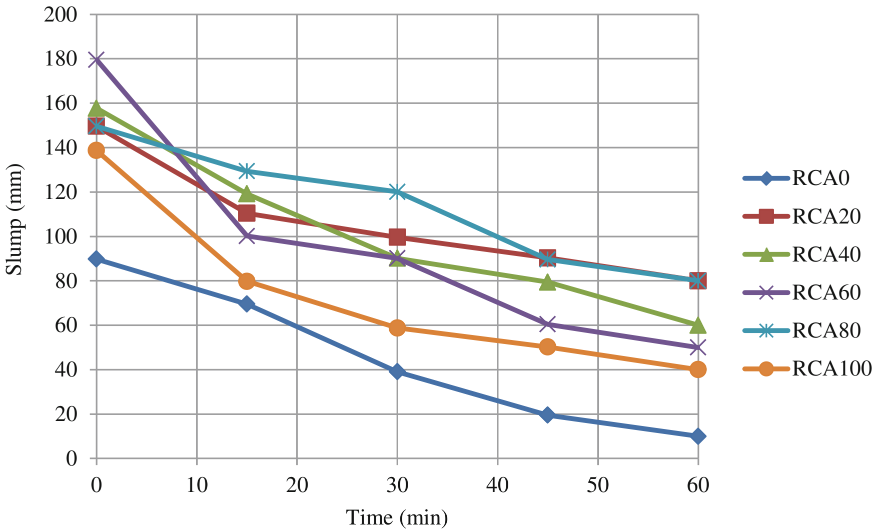

Fig. 4.4 Slump variation in concrete mixes of RCA in saturated-surface-dried condition (Mefteh et al. 2013)

recycled aggregate. For example, the slump of RCA20 mix was higher than RCA60 mix at 60 min. This might be associated with the amount of adhered mortar in recycled aggregate, which was haphazard.

4.2.1.3 Influence of Strength of Parent Concrete

The strength of parent concrete, the size of natural aggregates used in the parent concrete, and the crushing stages intensely affect the properties of recycled aggregates (Akbarnezhad et al. 2013). The amount of adhered mortar on recycled aggregate and hence the water absorption mainly depends on the quality of parent concrete (Rao 2016). The water absorption is one of the important parameters which influence the workability of concrete. Therefore, the strength of parent concrete also influences the workability of concrete.

Kou and Poon (2015) studied the influence of the strength of parent concrete on workability of normal-strength (Series I) and high-performance recycled aggregate concrete (Series II). The Series I mixes were prepared with a w/c ratio of 0.5 with a 28 day compressive strength of 45 MPa. Similarly, Series II mixes were prepared with a 28 day's compressive strength of 65 MPa with a w/c ratio of 0.35. All the mixes were prepared with saturated-surface-dry (SSD) aggregates. The details of RAC mixes are presented in Tables 4.2 and 4.3, respectively. Based on the moisture content of the aggregates, the actual concrete mixes were adjusted at the time of mixing.

Table 4.2 Mix proportions of normal-strength RAC (Series I) with RCA obtained from different strengths of parent concrete (Kou and Poon 2015)

Mix notation	W/C ratio	Proportion (kg/m^3)					Slump (mm)
		Cement	Water	Fine Aggregate	10 mm CA	20 mm CA	
NA	0.5	390	195	642	375	794	75
RCA-30					351	740	80
RCA-45					363	753	85
RCA-60					361	738	80
RCA-80					362	754	70
RCA-100					357	740	65

RCA-30, RCA-45, RCA-60, RCA-80, and RCA-100 indicate the RAC prepared with RCA obtained from 30, 45, 60, 80, and 100 MPa compressive strength, respectively

Table 4.3 Mix proportions of high-performance RAC (Series II) with RCA obtained from different strengths of parent concrete (Kou and Poon 2015)

Notation	W/C ratio	Proportion (kg/m^3)						ADVA 109 (l/m^3)	Initial slump (mm)
		Cement	Fly ash	Water	Fine Aggregate	10 mm CA	20 mm CA		
NA	0.35	420	105	184	668	334	668	3.7	245
RCA-30						294	622		255
RCA-45						306	632		260
RCA-60						303	619		255
RCA-80						305	635		245
RCA-100						301	622		230

It was found from that due to the presence of higher initial free water content in the mixes of RAC with RCA-30 and RCA-45 in both Series—I and II the slump of these mixes were more than that of natural aggregate concrete. The presence of higher initial free water content in RAC mixes was mainly because of higher water absorption capacity of recycled aggregate which was used at the air-dried condition with the aggregates moisture content was much lower than the water absorption at the time of mixing. Excess amount of water was added to maintain the mix proportions as presented in the Tables. That is slump of fresh concrete mixes greatly affected by the moisture states of aggregate. It was concluded that the slump of concrete mixes was highly dependent on the initial free water content of the concrete mixes. Further, it was concluded that the slump of normal-strength RAC mixes decreased with the increased strength of parent concrete from which the RCA obtained. A similar trend was observed in the initial slump of high-performance RAC mixes. This was due to the lower water absorption capacity of recycled coarse aggregate which was obtained from higher-strength parent concrete.

Rao (2016) studied the effect of strength of parent concrete (PC) on workability in terms of compaction factor of recycled aggregate concrete. The author considered two mixes of normal-strength parent concrete (20 MPa and 25 MPa) and three mixes of medium-strength parent concrete (30 MPa, 35 MPa, and 40 MPa) to produce five types of recycled coarse aggregates (RCA). Four strengths (20, 25, 30, and 35 MPa) of RAC mixes, each with two types of RCA obtained from the same strength and relatively higher-strength PC were considered; i.e., 20 MPa RAC mix was produced with RCA from 20 and 25 MPa strengths of PC separately. Similarly 25 MPa RAC with RCA from 25 and 30 MPa PC, 30 MPa RAC with RCA from 30 and 35 MPa PC and 35 MPa RAC with RCA from 35 and 40 MPa PC. A total of 13 mixes were considered in his investigation to study the workability of concrete. The details of mixes along with the compaction factor values are presented in Table 4.4.

It was observed that for a given w/c ratio to maintain the uniform workability the quantity of superplasticizer required in RAC was more than that of parent concrete. This was mainly due to the higher absorption capacity of old mortar adhered to aggregate in RCA. Further, it was observed that the quantity of superplasticizer increased with the increased strength of PC from which the RCA was obtained. This is obvious that the water absorption of RCA obtained from higher-strength parent concrete was more than that of RCA obtained from lower strength parent concrete due to the presence of higher quantity of adhered mortar in RCA obtained from higher-strength PC.

Table 4.4 Details of parent and recycled aggregate concrete mixes (quantities are per cubic meter of concrete) (Rao 2016)

	Grade	Cement (Kg)	Fine Aggregate (Kg)	CA(kg)	w/c ratio	Qty of sp (kg)	Compaction factor
Normal strength	PC20	387.5	568.73	1175.8	0.48	2.42	0.91
	PC25	420	530	1155	0.46	3.36	0.89
Medium strength	PC30	450	514	1119	0.43	4.5	0.86
	PC35	478.95	612.55	1081.8	0.4	5.75	0.84
	PC40	492.5	594.97	1069.4	0.38	7.39	0.85
Normal strength	RM20RCA20	387.5	568.7	1047.8	0.48	3.2	0.9
	RM20RCA25	387.5	568.7	1100.8	0.48	3.2	0.88
	RM25RCA25	420	530	1132.4	0.46	4.2	0.87
	RM25RCA30	420	530	1114.3	0.46	4.2	0.85
Medium strength	RM30RCA30	450	514	1079.5	0.43	5.4	0.86
	RM30RCA35	450	514	1044.4	0.43	5.4	0.85
	RM35RCA35	478.95	612.55	1009.64	0.4	6.23	0.84
	RM35RCA40	478.95	612.55	985	0.4	6.23	0.87

PC20, PC25, PC30, PC35, PC40 indicate the parent concrete of compressive strength 20, 25, 30, 35, 40 MPa, respectively. RM: recycled concrete mix; Number immediately after RM indicates the compressive strength; RCA: recycled coarse aggregate; Number immediately after RCA indicates the RCA obtained from the strength of PC

4.3 Mechanical Properties

The properties of RAC in hardened state such as compressive strength, modulus of elasticity, flexural strength, split tensile strength, density, and ultrasonic pulse velocity. The factors such as physical and mechanical characteristics of recycled coarse aggregate to be used in concrete, w/c ratio of mix, method, and age of curing and the microstructure significantly affects these properties of RAC. It has been found from the literature that due to poor bond between the recycled coarse aggregate and old mortar, the recycled coarse aggregates are less resistant against mechanical action, existence of transverse cracks and fissures in recycled coarse aggregate during crushing processes, and the presence of weak porous mortar attached on the surface of recycled coarse aggregate. The possibility of poorer mechanical properties of recycled aggregate concrete raise alarms about the proper assessment of the RCA properties before its use. In addition to this, the mechanical behavior of RAC will also depend upon the substitution level of RCA and the moisture condition of the RCA (Ajdukiewicz and Kliszczewkz 2002; Mc Neil and Kang 2013). Water–cement ratio also plays a very significant role in recycled aggregate concrete as it depends on many factors such as water absorption of RCA, aggregate-free moisture content and the extent of attached mortar in RCA. In spite of these, the wide disparity in RAC properties may be due to the variation of the quality of RCA and w/c ratio also. The most significant conclusion made from several studies is that the adhered old cement mortar in RCA influences the behavior of RAC, particularly the strength characteristics. In the recent past, many researchers have investigated various properties of RAC in hardened state, which brought the conclusion that the properties of concrete get affected by the substitution of higher quantities of RCA (Behera et al. 2014). These properties have been discussed briefly in the following subsections.

4.3.1 Compressive Strength

The compressive strength of concrete is directly related to the age after casting is completed and it increases with the age. The rate of development of compressive strength depends mainly on the hydration process of cement which in turn depends on the fineness and surface area of the cement, i.e., the presence of tricalcium silicate (C_3S) content of cement. If C_3S is more, the rate of hydration is much faster and thereby the strength development. In addition to this, the compressive strength of recycled aggregate concrete also depends on water–binder ratio, different properties of RAs, quantity of adhered mortar, amount of recycled aggregate, strength of parent concrete from which the recycled aggregate produced, mixing method and addition of secondary cementitious materials, etc. and are discussed in the subsequent sections.

4.3.1.1 Effect of Amount of Recycled Aggregate

The recycled aggregates are poised for natural aggregate and attached mortar. The properties of aggregate are significantly affected by the amount of adhered mortar. Therefore, the amount of recycled aggregate present in a concrete mix affects the properties of concrete.

Bairagi et al. (1993) attempted to get acceptable quality of concrete with maximum utilization of recycled aggregates in place of natural aggregates. The authors considered three mixes with different w/c ratios, and in each mix, the replacement ratios of coarse aggregates varied from 0 to 1.0 at an increment of 0.25. It has been observed that the compressive strength of RAC was reduced by 15% at the replacement ratio of 0.5 and it was 40% at replacement ratio of 1.0. Etxeberria et al. (2007) examined the influence of different amounts of recycled coarse aggregate on properties of RAC. The authors found that the RAC made with 25% RCA gives the same strength as that of normal concrete with the same cement content and effective w/c ratio, whereas at 100% RCA, the strength of RAC was 20–25% lower than that of normal concrete. However, the RAC made with 50% and 100% RCA requires 5–10% extra cement and 4–10% less w/c ratio than that was used in normal concrete to obtain the same compressive strength as that of normal concrete at 28 days. The authors also observed the strength gaining from 28 days to 6 months was slower in case of RAC made with 50% and 100% RCA when compared to normal concrete. This was due to the accumulation of cement on the aggregate surface, which produces low w/c ratio and effective interfacial transition zone (ITZ) and thereby the old ITZ becomes weaker. Topcu and Sengel (2004) attempted to attain 16 MPa and 20 MPa strength of RAC with different amounts of RCA obtained from 14 MPa strength concrete. The authors reported that it could be possible to produce 16 MPa strength of RAC with a maximum substitution of 30% RCA in place of natural coarse aggregate. In a study by Gonzalez-Fonteboa and Martinez-Abella (2008), it was reported that with 6.2% higher cement content than that was used in normal concrete, RAC with 50% RCA can achieve the same strength (30 MPa) as that of normal concrete. In addition, it was concluded that by the addition of 8% silica fume in RAC mixes, higher strengths can be achieved after 28 days compared to normal concrete. It was also reported that the development of strength with curing age was similar in both normal and recycled aggregate concretes with and without the addition of silica fume.

Yang et al. (2008) investigated the influence of quality and amount of recycled coarse and fine aggregates on properties of RAC. The authors considered three qualities of recycled aggregates: recycled coarse aggregates having specific gravity 2.53 and water absorption of 1.9% as RG-I, RCA having specific gravity 2.4 and water absorption of 6.2% as RG-III and recycled fine aggregate having specific gravity 2.36 and water absorption of 5.4% as RS-II. Three replacement percentages of 30, 50, and 100 recycled coarse and fine aggregates were used in separate mixes. The authors concluded that RAC made with lower absorption of RCA (RG-I) had similar strength as that of normal concrete. However, at 1 and 3 days curing age, the compressive strength of RAC made with higher absorption of both recycled fine

(RS-II) and coarse aggregate (RG-III) had 40% and 20%, respectively, lower than that of normal concrete; In contrast, at 56 and 90 days curing, the relative strength of RAC was increased. This indicates that the longer duration of curing was more advantageous in the development of compressive strength of RAC than the concrete with natural aggregates. This may be attributed to the additional hydration of remnant unhydrated cement paste on the surface of RA (Khatib 2005). In addition, the extra water absorbed by RA might have further assisted with internal curing as a source to react with the cement. Further, the relative compressive strength of RAC to that of normal concrete decreased with the rise in water absorption of RA. Particularly, though the recycled coarse aggregate (RG-III) had higher water absorption than recycled fine aggregate (RS-II), the relative strength of RAC prepared with recycled coarse aggregate had shown higher strength than that of RAC made with fine recycled aggregate. This might be due to the presence of several microcracks between cement paste and aggregate resulted by the uneven surface of recycled fine aggregate.

Many researchers have studied the compressive strength of RAC with 100% RCA and natural sand and it was observed that the reduction in compressive strength of RAC when compared to normal concrete varied from author to author. For example, Nixon (1978) concluded that the compressive strength of RAC was lower than that of normal concrete, and in some cases, it was 20% lower compared to normal concrete. BCSJ (1978) agreed with Nixon (1978) and the authors found that the reduction was 14–32%. Based on the results obtained by Buck (1977), Malhotra (1978), Frondistou-Yannas (1977) arrived a relationship between compressive strength of normal and recycled aggregate concrete and it was reported that the compressive strength of RAC was 10% lower than that of normal concrete. Forster (1986) observed that the strength of RAC was little lower than normal concrete. However, minimum normal strengths can be achieved easily with recycled coarse aggregate.

Rao et al. (2017) conducted a detailed investigation on the compressive strength of concrete made with RCA obtained from different demolished structures (typically used in India). Figure 4.5 shows the variation in compressive strength of concrete with different percentages of RCA at different curing periods.

Irrespective of the source of RCA, it was found that the variation in compressive strength of normal concrete and RAC made with 25% RCA is almost similar from 7 days to 28 days curing period, whereas a relatively slower strength gaining rate was observed for RAC made with higher percentage of recycled coarse aggregate (50% and 100%). There is an increase of approximately 25–27%, 17–23%, 13–24% of compressive strength in recycled aggregate concrete with 50 and 100% RCA obtained from sources 1, 2, 3, respectively, when the corresponding normal concrete strength and recycled aggregate concrete strength with 25% RCA increased by approximately 40, 33–36% in the last 21 days of 28 days curing. This indicates that the recycled aggregate concrete with higher percentage of RCA (50 and 100%) has attained relatively higher strength at 7 days of curing than normal concrete. However, the strength of RAC at all percentages of RCA is lower than the normal concrete at 28 days of curing. Similar results were reported in the literature

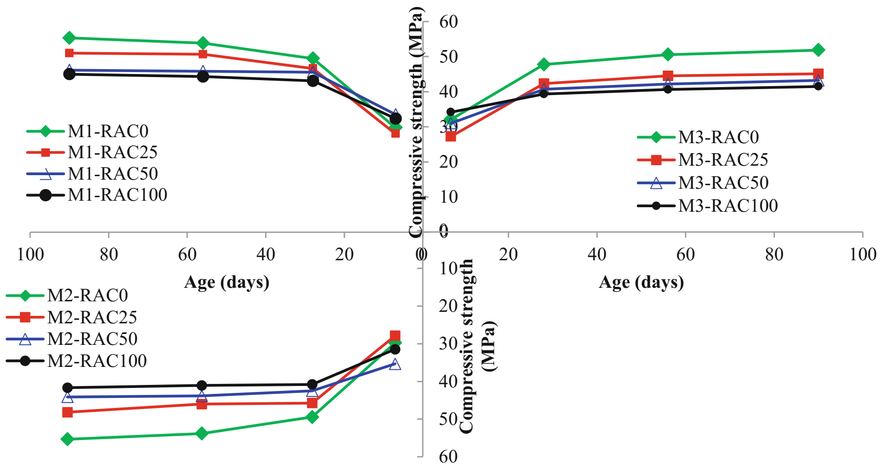

Fig. 4.5 Development of compressive strength with age in both normal and recycled aggregate concrete made with RCA obtained from different demolished structures (Rao et al. 2017)

(Etxeberria et al. 2007; Salem and Burdette 1998). It was reported in the literature that the increase in compressive strength of RAC at early age (7 days) is mainly due to high absorption capacity of old mortar adhered to the recycled aggregates and the rough texture of recycled aggregates that provide improved bonding and interlocking characteristic between the mortar and recycled aggregate themselves. After 28 days, no significant improvement was observed in compressive strength of recycled aggregate concrete made with RCA obtained from all the sources with curing age up to 90 days at 50% and 100% RCA compared to RAC with 25% RCA and 0% RCA (normal concrete). This might be due to the accumulation of cement paste on the surface of the aggregates which produces low w/c ratio and effective new interfacial transition zone (ITZ); i.e., the new ITZ of the cement paste and the aggregate has lower w/c ratio than the ITZ of the cement paste and the old mortar attached to the recycled aggregate. As a consequence, the new ITZ with aggregate becomes stronger. Similar observation is reported for curing period between 28 days and 6 months (Salem and Burdette 1998). The reduction in compressive strength of RAC at 25, 50, and 100% RCA obtained from sources 1, 2, and 3 are 5.7, 7.8, 12.9; 7.5, 14.1, 17.5, and 11.3, 14.7, 17.6%, respectively, compared to normal concrete. This indicates that 25% RCA content does not influence much on compressive strength of concrete. It was reported in the literature that up to C32/40 strength of concrete can be made by replacing 30% natural aggregate with recycled concrete aggregate (Corinaldesi 2010).

Kou and Poon (2008) studied the development of strength in RAC over five years curing with different amounts of RCA obtained from three different sources. The RCA obtained from C&D wastes were from Tseung Kwan O site (TKOS), old Kai Tak airport site (KTS) and waste obtained from Tseung Kwan O site further crushed in the laboratory (TKOL). All mixes were prepared with a constant w/c

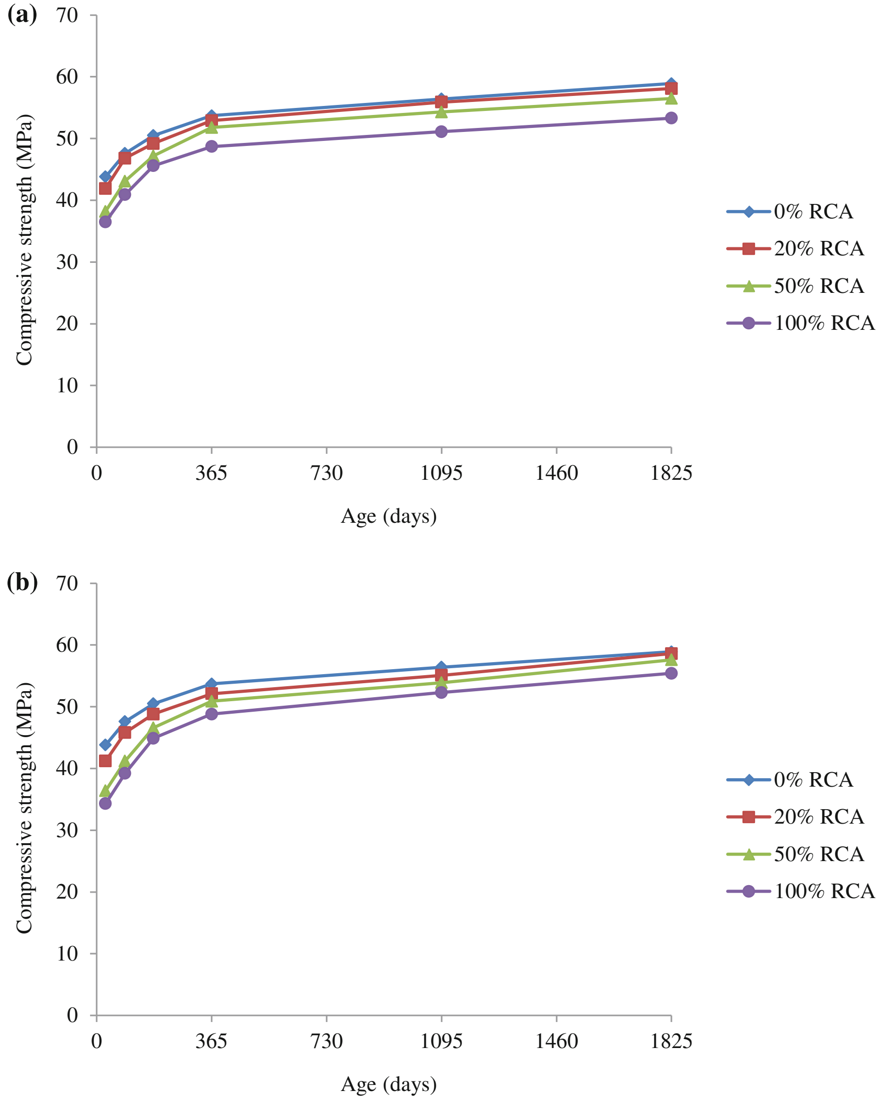

Fig. 4.6 Development of compressive strength with age in (**a**) Mixes with TKOS RA, (**b**) Mixes with KTS RA and (**c**) Mixes with TKOL RA (Kou and Poon 2008)

ratio of 0.55 and a cement content of 355 kg/m^3, and the results reported by the authors are presented in Fig. 4.6. The authors did not agree with the conclusion made by Rao et al. (2017) and Etxeberria et al. (2007), as the rate of strength gain in RAC made with all percentages (20, 50 and 100%) of RCA in their study was more after 28 days when compared to normal concrete. It was reported that the strength gain between 28 days and 5 years curing was 46–62% in RAC with 100% RCA

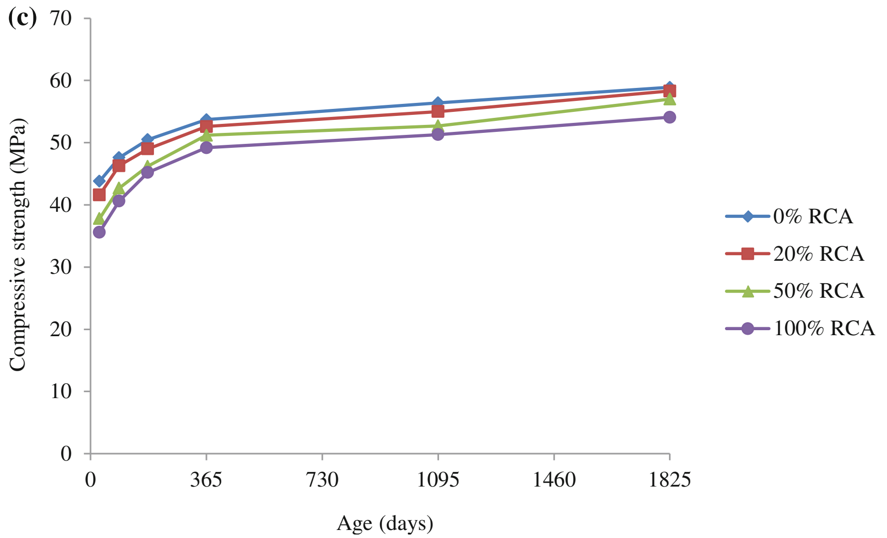

Fig. 4.6 (continued)

against 34% gain in normal concrete. However, the strength of RAC with all percentages of recycled coarse aggregate was always less than that of normal concrete and they were 17–22% and 6–11% less when RAC made with 100% RCA compared to normal concrete at 28 days and 5 years, respectively. Among all the three sources, it was reported that the concrete made with 100% RCA from TKOL had the maximum improvement in strength between 28 days and 5 years. This was due to RCA resulting from 100% crushed concrete. Further, it was stated that the RCA obtained from TKOL had a large percentage of residual cement mortar and was more porous when compared to the RCA obtained from other two sources and natural aggregate. The bond between the aggregate and cement might be improved due to the permeation of new cement hydrates into the recycled aggregate during the reutilization of the aggregate in concrete (Kou and Poon 2008).

The variation in compressive strength of concrete at 28 days with different percentages of recycled coarse aggregate reported by some of the researchers is presented in Fig. 4.7.

Figures reveal that for all coarse aggregate replacement ratios, the variation in compressive strength is almost similar except the results reported by Fonseca et al. (2011). It is further revealed that the compressive strength is decreased with the increased amount of recycled aggregate. However, 25% replacement of natural coarse aggregate by recycled aggregate does not influence much on the compressive strength of concrete.

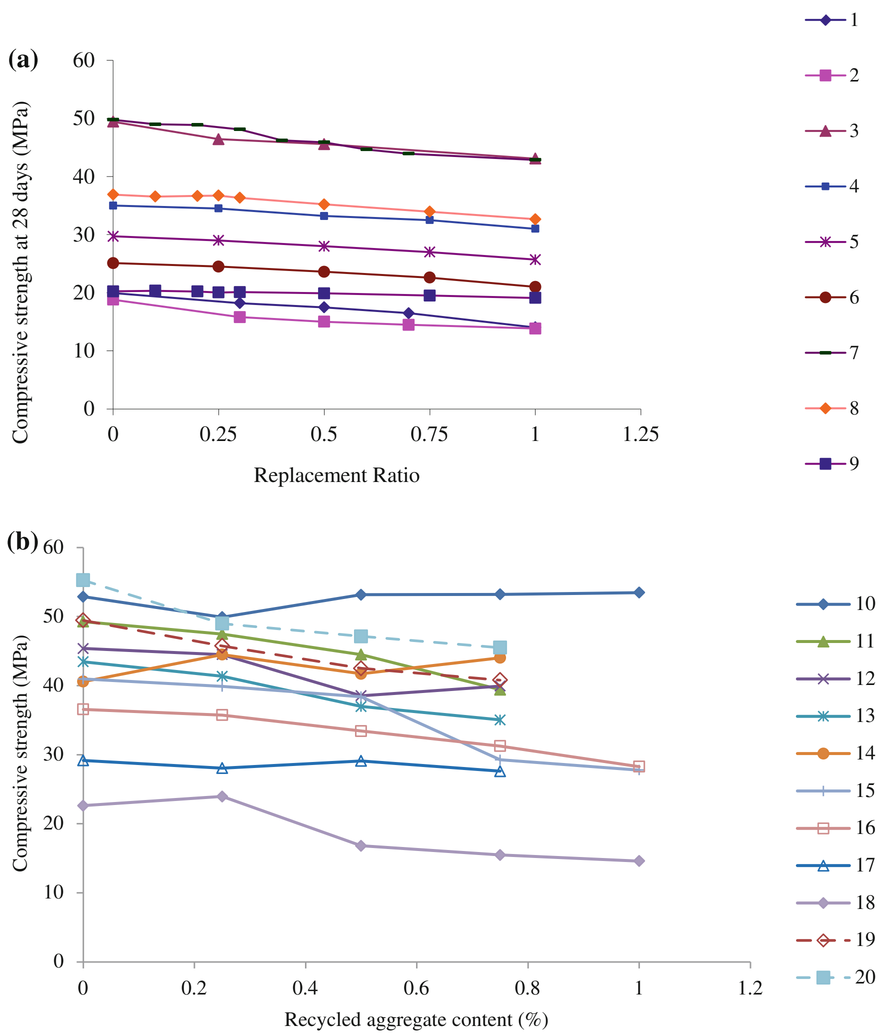

Fig. 4.7 Variation in 28 day compressive strength with percentage of RCA reported by different researchers. 1, 2 Topcu and Sengel (2004); 3 Rao et al. (2010); 4–6 Bairagi et al. (1993); 7–9 Limbachiya et al. (2004) 10 Fonseca et al. (2011); 11 Kou et al. (2012); 12-14 Poon et al. (2004); 15 Kwan et al. (2012); 16 and 18 Elhakam et al. (2012); 17 Etxeberria et al. (2007); 19 and 20 Rao et al. (2017)

4.3.1.2 Effect of Method of Curing

Like the curing period, the method of curing also affects the compressive strength of recycled aggregate concrete. Rao et al. (2017) investigated the effect of method of curing on compressive strength of recycled aggregate concrete made with different sources of RCA. The authors considered two types of curing methods viz. moist

Table 4.5 Effect of method of curing on compressive strength (Rao et al. 2017)

Method of curing	RCA (%)	Compressive strength (MPa)		
		Source 1	Source 2	Source 3
Wet curing	25	46.63	45.75	48.97
	50	45.58	42.5	47.12
	100	43.08	40.08	45.50
Partial wet curing	25	47.3	47.33	49.20
	50	46.83	44.25	50.53
	100	45.83	43.00	49.83

curing and partial wet curing; i.e., samples were initially cured in water for 7 days after demoulding and then kept in air till the 28 days testing. The test results of compressive strength of recycled aggregate concrete made with different percentages of recycled coarse aggregates cured in both continuously wet condition and partially wet condition are presented in Table 4.5. It indicates that when it is air dried after 7 days of wet curing, the compressive strength of recycled aggregate concrete for all coarse aggregate replacement percentages are more than those of wet cured till the testing age. In addition, the difference in compressive strength between wet cured and partial wet cured specimens increased with the increase in percentage of recycled coarse aggregate.

There is 6.5–9.5% increase in compressive strength of recycled aggregate concrete at 100% RCA when the same is air dried instead of wet cured. In case of recycled aggregate concrete, when the saturated-surface-dry (SSD) aggregate used, the water inside the recycled aggregate particles may move towards the bulk cement paste, creating relatively a local w/c ratio in the vicinity of the particles. This process may weaken the bond between the recycled aggregate and the cement matrix. During continuous moist curing condition, the local w/c ratio may further increase due to high absorption capacity of RCA and this further weakens the ITZ. Therefore, the strength of RAC in continuous wet curing condition is relatively lower than those in partial wet curing condition. The compressive strength of recycled aggregate concrete made with air-dried recycled aggregate had better strength than that of saturated-surface-dry recycled aggregate due to an improvement in the old ITZ quality is reported in the literature (Poon et al. 2004).

4.3.1.3 Effect of Strength of Parent Concrete

The amount adhered mortar on recycled aggregate mainly depends on the strength of parent concrete from which they derived. The adhered mortar is porous in nature which influences the properties like water absorption, density, porosity of recycled aggregate, and hence the compressive strength of recycled aggregate concrete. Hansen and Narud (1983) have studied the high-, medium-, and low-strength recycled aggregate concrete made with RCA obtained from high-, medium-, and low-strength concrete, and the results reported by the authors are presented in Table 4.6.

Table 4.6 Compressive strength after 14 days of standard curing and 38 days of accelerated curing of both RAC and original concrete (Hansen and Narud 1983)

Series	Curing regime	Compressive strength of original and recycled aggregate concrete (MPa)											
		H	H/H	H/M	H/L	M	M/H	M/M	M/L	L	L/H	L/M	L/L
1	14 days normal	49.5	54.4	46.3	34.6	26.2	27.7	27	23.2	9.1	10.2	10.3	9.6
	38 days accelerated	56.4	61.2	49.3	34.6	34.4	35.1	33.0	26.9	13.8	14.8	14.5	13.4
2	14 days normal	51.2	50.7			25.2		27.1		8.0			8.5
	38 days accelerated	61.2	60.7			36.0		36.2		14.5			13.6

*Symbols H, M, L represent original high-, medium-, and low-strength concretes made with natural gravel. Symbol H/M represents a high-strength recycled concrete made with recycled aggregate produced from medium-strength concrete, etc.

The authors concluded that the strength of RAC depends on the strength of parent concrete from which the recycled aggregate derived. That is the w/c ratio of the original concrete controls the strength of RAC when all other factors were essentially identical. When the w/c ratio of original concrete from which the recycled aggregates derived was the same or lesser than the w/c ratio of RAC, then the strength of RAC was the same or even higher than the strength of original concrete and vice versa. In addition, the authors concluded that higher strength of RAC than strength of original concrete from which recycled aggregates derived could be produced with higher cement content than that was used in normal concrete. Tavakoli and Soroushian (1996) studied the influence of two field demolished concrete as coarse aggregates on strength of RAC. The authors concluded that the strength of RAC made with RCA was higher than normal concrete when the strength of original concrete from which the recycled aggregates derived is higher than the strength of normal concrete. This was particularly true at lower w/c ratio.

Padmini et al. (2009) examined the influence of parent concrete made with different maximum sizes of coarse aggregate (10, 20 and 40 mm) of different strengths as recycled coarse aggregate on strength of RAC. It was concluded that the RAC needs lower w/c ratio than the parent concrete from which the recycled coarse aggregate derived to achieve a particular compressive strength and the difference in strength between RAC and parent concrete increased with higher strength. This means the presence of adhered mortar does not have significant effect on lower strength of RAC. In addition, the authors concluded that for a given target mean strength, with an increase in maximum size of RCA the achieved strength increased.

Pedro et al. (2014) investigated the target compressive strength of low- (20 MPa), intermediate- (45 MPa) and high-strength (65 MPa) recycled aggregate concrete using 100% RCA derived from two different sources, i.e., laboratory tested samples and precast rejected elements of the same target strengths (20, 45, 65 MPa). The recycled aggregates were subjected to two types crushing processes viz. primary crushing, impact crusher, and primary plus secondary crusher using an impact crusher followed by a hammer mill. It was reported that in lower strength RAC, the reduction in compressive strength was more. It was found 3.0–8.1, 3.2–7.6, and 9.0–17.7% reduction in 65, 45, and 20 MPa RAC mixes, respectively, compared to their corresponding normal concrete. The higher loss occurred in lower strength RAC family might be justified due to the lower quality of recycled aggregate used. In case of normal concrete, there was only one interface, i.e., interface between the aggregate and the cement mortar, whereas in RAC, there are two interfaces, i.e., one interface between cement mortar and recycled aggregate and the other between old adhered mortar and the recycled aggregate. These interfaces greatly ailments the behavior of concrete (Guedes et al. 2013). With respect to these interfaces, further it was reported that the greater loss in 20 MPa concrete mixes looks to be caused by the fact that the failure occurred at the interface between old cement mortar and recycled aggregate or through the mortar itself against the failure of ITZ between RA and new cement mortar in case of RAC mixes made with better a quality of RA.

Kou and Poon (2015) investigated the influence of RA obtained from 30, 45, 60, 80, and 100 MPa strength parent concretes on compressive strength of 45 MPa and 65 MPa of recycled aggregate concretes. The authors prepared two series of RAC mixes: Series I (compressive strength of 45 MPa with w/c 0.50) and Series II (compressive strength of 65 MPa with w/c 0.35) each with RA derived from 30, 45, 60, 80, and 100 MPa strength parent concretes. The development of compressive strength with age for both the series of mixes is presented in Figs. 4.8 and 4.9. It was found that the compressive strength of both series of RAC mixes prepared with 30 and 45 MPa strength parent concrete aggregates was much lower than the corresponding natural aggregate concrete mixes at all curing periods, whereas in RAC mixes prepared with RA from 80 MPa and 100 MPa strength parent concretes, the 28 days compressive strength was the same or even slightly more when compared to natural aggregate concrete. Further, it was reported that in Series I, the

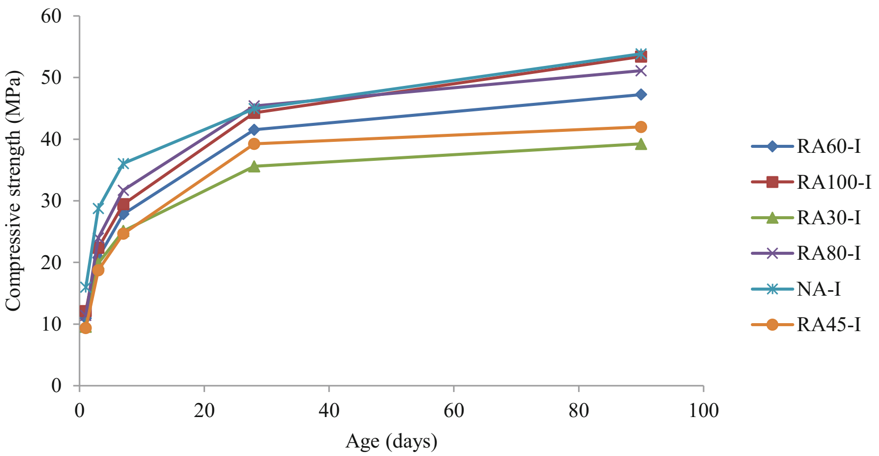

Fig. 4.8 Development of compressive strength with age in Series I mixes (Kou and Poon 2015)

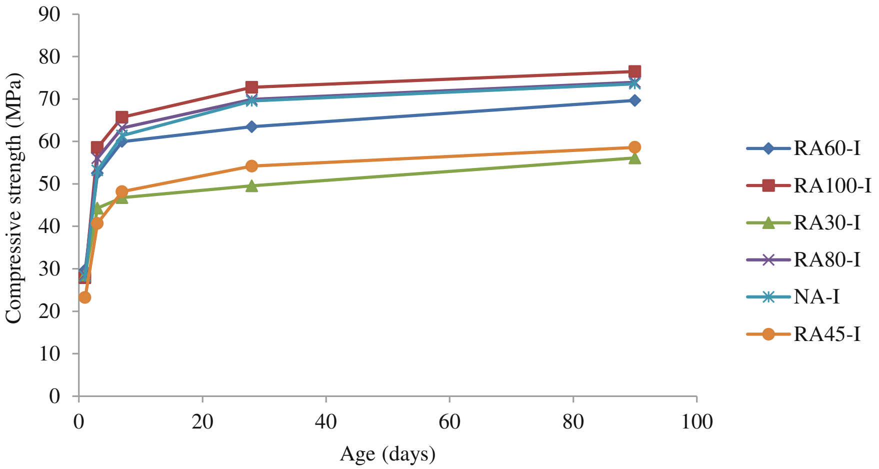

Fig. 4.9 Development of compressive strength with age in Series II mixes (Kou and Poon 2015)

compressive strength of RAC mixes made with RA derived from 30, 45, and 60 MPa were 21.1, 12.6, and 8.6%, respectively, lower than that of concrete prepared with natural aggregate. Whereas, only 1.1 and 0.4% reduction in compressive strength occurred in RAC mixes prepared with 80 MPa and 100 MPa parent concrete aggregates respectively, compared to natural aggregate concrete. Similarly in Series II, the compressive strength of RAC mixes made with RA obtained from 30, 45, and 60 MPa were 29.2, 22.5, and 9.4%, respectively lower when compared to their corresponding natural aggregate concretes. However, in mixes made with 80 MPa and 100 MPa strengths of concrete aggregates, the compressive strength was equal to or 3.9% more than that of the concrete with natural aggregate. It was reported that this might be ascribed to (i) the RA obtained from 80 and 100 MPa had higher original strength; and (ii) the enhanced hydration process by the internal water curing due to the inclusion of recycled aggregate.

In fact, the entrained water from RA might contribute to the creation and development of extra C-S-H in the capillary pores. Further, its refinement, filling, and transmutation of coarse capillary pores into smaller pores may result the noticed compressive strength enhancement (Kou and Poon 2015)

Rao (2016) examined the influence of normal- and medium-strength parent concrete (PC) aggregates on the compressive strength of recycled aggregate concrete. The variation of compressive strength with reference to curing period in both normal- and medium-strength parent concretes and recycled aggregate concretes is presented in Fig. 4.10. It was observed that the compressive strength attained in normal-strength parent concretes at 3 and 7 days curing period was ranging from 50–55% and 68–70%, respectively, than those of 28 days compressive strength. In medium-strength PC, the strength attained at 3 and 7 days curing was in the order of 59–60% and 73–78% of that of 28 days compressive strength, respectively. In normal- and medium-strength RAC, the strength development at 3 and 7 days were

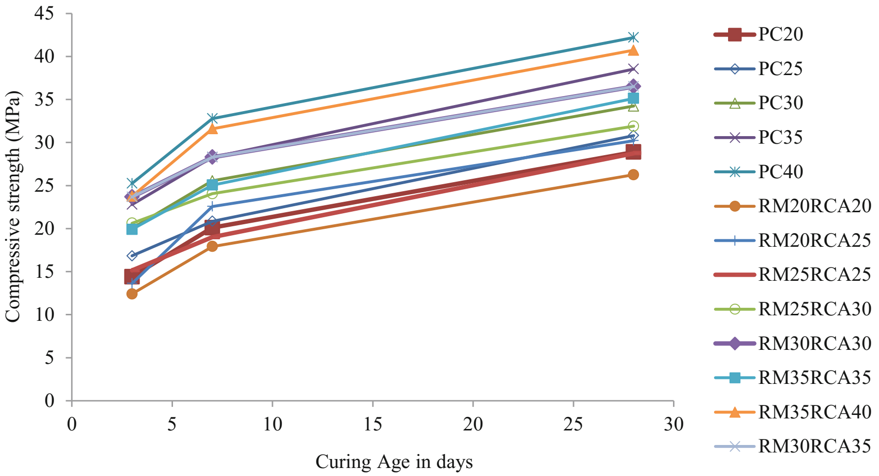

Fig. 4.10 Development of compressive strength of both parent concretes and recycled aggregate concretes with curing period (Rao 2016). PC parent concrete; RM recycled mix; Next number 28 day compressive strength; RCA recycled coarse aggregate; Last number strength of PC from which RCA derived

45–53% & 66–75% and 53–65% & 72–80%, respectively, of those of corresponding 28 days strength. These developments of strength in RAC were in tune with the strength development in normal concrete at all curing periods. In general, the strength of normal concrete at 7 days curing period is approximately 60–70% of that of 28 days compressive strength.

Figure 4.11 shows the 28 day cube compressive strength results of both normal- and medium-strength recycled aggregate concrete. It was found that the cube compressive strength of RAC prepared with RCA obtained from the same strength of PC is lower than those of corresponding parent concrete in both normal-strength and medium-strength concretes. Normal-strength RAC with RCA from 20 & 25 MPa and medium-strength RAC with RCA from 30 MPa & 35 MPa families attained 26.26 & 28.75 MPa and 29.32 & 35.13 MPa, respectively, against 28.89 & 30.81 MPa and 34.23 & 38.54 MPa of their corresponding parent concretes. Figure 4.12 presents the percentage of 28 days compressive strength of RAC prepared with RCA from different strengths of PC to that of concrete with natural aggregates. The figure reveals that the RAC with RCA from 20 MPa, 25 MPa, 30 MPa, and 35 MPa PCs lowered by 9.1, 6.69, 14.34, and 8.85%, respectively, than those of corresponding parent concrete with natural aggregates. Pedro et al. (2014) were reported that 9.0–17.7%, 3.2–7.6%, and 3.0–8.1% reduction in compressive strength of RAC with RCA from 20 MPa, 45 MPa, and 65 MPa laboratories made concrete mixes and precast concrete cores, respectively, than those of corresponding normal concrete with natural aggregates. These reductions were significantly lower in 30 and 45 MPa RAC mixes than those of normal concrete mixes (Kou and Poon 2015). It was reported that the 28 day compressive strength

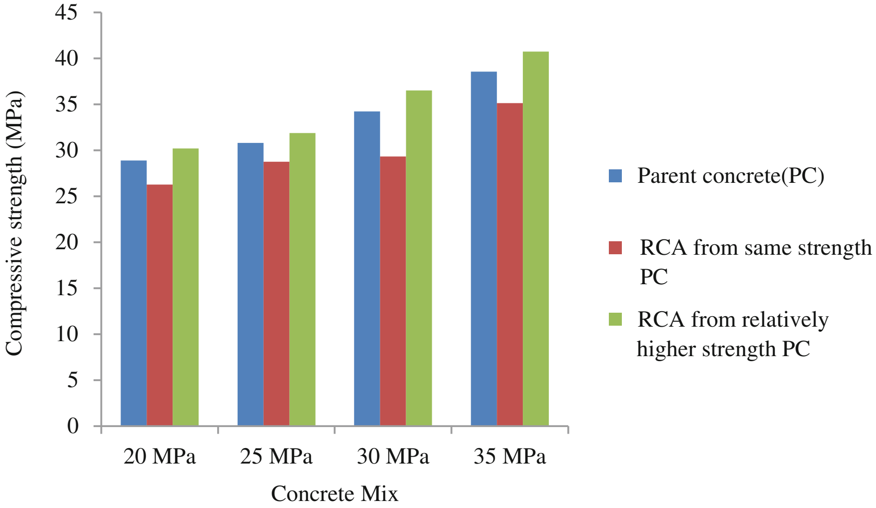

Fig. 4.11 28 day compressive strength of cubes in different concrete families (Rao 2016)

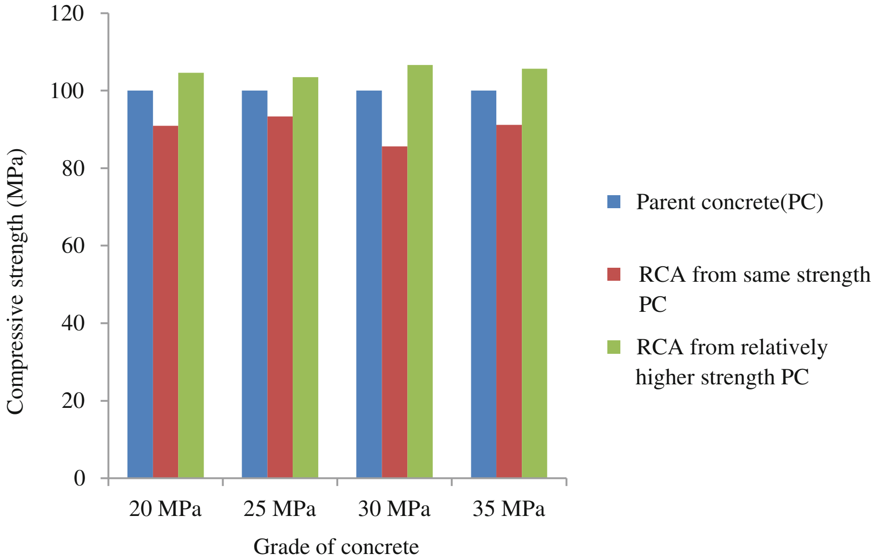

Fig. 4.12 Percentage of compressive strength of RAC to that of normal concrete at 28 days curing period (Rao 2016)

of RA30 (30 MPa), RA45 (45 MPa) and RA60 (60 MPa) with 100% recycled coarse aggregate losses 21.1, 12.6, and 8.6%, respectively, than that of natural aggregate concrete. These reductions may be attributed to the presence of old cement mortar in RCA, which results an increased porosity and weak interfacial bond between the mortar and the aggregate.

The development of compressive strength in both normal- and medium-strength RAC with RCA from relatively higher-strength parent concrete is more than those of corresponding parent concrete with natural aggregate at all curing periods. The RAC with RCA from relatively higher-strength PCs, i.e., RM20RCA25, RM25RCA30, RM30RCA35, and RM35RCA40, has attained the compressive strength at 28 days curing period are 30.21, 31.89, 36.5, and 40.73 MPa, respectively, against 28.89, 30.81, 34.23, and 38.54 MPa of corresponding concretes with natural aggregate. There was approximately 3.5–4.7 and 5.7–6.7% increase in compressive strength of normal-strength and medium-strength RAC when they made with RCA obtained from relatively higher-strength PC than corresponding natural aggregate concretes.

The test results of recycled aggregate concrete reveal that the compressive strength of recycled aggregate concrete made with RCA obtained from the same strength of parent concrete was always lower than that of the parent concrete at all the curing periods, whereas in RAC made with RCA produced from relatively higher-strength parent concrete, the compressive strength at 28 days testing was slightly higher than that of normal concrete. That means the RAC made with relatively higher strength of parent concrete aggregate may produce similar strength as that of normal concrete of the same strength. The improvement in compressive strength might be due to relatively higher strength (bond) of cement mortar adhered to aggregate in RCA obtained from higher-strength PC compared to RCA obtained from the same strength PC and hence the old interfacial transition zones in RM20RCA25, RM25RCA30, RM30RCA35, and RM35RCA40 are relatively stronger than those of RM20RCA20, RM25RCA25, RM30RCA30, and RM35RCA35, respectively. A similar result was reported in the literature. Kou and Poon (2015) reported that the compressive strength of 45 MPa RAC made with recycled aggregate obtained from 80 MPa and 100 MPa normal concrete attained the same or even slightly higher than that of the concrete with natural aggregate. This may be attributed to RCA obtained from 80 MPa and 100 MPa concrete which have higher original strength, and by the addition of RCA, there may be improvement in the hydration process due to internal curing leads to the formation of additional C-S-H in capillary pores. Hence, the coarse capillary pores transform into smaller ones which may lead to the improvement in compressive strength.

4.3.1.4 Effect of W/C Ratio

In a study by Rasheeduzzafar and Khan (1984), it was reported that the w/c ratio below 0.4 has no significant improvement on compressive strength of RAC. At lower w/c ratios, the interfacial transition zone (ITZ) between old mortar and

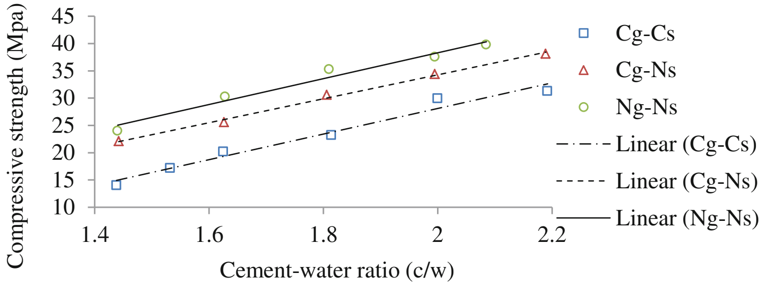

Fig. 4.13 Relationship between cement–water ratio and compressive strength of concretes made with natural and recycled aggregates (Mukai et al. 1978). Cg-Cs Coarse and fine recycled aggregate, Cg-Ns Recycled coarse aggregate and natural sand, Ng-Ns Natural gravel and natural sand

aggregate became the weaker link which controls the failure. In addition, it was reported that 30% reduction in strength in RAC compared to normal concrete at typical 0.35 w/c ratio. Further, it was observed that with the increase in w/c ratio, the difference in strength between normal and recycled aggregate concrete was decreased up to a w/c ratio of 0.55 and thereafter the strengths were equal, whereas Ravindrarajah et al. (1988) found that the strength of both RAC and normal concrete continuously decreased with the increase in w/c ratio and the difference in strength between normal and recycled aggregate concrete was more at lower w/c ratios. Ryu (2002) in his study concluded that the quality of RCA does not affect the strength of RAC at low w/c ratio. However, at high w/c ratio, the strength of RAC depends on the quality of RCA. That means the strength of RAC depends on the relative quality of old and new interfaces between aggregate and mortar. Mukai et al. (1978) in their investigations found an excellent relationship between the free cement–water ratio and compressive as well as tensile strengths of RAC made with both recycled coarse aggregate and natural and recycled fine aggregate as well (Fig. 4.13). The basic water–cement ratio law which is applicable to all concrete mix designs was applied to all types of RAC without any modifications.

Tavakoli and Soroushian (1996) observed that at lower w/c ratio, the strength of RAC made with recycled coarse aggregate had higher compressive strength than normal concrete. At the same water–cement ratio, the compressive strength of RAC may be lower than normal concrete.

4.3.1.5 Effect of Moisture State of RA

De Oliveira and Vazquez (1996) studied the influence of retained moisture in RCA on strength of RAC, it was concluded that the strength of RAC made with dry and saturated recycled aggregates were little lower when compared to normal concrete. Katz (2003) studied the influence of partially hydrated concrete as aggregate on properties of RAC. The authors adopted RCA crushed from 1, 3 and 28 days strength

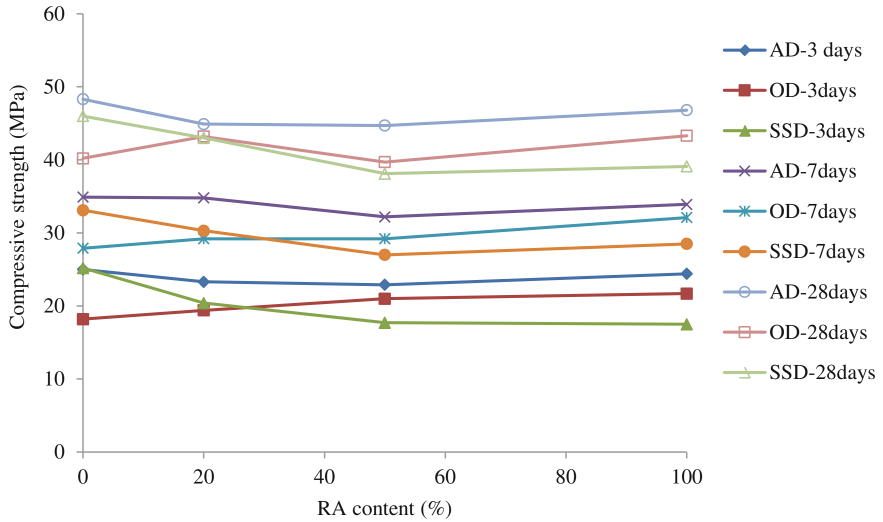

Fig. 4.14 Compressive strength of concrete mixes with different proportions of RA at various moisture conditions of aggregates (Poon et al. 2004)

concrete in making RAC with white Portland cement (WPC) and ordinary Portland cement (OPC) separately. It was concluded that irrespective of the crushing age, the strength of RAC was lower than that of normal concrete with both WPC and OPC cements. However, in WPC mixes, the RAC made with 3 days crushed aggregates showed better strength than those with 1 and 28 days crushed aggregates, whereas in OPC, the crushing age effect on strength of RAC was smaller. The influence of moisture conditions of recycled aggregate on properties of RAC reported by Poon et al. (2004) is presented in Fig. 4.14. It represents that the compressive strength of concrete mixes made with crushed granite aggregate in OD condition was much lower than that of concrete made with AD and SSD crushed granite aggregate. But this situation was different when the recycled aggregate adopted in the concrete mixes. When the crushed granite aggregate replaced with recycled aggregate, looks to have a positive effect on concrete mixes prepared with the OD aggregates but, a negative effect on concrete mixes made with SSD aggregates. It shows that with the increase in the percentage of RA, the strength of concrete mixes prepared with AD aggregates is almost constant at all testing periods, whereas the strength of RAC mixes prepared with SSD aggregates decreased while the strength of OD mixes increased with the increased percentage of RA.

It was found that the concrete prepared with air-dry (AD) recycled aggregate had higher strength than those with oven-dry (OD) and saturated-surface-dry (SSD) recycled aggregates. This was due to the movement of water from bulk cement matrix toward the recycled aggregates and cement particles may accumulate around the RCA; thereby, stronger bond may be formed between the cement mortar and aggregate particularly at early age. In contrary, it was reported that the concrete made with SSD RCA had negative effect on strength, which might be attributed to the bleeding of excess water in the pre-wetted aggregates in fresh concrete. A relatively high local w/c ratio giving higher porosity in the locality of aggregates might be developed due to the movement of water towards the cement matrix from the inside of RA during the vibration. This process can abate the bond between the cement matrix and the recycled aggregate. In addition, the authors suggested that the RAC prepared with 50% AD recycled aggregate was optimum for normal strength. Mefteh et al. (2013) reported similar results when the authors studied the concrete mixes prepared with dry, pre-wetted and SSD states of recycled aggregates. The compressive strength results reported by authors for various replacement of NA by RA are presented in Fig. 4.15.

It was shown in the figure that the mixes prepared with 20 and 40% RCA in dry state had slightly higher strength than that with natural aggregate at both 7 and 28 days. This increase may be due to the recycled aggregates having higher absorption capacity. However, the compressive strength decreased with the inclusion of 60–100% RCA in the concrete mixes.

In addition to the above crushing methodology also influence the compressive strength of RAC. Rahal (2007) studied the influence of recycled coarse aggregate on different target cube strengths ranging from 20–50 MPa. The authors also studied the development of strength with age. It was concluded that the target strength was achieved in all mixes except in 40 and 50 MPa target strength mixes, where the average strengths were 1.4 and 7%, respectively, lower than the target strength. It was reported that the strength of RAC for all mixes at 28 days was around 10% lower than that of normal concrete. In addition, the authors reported that the strength development in all RAC mixes similar to the normal concrete. However, relatively slower strength gaining rate in RAC compared to normal concrete in the first 7 days of 28 days strength curing. Tabsh and Abdelfatah (2009) concluded that the percentage loss in compressive strength of RAC made with recycled coarse aggregate was more significant in case of weaker concrete mixes than in stronger mixes. Rashwan and Abourzk (1997) studied the influence of crushing age on the strength of RAC. The authors concluded that the strength of RAC made with RCA obtained after 24 h crushing gives 25% higher than normal concrete. However, RAC made with RCA that remained in stockpiling for 7 days after crushing produces 7% lower strength than the concrete made with natural aggregate. The authors suggested that the strength of RAC improved if the early age crushed concrete used as RCA. In a study by Sagoe-Crentsil et al. (2001), it was concluded that not much significant difference was observed in strength of RAC made with commercially produced RCA and normal concrete made with natural basalt aggregate when the volumetric mixture proportions and workability were similar.

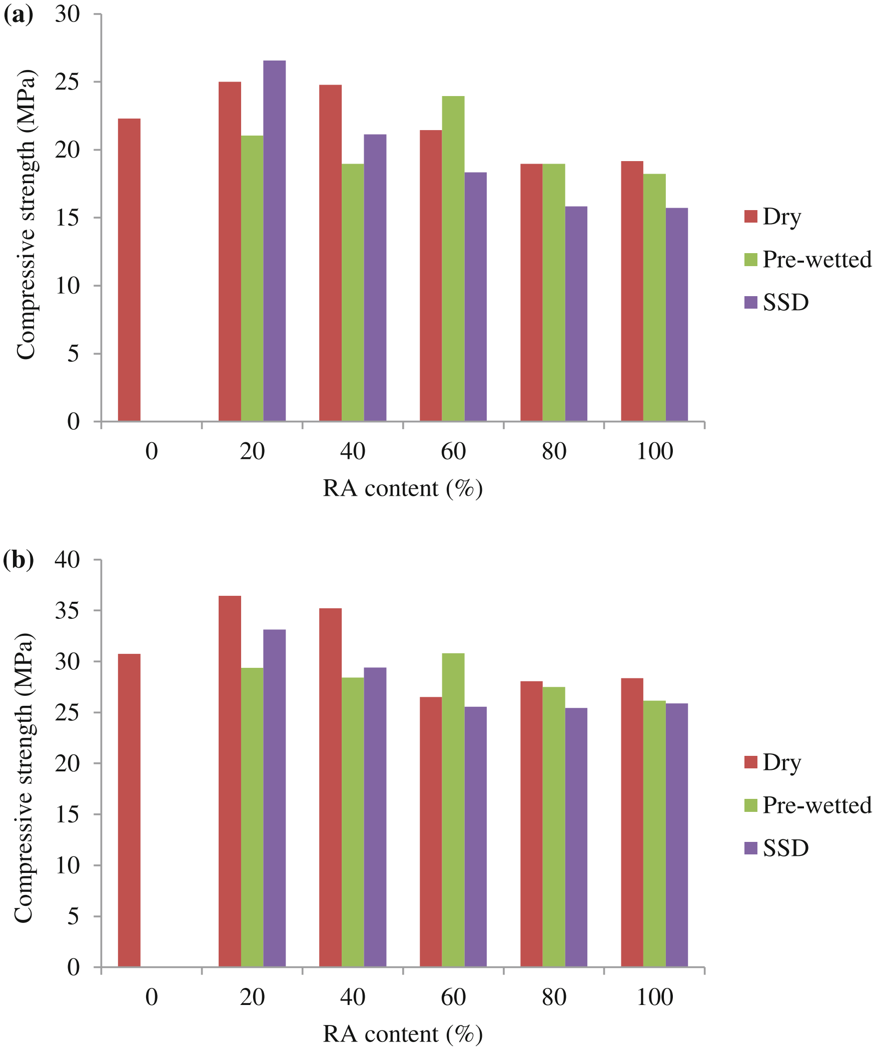

Fig. 4.15 Effect of recycled aggregate at different moisture conditions on compressive strength of concrete mixes (**a**) at 7 days and (**b**) 28 days (Mefteh et al. 2013)

4.3.2 Static Modulus of Elasticity

The stiffness of concrete is mainly indicated by the modulus of elasticity which is another important mechanical property of concrete. Recycled aggregate is a natural aggregate adhered with old cement mortar. Therefore, the recycled aggregates are loose and porous in nature. This decides the stiffness of the bulk cement paste. Because of this, the parameters such as the porosity and cement mortar, nature of

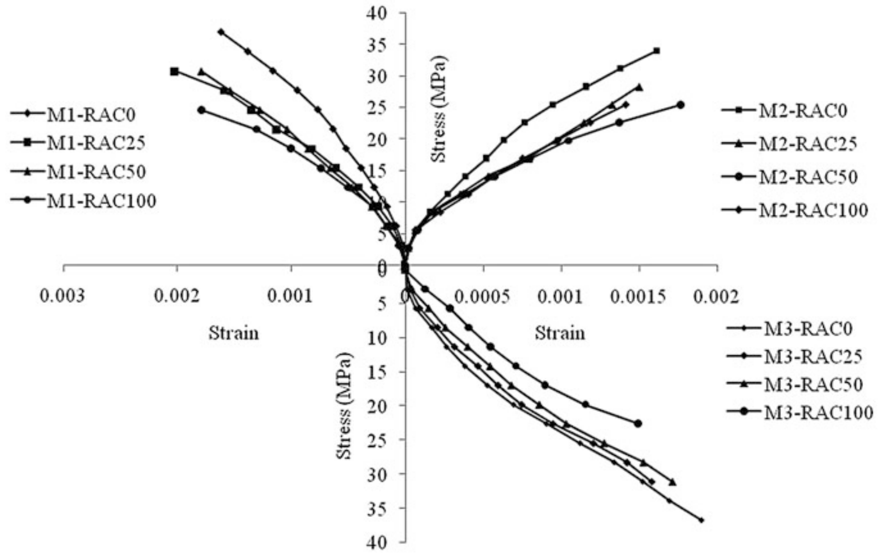

Fig. 4.16 Stress–strain variation of RAC for different percentages of RCA obtained from Sources 1, 2, and 3 (Rao et al. 2017)

aggregate, and the interfacial transition zone characteristics affect the modulus of elasticity of concrete. The modulus of elasticity also affected by the amount of natural aggregate replaced by recycled aggregate. Unlike the natural aggregate concrete, the recycled aggregate concrete has two ITZs: one the ITZ between the old cement mortar and natural aggregate and the other between the new cement mortar and recycled aggregate. Due to these reasons, the content of RA has more severe effect on modulus of elasticity than the compressive strength (Behera et al. 2014). The modulus of elasticity has followed the similar trend as the compressive strength with the percentage of substitution of NA by RA in concrete. The modulus of elasticity of recycled aggregate concrete was lower than that of normal concrete due to the lower modulus of old cement mortar adhered to recycled aggregate.

Rao et al. (2017) investigated the effect of RCA from different demolished old structures on the static elastic modulus of RAC. The authors considered 0, 25, 50 and 100% RCA in the mixes. The deformation characteristics of RAC mixes are presented in Fig. 4.16.

It reveals that the trend is almost similar for all the percentages of recycled coarse aggregate obtained from all the three sources. In addition, in case of recycled aggregate concrete, at an applied stress the strain increases at a faster rate than normal concrete. Therefore, the curvature of the stress–strain curve continues to increase in case of RAC. This may be due to the presence of interfaces between old cement mortar and aggregate, new cement mortar and old cement mortar, and new cement mortar and aggregates, which help crack propagation during loading. The

Table 4.7 Test results of static modulus of elasticity of both normal and recycled aggregate concretes (Rao et al. 2017)

Source of RCA	Mix designation	RCA (%)	Modulus of elasticity (GPa)	Reduction in modulus of elasticity of RAC w.r.t. normal concrete (%)
Source 1: RCC culvert near Medinipur	M1-RAC0	0	31.220	–
	M1-RAC25	25	23.567	24
	M1-RAC50	50	21.536	31
	M1-RAC100	100	20.350	35
Source 2: RCC culvert near Kharagpur	M2-RAC0	0	31.220	–
	M2-RAC25	25	26.751	14
	M2-RAC50	50	26.710	14
	M2-RAC100	100	26.408	15.5
Source 3: RCC slab of an old residential building near Vizianagaram	M3-RAC0	0	32.101	–
	M3-RAC25	25	25.177	21
	M3-RAC50	50	24.071	25
	M3-RAC100	100	22.024	31

test results of static modulus of elasticity of both normal concrete and RAC made with different percentages of RCA are presented in Table 4.7.

It reveals that the modulus of elasticity of RAC decreased with the increase in coarse aggregate replacement percentage in all the sources of mixes. In addition, it was observed that the reduction in modulus of elasticity of RAC in sources 1 and 3 mixes was relatively more than the reduction in source 2 mixes when compared to normal concrete. This is expected due to the fact that the recycled coarse aggregates were more prone to deformation due to the presence of porous mortar. Padmini et al. (2009) reported a similar result in the literature. In addition, the authors reported that the presence of higher porosity of smaller particles further reduces the modulus of elasticity of RAC. This aspect is also expected because of the lower modulus of elasticity of recycled aggregates than natural aggregates (Ravindrarajah and Tam 1985), and in addition, it is well known that the modulus of concrete depends on the modulus of the aggregates (Neville 2006) and stiffness of a coarse aggregate has a significant effect on the modulus of high-performance concrete (HPC) (Suzuki et al. 2009). The modulus of elasticity of concrete is proportional to its compressive strength and an appropriate granular fractions combination has a significant influence on the modulus of elasticity (Meddah et al. 2010).

Corinaldesi (2010) has observed that the modulus of elasticity of RAC was lowered by 15 at 30% substitution of RA. Pereira et al. (2013) reported that at 30% replacement level, the modulus of elasticity of RAC remained unaffected, whereas Kou and Poon (2013) reported that the modulus of elasticity of RAC was 12.6 and 25.2%, respectively, lower at 50 and 100% RA substitution than conventional concrete. Casuccio et al. (2008) found that the modulus of elasticity of RAC reduced with the inclusion of RA in concrete. This was due to that the RA increases the brittleness and reduces the stiffness of concrete.

Frondistou-Yannas (1977) observed that the modulus of elasticity of RAC made with recycled coarse aggregate and natural sand was about 40% lower than that of concrete made with natural aggregate. Rasheeduzzafar and Khan (1984) reported that the modulus of elasticity of RAC was 18% lower than that of normal concrete and this difference was decreased with the increase in w/c ratio. Hansen and Boegh (1986) studied high-, medium-, and low-strength of RAC made with RCA obtained from high-, medium-, and low-strength concrete. It was observed that the reduction in static and dynamic modulus of elasticity of RAC was 15–30% lower when compared to parent concrete from which the recycled aggregates derived. This reduction was increased to 50% when the high-strength RAC made with RCA obtained from low-strength RAC, this being an agreement with Frondistou-Yannas (1997). Bairagi et al. (1993) and Topcu and Guncan (1995) observed that the modulus of elasticity of RAC was 80% and 71% of the normal concrete, respectively. It was found from the failure pattern of RAC that the recycled aggregate concrete performs in a more brittle way than natural concrete. De Oliveira and Vazquez (1996) reported that the modulus of elasticity of RAC was always less than that of normal concrete. Katz (2003) in a study concluded that the modulus of elasticity of RAC made with RCA obtained from different curing ages of concrete was lower than that of normal concrete. Belen et al. (2011) reported that the recycled aggregate has hostile effect on both transverse and longitudinal elastic modulus of concrete. This is mainly responsible for the increase in the ultimate stain of recycled aggregate concrete which leads to higher amount of deformations.

Xiao et al. (2005) reported that the elastic modulus decreased with the increased percentage of RCA and at 100% RCA, the modulus of elasticity of RAC was 45% lower than that of normal concrete due to the old attached mortar on the surface of aggregate in recycled aggregate, the modulus of elasticity of it is comparatively lower than the natural aggregate. Prasad and Kumar (2007) concluded that the modulus of elasticity of both normal and recycled aggregate concrete was improved with the addition of glass fibers. However, the modulus of elasticity of RAC was lower than that of normal concrete with and without the addition of glass fibers. Etxeberria et al. (2007) observed that the modulus of elasticity of RAC decreased with the increase in percentage of RCA and always it was less than that of normal concrete. In a study by Gonzalez-Fonteboa and Martinez-Abella (2008), the modulus of elasticity of RAC was less than that of normal concrete was reported. No improvement was observed in modulus of elasticity even after 28 days by the addition of silica fume in RAC mixes.

Kou and Poon (2008) studied the influence of different amounts of RCA obtained from three different sources on modulus of elasticity of RAC over 5 years curing period. It was observed that the modulus of elasticity of RAC was decreased with the increase in percentage of RCA and it was always less than normal concrete. However, with the increased curing period the modulus of elasticity of RAC was improved. The gain in modulus of elasticity in RAC at 100% RCA was 33–41% against 20% in normal concrete between 28 days and 5 years curing period. Kou et al. (2008) concluded that the modulus of elasticity of RAC decreased with the increased percentage of recycled coarse aggregate (0–100) with all w/c ratios ranging from 0.4 to 0.55. In addition, it was reported that the modulus of elasticity was improved with the addition of fly ash as a partial replacement to cement. Yang et al. (2008) in their study compared the modulus of elasticity of RAC made with both recycled coarse and fine aggregates of different quality with the equations given by ACI 318-05 for normal concrete. The authors concluded that when the water absorption was above 3%, the modulus of elasticity of RAC is lower than those specified by American Concrete Institute (ACI) for normal concrete.

Kou et al. (2012) reported that the modulus of elasticity of RAC made with low-grade RA was significantly lower than conventional concrete. Further, it was reported that the elastic modulus of RAC prepared with commercially produced RA was more than that of RAC produced with low-grade recycled aggregate. This was expected as the recycled aggregates are more susceptible to deformation than natural aggregate. This result was anticipated due to the lower modulus of elasticity of low-grade recycled aggregate than natural and commercially produced recycled aggregate, and further, it is well known that the modulus of aggregates significantly affects the elastic modulus of concrete (Neville 2006; Etxeberria et al. 2007). A similar result was reported by Duan and Poon (2014) that at 28 and 90 days curing, the modulus of elasticity of RAC prepared with RA having higher amount of adhered mortar was lower than those prepared with RA having low amount of attached mortar and natural aggregate. This was further proved that the RA having lower amount of adhered mortar and natural aggregate had better mechanical properties (TFV and ACV), which can produce even high-strength concrete. Limbachiya et al. (2012) found that the modulus of elasticity of RAC was marginally decreased with the increased percentage of RCA for a given concrete mix and age. Further, the modulus of elasticity of concrete was closely associated with the aggregate skeleton rather than the cement paste. The lower values of modulus of elasticity of RAC might be due to the weaker strength characteristics of RCA than the natural aggregate.

Poon et al. (2006) investigated the influence of method of curing (steam curing and normal curing) on static modulus of elasticity of RAC with different proportions of RCA (0, 20, 50 and 100%). In their study the authors considered two series of mixes: Series I mixes were prepared with a w/c ratio of 0.55 and a cement content of 410 kg/m^3 and in Series II mixes, the w/c ratio was 0.45 and 400 kg/m^3 cement. The modulus of elasticity of all the mixes at w/c ratio 0.45 and w/c ratio 0.55 at 28 and 90 days curing is presented in Figs. 4.17 and 4.18, respectively.

It was observed from the figures that, irrespective of the method of curing, the elastic modulus of both series I and II mixes reduced with an increased percentage of RA at both 28 days and 90 days curing. At 28 days, the static modulus of steam cured concrete mixes with 100% RA in Series I and II, decreased by 14 and 29%, respectively, compared to those of conventional concrete mixes, whereas in water cured concrete mixes made with 100% RA, the reduction in modulus of elasticity was 20 and 40%, respectively, in Series I and II, compared to than those of corresponding natural aggregate concrete mixes. This shows that the reduction in static modulus of elasticity of RAC mixes due to the inclusion of RA in water curing was more significant than those with steam curing. It was further observed that the

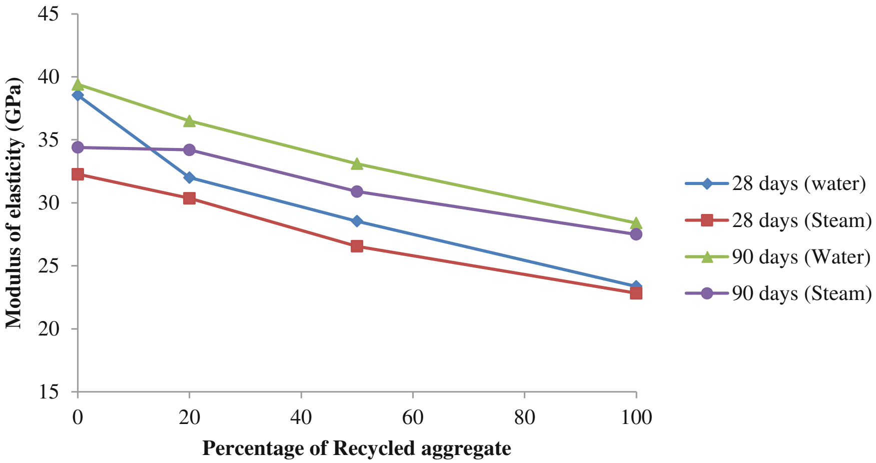

Fig. 4.17 Modulus of elasticity of concrete mixes with w/c 0.45 (Poon et al. 2006)

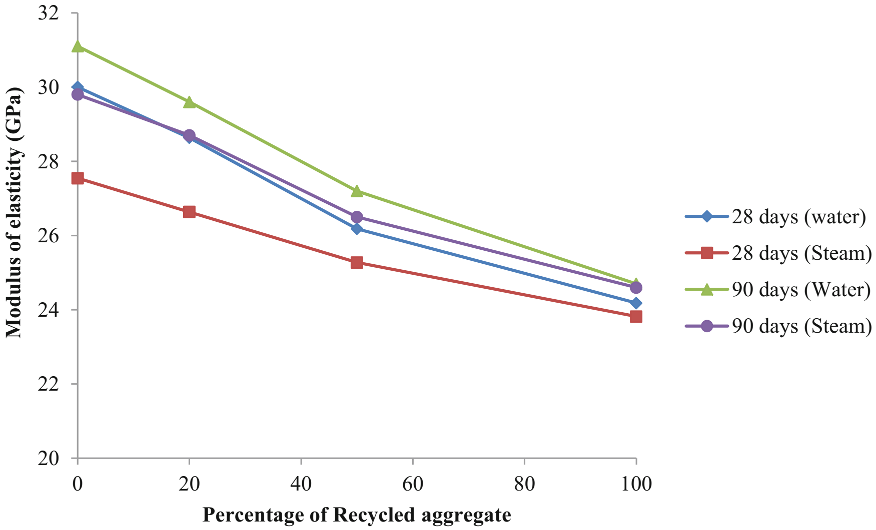

Fig. 4.18 Modulus of elasticity of concrete mixes with w/c 0.55 (Poon et al. 2006)

modulus of elasticity of concrete mixes with normal water curing was more than those with steam curing at both 28 and 90 days. In steam curing, 8 and 17% reduction in modulus of elasticity was observed in concrete mixes with 0% RA in Series I and II, respectively.

Padmini et al. (2009) in a study concluded that for a given compressive strength, the modulus of elasticity of parent concrete from which the recycled coarse aggregate derived was higher than that of RAC. Also, it was observed that the

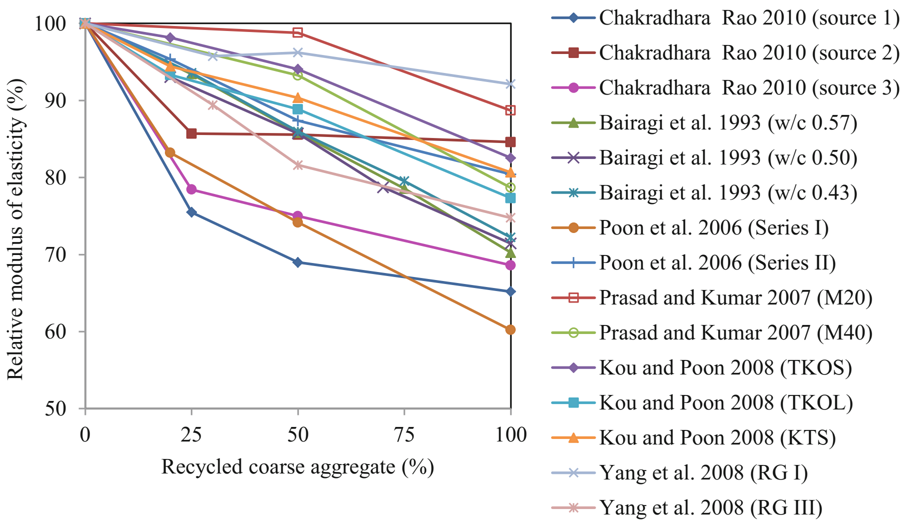

Fig. 4.19 Relative modulus of elasticity with different proportions of RCA

percentage reduction in modulus of elasticity of RAC made with smaller size aggregate was larger. Kou and Poon (2015) conducted a series of experiments on the influence of strength of parent concrete on modulus of elasticity of recycled aggregate concrete. The authors considered two series of mixes: Series I and series II with w/c of 0.50 and 0.35, respectively. In each series five mixes were prepared with RCA obtained from different strengths (30, 45, 60, 80, and 100 MPa) of parent concrete. It was found that irrespective of strength of parent concrete, the modulus of elasticity of concrete with recycled aggregate was lower than the conventional concrete. However, with the increased strength of parent concrete from which the RCA derived, the decrease in static modulus was reduced.

Figure 4.19 depicts the ratio of modulus of elasticity of RAC for all RCA values with the modulus of elasticity of normal concrete at 28 days of curing period reported in the literature. It unveils that the variation in modulus of elasticity of RAC with RCA content is almost similar. The reduction in modulus of elasticity of RAC with 100% RCA compared to normal concrete reported by different researchers is presented in Table 4.8. These reductions may be due to the lower modulus of elasticity of recycled coarse aggregate and the presence of weaker interfaces between aggregate and old cement mortar.

4.3.3 Split Tensile Strength

The concrete is mainly used from the compressive strength point of view; there is also a necessity for structural engineers to be able to determine other properties like tensile strengths. The direct tensile strength of concrete is not often carried out due

Table 4.8 Reduction in modulus of elasticity of RAC at 100% RCA

Author(s)	% reduction
Rao et al. (2017)	15–35
Bairagi et al. (1993)	39
Poon et al. (2006)	20–40
Yang et al. (2008)	8–25
Frondistou- Yannas (Frondistou-Yannas 1977)	40
Acker (Acker A 1998)	23
Katz (2003)	37–50
Li (2008)	45
Hansen and Boegh (1986)	19
Kheder and Al-Windawi (2005)	20–25
Limbachiya et al. (2012)	35

Table 4.9 Tensile strengths of both normal and recycled aggregate concrete (Rao et al. 2017)

Source of RCA	Mix designation	RCA (%)	Split tensile strength (MPa)	Flexural strength (MPa)
Normal concrete	M-RAC0	0	2.67	5.23
Source 1: RCC culvert near Medinipur	MM-RAC25	25	2.33	4.17
	MM-RAC50	50	2.05	4.42
	MM-RAC100	100	2.03	4.97
Source 2: RCC culvert near Kharagpur	MK-RAC25	25	2.3	4.86
	MK-RAC50	50	2.19	4.64
	MK-RAC100	100	2.05	4.39
Normal concrete	M-RAC0	0	2.57	5.36
Source 3: RCC slab of an old residential building near Vizianagaram	MV-RAC25	25	2.22	5.05
	MV-RAC50	50	2.1	4.77
	MV-RAC100	100	1.96	4.30

to development of secondary stresses induced by specimen holding devices which cannot be ignored. Hence, there are indirect methods to find out the tensile strength of concrete. Split tensile strength is one of the indirect measurements of tensile strength of concrete. Quality of RA, w/c ratio, mixing method, amount of RA, type of cement, and curing age are some of the factors influences the split tensile strength of RAC (Kisku et al. 2017). It was reported in the literature that the split tensile strength decreased with the increased amount of recycled aggregate. Bairagi et al. (1993) reported that the reduction in tensile strength of RAC with 25 and 50% RCA were 6 and 10%, respectively, than that of normal concrete and it was 40% when RAC made with 100% RCA. Rao et al. (2017) examined the split tensile strength of RAC made with different demolished structures concrete as coarse aggregate with different proportions. The 28 days split tensile and flexural strength test results reported by Rao et al. (2017) are presented in Table 4.9.

It revealed that the split tensile strength of RAC decreased with the increase of RCA percentage in all the three Sources of mixes. Further, it revealed that the split tensile strength of RAC was reduced by 13.6, 18.3, and 23.3% at 25, 50, and 100% RCA, respectively, than the conventional concrete.

Yang et al. (2008) found that the split tensile strength of RAC was reduced by 0.24, 9.6, and 13.6% at 30, 50, and 100% RCA, respectively, when compared to control concrete, whereas these reductions are further increased to 23.7, 30.6, and 36.9% when natural coarse aggregate and 30, 50, and 100% recycled fine aggregate was used. However, when both recycled fine and coarse aggregates used in the mix, there was an improvement in tensile strength of RAC than the later. In a study conducted by BCSJ (1978), Mukai et al. (1978), and Ravindrajah and Tam (1985), it was observed that not much significant difference in split tensile strength of RAC and normal concrete when RAC prepared with RCA and natural sand. However, the tensile strength of RAC was 20% less than that of normal concrete when it was made with both recycled fine and coarse aggregates. Gerardu and Hendriks (1985) have observed similar result when both fine and coarse recycled aggregates used in RAC. However, when RAC made with recycled coarse aggregate and natural sand, the split tensile strength of RAC was 90% of that of normal concrete. Tavakoli and Soroushian (1996) concluded that depending on w/c ratio and dry mixing period the tensile strength of RAC can be higher or lower than that of normal concrete.

In a study by Sagoe-Crentsil et al. (2001), concluded that the development of tensile strength depends on the binding mortar rather than on aggregate type. The authors observed that with the use of slag cement, the tensile strength of RAC could be improved with the curing age and after one year; the tensile strength of RAC made with slag cement was 25% higher than those made with OPC. In addition, it was reported that the RAC made with OPC has no significant reduction in tensile strength from 90 days to one year curing period. Prasad and Kumar (2007) studied the influence of glass fiber on tensile strengths of recycled aggregate concrete with different amounts of RCA. The authors found that at 100% RCA, the tensile strength of RAC with glass fiber has increased by 13.03 and 10.57% in M20 and M40 grade concretes than those made without fibers. Etxeberria et al. (2007) attempted to study the influence of different amounts (25, 50 and 100%) of RCA on tensile strength of concrete over a period of 6 months curing. The authors concluded that the tensile strength of RAC made with RCA was higher than that of normal concrete, except for RAC with 100% RCA. In addition, it was observed that the tensile strength of RAC was improved with the curing age from 28 days to 6 months. The authors found that, in general, the interface was the weak zone in concrete and most of the medium-strength concretes fail through the interface, whereas this was not so happened in RAC, as the failure occurred through the recycled aggregate in addition to the failure at the interface.

Kou and Poon (2008) examined the influence of curing age on tensile strength of RAC made with different percentages (0, 20, 50 and 100) of RCA obtained from three different sources. The authors found that the tensile strength of RAC with all percentages of RCA was lower than that of normal concrete at 28 days curing period. However, after one year period of curing the tensile strength of RAC with

100% RCA obtained from all sources were higher than the normal concrete. In addition, it was observed that after 5 years curing period, the tensile strength of RAC made with all percentages of RCA obtained from different sources were higher than that of normal concrete and the RAC made with 100% RCA had highest tensile strength and strength gain. This gain may be attributed to the presence of recycled aggregate which improved the microstructure of interfacial transition zone (ITZ) and increased bond between recycled aggregate and new cement mortar. Kou et al. (2008) examined the influence of the addition of fly ash as cementing material on tensile strength of RAC made with different amounts of RCA for different w/c ratios. The authors concluded that the tensile strength of RAC decreased with the increase in RCA content and at 100% RCA the tensile strength of RAC was 9–12% lower than that of normal concrete at 90 days curing period. However, the addition of fly ash improves the tensile strength of RAC by 3–8% than those without fly ash. This could be attributed to the pozzolanic reaction of fly ash and possible reduction in porosity due to the compactness of the fine fly-ash particles.

Padmini et al. (2009) concluded that the tensile strength of RAC made with RCA was lower than that of normal concrete at a given compressive strength. This difference was lower when compressive strength of concrete is low. The authors agreed with Etxeberria et al. (2007), as the failure is through the recycled aggregate in addition to the interface bond between aggregate and mortar. Tabsh and Abdelfatah (2009) studied the tensile strength of two different RAC mixes: Mix 1 with a target strength of $f_c^1 = 30$ MPa and Mix 2 with a target strength of $f_c^1 = 50$ MPa made with RCA obtained from three different sources viz. waste from dumping site, crushing of low-strength (30 MPa) concrete and crushing of medium-strength (50 MPa) concrete samples. It was reported that the split tensile strength of RAC mixes prepared with RCA obtained from 50 MPa concrete was almost the same as that prepared with natural aggregate. However, the tensile strength was lowered by 25–30% and 10–15% in RAC mixes 1 and 2, respectively, when they made with RCA obtained from dumping site and 30 MPa strength concrete compared to those of conventional concrete. It was concluded that the tensile strength of RAC depends on the strength of concrete from which the RCA was derived and it was more significant in case of lower strength concrete. Rao (2016) had found a similar observation in his investigation. The author had investigated the influence of RCA obtained from different strengths (20, 25, 30, 35, 40 MPa) of parent concrete on the split tensile strength of different RAC mixes and is presented in Fig. 4.20. It was reported that the split tensile strength of RAC prepared with RCA obtained from the same strength of PC was lower than those of corresponding natural aggregate concrete in both normal-strength (20 and 25 MPa) and medium-strength (30 and 35 MPa) concretes. It was further reported that with an increase in strength of PC from which the RCA produced, the split tensile strength of RAC was relatively improved and even reached to the values of corresponding parent concretes. This improvement may be due to the presence of relatively stronger bond between adhered mortar and aggregate surface in higher-strength parent concrete aggregate comparatively lower strength parent

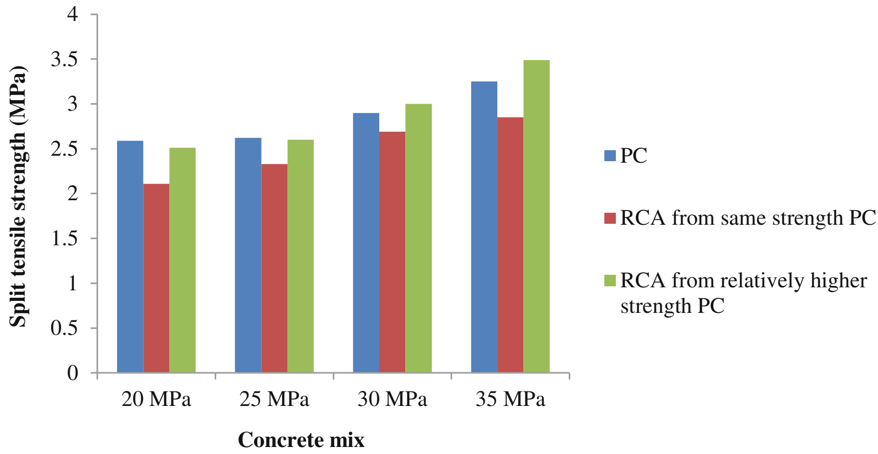

Fig. 4.20 Split tensile strength of different concrete families (Rao 2016)

concrete aggregate, which makes the surface more rough and hence an improvement in the bond between RCA and new cement mortar.

Kou and Poon (2015) investigated the influence of RA obtained from 30, 45, 60, 80, and 100 MPa strength parent concretes on split tensile strength of RAC. The authors considered two series of RAC mixes: Series I (compressive strength of 45 MPa with w/c 0.50) and Series II (compressive strength of 65 MPa with w/c 0.35) each with RA derived from 30, 45, 60, 80, and 100 MPa strength parent concretes separately. The development of split tensile strength at different curing periods in both Series I and II mixes is presented in Figs. 4.21 and 4.22, respectively.

It was found that in both I and II Series mixes, regardless of the parent concrete from which RA obtained, the split tensile strength of RAC was lower than the corresponding natural aggregate concrete at 7 and 28 days curing. However, at 90 days, the tensile strength of RAC mixes (both Series I and II) was more than the tensile strength of conventional concrete at 28 days. The ratio of split tensile strength of RAC (both Series I & II) to that of 28 days controlled concrete is depicted in Fig. 4.23.

It was found from the figure that in Series I mixes, the split tensile strength of RAC made with RA obtained from 30, 45, 60, 80, and 100 MPa strength of parent concretes were 2.1, 3.8, 4.9, 7.0 and 12.2%, respectively, more than that of the corresponding concrete prepared with natural aggregate. Similarly, in Series II mixes, the splitting tensile strength of concrete made with RA produced from 30, 45, 60, 80, and 100 MPa strength parent concretes were 11.1, 12.6, 13.5, 18.6, and 26.0%, respectively, greater than that of corresponding natural aggregate concrete. The improved split tensile strength of RAC might be due to the porous and rough surface of recycled aggregates which enhanced the ITZ between the recycled aggregate and new cement paste that leads to the enriched bond strengths (Kou and Poon 2015).

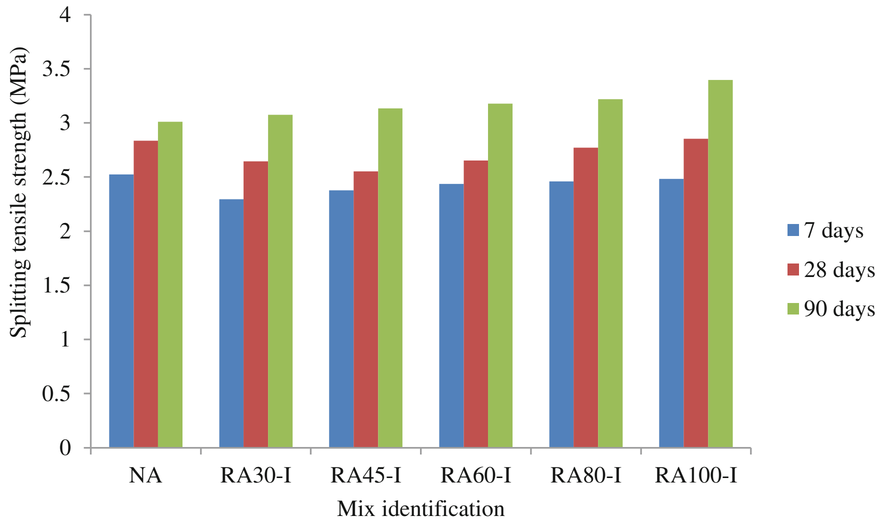

Fig. 4.21 Split tensile strength of concrete mixes at different curing periods in Series I (Kou and Poon 2015)

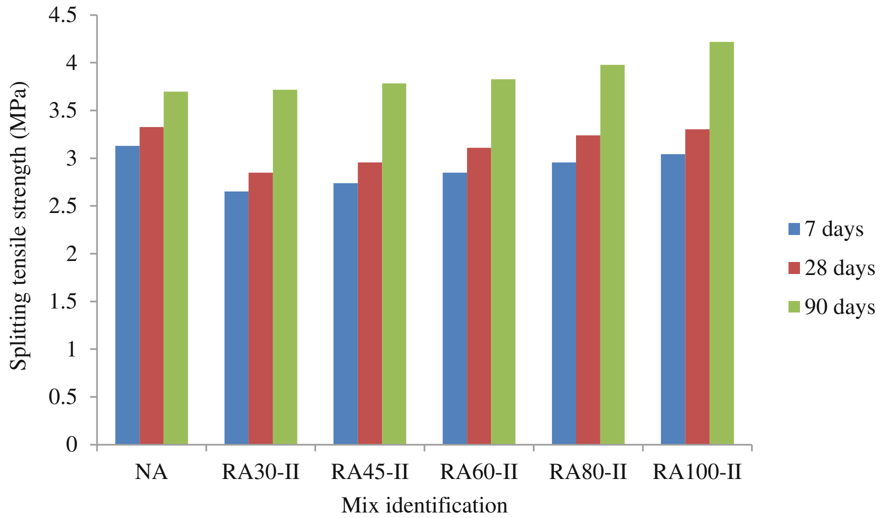

Fig. 4.22 Split tensile strength of concrete mixes at different curing periods in Series II (Kou and Poon 2015)

The influence of fly ash on long-term properties of recycled aggregate concrete has been investigated by Kou and Poon (2013). The authors considered the controlled concrete mix with a water/binder (w/b) ratio of 0.55 and a cement content of 410 kg/m^3. The cement was replaced by 0, 25, 35, and 55% fly ash (FA) by weight

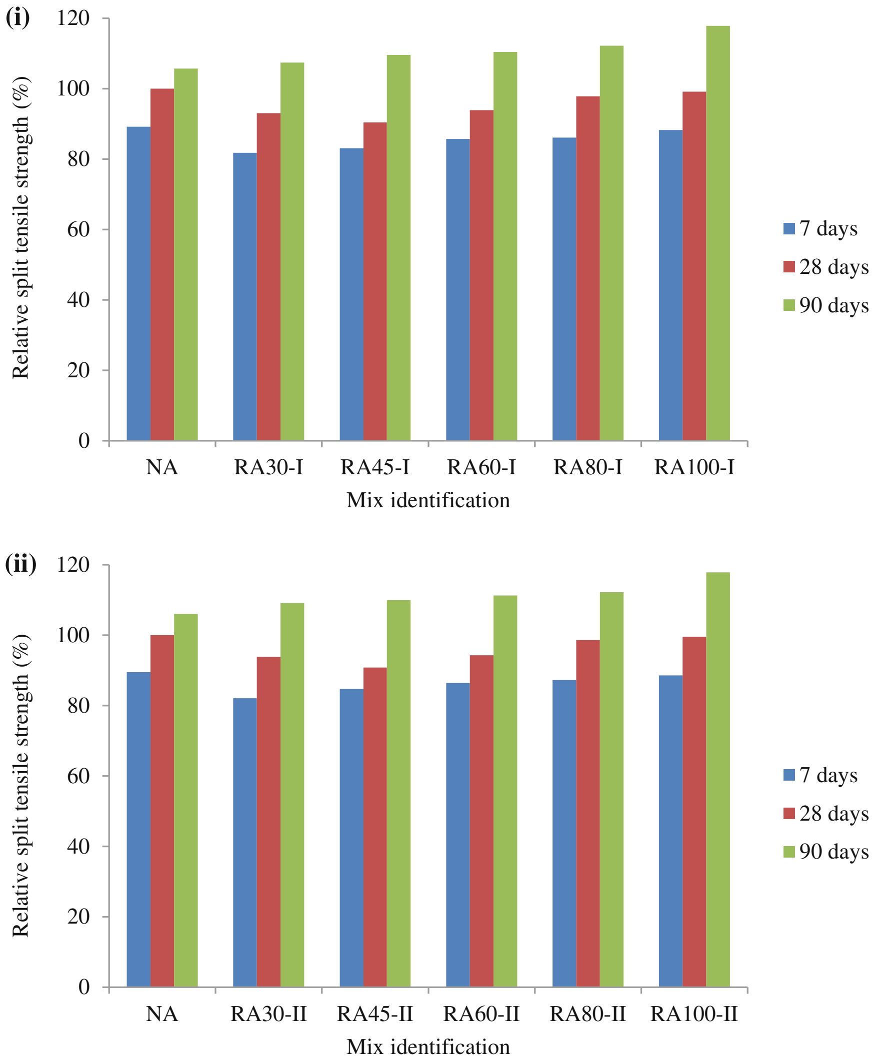

Fig. 4.23 Relative split tensile strength of concrete mixes in (i) Series I and (ii) Series II (Kou and Poon 2015)

and the natural aggregates were replaced with 0, 50 and 100% by weight. Two different methods of curing viz. normal water curing and air dry (samples exposed to outdoor in the laboratory) were adopted for 28 days, 1, 3, 5, and 10 years curing period. The splitting tensile strength results of all the concrete mixes reported by Kou and Poon (2013) is depicted in Table 4.10. The percentage gain in split tensile strength of mixes at different curing periods with respect to standard water curing at 28 days is shown in Fig. 4.24.

Table 4.10 Split tensile strength of different concrete mixes (Kou and Poon 2013)

Notation	Fly ash (%)	RA (%)	Split tensile strength (MPa)									
			28 days		1 year		3 years		5 years		10 years	
			WC	AC	WC	AC	WC	AC	WC	AC	WC	AC
R0	0	0	3.32	3.21	3.45	3.31	3.76	3.54	4.23	4.01	4.61	4.25
R50	0	50	3.16	3.09	3.51	3.41	3.92	3.62	4.41	4.14	4.71	4.32
R100	0	100	3.06	2.98	3.56	3.44	4.12	3.78	4.45	4.18	4.83	4.41
R0F25	25	0	3.28	3.14	3.65	3.42	3.89	3.58	4.25	3.92	4.69	4.27
R50F25	25	50	3.09	3.01	3.62	3.46	3.94	3.85	4.41	4.15	4.75	4.32
R100F25	25	100	2.96	2.91	3.75	3.54	4.12	3.91	4.4	4.21	4.81	4.49
R0F35	35	0	2.9	2.81	3.14	3.02	3.36	3.18	3.68	3.42	4.18	3.77
R50F35	35	50	2.78	2.72	3.24	3.11	3.38	3.21	3.72	3.53	4.24	3.88
R100F35	35	100	2.56	2.48	3.31	3.12	3.47	3.26	3.77	3.55	4.28	3.91
R0F55	55	0	2.66	2.58	2.89	2.73	3.04	2.91	3.22	3.01	3.72	3.3
R50F55	55	50	2.42	2.36	2.93	2.8	3.12	2.96	3.28	3.05	3.75	3.36
R100F55	55	100	2.23	2.19	3.01	2.89	3.24	3.11	3.41	3.14	3.81	3.48

WC water cured; AC air cured

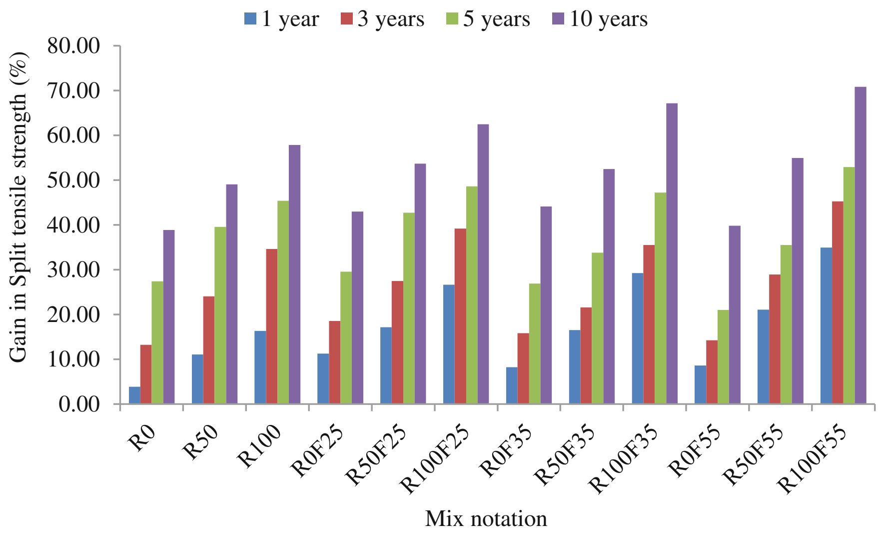

Fig. 4.24 Increase in split tensile strength of different concrete mixtures with respect to standard water curing at 28 days (Kou and Poon 2013)

From the above, the following observations were reported.

(i) At 28 days, the split tensile strength of concrete mixes decreased with the increased percentage of recycled aggregate and fly ash. The reduction in split tensile strength of RA50 and RA100 at 50 and 100% recycled aggregates, respectively, were 5 and 7.5% to that of controlled concrete, whereas the reduction in split tensile strength was further increased to 27 and 33% in RA50 and RA100, respectively, when they were made with 50 and 100% recycled aggregate with 55% fly ash.

(ii) After 28 days, the split tensile strength of RAC mixes was continuously increased with the curing age up to 10 years and this improvement was more considerable. At 1 year curing, the split tensile strength of concrete mixes with 100% recycled aggregate was more than that of conventional concrete. In addition, the split tensile strength of concrete mixes cured under standard water was more than those of cured in air dry at all curing periods.

(iii) When the controlled concrete mixes made with 0, 25, 35, and 55% fly-ash content, the split tensile strength was increased by 38.9, 43.0, 44.1, and 39.8%, respectively, from 28 days to 10 years curing and 3.9, 11.3, 8.3, and 8.6%, respectively, between 28 days and 1 year curing period.

(iv) The concrete mixes made with 100% recycled aggregate with 0, 25, 35, and 55% fly-ash content, the gain in split tensile strength were 57.8, 62.5, 67.2, and 70.9%, respectively, between 28 days and 10 years curing periods, whereas in first one year, the increase in split tensile strength of RA100 mixes was 16.3, 26.7, 29.3, and 35%, respectively, with 0, 25, 35, and 55% fly-ash content.

(v) After 10 years of curing period, the highest tensile strength gain (70.9%) occurred in mix with 100% RA and 55% fly-ash content and the maximum split tensile strength (4.81 MPa) developed in the mix made with 100% RA and 25% fly-ash content. The increase in split tensile strength could be ascribed to the pozzolanic reaction of fly ash and possible decrease in porosity results increased bond strength between aggregate and new cement paste and enhanced microstructure of interfacial transition zone (ITZ).

Cakir and Sofyanli (2015) reported that the split tensile strength of recycled aggregate concrete was continuously and significantly improved with time by the addition of silica fume. The mixes with 10% SF and 4/12 mm size fraction of recycled aggregate had shown better performance than other mixes made with 8/22 mm size fraction with SF 10% and 4/12 mm, 8/22 mm with 5% silica fume.

Matias et al. (2013) have studied the influence of superplasticizers on splitting tensile strength of concrete with different amounts of recycled aggregate. The authors considered two types of superplasticizers viz. a standard superplasticizer (SP1) (based on lignosulphonate polymers) and a high-performance superplasticizers (SP2), (based on modified polycarboxylic polymers). It was found that the splitting tensile strength of concrete was increased due to the incorporation of recycled aggregate. This was because of the rough surface of recycled aggregate which provides better bonding with the cement matrix. However, the splitting tensile strength of concrete decreased with the increased amount of recycled aggregate. This loss could be reduced by adding the superplasticizers in recycled aggregate concrete. It was reported that by the addition of SP2, the split tensile strength of RAC was improved and close to that of the reference concrete,

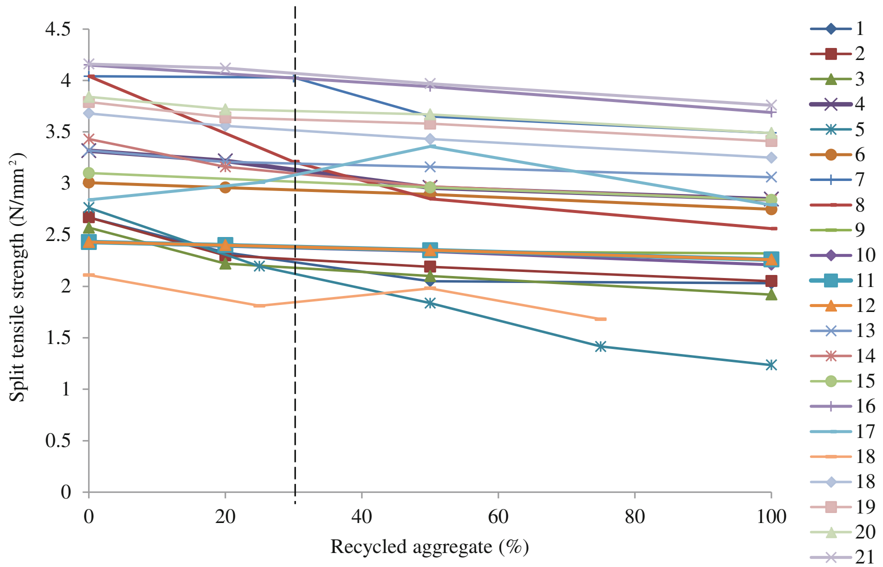

Fig. 4.25 28 days Split tensile strength vs percentage of recycled aggregate reported by different researchers [(1), (2), (3) Rao et al. (2017): Sources 1, 2, 3, respectively; (4), (5) Elhakam et al. 2012: w/c 0.45, w/c 0.60; (6) Xiao et al. 2005; (7), (8) Yang et al. 2008: RG-I and RG-II; (9), (10), (11) Kou and Poon (2008): TKOS, TKOL, KTS; (12), (13) Poon et al. (2006): Series I and Series II; (14), (15) Prasad and Kumar (2007): M20 and M40; (16) Etxeberia et al. 2007; (17) Mas et al. 2012; (18), (19), (20), (21) Kou et al. (2008): w/c 0.55, w/c 0.5, w/c 0.45, w/c 0.4, respectively

a difference of only 7.2%. But the split tensile strength of RAC with SP1 had a poor result, a reduction of 16.4% observed when compared to its reference concrete. Pereira et al. (2012) agreed that the concrete made with fine recycled aggregate, the addition of superplasticizers enhanced the split tensile strength. It was reported that the split tensile strength of concrete with RA was increased up to 26.6 and 52.8%, respectively, with the addition of SP1 (based on lignosulfate) and SP2 (based on polycarboxylate). Behera et al. (2014) in their review paper concluded that the bond strength among mortar matrix and aggregate surface significantly affect the RAC splitting tensile strength which rises age due to further hydration.

The variation in split tensile strength of RAC at 28 days with respect to different percentages of RA reported by different researchers is presented in Fig. 4.25. It shows that up to 30% replacement of natural aggregate by recycled aggregate, the reduction in split tensile strength of RAC is not much significant.

The failures of split tensile specimens of both normal concrete and recycled aggregate concrete made with 100% RCA obtained from all the three Sources reported by Rao et al. (2017) are presented in Fig. 4.26.

In general, the failure is through the interface between aggregate and cement mortar, as it is the weakest zone in case of normal concrete, whereas this is not so in case of recycled aggregate concrete. In case of recycled aggregate concrete, it was observed that the failure occurred not only through the interface, but also through the

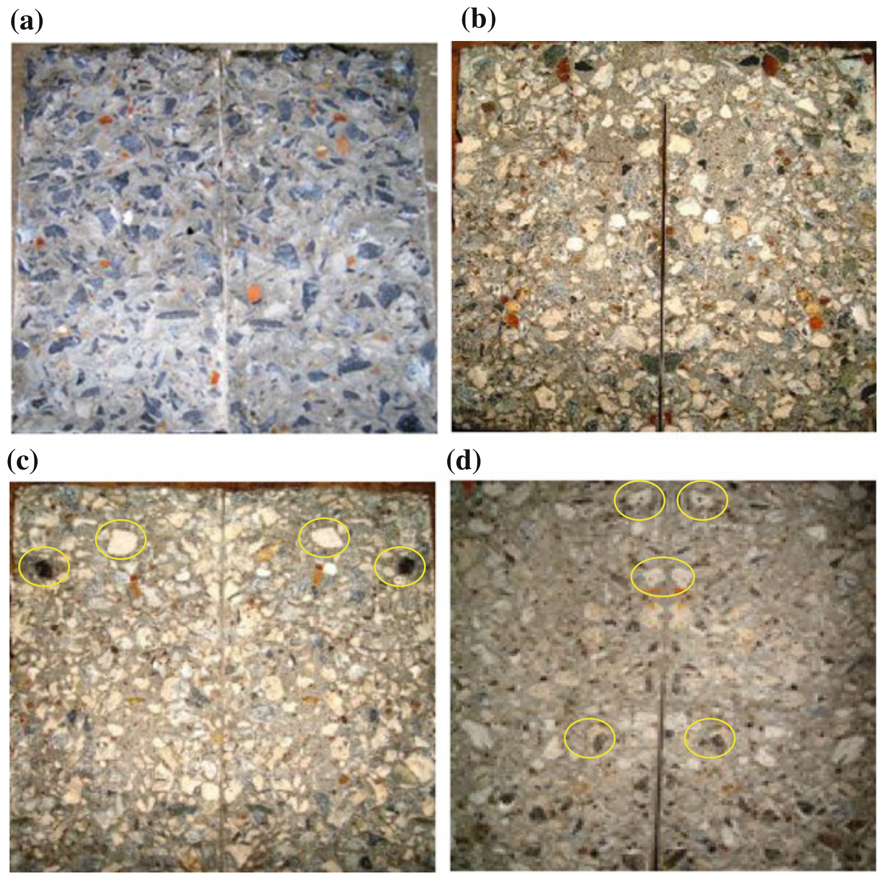

Fig. 4.26 Split tensile failure surfaces of (**a**) normal concrete and (**b**), (**c**), (**d**) RAC with 100% RCA obtained from Sources 1, 2, 3, respectively (aggregate failure) (Rao et al. 2017)

recycled aggregate. The failure surfaces showed that the crushed aggregates are the pieces of old cement mortar which was adhered to recycled coarse aggregate. In addition, the figure reveals that the failure surface was more tortuous in case of normal concrete, whereas the failure surface of RAC in all the three cases was more even.

4.3.4 Flexural Strength

Flexural strength or modulus of rupture is also another important factor which affects the structural behavior of concrete. In recycled aggregate concrete, it mainly depends on the replacement level of NA by RA, curing condition, moisture state of RA, water–binder ratio, etc. (Kisku et al. 2017).

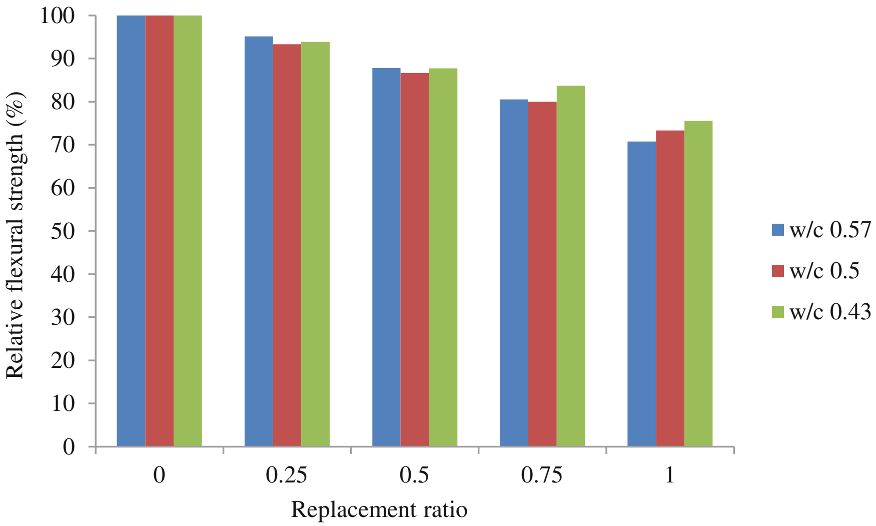

Fig. 4.27 Relative flexural strength of RAC mixes at 28 days (Bairagi et al. 1993)

Malhotra (1976) found that the flexural strength of RAC was 80–100% of that of normal concrete at 100% replacement of NA by RA. Ravindrarajah and Tam (1985) observed that the flexural strength of RAC made with RCA and natural sand was not much differed from normal concrete. Bairagi et al. (1993) studied the influence of w/c ratio on modulus of rupture of RAC with different replacement ratio of NA to RA. The relative flexural strength of RAC with normal concrete at 28 days is presented in Fig. 4.27.

It revealed that the flexural strength of RAC made with 25 and 50% RCA were around 94–87% of that of normal concrete. When RAC made with 100% RCA, the reduction in flexural strength was comparable with normal concrete and it was 29–24% less for w/c ratio 0.57–0.43. This may be ascribed to the inferior quality of new interfacial transition zone, i.e., interfacial bond between the recycled aggregate and new cement paste. It was further revealed that at a given replacement ratio, the reduction in flexural strength was more or less same at all w/c ratios. Mas et al. (2012) investigated the flexural strength of RAC mixes at w/c 0.45, 0.65, and 0.72 with different proportions of mixed recycled aggregate and observed that the flexural strength decreased by 30, 20, and 13%, respectively, when compared to the corresponding normal concrete. Rao et al. (2017) conducted a detailed investigation on flexural strength of RAC made with RA obtained from different sources of demolished old structures. The results of different mixes are depicted in vide Table 4.9. It can be seen that the flexural strength of RAC decreased with the increase in percentage of RCA from 25 to 100% for the sources 2 and 3 mixes, whereas the same increases with the increase in recycled coarse aggregate percentage in source 1 mixes. Nevertheless, the flexural strength of RAC for any percentage of RCA is less than that of normal concrete (0% RCA) in all the three

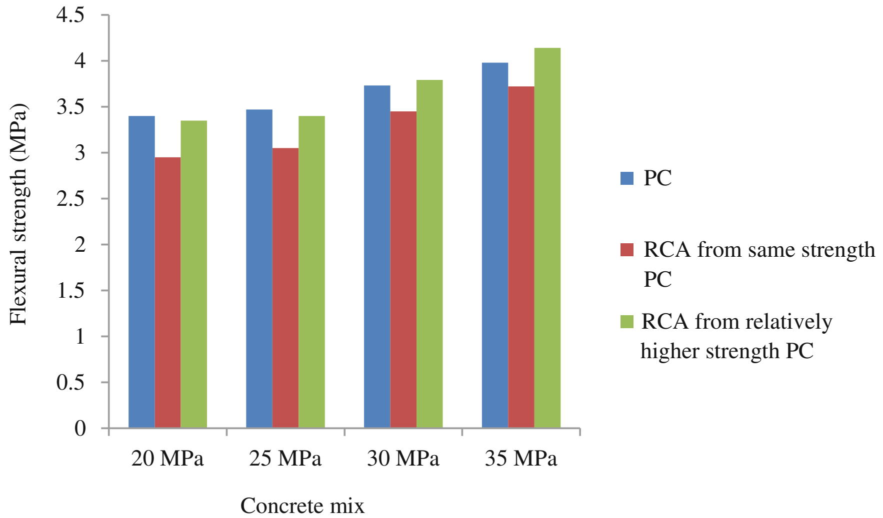

Fig. 4.28 Flexural strength of different concrete families (Rao 2016)

sources of mixes. Further, the reduction in flexural strength of RAC with 25, 50, and 100% RCA obtained from Source 1 was 20, 15, 5%; Source 2 was 7, 11, 16%; and Source 3 was 6, 11, 20%, respectively, when compared to concrete with natural coarse aggregate. In addition, it was observed from the test results that the flexural strength of RAC made with different percentages of RCA was in the order of 9 to 12%, 10 to 11%, and 9 to 10% of the corresponding compressive strength in case of sources 1, 2, and 3 mixes, respectively. Limbachiya et al. (2012) reported 30% recycled coarse aggregate as optimum substitute for natural aggregate in making RAC. At 25% RA, only 2.5% reduction in flexural strength at 28 days was observed in RAC with w/c 0.55 when compared to the normal concrete (James et al. 2011).

The quality of recycled aggregate also affects the flexural strength of recycled aggregate concrete. Rao (2016) had investigated the influence of RCA obtained from different strengths of parent concrete on flexural strength of RAC mixes and is shown in Fig. 4.28.

Like in split tensile strength the flexural strength of RAC with RCA from the same strength of PC was lower than that of corresponding natural aggregate concrete in both normal-strength and medium-strength concretes. The flexural strength of RAC made with RCA from similar strength of PC (RM20RCA20, RM25RCA25, RM30RCA30, and RM35RCA35) was lower by 13.24, 12.1, 7.51, and 6.53%, respectively, than those of corresponding natural aggregate concrete. However, there is a marginal improvement in flexural strength was observed in RAC with RCA from relatively higher-strength parent concrete. Except in RM20RCA25, the flexural strength of RM25RCA30, RM30RCA35, and RM35RCA40 was slightly more than that of natural aggregate concrete. These

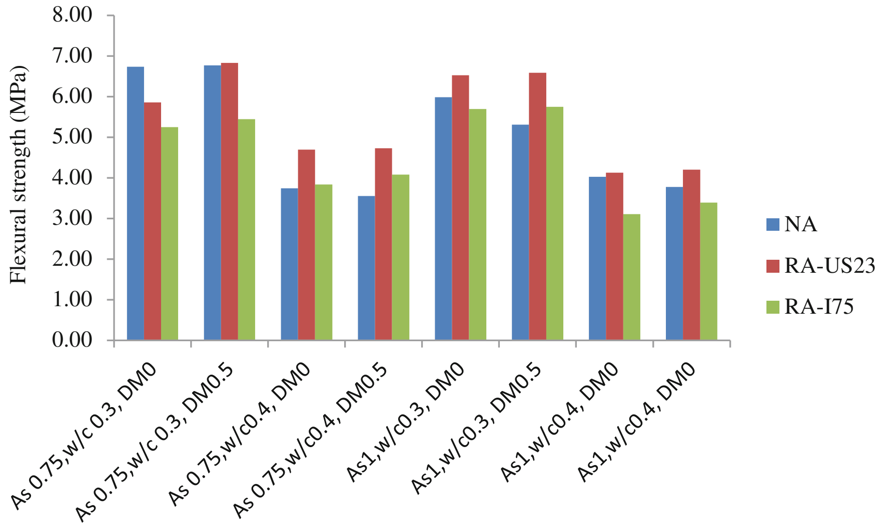

Fig. 4.29 Flexural strength of concrete mixes at 28 days (Tavakoli and Soroushian 1996)

improvements may be due to the improved new interfacial transition zones between RCA and new cement mortar. Further, the aggregate obtained from the higher-strength parent concrete was having a relatively stronger bond between it and the old cement mortar. Tavakoli and Soroushian (1996) investigated the influence of field demolished concrete as aggregate on flexural strength of RAC. The authors considered two different sources: crushed concrete pavements from US23 and I-75 projects in Michigan. The authors further considered two maximum sizes of aggregates (0.75 and 1 inch), two levels of w/c ratio (0.3, 0.4), and two levels of dry mixing time (0, 0.5 h) of coarse aggregates. The flexural strength of concrete mixes reported by the authors is presented in Fig. 4.29.

It revealed that the concrete made with lower maximum size of US23 recycled aggregate lead to more flexural strength than that of I-75 recycled aggregate. Further, at higher w/c ratio, concrete mix prepared with US23 RA had better performance than the natural aggregate, but at lower w/c ratio, its performance was poor except when dry mixing of RA was adopted.

De Oliveira and Vazquez (1996) conducted a study on the influence of moisture states of RCA (dry, semi-saturated, and saturated) on flexural strength of RAC. It was concluded that the flexural strength of RAC was lower than that of normal concrete irrespective of the moisture state of RCA. The reduction was more significant when RAC made with saturated recycled aggregates. Brand et al. (2015) found that the flexural strength of RAC made with RA at 80% SSD was better than those made with RA at OD and SSD. Katz (2003) studied the influence of partially hydrated concrete (1, 3, 7 days age) as aggregate on flexural strength of RAC with two different types of cements. The authors concluded that the flexural strength of RAC made with RCA obtained from 3 days cured concrete gives better strength with white cement than those with 1 and 7 days cured concrete. But in all cases, the flexural strength of RAC was lower than that of normal concrete. Prasad and Kumar (2007) in a study concluded that the flexural strength of RAC was improved by the addition of glass fibers. It was further reported that the addition of fly ash (10–15%) further enhances the flexural strength of RAC mixes at 0.55 w/c ratio. In a study conducted by Yang et al. (2008), it was concluded that the flexural strength of RAC decreased with the increase in relative water absorption capacity of recycled aggregates.

Figure 4.30 shows the flexural strength results reported by some of the researchers with respect to the RA content. The figure reveals that the flexural strength decreased with the increase in amount of recycled aggregate. Further, it reveals that up to 30% replacement of NA by RA, the flexural strength does not get affected. It is also evident from the figure that except at w/c = 0.72, with the decrease in the w/c ratio, the flexural strength of RAC increases at all percentages of RA.

4.3.5 Density

The test results of density of normal concrete and recycled aggregate concrete for different RCA values obtained from three different Sources are presented in Fig. 4.31.

Figure 4.31 reveals that in all the three Sources of mixes, the density of RAC decreased with the increase in percentage of recycled coarse aggregate. These results were in good agreement with the results reported in the literature (Topcu and Guncan 1995; Etxeberria et al. 2007; Gonzalez-Fonteboa and Martinez-Abella 2008). The decrease in density may be due to the fact that the density of recycled coarse aggregates was 1.12, 1.18, and 1.17 times lower in Sources 1, 2, and 3 than that of natural coarse aggregate. This may be attributed to the adherence of weak porous nature of old cement mortar to the recycled aggregates. The density of RAC made with 25–100% RCA obtained from Sources 1 and 2 were in the range of 2329 to 2175 kg/m^3 and 2349 kg/m^3 to 2148 kg/m^3, respectively, against 2415 kg/m^3 for normal concrete. Similarly, the density of RAC made with 25 to 100% RCA obtained from Source 3 is in the range of 2405 to 2238 kg/m^3 against 2435 kg/m^3 for normal concrete. In general, the density of normal concrete made with natural aggregate is in the range of 2200–2600 kg/m^3 (Neville 2006). The density of RAC made with Source 3 RCA was relatively higher than that of RAC made with other Sources of RCA. As discussed in Sect. 3.2.1, more particles (5%) finer than 4.75 mm in Source 3 RCA, the density of the mixes may increase relatively. The ratio of density of RAC with the density of normal concrete at 28 days curing period for all the three Sources of mixes is presented in Fig. 4.32.

It was found that the reduction in density of recycled aggregate concrete made with RCA obtained from Sources 1, 2, and 3 was in the range of 3.6–10%, 2.7–11.1%, and 1.2–8.1%, respectively, compared to the concrete with natural coarse aggregate. This reduction may be beneficial where dead weight is a major problem and where concrete of light weight is necessary. Generally, the self-weight role is a

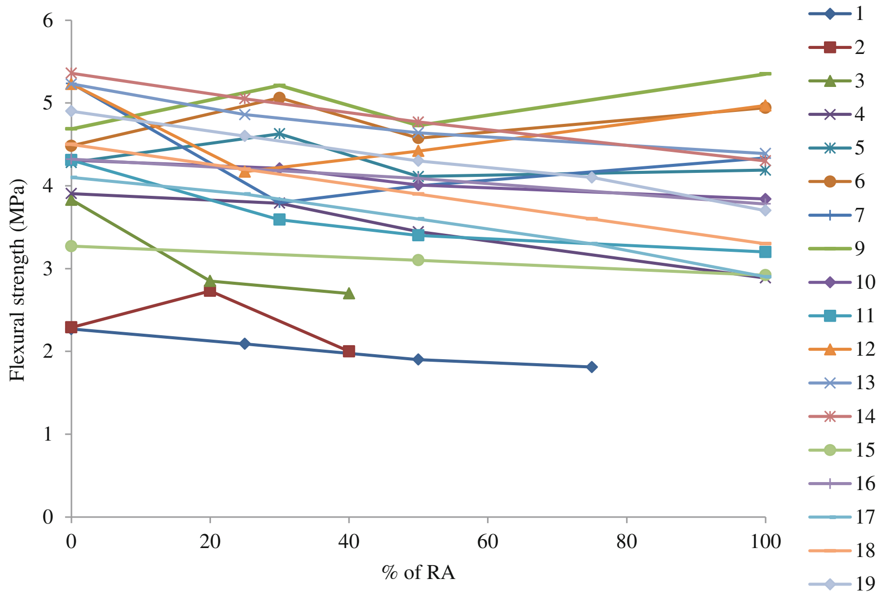

Fig. 4.30 Flexural strength of RAC with different amounts of RCA (1, 2, 3) Mas et al. (2012) w/c 0.65, 0.72, 0.45; (4, 6, 8) Limbachiya et al. (2012)(PC20, PC30, PC35); (5, 7, 9) Limbachiya et al. (2012) (PCFA20, PCFA30, PCFA35); (10, 11) Yang et al. (2008) (RG-I, RG-II); (12, 13, 14) Rao et al. (2017) (S1, S2, S3); (15, 16) Prasad and Kumar (2007) (M20, M40); (17, 18, 19) Bairagi et al. (1993) (w/c 0.57, 0.5, 0.43)

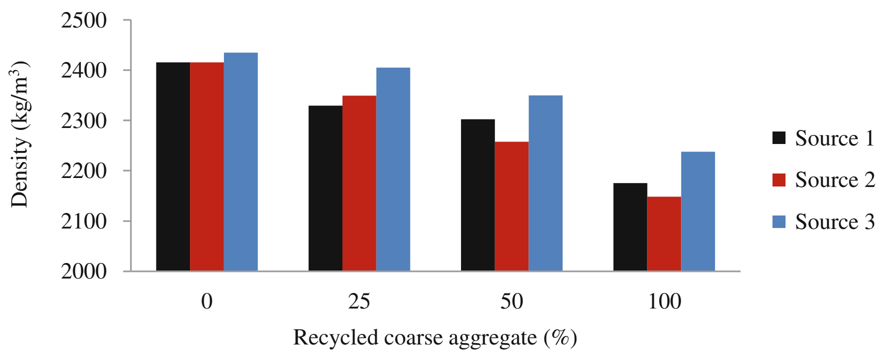

Fig. 4.31 Density of normal and recycled aggregate concrete prepared with RCA obtained from all the three Sources (Chakradhara Rao 2010)

key portion of the total load on the structure in case of concrete structures. Therefore, using low density with reasonable strength of concrete with recycled aggregates slighter sections may be accepted thereby foundation sizes can be reduced (Chakradhara Rao et al. 2011).

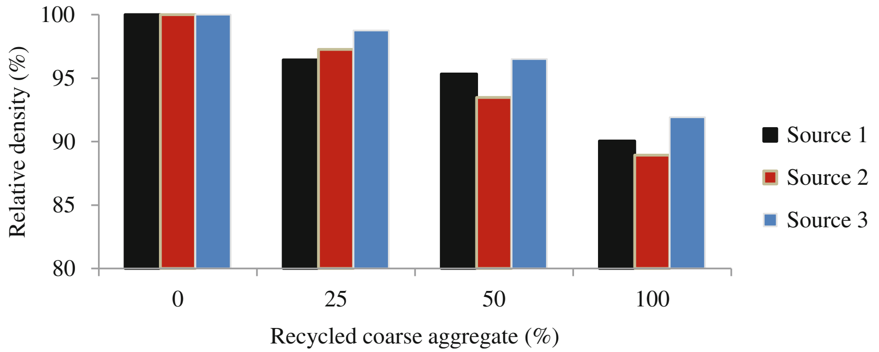

Fig. 4.32 Relative density of RAC made with RCA obtained from different Sources (Chakradhara Rao 2010)

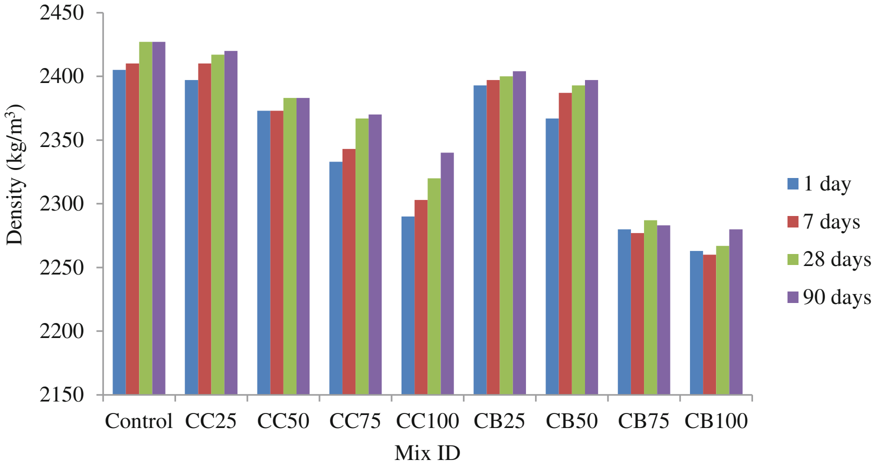

Fig. 4.33 Density of concrete mixes made with RFA from CC and CB (Khatib 2005)

Khatib (2005) investigated the influence of different proportions of fine recycled aggregate (FRA) made from crushed concrete (CC) and crushed brick (CB) on density of RAC. Total nine mixes were prepared. Mix (M1) was prepared with fully natural aggregates, Mixes 2 to 5 (M2 to M5) were prepared with 25, 50, 75 and 100% recycled fine aggregate obtained from crushed concrete (CC) and Mixes 6 to 9 (M6 to M9) were made with fine RA from crushed brick (CB). In all the mixes the w/c ratio was constant at 0.5. The density was measured at different curing periods, and the reported results are presented in Fig. 4.33. It reveals that like the recycled coarse aggregate, the incorporation of fine recycled aggregate also reduces the density of RAC with the increased percentage of FRA either from CC or CB. At 28 days, the density of normal concrete was found to be 2427 kg/m^3 against 2320

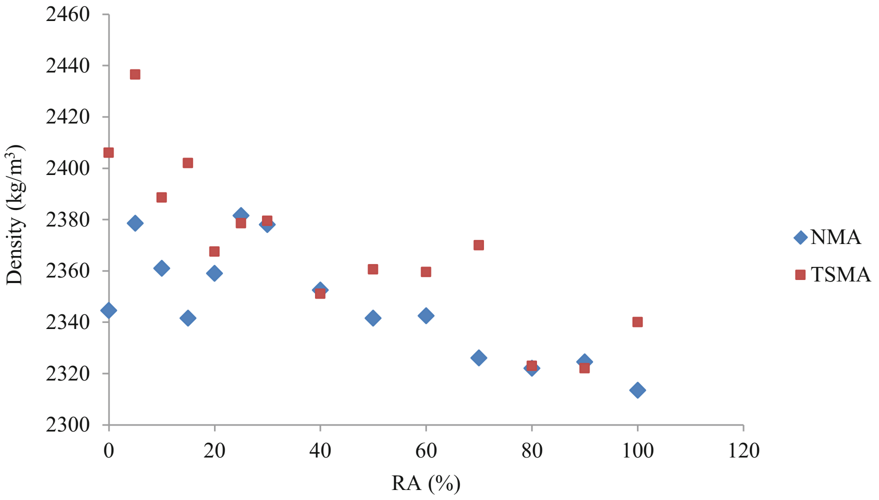

Fig. 4.34 Density of concrete mixes prepared with different proportions of RA using NMA and TSMA (Tam et al. 2007)

and 2267 kg/m^3, respectively, in RAC with 100% CC and CB recycled fine aggregates. Further it was found that there is a slight improvement in density with the curing period; nevertheless, the density of RAC with FRA from either CC or CB was lower than the normal concrete at all curing periods. Furthermore, it reveals that at the same replacement level, the density of RAC made with FRA from CB was lower than that of concrete containing CC.

This reduction in density was not related with the compressive strength reduction. It was found from the figure that the replacement of natural fine aggregate by either 50% CC FRA or 25% CB FRA does not show any significant difference in density of normal concrete and RAC.

Tam et al. (2007) studied the influence of method of mixing on density of RAC prepared with different proportions of RCA. The authors adopted two types of mixing approaches: NMA and TSMA. The results reported by the authors are presented in Fig. 4.34. It was found that the incorporation of RCA lowers the density of concrete and it decreases with the increase in RCA content. It was also found that the density of RAC prepared by TSMA does not show any significant difference from those prepared by NMA.

Ismail and Ramli (2013) examined the density of RAC with different amounts of untreated and treated RCA. The RCA was treated with different concentrations (0.1 M, 0.5 M, and 0.8 M) of HCl and for different durations (1, 3, and 7 days) as per the procedure laid down by Tam et al. (2007). Table 4.11 shows the results reported by the authors.

It reveals that the density of RAC decreased with the increased amount of RCA. This was due to the presence of old porous mortar attached on the surface of RCA. This lowers the specific gravity of RCA in comparison with the natural aggregate.

Table 4.11 Density of concrete prepared with different proportions of untreated and treated RCA (Ismail and Ramli 2013)

RCA (%)	Untreated RCA	Treated RCA								
		0.1 M HCl			0.5 M HCl			0.8 M HCl		
		1 day	3 day	7 day	1 day	3 day	7 day	1 day	3 day	7 day
15	2378	2393	2383	2388	2379	2376	2380	2384	2388	2383
30	2368	2404	2374	2383	2371	2374	2364	2366	2367	2362
45	2369	2435	2365	2380	2372	2378	2366	2356	2363	2358
60	2354	2347	2345	2346	2355	2348	2351	2359	2352	2349

Hence, higher level of replacement of natural aggregate by RCA has a significant effect on lowering the density of concrete. Further, it reveals no significant differences in density of concrete containing untreated RCA and treated RCA.

4.3.6 Ultrasonic Pulse Velocity (UPV)

The nondestructive tests are useful for the assessment of homogeneity and quality of concrete. Though there are many tests available, ultrasonic pulse velocity test is the most popular among them. Chakradhara Rao (2010) investigated the UPV of concrete mixes prepared with different proportions of RCA obtained from three different sources. On each sample, the pulse velocity and compressive strength tests were conducted. The test results of ultrasonic pulse velocity and compressive strength of RAC made with different percentages of recycled coarse aggregate obtained from all the three Sources are shown in Table 4.12.

Table 4.12 Ultrasonic pulse velocity and compressive strength of both normal and recycled aggregate concrete (Chakradhara Rao 2010)

Source of RCA	RCA (%)	Ultrasonic pulse velocity (km/s)	Compressive strength (f_{ck}) (MPa)
Normal concrete	0	4.748	49.45
Source 1: RCC culvert near Medinipur	25	4.658	46.63
	50	4.464	45.58
	100	4.3	43.08
Source 2: RCC culvert near Kharagpur	25	4.66	45.75
	50	4.358	42.5
	100	4.275	40.8
Source 3: RCC slab of an old residential building near Vizianagaram	0	4.690	56.67
	25	4.430	52.30
	50	4.430	49.33
	100	4.181	45.26

It reveals that irrespective of the Source of RCA the UPV of RAC decreased with the increase in percentage of RCA. This may be due to the change in porosity of RCA and the presence of more number of microcracks in recycled aggregates. Normally, the RCA consists of natural aggregates adhered with more porous cement mortar (Chakradhara Rao et al. 2011). Ravindrarajah et al. (1988) had reported a similar result in the literature. Compared to normal concrete, there is a reduction of 1.9–9.4% in UPV in RAC with 25–100% RCA obtained from Source 1. Similarly, the reduction in UPV in RAC with 25–100% RCA obtained from Sources 2 and 3 are 1.9–10% and 5.5–10.8%, respectively, compared to the corresponding normal concrete. In addition, it was observed that the pulse velocity of RAC for all the percentages of RCA obtained from all Sources are in the range of 4.181 km/s to 4.66 km/s and the same for normal concrete is 4.69 to 4.75 km/s. This indicates the uniformity of concrete mixes. According to IS: 13311-1992 (Part 1), the quality of concrete graded as excellent and good when the pulse velocity is more than 4.5 km/s and 3.5 to 4.5 km/s, respectively.

Ravindrajah et al. (1988) investigated the influence of different w/c ratios and method of curing on pulse velocity of concrete prepared with 100% recycled coarse and fine aggregates. The recycled coarse aggregates were produced by crushing 100 mm concrete cubes whose compressive strength was 60 MPa. The relationship between UPV and w/c ratio of both normal and recycled aggregate concrete at 7, 28, and 90 days curing periods is presented in Fig. 4.35a. It was reported that with an increase in w/c ratio, the UPV reduces in both normal and recycled aggregate concretes. This may be because of rise in the capillary porosity. In hardened state of the cement paste, the capillary porosity reduces when the hydration of cement paste rises with age, which results an increase in UPV of concrete. Also, the pulse velocity of RAC decreased when compared to normal concrete due to the change in porosity of natural and recycled aggregates. Normally, the recycled aggregates were adhered with old cement mortar of more porous in nature on the surface of aggregates. It was reported that the ultrasonic pulse velocity through granite is around 4.6 km/s or more, whereas, in a good quality mortar, the UPV is in the range of 3.50 and 3.95 km/s. Thus, for a given volumetric mix composition, it is obvious that the RAC has lesser UPV than normal concrete. The pulse velocity of both normal and recycled aggregate concrete with different curing conditions is presented in Fig. 4.35b. The UPV of normal concrete was higher than that of the recycled aggregate concrete in both wet and air-dry curing conditions. Furthermore, the UPV of concrete decreased when they were cured in air dry compared to those cured in water. The decrease in UPV may be because of the joint effect of reduction in hydration of cement paste and concrete drying. The drop in UPV was found to be 10% and 7%, respectively, in RAC and normal concrete in air-dry condition when compared to wet curing condition at 90 days of curing age. Tu et al. (2006) agreed with the results reported by Ravindrajah et al. (1988) that lower the w/b ratio, higher the UPV of concrete mixes. Further, it was reported that the UPV of

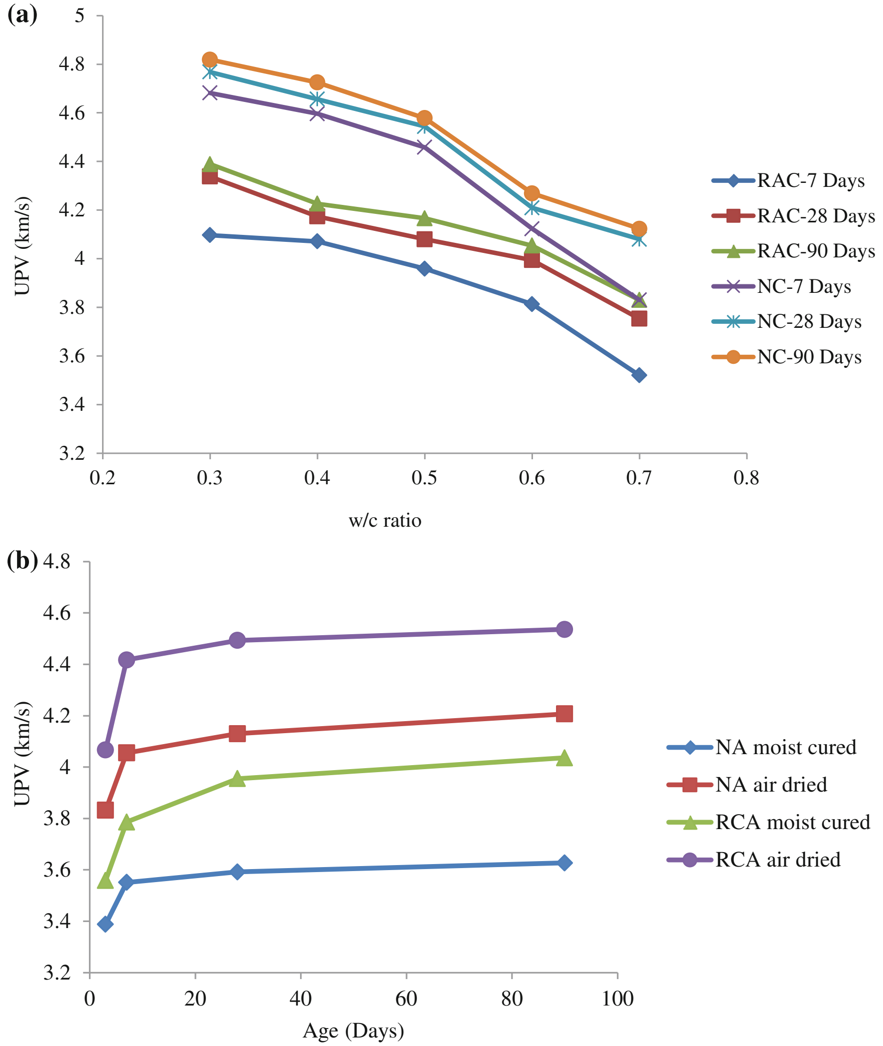

Fig. 4.35 **a** Relationship between ultrasonic pulse velocity and w/c ratio of concretes (Ravindrajah et al. 1988). **b** Development of pulse velocity with age (Ravindrajah et al. 1988)

high-performance RAC prepared with both recycled fine and coarse aggregate was lower than those of prepared with recycled coarse aggregate and natural fine aggregate.

Kou et al. (2011) reported the results of UPV of concrete mixes prepared with different percentages of RCA and different mineral admixtures (Fig. 4.36). It was reported that the UPV of RAC was lower than normal concrete. It was further reported that the UPV of both recycled aggregate concrete and normal concrete was improved with the addition of SF and MK, whereas concrete with FA and GGBS

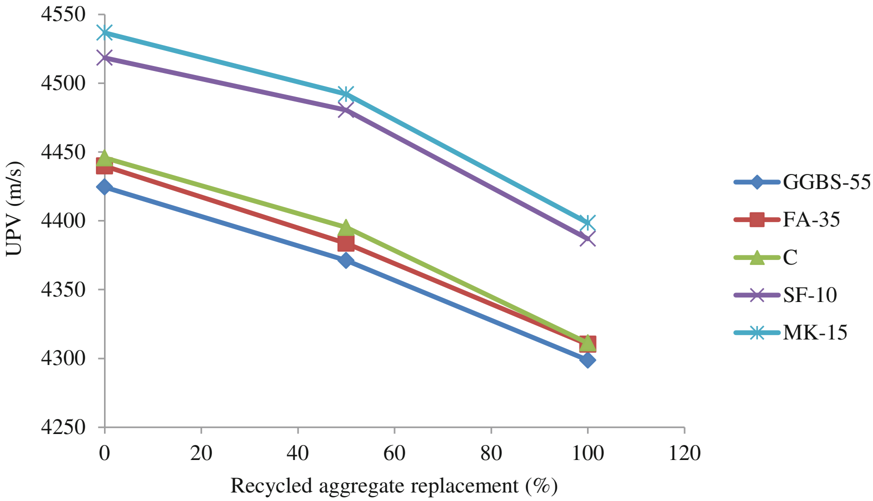

Fig. 4.36 Effect of recycled aggregates and mineral admixtures on UPV of concrete mixtures at 28 days (Kou et al. 2011)

has shown lower values of UPV than that of corresponding control concrete. Furthermore, it was reported that the UPV of RAC mixes with mineral admixtures exhibits higher percentage gain when compared to the concrete with natural aggregates. This may be attributed to the improved bond strength and microstructure of ITZ between RA and new cement paste due to the addition of mineral admixtures.

The test results of UPV variation with age of both normal and recycled aggregate concrete made with different w/c ratios reported by Latif-Al-Mufti and Fried (2012) is shown in Figs. 4.37, 4.38 and 4.39. It was found that in the first few hours after mixing, the UPV of RAC mixes were increasing at a rising rate than normal concrete but then it was slow down at later age, i.e., up to 28 days in RAC compared to normal concrete. As reported earlier in the literature by other researchers (Ravindrajah et al. 1988; Tu et al. 2006), the authors found that the reduction in w/c ratio increases the UPV of both normal and recycled concrete mixes; however, the difference was found to be less in RAC compared to normal concrete. This might be attributed to the variance in pore structure and surface characteristics of RCA and natural aggregate. Irrespective of the w/c ratio, the UPV of RAC was lower than normal concrete and the difference in UPV between the normal concrete and recycled aggregate concrete was found to be larger than the difference obtained in strength, particularly during the hardening stage. This indicates that the UPV was more influenced by the changes in the properties of RCA and it moves more through the concrete's paste, aggregate and boundaries than the overall strength of the concrete.

Kou and Poon (2015) examined the ultrasonic pulse velocity of normal-strength (45 MPa) and high-performance (HP) RAC prepared with 100% RCA obtained

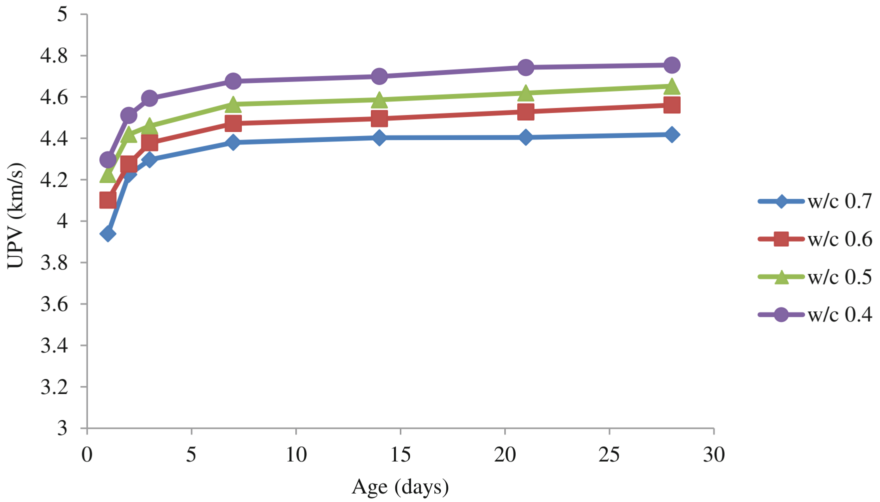

Fig. 4.37 UPV variation at early age for normal concrete with different w/c ratios (Latif-Al-Mufti and Fried 2012)

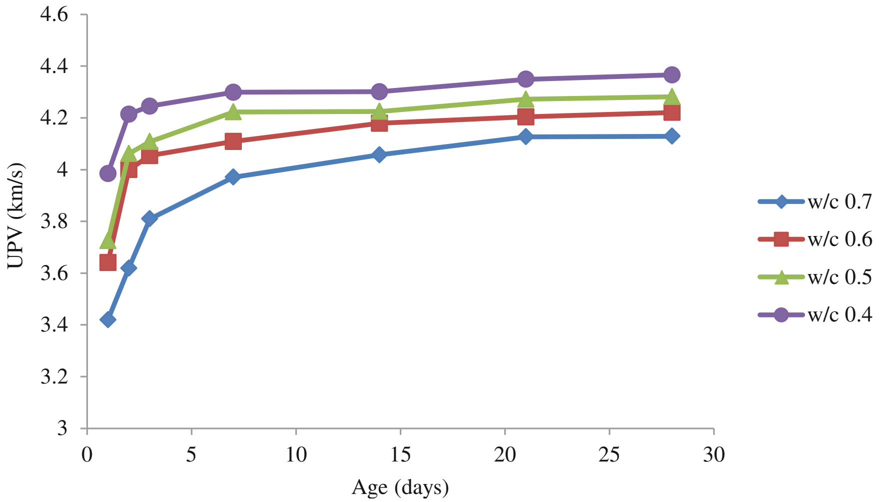

Fig. 4.38 UPV variation at early age for RAC with different w/c ratios (Latif-Al-Mufti and Fried 2012)

from different strengths of parent concrete (PC). Two series of mixes were considered: Series I mixes were made with a w/c of 0.35 and Series II mixes were prepared with a w/c of 0.50. In each series, RCA obtained from 30, 45, 60, 80, and 100 MPa strength of parent concretes was used. The experimental results of the investigation are presented in Fig. 4.40. It indicates that the UPV of RAC in both

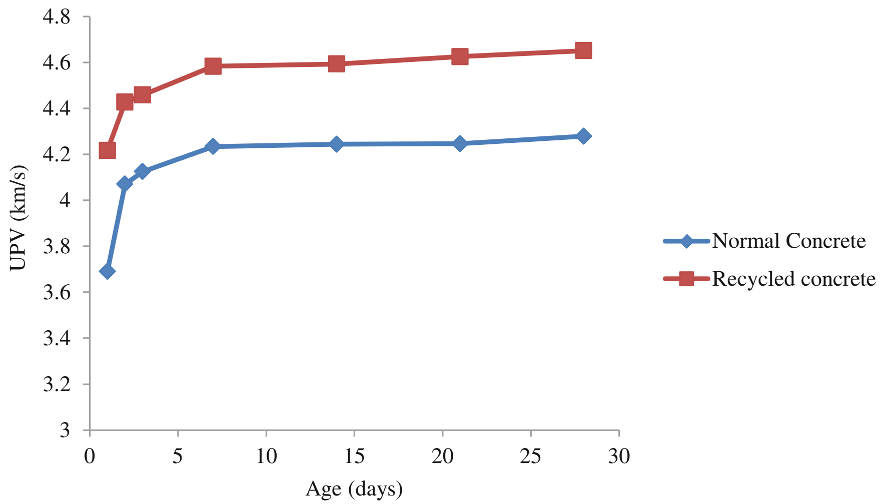

Fig. 4.39 UPV variation at early age for normal and RAC with w/c 0.5 (Latif-Al-Mufti and Fried 2012)

the series was lower than the corresponding concrete with natural aggregates. However, the difference was reduced with the use of RCA obtained from higher strength of parent concrete. When RAC made with 100% RCA obtained from the parent concrete of 100 MPa strength, the UPA was found to be only 2.5% lower than its corresponding control concrete at 90 days age. Further, as reported by previous researchers (Ravindrajah et al. 1988), with the increase in curing period, the UPV increased in both the concretes. The decrease in UPV in RAC was attributed to the greater porosity and lower density of RA when compared to NA and further in solid medium, the UPV depends on the elastic properties and density of materials.

4.4 Interrelationships Among Mechanical Properties

Similar to normal concrete, the interrelationships among different properties of RAC are also important for the analysis and design of reinforced cement concrete structures, particularly, when this type of new material is used. For normal concrete, different National Codes have published the well-established relationships between various properties of concrete. However, in case of RAC, limited attempts were made to establish the relationships among the important properties of RAC based on their own experimental investigations. Xiao et al. (2006) made a review on the interrelationships among the mechanical properties of RAC based on the experimental results published by researchers worldwide from 1985 to 2004. It was concluded that the existing relationships for normal concrete were not suitable for

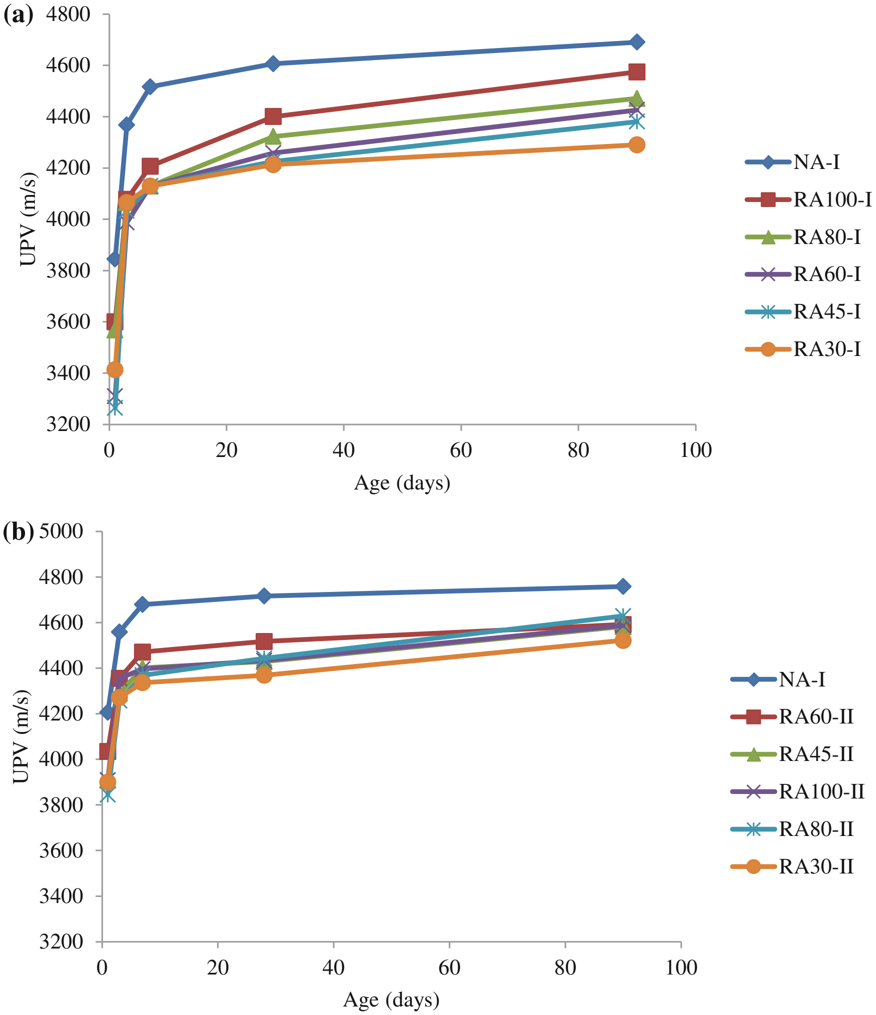

Fig. 4.40 Development of UPV of concrete mixes (**a**) in Series I and (**b**) in Series II (Kou and Poon 2015)

RAC. Further modified equations were suggested to establish the relationships among the important properties of RAC from the database available up to 2004. From 2004 to 2015, many researchers focused on the improvement of the properties of RAC by using different techniques; hence, further investigation is required to establish appropriate relationships between the important properties of RAC. Considering the findings of previous research, an attempt is made to establish appropriate relationships among important properties of RAC on the basis of a multitude of experimental results published globally during 1990–2015 including the experimental results, generated by the present authors. In the present context,

Table 4.13 Conditions for selection of experimental results for the preparation of database

Description of parameters	Test condition/range of values
Fine aggregate	Natural sand
Recycled coarse aggregate content	10–100%
Brick content	0–16%
Fly ash	0–35%
Silica fume	0–20%
Water–cement ratio	0.3–0.7%
Density	2100–2450 kg/m^3
Cube compressive strength	10–80 MPa
Age of testing	28 days
Curing condition	Standard/water
Method of mixing	Normal/two-stage
Source of recycled coarse aggregate	Laboratory samples/demolished concrete structures/precast elements/airport runways

based on the widely published results, particularly on important properties of RAC, a database is prepared. Using this database, a static regression analysis is conducted to propose the new improved equations for describing the relationships among the important properties of RAC. The conditions considered for selecting the test results for the preparation of database are as shown in Table 4.13.

4.4.1 *Relationship Between Compressive Strength and Split Tensile Strength*

Obtaining the direct tensile strength of concrete is difficult experimentally, and hence, tensile strength of concrete is normally obtained from the split tensile strength test of concrete. However, in practical conditions, the compressive strength of concrete is usually considered to calculate the tensile strength of concrete. The compressive strength (f_{ck}) and split tensile strength (f_{sp}) relationship in normal concrete was well established in the literature. As per the Chinese Code GB 50010 (2002) and the ACI 318 (2002) Code, the split tensile strength was related to compressive strength in case of normal concrete as in Eqs. 4.1 and 4.2.

$$f_{\mathrm{sp}} = 0.19 \times f_{ck}^{0.75} \tag{4.1}$$

$$f_{\mathrm{sp}} = 0.49 \times f_{ck}^{0.5} \tag{4.2}$$

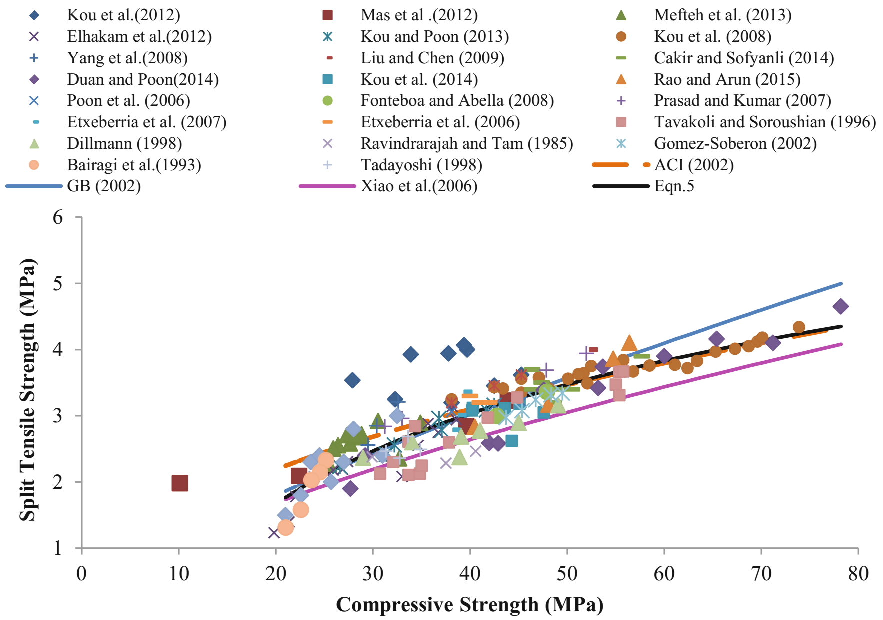

Fig. 4.41 Compressive strength and split tensile strength relationship

where f_{ck} and f_{sp} are in MPa and it should be noted that a conversion from cylinder to cube compressive strength has been considered in Eq. 4.2 according to Xiao et al. (2006).

Even though some attempts were made to correlate the split tensile strength with compressive strength for RAC, those attempts were limited to their particular study. Xiao et al. (2006) had suggested an equation which relates the compressive strength and split tensile strength of RAC based on the database available worldwide from 1985 to 2004 and it was stated as (Eq. 4.3)

$$f_{sp} = 0.24 \times f_{ck}^{0.65} \tag{4.3}$$

The split tensile strength of RAC in terms of compressive strength reported by different researchers available in the literature is presented in Fig. 4.41. The relation obtained from Eqs. 4.1–4.3 are also plotted in Fig. 4.41. It can be seen from Fig. 4.41 that the ACI Code underestimates the split tensile strength for lower values of compressive strength, whereas the Chinese Code GB:50010 (2002) overestimates the split tensile strength for higher values of compressive strength. Similarly, the equation given by Xiao et al. (2006) for RAC is underestimating the split tensile strength for almost all the values of compressive strength. In this study, an alternative statistical regression analysis is performed on the results presented in Fig. 4.41 by adopting the following regression equation in which the regression coefficients are A and B.

$$f_{sp} = A \times ln(f_{ck}) + B \tag{4.4}$$

The regression analysis yields the coefficients A and B as 1.9662 and −4.221 with an R value equal to 0.92. Therefore, the following equation is suggested to express the split tensile strength in terms of the compressive strength for RAC.

$$f_{sp} = 1.9662 \times ln(f_{ck}) - 4.221 \tag{4.5}$$

Equation 5 is presented in Fig. 4.41, and it is clearly demonstrated that the proposed equation provides a better description of the relationship between compressive strength and split tensile strength in comparison with Eqs. 4.1–4.3.

4.4.2 Compressive Strength and Flexural Strength or Modulus of Rupture Relationship

The modulus of rupture or flexural strength is also a measure of the tensile strength of concrete. Many researchers had carried out their studies on flexural strength and compressive strength of RAC. Based on the data gathered from the literature, the variation of flexural strength with compressive strength is presented in Fig. 4.42.

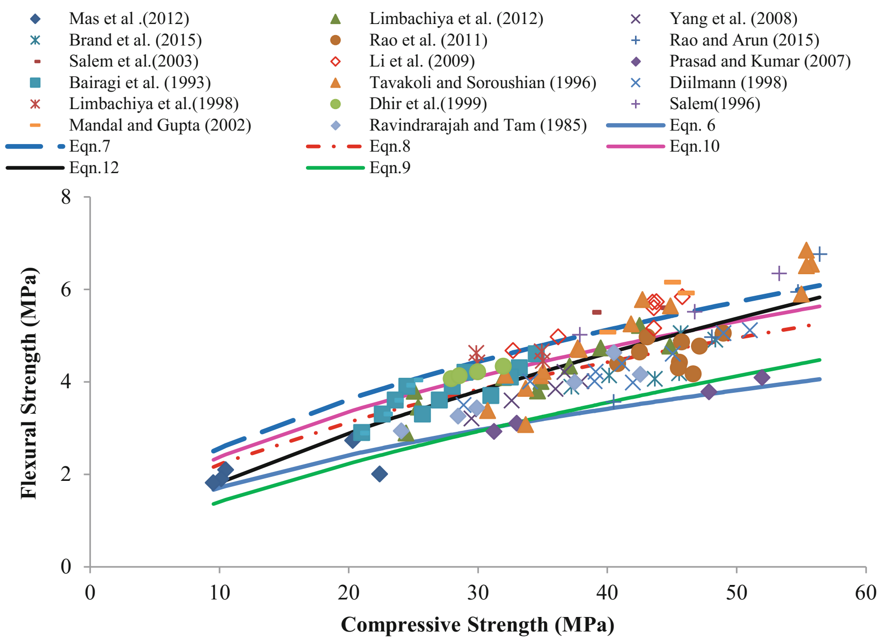

Fig. 4.42 Flexural strength and compressive strength relationship

It is found that even though the results are widespread, the flexural strength increases with the increase in compressive strength. Various National Standard Codes have published the relationship between compressive strength and flexural strength for NC. As per the ACI 318 (2002), CEB (1990), IS 456 (2000), and EC-02 (Beeby and Narayanan 1995), the variation of flexural strength (f_f) with reference to compressive strength (f_{ck}) is presented in Eqs. 4.6–4.9. It is to be mentioned that as per Xiao et al. (2006), a conversion factor of 0.76 has been taken to convert the cube compressive strength to cylinder compressive strength in Eqs. 4.6 and 4.7.

$$f_{\mathrm{f}} = 0.54 \times \sqrt{f_{\mathrm{ck}}} \qquad \text{As per ACI} \tag{4.6}$$

$$f_{\mathrm{f}} = 0.81 \times \sqrt{f_{\mathrm{ck}}} \qquad \text{As per CEB} \tag{4.7}$$

$$f_{\mathrm{f}} = 0.7 \times \sqrt{f_{\mathrm{ck}}} \qquad \text{As per IS:456} \tag{4.8}$$

$$f_{\mathrm{f}} = 0.3 \times f_{\mathrm{ck}}^{0.67} \qquad \text{As per EC} - 02 \tag{4.9}$$

Further, Xiao et al. (2006) had suggested an equation which relates the flexural strength to the compressive strength of RAC and is presented in Eq. 4.10.

$$f_{\mathrm{f}} = 0.75 \times \sqrt{f_{\mathrm{ck}}} \qquad \text{As per Xiao etal.} \tag{4.10}$$

where flexural strength (f_f) and compressive strength (f_{ck}) are in MPa.

The variation of the flexural strength with compressive strength for recycled aggregate concrete based on Eqs. 4.6–4.10 is shown in Fig. 4.42. From the figure, it is observed that Eqs. 4.6 and 4.9 yield underestimation and Eq. 4.7 provides overestimation of the flexural strength of the RAC. Further both Eqs. 4.8 and 4.10 overestimate the flexural strength of recycled aggregate concrete for relatively lower values of compressive strength. Hence, for better description of the relationship, using the database prepared from the experimental results published by different researchers, a regression analysis is carried out by using the following regression Eq. 4.11.

$$f_{\mathrm{f}} = C.\,(f_{\mathrm{ck}})^{D} \tag{4.11}$$

where C and D are the regression coefficients. The regression analysis gives the values of C and D as 0.3803 and 0.6768, respectively, with R = 0.87. Therefore, the flexural strength and compressive strength relationship in RAC may be stated as

$$f_{\mathrm{f}} = 0.3803 \times (f_{\mathrm{ck}})^{0.6768} \tag{4.12}$$

Eq. 4.12 is also presented in Fig. 4.42. It is clearly shown that Eq. 4.12 is more effective for describing the variation of the flexural strength with the compressive strength for RAC when compared to Eqs. 4.6–4.10.

4.4.3 *Compressive Strength and Static Modulus of Elasticity Relationship*

The elastic modulus is another important property of concrete. In the present study, a database of elastic modulus and compressive strength is prepared from the results published by different researchers worldwide. The relationship of elastic modulus with compressive strength is plotted in Fig. 4.43 based on the prepared data. Different National Standards viz. ACI 318 (2002), BS 8110 (BS (British Standards) 1997), GB 50010 (2002), IS 456 (2000) have given the relationship between elastic modulus and compressive strength for NC and Ravindrarajah and Tam (1985), Dhir et al. (1999), Xiao et al. (2006) had proposed the relationship of elastic modulus with compressive strength in RAC and are shown in Eqs. 4.13–4.19.

$$E_c = 4127 \times f_s^{0.5} \qquad \text{As per ACI} \tag{4.13}$$

$$E_c = 20000 + 0.2 \times f_{ck} \qquad \text{As per BS 8110} \tag{4.14}$$

$$E_c = \frac{10^5}{2.2 + \frac{34.7}{f_{ck}}} \qquad \text{As per GB} \tag{4.15}$$

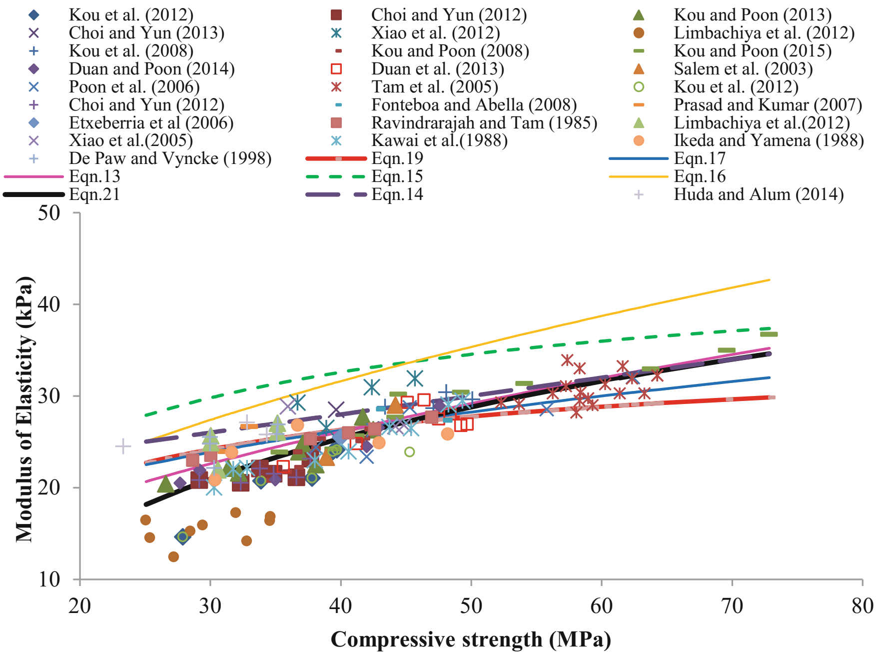

Fig. 4.43 Modulus of elasticity and compressive strength relationship

$$E_c = 5000 \times f_{ck}^{0.5} \qquad \text{As per IS : 456} - 2000 \tag{4.16}$$

$$E_c = 7770 \times f_{ck}^{0.33} \qquad \text{As per Ravindrarajah and Tam} \tag{4.17}$$

$$E_c = 370 \times f_{ck} + 13100 \qquad \text{As per Dhir etal.} \tag{4.18}$$

$$E_c = \frac{10^5}{2.8 + \frac{40.1}{f_{ck}}} \qquad \text{As per Xiao etal.} \tag{4.19}$$

where compressive strength (f_{ck}) and static modulus of elasticity (E_c) are in MPa. In Eq. 4.13, the cylinder compressive strength is taken as 0.76 times the cube compressive strength (f_{ck}) as per Xiao et al. (2006).

Equations 4.13–4.19 are also presented in Fig. 4.43. The figure reveals that the relationship suggested by ACI 318 (Eq. 4.13) between compressive strength and elastic modulus of NC is more effective for RAC. Further, it is found that BS 8110 (1997) (Eq. 4.14) and IS 456 (2000) (Eq. 4.16) overestimate the elastic modulus for all the values of compressive strength of RAC. Further for relatively lower values of compressive strength, Eq. 4.14 overestimates the elastic modulus of RAC. Similarly, equation presented by Xiao et al. (2006) (Eq. 4.19) leads to an underestimation of elastic modulus for relatively higher values of compressive strength of recycled aggregate concrete, whereas the equation suggested by Ravindrarajah and Tam (1985) (Eq. 4.17) overestimates the elastic modulus for relatively lower values of compressive strength of recycled aggregate concrete. To improve the relationship between elastic modulus and compressive strength of RAC, a statistical regression analysis is carried out on the data from the results published in the literature using Eq. 4.20.

$$E_c = g \times \ln(f_{ck}) - h \tag{4.20}$$

where g and h are the regression coefficients. The regression analysis yields R equal to 0.87 with regression coefficients g = 15.438 and h = 31.586. Hence, the relationship between compressive strength and elastic modulus of RAC may be represented as

$$E_c = 15.438 \times \ln(f_{ck}) - 31.586 \tag{4.21}$$

Equation 4.21 is also presented in Fig. 4.43 for comparison. It is observed that when compared to Eqs. 4.14–4.19, Eq. 4.17 offers closer relationship between compressive strength and elastic modulus of RAC with Eq. 4.21. Further, it is observed that the relationship proposed by ACI 318 (Eq. 4.13) also predicts the results closer to Eq. 4.21. Therefore, both Eqs. 4.13 and 4.21 may be used to estimate the elastic modulus from the compressive strength of RAC.

4.4.4 *Compressive Strength and Ultrasonic Pulse Velocity (UPV) Relationship*

In existing/damaged concrete structures the nondestructive testing (NDT) plays a vital role in evaluating the concrete quality. The ultrasonic pulse velocity (UPV) is more attractive and widely used NDT test for assessing the quality particularly the compressive strength of concrete. Therefore, in this sec, the authors tried to establish a relationship between the UPV and compressive strength of RAC based on the results published by different researchers in the literature worldwide. The experimental results published by different authors are presented in Fig. 4.44. Ravindrarajah et al. (1988) had proposed an empirical formula which relates the UPV and compressive strength of RAC and is given in Eq. 4.22 and also shown in Fig. 4.44.

$$f_{ck} = 0.008 \times e^{2.06 \times V} \qquad \text{As per Ravindrarajah etal.(1988)} \tag{4.22}$$

The figure reveals that Eq. 4.22 yields an overestimation of compressive strength for a given value of ultrasonic pulse velocity of RAC. In the present study, a regression analysis is carried out based on the experimental results published using the relationship shown in Eq. 4.23.

$$f_{ck} = i \times \left(V^{j}\right) \tag{4.23}$$

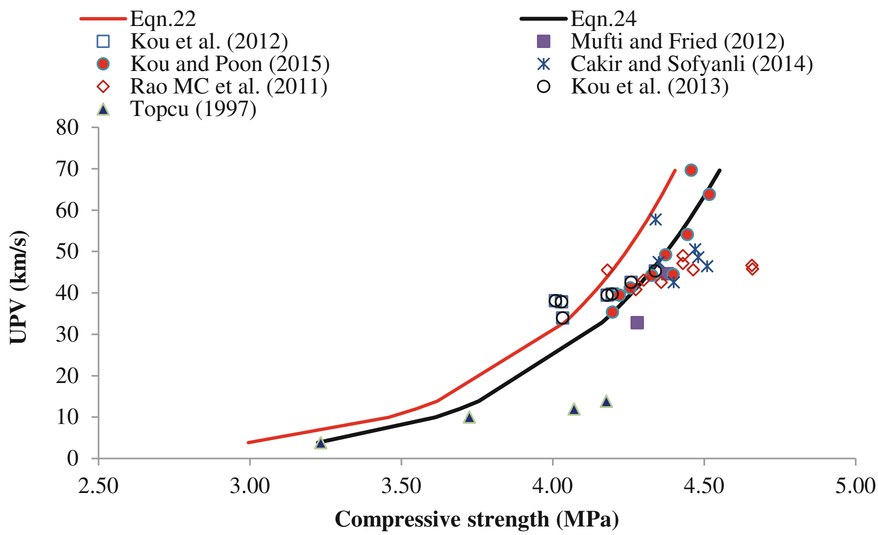

Fig. 4.44 Compressive strength and UPV relationship

where i and j are the regression coefficients. The analysis gives i = 0.0006 and j = 7.6164 and R equal to 0.85. Therefore, Eq. 4.24 is suggested for better description of the relationship between UPV and compressive strength of RAC, where f_{ck} and V are in MPa and km/s, respectively.

$$f_{\mathrm{ck}} = 0.0006 \times \left(V^{7.6164}\right) \tag{4.24}$$

Eq. 4.24 is plotted in Fig. 4.44. It is found that the proposed equation in the present study describes more effective relationship between UPV and compressive strength of RAC.

4.4.5 *Compressive Strength and Density Relationship*

The compressive strength of concrete is significantly influenced by w/c ratio and relative volume of air in the mix. Since the cement, water, and aggregates have different densities, the overall density of concrete mix will depend on the relative amounts of the ingredients of concrete. Hence, a relationship between density and compressive strength of concrete may be explored. In this study, the relationship between the compressive strength and density of RAC based on the data collected from the literature available is explored. With the available data, the variation of compressive strength with density in RAC is presented in Fig. 4.45. It can be seen

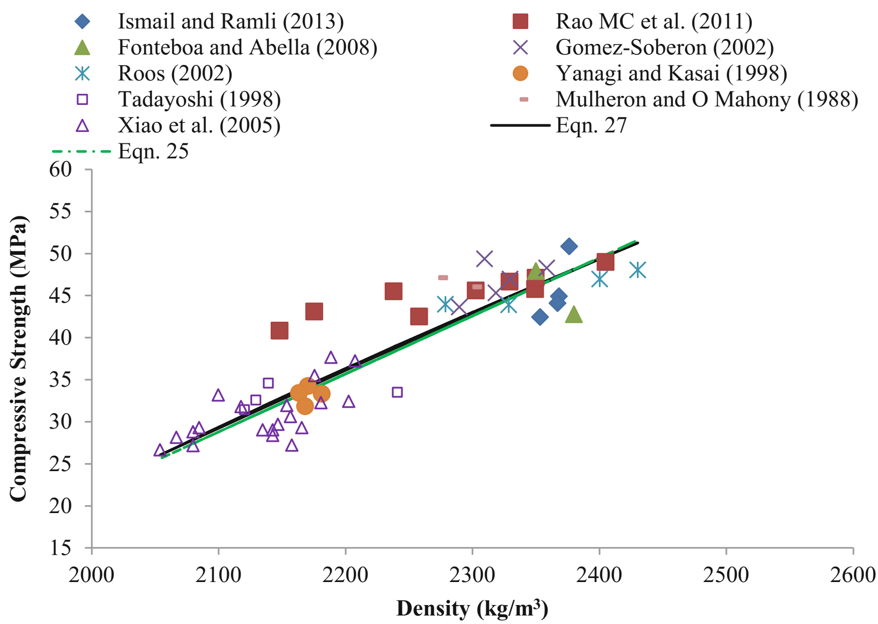

Fig. 4.45 Compressive strength and density relationship

from the figure that the compressive strength of RAC increased as the density increased. Xiao et al. (2006) described the relationship between density and compressive strength of RAC as given in Eq. 4.25.

$$f_{\mathrm{ck}} = 0.069\rho - 116.1 \qquad \text{As per Xiao } et\,al. \tag{4.25}$$

where compressive strength f_{ck} is in MPa and mass density ρ in kg/m^3.

In the present investigation, based on the database developed, a static regression analysis is carried out using the following relationship with the regression coefficients k and m.

$$f_{\mathrm{ck}} = k \times \ln(\rho) + m \tag{4.26}$$

The regression analysis yields a value of R = 0.94 and k = 149.91 and m = −1117.4. Thus, Eq. 4.27 gives the relationship between density and compressive strength of RAC and Eq. 4.27 is also plotted in Fig. 4.45 for comparison.

$$f_{\mathrm{ck}} = 149.91 \times ln(\rho) - 1117.4 \tag{4.27}$$

It is observed that the results predicted by Eq. 4.25 are very close to the results predicted by the present Eq. 4.27. Therefore, both Eqs. 4.25 and 4.27 may be adopted for estimating the relationship of compressive strength with density of RAC.

4.5 Summary

The properties of recycled aggregate concrete at fresh and hardened state are discussed. The influence of various factors on these properties is also described with illustrations. Further, the interrelationships among the important properties of RAC are illustrated. Based on these discussions the following important aspects observed.

- To attain the same workability, the RAC made with recycled coarse aggregates and natural sand needs 5% extra water than concrete with natural aggregate. If sand is also recycled one, it needs 15% more water. This was due to the rough surface and more angularity of recycled aggregates against to smooth and round surfaces of natural aggregates. There was a contradiction to the above statement, as the presence of old cement mortar, the recycled aggregate concrete mixes are more cohesive than normal concrete mixes. Few researchers suggested that the recycled aggregates should be pre-wetted to prevent a rapid reduction in workability.
- The moisture conditioning of the aggregates severely affects the workability of RAC. Aggregates under oven-dry condition headed to a larger initial slump and faster slump loss when compared to SSD and AD aggregates. The initial slump

of concrete greatly depends on the initial free water content of concrete mixes. The slump of normal-strength RAC mixes decreased with the increased strength of parent concrete from which the RCA obtained. A similar trend was observed in the initial slump of high-performance RAC mixes due to the lower water absorption capacity of recycled coarse aggregate which was obtained from higher-strength parent concrete.
- The compressive strength of RAC made with 100% recycled coarse aggregate is less than that of normal concrete. However, the variation is not unique.
- The strength of RAC mainly depends on the strength of parent concrete (w/c ratio) form which the recycled aggregates derived. When the w/c ratio of parent concrete from which the RCA derived is the same or lower than the w/c of RAC, then the strength of RAC was the same or even better than the strength of parent concrete and vice versa. Higher strength of RAC than parent concrete could be produced with higher cement content than that was used in normal concrete. The compressive strength of HPC made with RCA obtained from lower strength parent concrete significantly reduced, whereas the strength of the same prepared with high-strength (80 MPa and 100 MPa) parent concrete aggregates was equal to or even slightly more than that of the concrete with natural aggregate.
- The difference in compressive strength between normal concrete and recycled aggregate concrete decreased with the increase in w/c ratio up to 0.55 and thereafter both strengths are almost equal. At lower w/c ratio (<0.35), the failure occurred at the old ITZ, therefore, the difference in strengths between normal and RAC was higher.
- The properties of recycled aggregate concrete made with 0–100% recycled coarse aggregates have been studied and concluded that the recycled aggregate concrete made with 20–30% recycled coarse aggregates does not significantly affect the properties of concrete.
- The variation in the development of strength of recycled aggregate concrete with curing age follows the similar trend as that of normal concrete. However, the percentage gain in strength from 7 days to 28 days is slower than normal concrete.
- With 5–10% extra cement and 4–10% lower effective w/c ratio, one may produce equal strength of RAC with 50% and 100% RCA to that of normal concrete.
- The moisture condition of RCA also affects the compressive strength and tensile strength of concrete. The recycled aggregates under SSD condition seem to have more negative affect on compressive strength due to the presence of water in SSD aggregate, whereas AD aggregates have shown the highest compressive strength of RAC.
- Like in compressive strength, the percentage reduction in modulus of elasticity of RAC when compared to normal concrete was not unique. The modulus of elasticity of RAC made with any percentage of RCA was always lower than that of normal concrete. This was mainly due to the lower modulus of elasticity of

RA than natural aggregates. The difference between the modulus of elasticity of RAC and normal concrete is much larger, when both fine and coarse recycled aggregates are used in concrete.
- Irrespective of the strength of parent concrete from which the RCA derived, the modulus of elasticity of concrete made with recycled aggregate was lower than the conventional concrete. However, with the increased strength of parent concrete from which the RCA derived, the decrease in static modulus was reduced.
- Irrespective of the percentage of RCA inclusion, the flexural and split tensile strengths of RAC are always less than those of normal concrete and it is more significant at 100% inclusion of RCA.
- The improvement in tensile strengths of RAC with longer curing period (after 28 days) was more when compared to normal concrete.
- With the addition of pozzolanic materials (fly ash) in recycled aggregate concrete, the tensile strength of RAC may be improved due to the pozzolanic reaction of fly ash and possible reduction in porosity due to the compactness of the fine fly-ash particles.
- The strength of parent concrete from which the RCA produced strongly affects the tensile strength of RAC. The tensile strength of RAC was relatively improved and even reached to the values of the corresponding parent concretes when RAC made with an increase in strength of PC from which the RCA produced. This improvement may be due to the presence of relatively stronger bond between adhered mortar and aggregate surface in higher-strength parent concrete aggregate comparatively lower strength parent concrete aggregate, which makes the surface more rough and hence an improvement in the bond between RCA and new cement mortar.
- Recycled aggregates produced from HPC had better interface layer than those produced from the normal-strength concrete. This means that the strength of concrete from which the recycled aggregates derived influence the properties of ITZ. Binder also plays a vital role in the properties of ITZ.
- The density of RAC decreased with the increased amount of RCA due to the presence of old porous mortar attached on the surface of RCA. Further, method of mixing does not show any significant change in the density of RAC.
- The ultrasonic pulse velocity of concrete mixes increased with the decrease in w/c ratio. But irrespective of the w/c ratio, the UPV of RAC was always lower than normal concrete and the difference in UPV between the normal and recycled aggregate concretes were found to be larger than that obtained in strength requirements, particularly during the hardening stage. Further, the UPV of RAC made with RCA obtained from higher-strength parent concrete was little lower than the corresponding normal concrete.
- Based on a multitude of experimental results collected from the literature published worldwide a database is prepared mainly related to five important properties viz. compressive strength, static modulus of elasticity, flexural strength, split tensile strength, density and UPV of RAC. Using this database, a static regression analysis with least square method is carried out to suggest some

new relationships among the important properties of RAC. Further, the suitability of the well-established interrelationships published by different National Standards among the mechanical properties of normal concrete (NC) to RAC is examined. The same is summarized herein.

- The existing interrelationships between the mechanical properties of NC are not suitable for RAC as these are quite different from those of RAC. However, Eq. 4.13 proposed by ACI 318 for relating the static modulus of elasticity and compressive strength of normal concrete may be used for recycled aggregate concrete.
- Equation 4.25 suggested by Xiao et al. may also be effectively applied for relating the density and compressive strength of RAC.
- New improved equations are recommended to express the relationship between the following properties of RAC:

  (i) Split tensile strength and compressive strength,
  (ii) Flexural strength and compressive strength,
  (iii) Static modulus of elasticity and compressive strength,
  (iv) Compressive strength and ultrasonic pulse velocity (UPV), and
  (v) Compressive strength and mass density.

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Chapter 5
Long-Term and Durability Properties

5.1 Introduction

In previous chapter, mainly the discussion referred to the short-term properties particularly the strength aspects and elastic modulus of recycled aggregate. When concrete is subjected to sustained loading, the strain increases with respect to time. Further, whether subjected to load or not, concrete contracts on drying, undergoing shrinkage (Neville 2006). Therefore, the long-term properties like shrinkage and creep of recycled aggregate concrete reported by different researchers are discussed in the subsequent sections. It is necessary that all concrete structures should continue to perform its intended function, which maintained its required strength and serviceability, during its service life. Therefore, concrete must be able to sustain the deterioration process to which it can be expected to be exposed (Neville 2006). Hence, the important aspects of durability such as permeability, chloride penetration, and depth of carbonation of recycled aggregate concrete are discussed in this chapter. Drying shrinkage is a salient feature of recycled aggregate concrete (RAC), which is the reduction in volume of concrete due to the loss of capillary moisture from concrete that leads to the progress of capillary tension cracks inside the meso-pore structure of cement mortar (Behera et al. 2014). On the other hand, creep is the increase in strain under a constant stress. The important factors like amount of RA, w/c ratio, residual cement paste significantly affect these long-term properties and are discussed with illustrations in the subsequent sections. The research findings reported by different researchers reveal that both shrinkage and creep of RAC are significantly higher than that of conventional concrete due to the high absorption capacity of RA. But, the incorporation of fly ash in RAC can minimize these to a certain extent, and the results reported by different researchers are described. The durability performance of recycled aggregate concrete is the ability of concrete to withstand the external environmental agents such as ingression of chlorides, sulfates, acids, oxygen, carbon dioxide. The research results of various researchers reveal that the durability performance of RAC is more hapless than

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normal concrete. The poor durability performance of RAC is associated with many factors such as poor quality of RA, the presence of numerous microcracks, the presence of old and new interfacial transition zones, w/c ratio. The influence of these factors on the durability of the performance of RAC in terms of permeability, carbonation, and chloride penetration is discussed in this chapter too.

5.2 Shrinkage

Drying shrinkage is the decrease in concrete volume due to the loss of capillary moisture from concrete that leads to the progress of capillary tension cracks inside the meso-pore structure of cement mortar (Behera et al. 2014). The results presented by various researchers in the literature show that the drying shrinkage of recycled aggregate concrete was more than that of concrete with natural aggregate, and the magnitude of shrinkage of RAC depends on many factors such as the amount of recycled aggregate, method of mixing, method of crushing procedure, quality of recycled aggregate, curing conditions, use of mineral admixtures. The influence of each of the factors is discussed subsequently in the following sections.

5.2.1 Influence of Amount of Recycled Aggregate

A number of experimental investigations revealed that the drying shrinkage of recycled aggregate concrete increases with the increase in amount of recycled aggregate (Kou et al. 2008, 2011; Yang et al. 2008; Domingo-Cabo et al. 2009; Limbachiya et al. 2012a; Tam et al. 2015; Matias et al. 2013; Sagoe-Crentsil et al. 2001; Kou and Poon 2012; Kwan et al. 2012; Tam and Tam 2007). This was due to the increased volume of total cement paste by the contribution of attached cement mortar to the recycled aggregate (Tavakoli and Sorousian 1996). Sanchez de Juan and Gutierrez (2004) found that the shrinkage strain ranges between 15 and 60%. Domingo-Cabo et al. (2009) observed that the shrinkage of RAC with 50% and 100% RA were 20% and 70%, respectively, higher than that of normal concrete at 180 days. Further, it has shown that at 20% substitution of RA, the shrinkage of RAC was equivalent to normal concrete at the initial stage. The drying shrinkage of RAC with 100% RA was 25% more than the conventional concrete (Sagoe-Crentsil et al. 2001). Limbachiya et al. (2012b) found that 30% replacement of NA by RA does not show any significant effect on the drying shrinkage of RAC, but it significantly increases with the increase in RCA content. The summary of the past research findings presented by Tam et al. (2015) is shown in Table 5.1.

The results of drying shrinkage of concrete mixes prepared with different percentages of RCA over a period of 252 days reported by Domingo-Cabo et al. (2009) are presented in Fig. 5.1. It was reported that the shrinkage deformation of RAC increased with the increase in the percentage of RCA. At early age, the

Table 5.1 Summary of the research findings of the previous researchers (Tam et al. 2015)

Source	% RCA	Shrinkage (%)	
Dhir et al. (2004)	30	0.67	Larger
	50	4.86	Larger
	100	12.92	Larger
Dumitru et al. (2000)		36.36	Larger
Poon and Kou (2004)	20	5.91–10.37	Larger
	50	11.82–17.53	Larger
	100	20.82–33.33	Larger
Sagoe-Crentsil et al. (2001)	100	35	Larger
Selih et al. (2003)	100	57.14	Larger
Tam (2005)	0% with TSMA	0.02	Larger
	20 with TSMA	0.01%	Smaller
	100% with TSMA	0.08	Smaller
Teranishi et al. (1998)	50%	53.40	Larger
Yanagi et al. (1993)	30	0.4–30.9	Larger
	50	0.1–29.9	Larger
	100	5.9–40	Larger

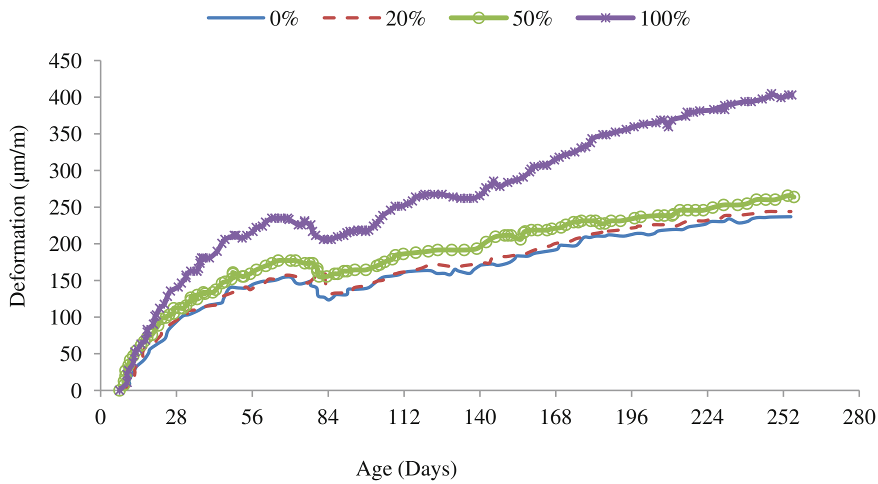


Fig. 5.1 Shrinkage deformation of concrete mixes with age (Domingo-Cabo et al. 2009)

shrinkage deformation of RAC prepared with 20% RCA was same as control concrete, and at 180 days, the shrinkage deformation was 4% more than the conventional concrete. In case of 50 and 100% RCA, the shrinkage deformations were 12 and 70% more than the normal concrete. The authors finally concluded that below 50% substitution level of RCA, the shrinkage trend was similar to the conventional concrete.

Table 5.2 Correction factors for shrinkage of RAC (Silva et al. 2015)

Shrinkage correction factors		
20% RCA	50% RCA	100% RCA
1.2	1.4	1.8

Based on the results reported by different researchers in the literature, Silva et al. (2015) suggested a new set of correction factors for different substitution levels of RCA in place of NA, and it is presented in Table 5.2. Using these correction factors, the shrinkage strain of recycled aggregate concrete for a given substitution level of RCA can be determined by multiplying the shrinkage strain of normal concrete.

5.2.2 Effect of Quality of Recycled Aggregate

Hansen and Boegh (1985) studied the drying shrinkage of high, medium, and low strength RAC made with recycled coarse aggregate obtained from high, medium, and low strength parent concretes. Four 100 × 100 × 600 mm specimens from each concrete mix were dried at 25º C, and 40% relative humidity for 440 days and drying shrinkage was measured. The authors concluded that the drying shrinkage of RAC made with recycled coarse aggregate and natural sand was 40–60% higher than the concrete from which the recycled aggregate derived, and in case when recycled fine aggregate was used in addition to recycled coarse aggregate, the shrinkage increases further. Use of recycled aggregate obtained from low strength concrete can result in shrinkage values several times higher than that of normal concrete, and therefore, care should be taken while selecting the concrete rubble for production of RCA.

Tavakoli and Sorousian (1996) in their study reported that the drying shrinkage of RAC more than that of normal concrete and the amount of increase depends on the quality of concrete from which the RCA obtained. The drying shrinkage of RAC increased with the increase in w/c ratio as in normal concrete. In case of normal concrete, the shrinkage decreased with the increased size of coarse aggregate, whereas, in case of RAC, effect of size of coarse aggregate on drying shrinkage depends on the quality of source concrete from which the RA derived, properties of RA and RAC. Further, the drying shrinkage of RAC was larger with the larger content of cement mortar adhered to recycled aggregates. Water absorption of recycled aggregate gives good indication for the content of cement mortar adhered to recycled aggregate.

Yang et al. (2008) investigated the influence of different amounts of recycled coarse and fine aggregates of different quality on the drying shrinkage of RAC mixes. The quality of RA was decided based on the water absorption capacity: Lower the water absorption better the quality of recycled aggregate. The authors found that the rate of development of shrinkage strain was more in the first 10 days and slowed down in the later ages. It was further observed that in the first 10 days the shrinkage of RAC mixes was lower than the control/normal concrete mixes due to the initial higher water absorption of RA. However, the shrinkage of RAC mixes was more than the control concrete mixes at later ages. Particularly, this was more noticeable with 100% recycled fine aggregate and recycled coarse aggregate of higher water absorption capacity compared to lower absorption capacity of recycled coarse aggregate. The influence of the quality and replacement level of RA on shrinkage strain from 10 to 91 days reported by the authors revealed that the long-term shrinkage was increased with the increase in water absorption of RA.

The influence of w/c ratio and fly-ash addition (by weight of cement) on the drying shrinkage of RAC mixes made with different percentages of RCA has been studied by Kou et al. (2008). The authors considered four Series of mixes, viz: Series I, II, III, and IV with w/c ratios 0.55, 0.50, 0.45, and 0.40, respectively. In each series 0, 20, 50, and 100% recycled coarse aggregate in place of natural aggregate and 25% fly ash by weight of cement has been incorporated. The results reported by the authors are depicted in Fig. 5.2.

It was found that with the increased content of RCA, the shrinkage of RAC increased regardless of w/c ratio. This was due to the increased volume of total cement paste by the contribution of attached cement mortar in recycled aggregate (Tavakoli and Sorousian 1996). However, the authors reported that the drying shrinkage could be minimized by the incorporation of fly ash or by lowering the w/c ratio. Furthermore, it was reported that reducing w/c ratio from 0.55 to 0.40 was more effective method to alleviate the drying shrinkage than by incorporation of 25% fly ash by weight of cement. Since the compressive strength increased with the

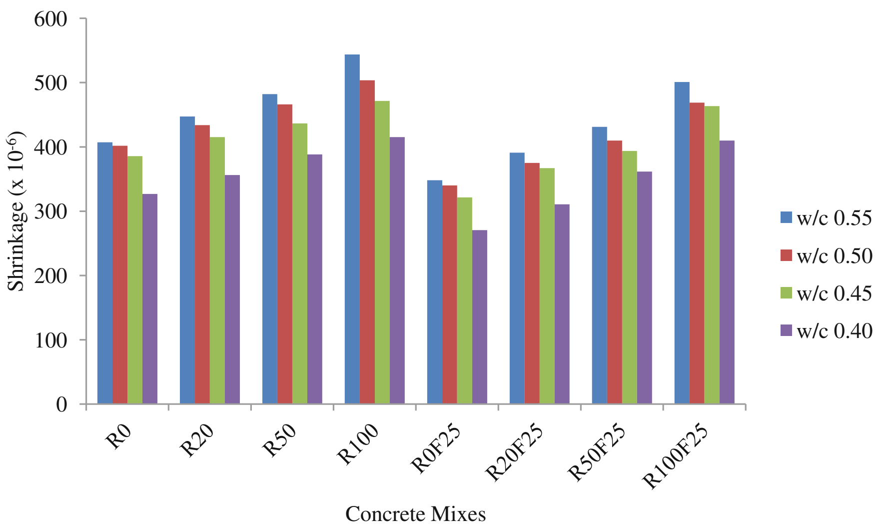


Fig. 5.2 Drying shrinkage of concrete mixes at 112 days (Kou et al. 2008)

decrease in w/c ratio and or addition of fly ash in concrete mixes, a negative relationship was established between compressive strength and drying shrinkage and it was submitted that drying shrinkage decreased with the increase in compressive strength (Kou et al. 2008).

Tam et al. (2015) investigated the shrinkage behavior of RAC of different mix proportions, which include different amounts of RCA (0, 30, and 100%), different w/c ratios (0.35, 0.45, and 0.6) and various aggregate-to-cement ratios (3, 4.5, and 6). It was found that for a given w/c ratio and aggregate–cement ratio, the drying shrinkage increased with the increased percentage of RCA incorporation. The highest shrinkage of 0.12108% was found at 100% RCA with w/c ratio 0.45 and aggregate-to-cement ratio 6, while the lowest shrinkage was about 0.06137% developed in normal concrete (0% RCA) at w/c ratio 0.45 and aggregate-to-cement ratio 4.5. However, there was no significant difference observed between 0% and 30% RCA samples. For a given w/c ratio, the drying shrinkage of RAC increased with increase in recycled aggregate content at different aggregate-to-cement ratios. At aggregate-to-cement ratio 3, the drying shrinkage of RAC with 0%, 30%, and 100% RCA was 0.08519%, 0.08319%, and 0.09978%, respectively, and at aggregate-to-cement ratio 6, the shrinkage of RAC was 0.08344%, 0.09741%, and 0.10491%, respectively, with 0, 30, and 100% RCA at 182 days. A saturated cement paste will remain dimensionally unstable, when concrete is exposed to humidity below saturation, this is partially due to the forfeiture of physically absorbed water from calcium silicate hydrate [CaO. SiO_2–H_2O, CSH], which consequences in shrinkage strain (Neville 1995; Mehta 1993). Table 5.3 shows the factors affecting drying shrinkage and creep. Based on these, it was reported that as the recede of physically absorbed water to fulfill the recycled aggregate requirement was increased with the increased amount of RA, the drying shrinkage increased. Further, it was found that the effect of w/c ratio and aggregate-to-cement ratio on drying shrinkage behavior of RAC was not clear (Figs. 5.3 and 5.4). Although, it was not acquitted that the drying shrinkage can lower whether with the higher or lower w/c ratio. It was observed that the lowest drying shrinkage occurred with medium w/c ratio of 0.45. Also it was observed that the drying shrinkage with aggregate-to-cement ratio 3 was close in touch with aggregate-to-cement ratio 6 and the maximum drying shrinkage was noted with aggregate-to-cement ratio 6.

Ajdukiewicz and Kliszczewicz (2002) studied the shrinkage of high strength RAC made with 100% recycled fine and coarse aggregate obtained from granitic

Table 5.3 Parameters affecting the shrinkage and creep (Neville 1995; Wu and Zhou 1988)

Paste cement parameters	Porosity, age of paste, curing temperature, cement composition, moisture content, admixture
Concrete parameters	Aggregate stiffness, aggregate content, volume surface ratio, thickness
Environmental parameters	Applied stress, duration of load, relative humidity, rate of drying, time of drying

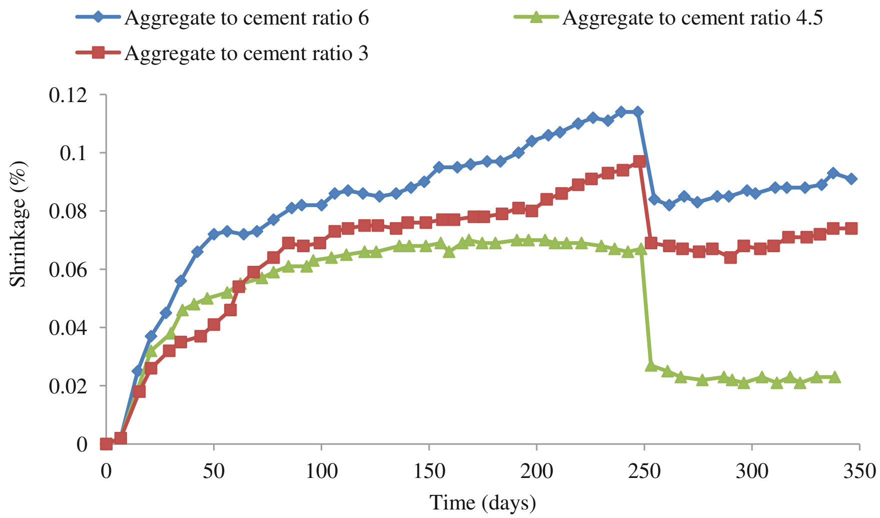


Fig. 5.3 Shrinkage behavior of RAC with 30% of RCA replacement ratio, w/c ratio of 0.45 and different aggregate-to-cement ratios (Tam et al. 2015)

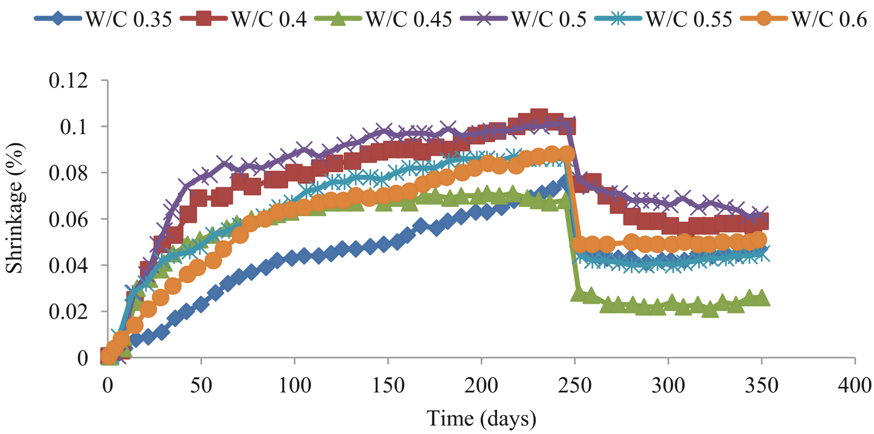


Fig. 5.4 Shrinkage behavior of RAC with 30% of RCA replacement ratio, 4.5 aggregate-to-cement ratio and different w/c ratios (Tam et al. 2015)

and basaltic origins. Ten percentage silica fume and 3% super plasticizer by weight of cement were added in the mixes. It was found that the shape of shrinkage deformation curves of RAC and normal concrete for one year was almost similar in both the aggregate origins. But the effect of recycled aggregate was significant on shrinkage of RAC. It was reported that regardless of aggregate origin, the shrinkage of RAC was 10–30% higher when 100% recycled coarse aggregate was used compared to normal concrete. But when recycled coarse and fine aggregates were used, the shrinkage of RAC was 35–45% higher than that of normal concrete. These results indicate that the origin of natural aggregate has little effect on the shrinkage behavior of RAC.

5.2.3 *Effect of Mineral Admixtures*

Liu and Chen (2008) investigated the influence of mineral admixtures on the shrinkage of high strength RAC made with 100% recycled coarse and fine aggregates. The results reported by the researchers are presented in Fig. 5.5.

It was concluded that the shrinkage of RAC mixes increased significantly with the incorporation of both recycled coarse and fine aggregates, and it was 10% higher when compared to the concrete made with natural aggregate at 56 days age. Due to the addition of silica fume, the shrinkage resistance of RAC mixes significantly improved and it was found that with the incorporation of 10% silica fume, the shrinkage of RAC made with RCA was even lower than that of normal concrete made without silica fume. The concrete interfacial bond strength and interfacial fracture energy were improved by about 100% with the addition of silica fume (Pope et al. 1992; Goldman and Bentur 1989; Rao and Prasad 2002). The improvement in shrinkage resistance was due to its smaller particle size and pozzolanic reaction, which results in the exclusion of water on the surface of aggregate in non-coated aggregate–cement system, denser microstructure, and rich interfacial transition zone.

Kou et al. (2011) conducted a series of experiments on the influence of different mineral admixtures such as silica fume (10%), metakaolin (15%), fly ash (35%), and GGBS (55%) by weight of cement on the shrinkage of concrete mixes made with different proportions of natural and recycled coarse aggregates (Fig. 5.6).

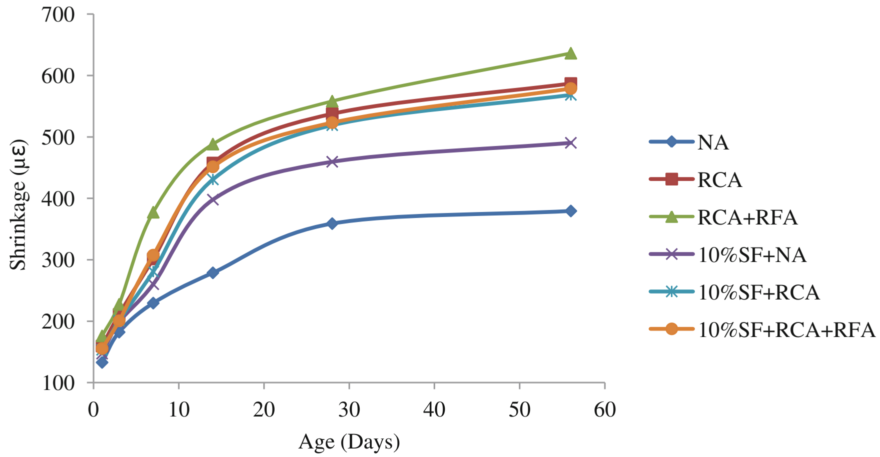


Fig. 5.5 Shrinkage of both normal and recycled aggregate concrete mixes (Liu and Chen 2008)

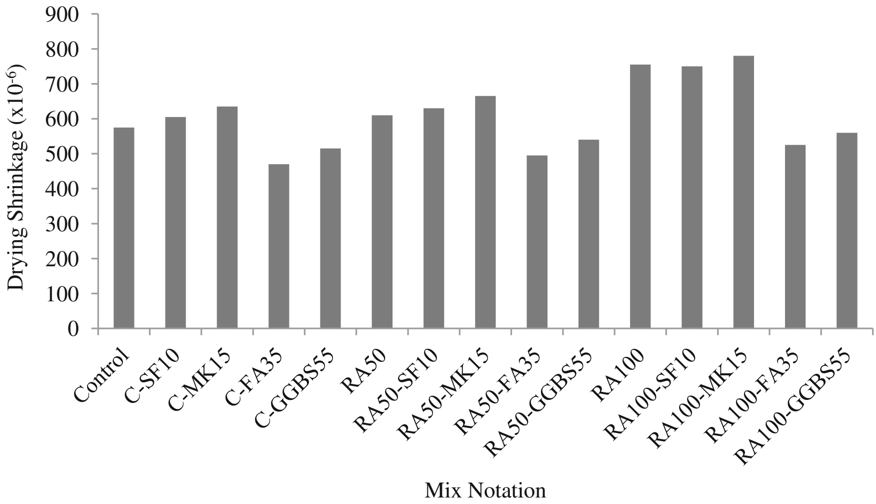


Fig. 5.6 Drying shrinkage of concrete mixes at 112 days (Kou et al. 2011)

It was found that the shrinkage of concrete mixes increased with the incorporation of recycled aggregates due to the lower stiffness of RA and the presence of attached mortar. It was further found that the shrinkage of concrete mixes made with SF and MK was more than that of corresponding control mixes. This was due to the presence of higher content of C–S–H gel in the cement paste that resulted by the pozzolanic reaction between $Ca(OH)_2$ and MK/SF. However, the concrete mixtures made with FA and GGBS were lower than the controlled concrete mixes. Possibly, this could be attributed to the lower hydration rates of GGBS and fly ash and the unhydrated powder particles in paste due to possible restraining effect.

Limbachiya et al. (2012a) studied the effect of fly ash (30% by weight of cement) on RAC mixes prepared with different proportions (0, 30, 50, and 100%) of RCA. The authors considered three different grades of concrete, whose 28 days compressive strengths are 20, 30, and 35 MPa, respectively. The mixes prepared with fly ash had shown lower shrinkage strains and higher swelling compared to those without fly ash (Table 5.4). It is well known that the major contribution to drying shrinkage is the water content in the mixes. The amount of water required to make the paste would reduce, due to the lubricating action of fly ash and consequently the magnitude of drying shrinkage. Further, it is well known that the high capacity of retention of water in fly ash during the initial stage of hydration of cement could contribute to reduce the water available in pore structure for any external drying (Lee et al. 2003; Meddah and Tagnit-Hamou 2009).

Furthermore, the drying shrinkage increases proportionately with the increase in amount of RCA in both the mixtures prepared with and without fly ash, and the increase was more significant particularly at higher percentages of RCA in the mixes. However, the effect of increase in the percentage of RCA on drying

Table 5.4 Drying shrinkage and expansion of concrete mixes after 91 days of curing (Limbachiya et al. 2012a)

Mixture code	RCA (%)	Strains (με)			
		Drying shrinkage		Expansion	
		PC	PCFA	PC	PCFA
C20	0	290	190	125	145
	30	320	260	110	190
	50	450	250	100	190
	100	650	450	60	220
C30	0	340	215	100	90
	30	340	240	120	145
	50	520	430	80	110
	100	630	550	80	135
C35	0	280	195	120	140
	30	320	250	130	140
	50	425	425	130	65
	100	810	695	140	100

Note PC Portland cement; *FA* fly ash; C20, C30, C35 (28 days compressive strengths are 20, 30, 35 MPa, respectively)

shrinkage of the concrete mixes without fly ash was more noticeable than those with fly ash. This increase in drying shrinkage in RAC mixes could be ascribed to the additional water provided by the RCA, and also, the old mortar adhered on the surface of the RCA could increase the amount of cement paste which may result in increase in shrinkage strain. But the incorporation of 30% RCA in concrete mixes with and without fly ash does not show any significant effect on the drying shrinkage.

5.2.4 *Effect of Source Concrete, Crushing Method, and Age of Crushing*

Kou and Poon (2015) reported the results of the drying shrinkage of RAC mixes prepared with RCA obtained from different strengths of parent concrete and are presented in Fig. 5.7. It was reported that the drying shrinkage of RAC mixes was more than that of control mixes regardless of their strength. Further, it was reported that the drying shrinkage of RAC prepared with RA produced from lower strength parent concrete was more than those of prepared with RA derived from higher strength parent concrete. This is fact that the RA resulted from the lower strength parent concrete had higher water absorption than that of RA resulted from higher strength parent concrete.

Pedro et al. (2014a) investigated the influence of RA obtained from laboratory crushed and precast rejected concretes of same strength on shrinkage of low,

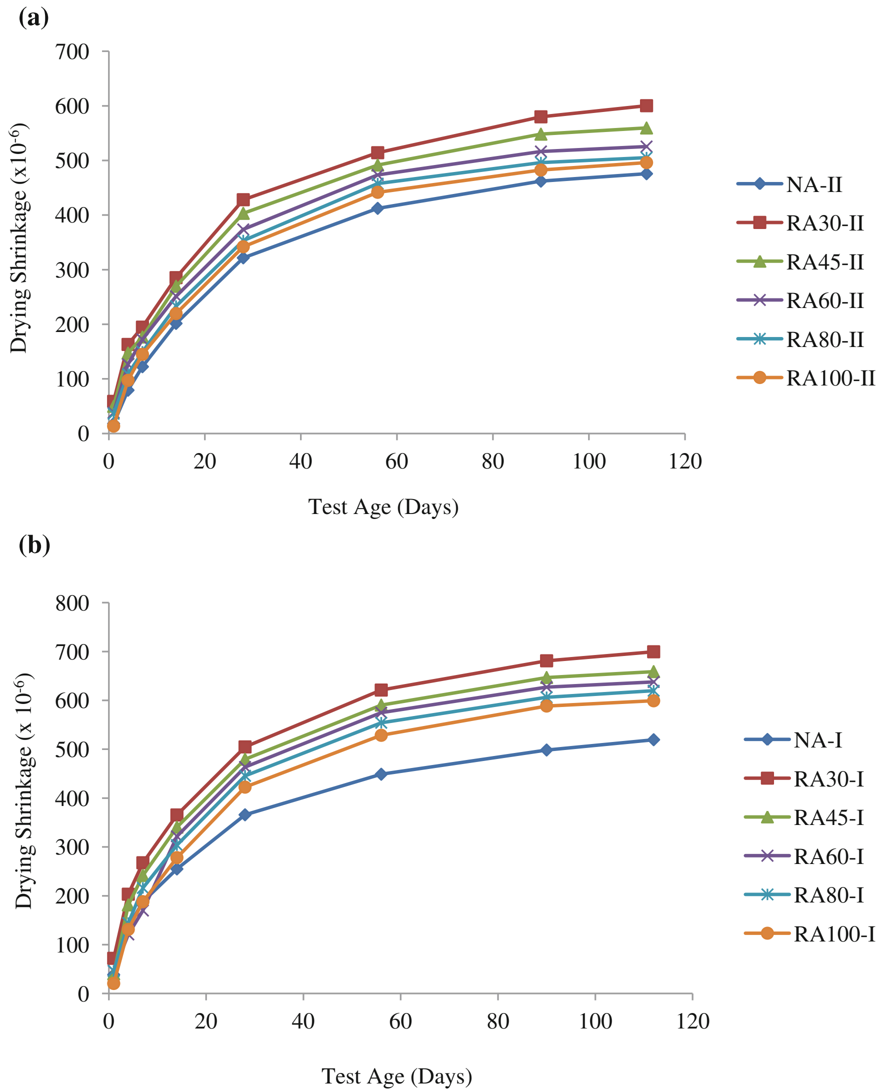


Fig. 5.7 Drying shrinkage of concrete mixes of (**a**) Series I and (**b**) Series II (Kou and Poon 2015)

medium, and high target compressive strengths (20, 45, and 65 MPa) recycled aggregate concrete mixes over a period of 91 days. The recycled aggregates were produced by two types of crushing methods: One was only primary crushing, and the other was primary plus secondary crushing. It was found that the shrinkage deformation nonlinearly increases with time; i.e., at the early age, the rate of increase of shrinkage deformation was more, and the trend was alleviated at later age. The shrinkage deformation of RAC mixes of low, medium, and high target strengths at 7 days were 12%, 31%, and 21%, respectively, higher than those of concrete prepared with natural aggregates, whereas these increases were 47%, 43%, and 68%, respectively, at 91 days. The increase in RAC mixes was expected due to the lower modulus of elasticity of RA and the presence of a large amount of voids resulted by the old attached cement mortar in RA. As long as there is water present in RA, the changes in the dimensions were relatively small due to the loss of water by evaporation that gets compensated by the stored water inside the RA by their internal curing phenomenon which resulted in these greater increases in shrinkage deformation at 91 days (Amorim et al. 2012). Further, the authors concluded that the shrinkage of RAC was independent of the quality of the concrete origin, and it depends solely on the substitution of RA and little on the concrete composition. Pedro et al. (2014b) in their study concluded that the primary plus secondary crushing method produces relatively round recycled coarse aggregate with less amount of attached old mortar on the surface when compared to only primary crushing, which leads to the higher shrinkage resistance. Katz (2003) reported the shrinkage of recycled aggregate made with RA was significantly higher than that of normal concrete. Further, it was reported that the shrinkage of recycled aggregate concrete made with RA crushed at the 28 days was higher than those made with at 1 and 3 days, whereas no significant difference was observed between 1 and 3 days crushing age.

5.2.5 *Effect of Method of Curing*

Poon et al. (2006) studied the influence of steam curing on the drying shrinkage of RAC. The authors considered two methods of curing: Method I—normal water curing, i.e., after casting, all the specimens were covered with a plastic sheet and kept in air for 24 h before they were demoulded. Immediately after demoulded the samples were kept in water at 27 ± 1 °C till the testing age and Method II—steam curing, i.e., all the concrete samples immediately after casting (without demoulding) underwent a 24 h steam curing regime. The duration of preheating, heating, treatment, and cool periods are 4, 4, 8, and 8 h, respectively. After the steam curing, all the samples were demoulded and kept in water till the testing period. Further, two Series of mixes were considered. Series I mixes were prepared with w/c ratio of 0.55, and in Series II mixes, 0.45 w/c ratio was adopted. In each series 0, 20, 50, and 100% recycled coarse aggregate were adopted. The drying shrinkage results reported by the authors are presented in Fig. 5.8.

It reveals that irrespective of the method of curing, the drying shrinkage of concrete mixes decreased with the decrease in w/c ratio. Since the rate of water movement toward the surface specimen and the amount of water that is evaporated from the cement paste directly imitates the w/c ratio, a lower w/c ratio leads to a lower shrinkage (Neville 2006). In case of normal water curing, the drying shrinkage of RAC prepared with 0–100% RCA increased by about 33% and 20%, respectively, in Series I and II mixes. Whereas, the drying shrinkage of these

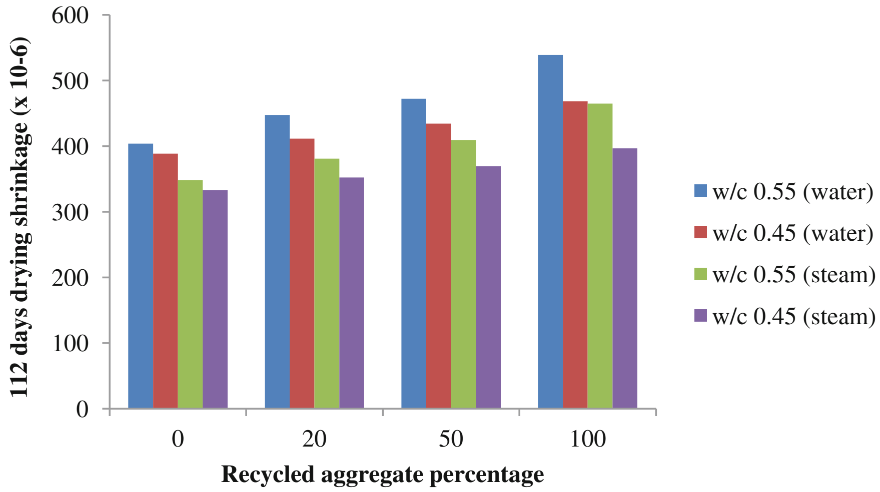


Fig. 5.8 Effect of method of curing on drying shrinkage at 112 days (Poon et al. 2006)

concrete samples were reduced when they cured in steam curing compared to normal curing. When they were cured in steam, the reduction in drying shrinkage of RAC with 100% RCA in Series I and II was 14% and 15%, respectively. The reduction in drying shrinkage was ascribed to the less absorbed water on the C–S–H surfaces after steam curing (Bakharev et al. 1999).

Silva et al. (2015) presented the results reported by Amorim et al. (2012) on the influence of the environmental conditions on the durability performance of recycled aggregate concrete with different proportions of recycled coarse aggregate (Fig. 5.9). It was reported that the concrete specimens cured in laboratory environment had the highest drying shrinkage than those cured in other environmental conditions, as the laboratory environment was wryest with a temperature of 20 °C and relative humidity of 60%. The specimens cured under this environment had shown 60% increase in drying shrinkage when RAC made with 100% RCA. The specimens cured in other environmental conditions do not show such a harmful effect on drying shrinkage of RAC with the incorporation of RCA due to less loss of water by the evaporation at higher level of humidity in these environmental conditions.

5.2.6 *Effect of Method of Mixing*

Generally, in normal concrete mixing, the aggregates are placed in the mixer in a dry state, since the water absorption of natural aggregates are normally very low (0.5–1.5%), and therefore during mixing less quantity of water is required to compensate the water absorbed by the natural aggregate. However, it is well known

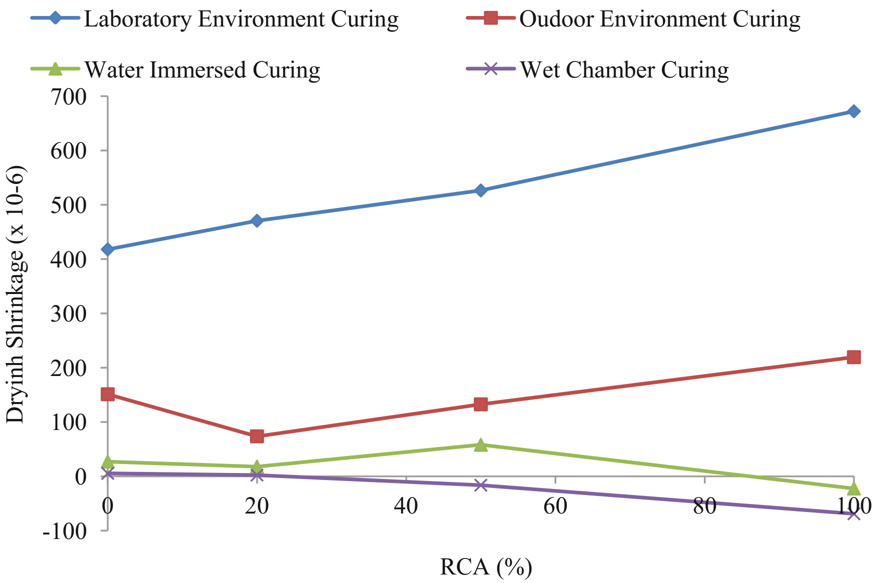


Fig. 5.9 Drying shrinkage of RAC with RCA content cured under different environmental conditions (Silva et al. 2015)

that the recycled aggregate has high absorption capacity due to the old attached mortar in recycled aggregate (Silva et al. 2015). Ferreria et al. (2011) compared the RAC mixes made with pre-saturated and water compensated RCA. It was found that the RAC mixes prepared with pre-saturated RCA had shown higher shrinkage than the RAC with water compensated RCA. After 90 days demoulding, RAC prepared with 100% pre-saturated RCA had presented 30% higher shrinkage than that of mixes prepared with same amount of water compensated RCA. Tam and Tam (2007) have adopted a different method of mixing called two-stage mixing method. The authors investigated the influence of the method of mixing on durability performance of RAC with different proportions of RCA. The authors considered two methods of mixing: Method I—normal mixing method, in which first half of the amount of coarse aggregate, then with fine aggregate and then with cement and finally the remaining coarse aggregates were loaded in the mixer and then the water is added to the all ingredients in the mixer before the rotation of mixer and Method II—two-stage mixing approach (TSMA) (Tam et al. 2005) in contrast to the Method I, TSMA divides the whole mixing into two parts and accordingly the water is divided into two parts, which are added at different stages during the mixing. Figure 5.10 reveals that no significant difference in shrinkage of RAC with lower percentage of RCA (20%) and normal concrete (0% RCA) could be observed between NMA and TSMA. However, the shrinkage deformation of RAC with 100% RCA was improved by 68.09% at 14 days in TSMA than that of by NMA.

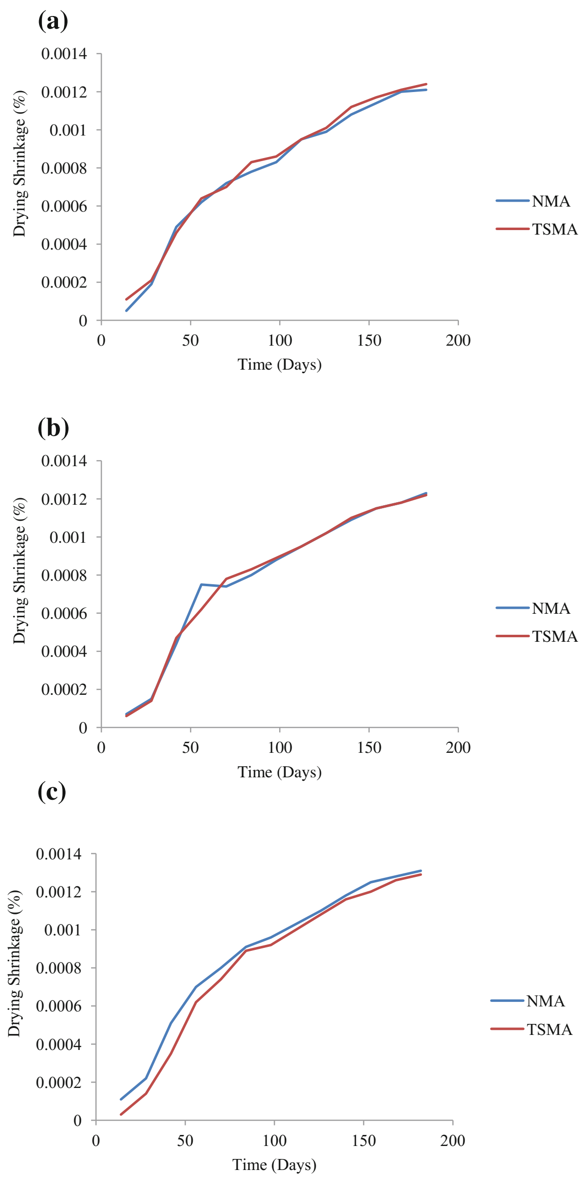


Fig. 5.10 Shrinkage deformation behavior of RAC with (**a**) 0% RCA, (**b**) 20% RCA and (**c**) 100% RCA by NMA and TSMA (Tam and Tam 2007)

5.3 Creep

A large number of investigations in the literature reported that the creep of recycled aggregate concrete increased with the increase in amount of recycled aggregate. This is due to the presence of old adhered mortar in RA which results in an increase in the total volume of cement mortar content in RAC compared to normal concrete.

Sato et al. (2007) in their study observed that the specific creep of RAC made with recycled fine and coarse aggregate was 1.5 and 2.5 times that of normal concrete in wet and dry conditions, respectively. Ajdukiewicz and Kliszczewicz (2002) found the creep of RAC made with fully recycled aggregate was 20% lower than that of normal concrete after one year. This may be due to the decrease in the effective w/c ratio of the mix by the high absorption rate of RA, which were derived from higher strength of concrete. Domingo-Cabo et al. (2009) observed that the creep of RAC with 100% RA was more than 50% of that of conventional concrete.

5.3.1 Effect of Water to Cement (w/c) Ratio

Tam et al. (2015) investigated the long-term behavior of RAC mixes prepared with different proportions of RCA (0, 30, and 100%), with different w/c ratios (0.35, 0.45, and 0.6) and with various aggregate-to-cement ratios (3, 4.5, and 6). It was found that for a given w/c ratio and aggregate-to-cement ratio, the creep strain or initial elastic strain increased or creep coefficient decreased in RAC when the recycled aggregate replacement ratio increased. The cement matrix will show a creep strain due to lose of greater amount of the physically absorbed water in CSH when the hydrated cement paste is under stress over an unremitting period. The parameters affecting the creep are presented in vide Table 5.2. On the basis of this, it was concluded that as the recede of physically absorbed water to fulfill the recycled aggregate requirement is increased with the increase in amount of RA, the creep of RAC increased. Like shrinkage, the authors did not find a clear trend on the creep behavior of RAC when it was made with different w/c ratios and aggregate-to-cement ratios (Figs. 5.11 and 5.12).

The authors found an opposite trend in creep from shrinkage as the medium w/c ratio of 0.45 and medium aggregate-to-cement ratio of 4.5 shown highest creep strains. At 0.45 w/c ratio with 30% recycled aggregate replacement ratio, the creep of RAC was noted as 0.001295, 0.001780, and 0.001347, respectively, at aggregate-to-cement ratio of 3, 4.5, and 6 at 245 days. Therefore, the authors reported that at this stage it was difficult to conclude the effect of w/c ratio and

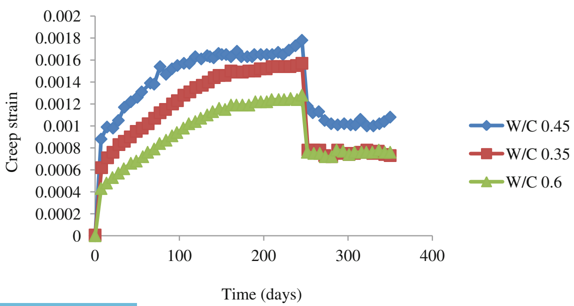


Fig. 5.11 Creep strain of RAC with 30% RCA and 4.5 aggregate-to-cement (a/c) ratio with different w/c ratios (Tam et al. 2015)

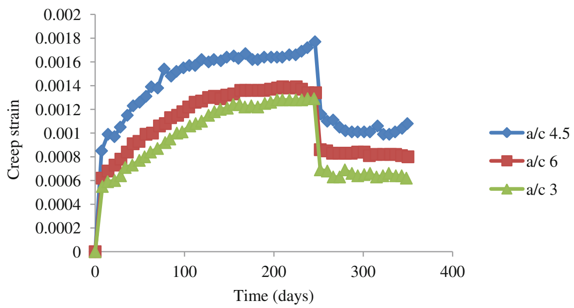


Fig. 5.12 Creep strain of RAC with 30% RCA and 4.5 aggregate-to-cement ratio with different w/c ratios (Tam et al. 2015)

aggregate-to-cement ratio on the creep behavior of RAC. Whereas, Lye et al. (2016) (based on the results of Castano et al. 2009 and Ravindrarajah and Tam 1985) concluded that the effect of w/c ratio on the specific creep of recycled aggregate concrete made with up to 100% recycled coarse aggregate was same or little lower than the effect on concrete made with natural aggregate (Fig. 5.13).

5.3.2 *Effect of Mineral Admixtures*

In general, the mineral admixtures such as fly ash and silica fume can be expected to develop better particle packing in a concrete mix and long-term strength due to their shape, particle size distribution, and fineness. As it was observed in the literature that the recycled aggregate concrete is being most sufferer than the conventional concrete, hence, the use of the fly ash and silica fume in RAC may be more beneficial (Lye et al. 2016). Kou and Poon (2012) investigated the influence of fly ash on the durability of RAC. The authors considered two series of mixes: Series I mixes with a w/c ratio of 0.55 and Series II mixes contains the w/c ratio of 0.42. In each series 0, 20, 50, and 100% recycled coarse aggregates and 0, 25, 35% fly ash by weight of cement was used. The creep strain of concrete mixes of both Series I and II reported by the authors is shown in Fig. 5.14 and the percentage increase or decrease of creep strain in Table 5.5. As it was reported earlier in the literature, the creep strain of RAC mixes (both Series I & II) increased with the increase in the percentage of RCA. This was ascribed to the increase in the total volume of the cement paste in RAC compared to the normal concrete. It was further reported that this negative effect can be minimized with the addition of fly ash in the concrete mixes. Additionally, the creep strain of Series I concrete mixes was more than that of Series II mixes. This might be due to the reduction in w/b ratio by the replacement of cement by fly ash which resulted in increased compressive strength. The concrete made with fly ash had lesser creep strains, as the fly-ash addition

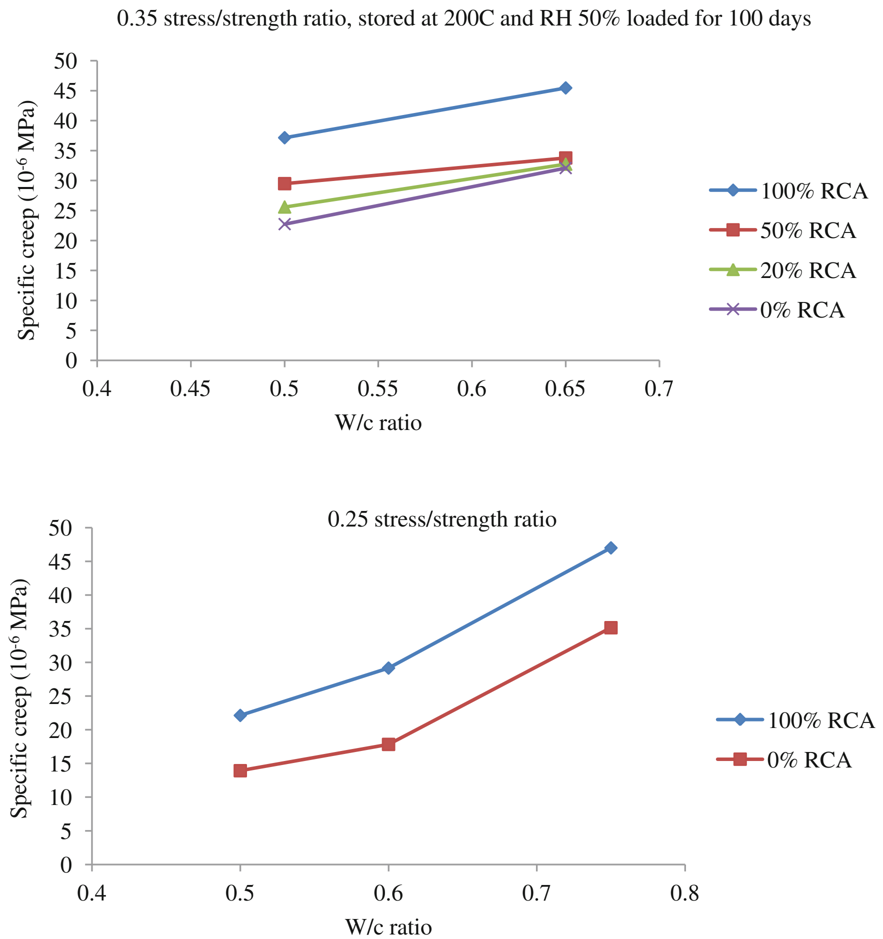


Fig. 5.13 Specific creep of concrete at different w/c ratio (Lye et al. (2016) based on the results of Castano et al. (2009) and Ravindrarajah and Tam (1985))

hinged on increase in compressive strength of concrete following the load application (Dhir et al. 1999). The authors found in their investigation that the increase in compressive strength of concrete with the addition of fly ash was significant; therefore, the actual stress/strength was lesser than that made without fly ash during which the creep test was conducted. Therefore, the lesser recorded values of creep strain for concrete made with fly ash ascribed to the lesser stress/strength ratio during the creep test period.

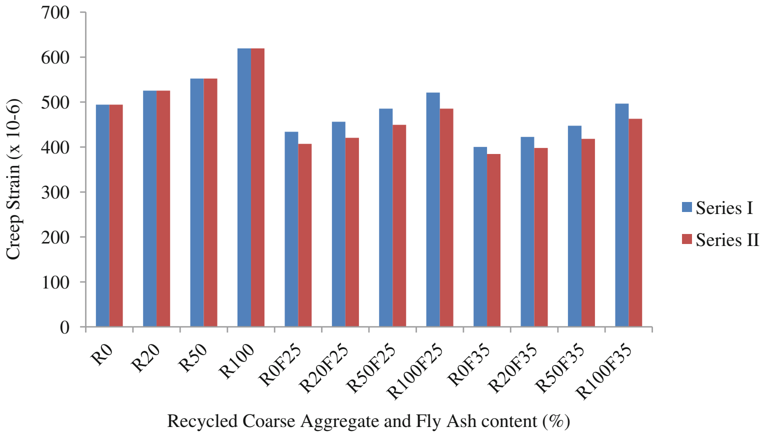


Fig. 5.14 Creep strain of concrete mixes in Series I and II at 120 days (Kou and Poon 2012)

Table 5.5 % increase or decrease of creep strain in concrete mixes at 120 days (Kou and Poon 2012)

Mix notation	Fly ash (%)	RCA (%)	% increase or decrease in creep strain from concrete mixture R0			
			% Increase		% decrease	
			Series I	Series II	Series I	Series II
R0	0	0	–	–	–	18.3
R20	0	20	5.6	–	–	14.7
R50	0	50	11.5	–	–	9.7
R100	0	100	24.6	–	–	2.6
R0F25	25	0	–	–	12.3	22.4
R20F25	25	20	–	–	8.1	19.8
R50F25	25	50	–	–	2.2	16.1
R100F25	25	100	4.8	–	–	6.5
R0F35	35	0	–		19.0	
R20F35	35	20	–		14.9	
R50F35	35	50	–		10.3	
R100F35	35	100	–		–	

5.3.3 *Effect of Method of Curing and Period of Curing*

Figure 5.15 shows the effect of wet and dry curing conditions on specific creep of concrete reported by Sato et al. (2007). The fig reveals that the specific creep of concrete made with both natural and recycled aggregate under wet condition was lower than those of dry condition. Similarly, the specific creep of concrete

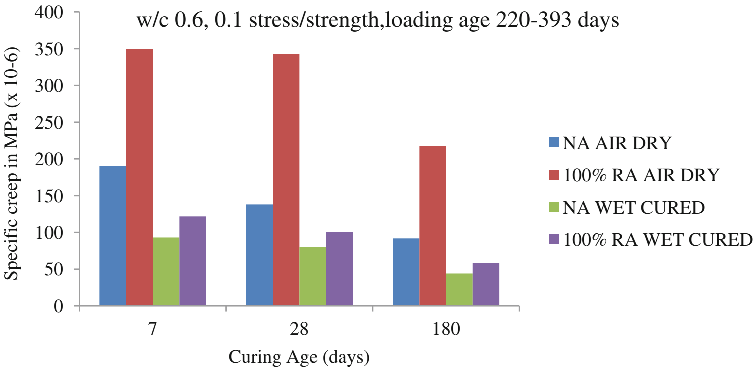


Fig. 5.15 Specific creep of concrete mixes with different curing conditions and age (Sato et al. 2007)

decreased as the age increased. Further, it shows that the recycled aggregate concrete is found to be more sensitive to curing than concrete with natural aggregate with respect to creep.

5.3.4 Estimation of Creep of RAC

Lye et al. (2016) reported the ACI 209, BS EN 1992-1-1 Eurocode 2 (2004) and Bazant-Bawega B3(1995) models were sufficiently accurate to predict the creep of RAC made with different proportions of RCA and NA up to a creep coefficient of less than 2.0 probably as these models do not take the aggregate properties into account during the process. Further, based on the data available in the literature and strength ranges from 30 to 100 MPa, the authors proposed an empirical relationships of RAC relative to NC, and using these empirical relationships, the authors derived the correction factors (Table 5.6) for easy to use in concurrence with any code of practice like Eurocode 2 for calculating the creep of RAC for a given strength (30–100 MPa) and amount of RCA (0–100%). As an example, using Eurocode 2, concrete (characteristic cube strength of 37 MPa) prepared with natural aggregates and CEM 42.5 N cement, having notional size of 820 mm, loading at 7 days in 50% RH environment, was estimated to have creep coefficient of 2.5. If the replacement level of NA by RA is 50%, a creep multiplying factor corresponding to this would be 1.275. Therefore, for 50% RCA, the creep coefficient estimated to be 3.19 against 2.5 for 100% natural aggregate concrete.

Table 5.6 Proposed creep multiply factor of concrete made with different contents of RCA at various strength level (Lye et al. 2016)

Cube strength (MPa)	Creep multiply factor of RAC							
	RCA replacement level							
	20	30	40	50	60	70	80	100
30	1.14	1.2	1.26	1.3	1.34	1.37	1.39	1.41
40	1.13	1.18	1.23	1.27	1.3	1.32	1.34	1.36
50	1.11	1.16	1.2	1.24	1.27	1.29	1.31	1.32
60	1.1	1.15	1.18	1.22	1.24	1.26	1.28	1.29
70	1.09	1.13	1.16	1.19	1.22	1.24	1.26	1.27
80	1.08	1.12	1.15	1.18	1.21	1.23	1.24	1.26
90	1.08	1.11	1.14	1.17	1.19	1.21	1.23	1.24
100	1.07	1.1	1.13	1.16	1.18	1.2	1.21	1.22

5.4 Durability Performance of RAC

Normally, the durability performance of concrete is a measure of the permeation characteristics of concrete, as well as the integrity of concrete against the aggressive agents like chlorides, acids, sulfates, oxygen, carbon dioxide present in the environment (Behera et al. 2014). Durability of concrete may be defined as its ability to resist the process of deterioration due to weathering action or chemical attack such as carbonation, chloride attack, or abrasion. The studies on the durability performance of RAC reveal that it is inferior to normal concrete. The meager performance of RAC in durability is mainly related with the poor quality of RA because of the presence of a large number of pores, cracks, and fissures present inside the RA, thus making it more prone to the permeation (Olorunsogo and Padayachee 2002; Kou and Poon 2012). The durability of RAC in terms of water absorption, permeability, chloride penetration, and carbonation depth is mainly discussed in the following subsections.

5.4.1 Permeability

Permeability is the passage of foreign materials through the concrete pores. Permeability can be indicated by many ways like water permeability, air permeability, oxygen permeability, capillary water, and water absorption (Kisku et al. 2017). Most of the investigators found that the permeability of RAC made with partial or fully recycled aggregate was larger than that of concrete prepared with natural aggregate and it increases with the increase in the percentage of RA. This is because of the fact that the presence of higher amount of pores, cracks and fissures on the attached mortar in RA during the production. The water absorption of recycled aggregate concrete is directly related with the recycled aggregate water

Table 5.7 Test results of water absorption and volume of voids of both normal and recycled aggregate concretes (Rao et al. 2017)

Source of RCA	RCA (%)	Water absorption (%)	Volume of voids (%)
Normal concrete	0	5.55	13.41
Source 1: RCC culvert near Midnapur	25	6.41	14.93
	50	6.54	15.06
	100	7.37	16.02
Source 2: RCC culvert near Kharagpur	25	6.13	14.43
	50	6.74	15.21
	100	7.25	15.57
Source 3: RCC slab of an old residential building near Vizianagaram	0	4.15	10.07
	25	4.68	11.32
	50	4.91	11.52
	100	5.40	12.07

absorption. Hence, a larger amount of water absorption of RA increases the jeopardy toward the durability of RAC. It was reported that the water absorption of RAC was significantly higher than the concrete with natural aggregate (Rao et al. 2017; Ryu 2003; Levy and Helen 2004). The water absorption and volume of voids for both normal concrete and RAC made with different percentages of RCA obtained from different Sources reported by Rao et al. (2017) is presented in Table 5.7.

It reveals that irrespective of the Source of RCA, the water absorption and volume of voids of RAC increased with the increase in the percentage of recycled coarse aggregate. Figures 5.16 and 5.17 show the ratio of water absorption and volume of voids of RAC with the normal concrete at 28 days curing period.

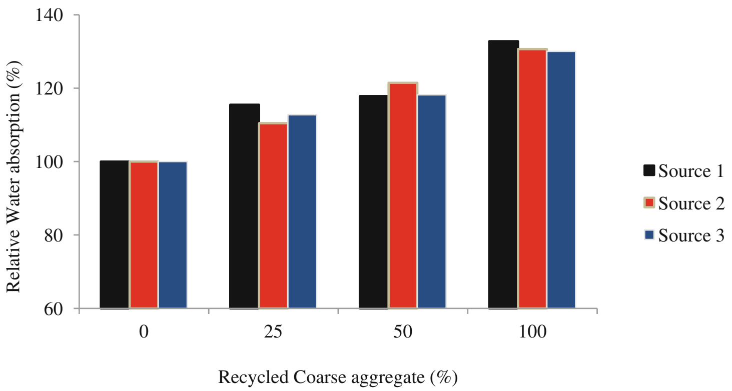


Fig. 5.16 Relative water absorption of RAC made with RCA obtained from different Sources (Rao et al. 2017)

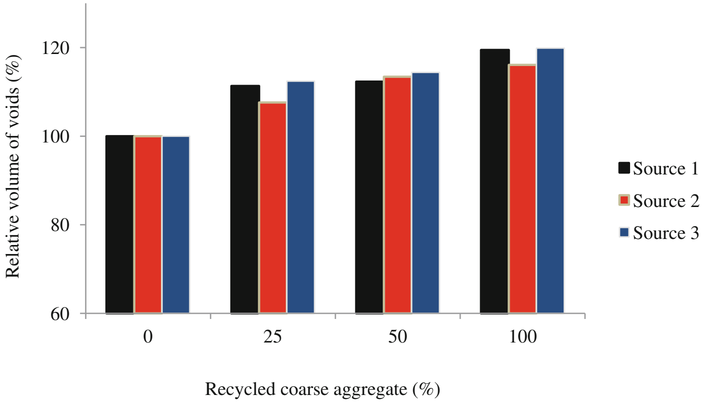
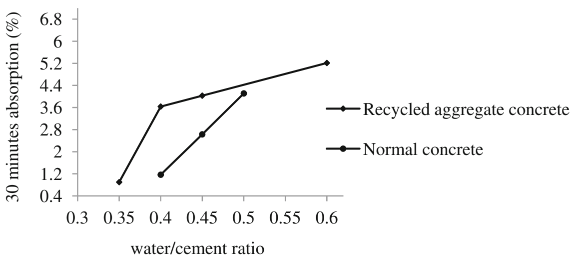


Fig. 5.17 Relative volume of voids of RAC made with RCA obtained from different Sources (Rao et al. 2017)

It indicates that the percentage increase in water absorption and volume of voids are increased with the increase in the percentage of recycled coarse aggregate in all the three Sources of mixes. The water absorption of RAC made with 25–100% RCA obtained from Sources 1, 2, and 3 is 11.6–32%; 10.5–30.6%, and 12.7–30%, respectively, higher than those of normal concrete. Similarly, the volume of voids in RAC made with 25–100% RCA obtained from the Sources 1, 2, and 3 is 11–19%; 11.3–16.1%, and 12.4–19.8%, respectively, higher than those of normal concrete. The test results are in good agreement with the results reported in the literature for all coarse aggregate replacement percentages, except 25% RCA (Levy and Helene 2004). The higher water absorption seems justified as the water absorption of recycled coarse aggregate obtained from Sources 1, 2, and 3 are 2.8, 3.5, and 3.5 times higher than that of natural coarse aggregates and also because of more porosity of RCA. From the test results of water absorption and pores, it may be said that the recycled aggregate concrete is vulnerable to the permeation of the fluids.

Rasheeduzzafar and Khan (1984) studied the durability of RAC with different w/c ratios in terms of water absorption (Fig. 5.18). From Fig. it was assessed that the water absorption (and thus permeability) of RAC and normal concrete were almost similar when both concretes have w/c ratio higher than the concrete from which the

Fig. 5.18 Water absorption (30 min) of RAC and normal concrete made with different w/c ratios (Rasheeduzzafar and Khan 1984). All the RACs made with recycled aggregates obtained from a original concrete which was produced with a w/c ratio of approximately 0.55



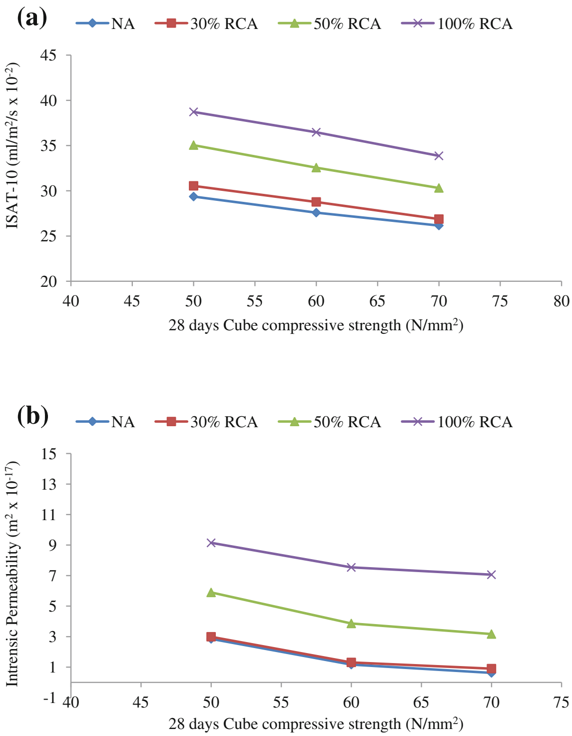


Fig. 5.19 Permeation properties of RAC and normal concrete (**a**) ISAT and (**b**) air permeability (Limbachiya et al. 2000)

recycled aggregates produced. But, the water absorption of RAC may be 3 times more than that of normal concrete when both concretes were made with lower w/c ratio than the concrete from which recycled aggregate was derived. This was due to a large amount of porous nature of old cement mortar adhered to recycled aggregate.

Limbachiya et al. (2000) reported the results of initial surface absorption (ISAT) and air permeability of high strength concrete made with different amounts of RCA (Fig. 5.19). It was found that the ISAT measured at 10 min (ISAT-10) of RAC increased with the increase in amount of RCA. However, ISAT of RAC with 30% RCA has no significant effect on the ISAT-10. The increase in ISAT-10 in RAC with higher amounts of RCA may be due to the increased volume of cement paste, as in RAC with 100% RCA, the volume of the cement paste is three times augmented than RAC prepared with 30% RCA. Further, it was found that as the RCA content increased, the decay in the rate of absorption with respect to time increased. It was also reported that with the increased design strength of mix, initial surface absorption test (ISAT-10) and rate of decay of both normal and recycled aggregate concretes increased. Similar to ISAT results, irrespective of the strength of concrete, the air permeability of RAC increased with the increase in content of RCA more than 30%. However, RAC with 30% RCA does not show any negative effect on the air permeability.

Olorunsogo and Padayachee (2002) studied the durability in terms of oxygen permeability, chloride conductivity, and water sorptivity of recycled aggregate concrete made with different percentage of recycled coarse aggregates. The oxygen permeability index (OPI) decreased with the increase in recycled aggregate content at a given curing age, and the OPI of concrete increased with the curing age for a given value of RCA. It was found (Fig. 5.20a) that the OPI of RAC with 0–100% RCA at curing age of 3, 7, 28, and 56 days decreased by 15%, 16%, 10%, and 10%, respectively. The reduction in OPI with the increase in RCA was due to fact that the adhered mortar in recycled aggregates was subjected to a large number of cracks and fissures during the crushing process, thereby these cracks and fissures provide the vent for the passage of fluids in the concrete mix. Further, it was found (Fig. 5.20b) that the curing period increased from 3 to 56 days, and the increase in OPI in RAC with 0%, 50%, and 100% RCA is 33.6%, 37.6%, and 38.2%, respectively. As per the durability classification given by Alexander et al. (1999a, b) (Table 5.8), the authors assessed the durability class of the mixes, and it was reported that the normal concrete with OPI 9.6 was found to be ‘good’ at 28 days curing age, while RAC with 50% RCA attained the same class at 56 days with OPI of 9.69. However, the RAC with 100% RCA reported only poor class at 56 days

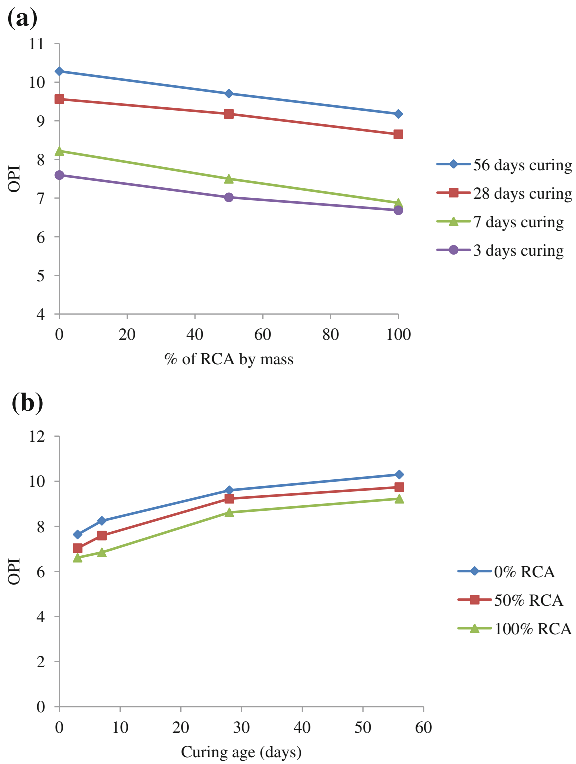


Fig. 5.20 Variation of OPI with (**a**) aggregate content and (**b**) curing age (Olorunsogo and Padayachee 2002)

Table 5.8 Suggested values of durability classification (Alexander et al. 1999a, b)

Durability class	OPI (log scale)
Excellent	>10
Good	9.5–10
Poor	9.0–9.5
Very poor	<9.0

curing age with OPI of 9.22. Further, it was reported that the value of OPI increased with the curing age.

Buyle-Bodin and Hadjieva-Zaharieva (2002) concluded that the water absorption and air permeability of RAC made with both recycled fine and coarse aggregates were more than that of normal concrete due to higher porosity of old cement paste adhered to RCA and the rate of carbonation was also higher for RAC. This leads to less resistance against environmental attacks. The performance of RAC made with RCA and natural sand was in between the performance of RAC with full recycled aggregates and normal concrete. The authors concluded that the use of fine recycled aggregate is the main reason for weakening the performance of RAC. Zaharieva et al. (2003) studied the durability of RAC made with both recycled fine and coarse aggregate and recycled coarse aggregate and natural sand separately in terms of surface permeation properties. The authors concluded that the permeability of RAC made with both recycled fine and coarse aggregate was more than that of normal concrete. The rate of carbonation of RAC was also faster than normal concrete. This effect restricts the yield of reinforced concrete elements using recycled aggregate. Whereas, the authors found an intermediate result between normal concrete and fully recycled concrete when RAC made with RCA and natural sand. This indicates that the recycled fine aggregate significantly affects the durability of RAC. Therefore, the recycled fine aggregate should be restricted. The authors finally concluded that the permeability is more dependent on mix design and curing conditions and it can be improved with the curing.

Somna et al. (2012) investigated the influence of ground fly ash (GFA) and ground bagasse ash (GBA) on the durability of recycled aggregate concrete in terms of water permeability, chloride depth, and expansion by sulfate attack. Recycled aggregate concretes were produced by replacing the natural coarse aggregate with 100% recycled coarse aggregate. GFA and GBA were used partially to replace the cement at the rate of 20, 35, and 50% by weight of binder in all RAC mixes. The investigation results are presented in Fig. 5.21. It was observed that the coefficient of water permeability at 28 and 90 days of RAC was more than that of control concrete. The 28 and 90 days permeability coefficient of RAC was approximately 80×10^{-13} and 21×10^{-13} m/s, respectively, against 16×10^{-13} and 9×10^{-13} m/s of control concrete. This is because of the recycled aggregate has greater porosity than natural aggregate (Gomez-Soberon 2003). The coefficient of water permeability of RAC mixes was significantly decreased and even lower than the control concrete when the cement was partially replaced with GFA and GBA in RAC mixes. However, the higher percentage (up to 50%) replacement of cement by

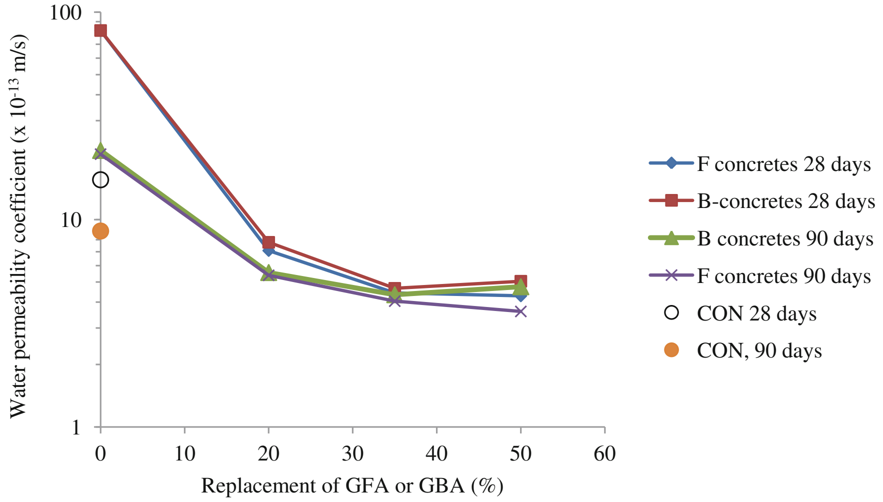


Fig. 5.21 Relationship between water permeability coefficient and replacement of GFA or GBA (Somna et al. 2012)

GFA and GBA increases the water permeability coefficient of RAC mixes. Though the curing time extended from 28 days to 90 days reduces the water permeability coefficient, the effect of utilization of GFA and GBA in RAC mixes had shown larger reduction in coefficient of water permeability. This higher reduction in permeability coefficient in RAC is due to that the RAC becomes denser, as the pores of the concrete matrix get filled by the finer particles of GFA and GBA. The authors recommended using GFA and GBA as a partial replacement of cement by weight in RAC mixes to obtain lower values of coefficient of water permeability and 20% replacement was more appropriate.

Tam and Tam (2007) studied the influence of two different methods of mixing, namely single-stage mixing approach (SMA) and two-stage mixing approach (TSMA) on the permeability (water, air, and chloride) of RAC prepared with different proportions of RCA. It was reported that the introduction of RCA in concrete reduces the resistance against the permeability (Tables 5.9, 5.10, and 5.11). It was found that the water, air, and chloride permeability of RAC with 100% RCA in NMA are 0.001642 mm^2/s BAR, 4.5470 s/mL, and 2906.60 Å s compared to 0.001284 mm^2/s BAR, 9.0082 s/mL, and 2231.56 Å s in concrete with natural aggregate at 180 days curing. However, the difference observed with 20% RCA was not significant. Due to TSMA, the resistance against permeability of RAC mixes was improved significantly, as the cement gel around the RCA in the premixing stage of TSMA reduces the porosity of RA. It was found that the reduction in water and chloride permeability was around 35.41% and 29.98%, respectively, (at 100% RCA substitution after 26 days curing) and 51.81% in air permeability (20% RCA @ 56 days) when compared to SMA. In TSMA, the initial half of the water added in the first stage of mixing was able to form a thin layer of

Table 5.9 Water permeability of RAC (Tam and Tam 2007)

Curing days	Water permeability (mm^2/s Bar)					
	SMA			TSMA		
	0%	20%	100%	0%	20%	100%
14	0.00786	0.00723	0.00887	0.00716	0.00737	0.00722
28	0.00734	0.00832	0.00719	0.00697	0.00813	0.00678
42	0.00733	0.00737	0.00723	0.00673	0.0077	0.00744
56	0.00747	0.00855	0.00786	0.00728	0.00793	0.00866
70	0.00829	0.00114	0.0015	0.00131	0.00121	0.00146
84	0.0012	0.00121	0.0014	0.00109	0.00122	0.00113
98	0.00152	0.00117	0.00137	0.00111	0.00121	0.0012
112	0.00132	0.00163	0.00144	0.00101	0.00117	0.00122
126	0.00134	0.00128	0.00172	0.00132	0.00113	0.00111
140	0.00173	0.00152	0.00142	0.00124	0.00144	0.00162
154	0.00139	0.00144	0.0014	0.00129	0.00152	0.0014
168	0.00138	0.00162	0.0014	0.00147	0.00147	0.00164
182	0.00128	0.00154	0.00164	0.00139	0.0015	0.00147

Table 5.10 Air permeability of RAC (Tam and Tam 2007)

Curing days	Air permeability (s/mL)					
	SMA			TSMA		
	0%	20%	100%	0%	20%	100%
14	9.64	10.81	6.73	12.54	12.72	7.91
28	13.9	11.12	7.1	14.15	14.7	8.77
42	13.65	10.04	7.6	12.18	12.61	10.94
56	10.603	8.28	4.72	12.01	12.57	6.99
70	10.55	9.74	5.75	11.67	10.34	6.26
84	8.84	10.77	6.35	11.67	11.97	5.4
98	9.65	11.45	7.08	11.28	10.55	6.61
112	11.28	9.27	4.8	12.18	11.28	6.31
126	11.5	9.1	6.35	12.05	11.32	5.83
182	9.01	9.74	4.55	11.24	11.28	6.56

cement slurry on the RCA surface, which will try to penetrate and fill up the pores and cracks in the old attached mortar, thereby the ITZ becomes stronger when compared to SMA (Figs. 5.22, 5.23, 5.24, and 5.25). Therefore, the improved durability performance of RAC in TSMA was observed.

The permeability of RAC with respect to duration reported by different researchers in the literature is presented in Fig. 5.26. It can be seen from the figure, the increase in permeability from 20 days to 80 days is very minimal except in few cases.

Table 5.11 Chloride permeability of RAC (Tam and Tam 2007)

Curing days	Chloride permeability (Å s)					
	SMA			TSMA		
	0%	20%	100%	0%	20%	100%
14	3753.29	3153.53	3800.54	3248.91	3827.9	3617.31
28	2468.93	3054.11	2931.48	2436.47	2487.39	3394.85
42	2475.68	3025.79	3199.16	2409.26	2301.6	2697.59
56	2636.46	2269.97	2629.22	2100.35	2911.52	2778.94
70	2405.66	2606.84	2243.48	2137.37	2414.94	2455.22
84	2337.51	2410.25	2947.68	2097.71	2728.77	2481.16
98	2427.69	2615.16	2257.86	1927.43	2596.97	2313.65
112	2627.2	2766.39	3155.49	2031.45	2739.06	2683.31
126	2741.46	2211.87	3330.5	2021.21	2811.89	2331.85
182	2231.56	2703.69	2906.6	1869.96	2121.95	2578.04

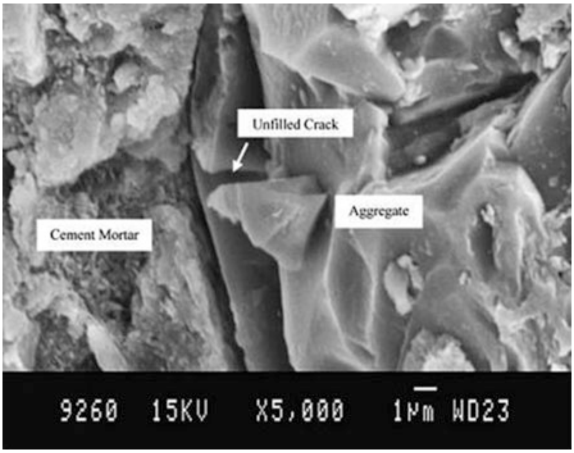


Fig. 5.22 Unfilled crack in RA using SMA (Tam and Tam 2007)

5.4.2 *Chloride Penetration*

Steel corrosion is one of the major causes for deterioration of reinforced concrete structures. Corrosion in steel increases the volume of steel and thereby the concrete gets cracked and allowing more harmful substances inside the concrete to assist in further deterioration. In most of the cases, the reinforcement is corroded due to ingress of chlorides through the concrete cover penetrating to the reinforcement and thereby causing corrosion to the reinforcement. Concrete deterioration does not occur directly due to ingress of chlorides but indirectly due to corrosion of reinforcing bars. The chloride penetration can be measured either by constant immersion in sodium chloride solution or by cyclic immersion/drying.

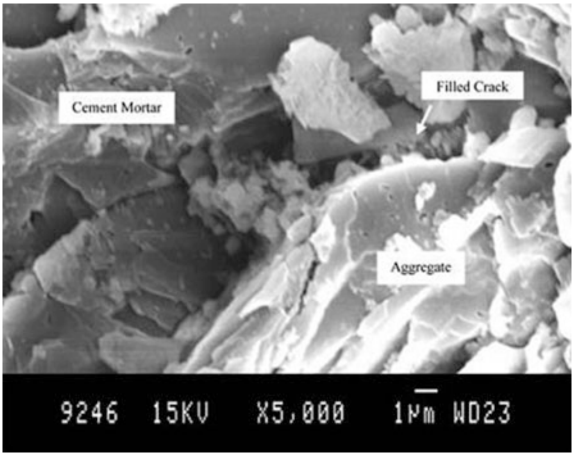


Fig. 5.23 Filled crack in RA using TSMA (Tam and Tam 2007)

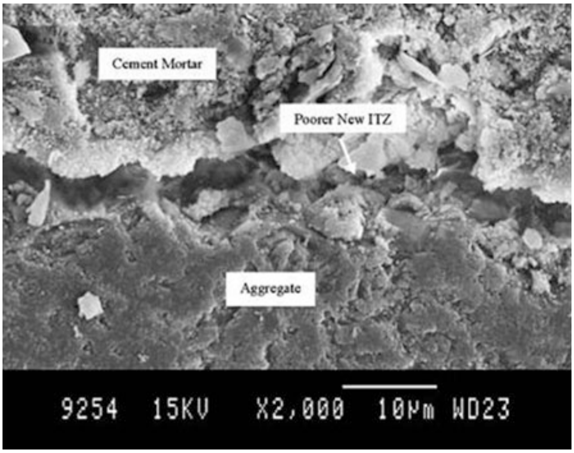


Fig. 5.24 Poorer new ITZ in SMA (Tam and Tam 2007)

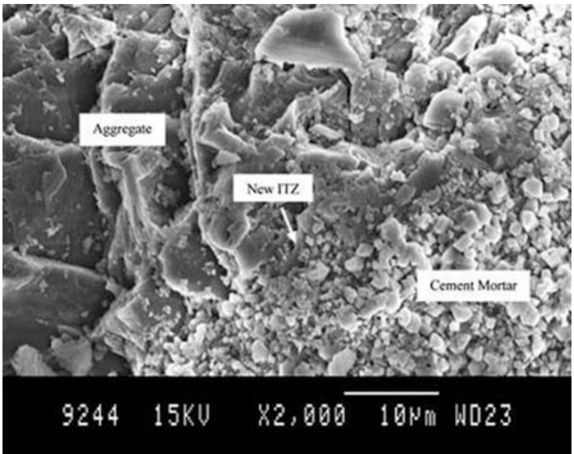


Fig. 5.25 New ITZ in TSMA (Tam and Tam 2007)

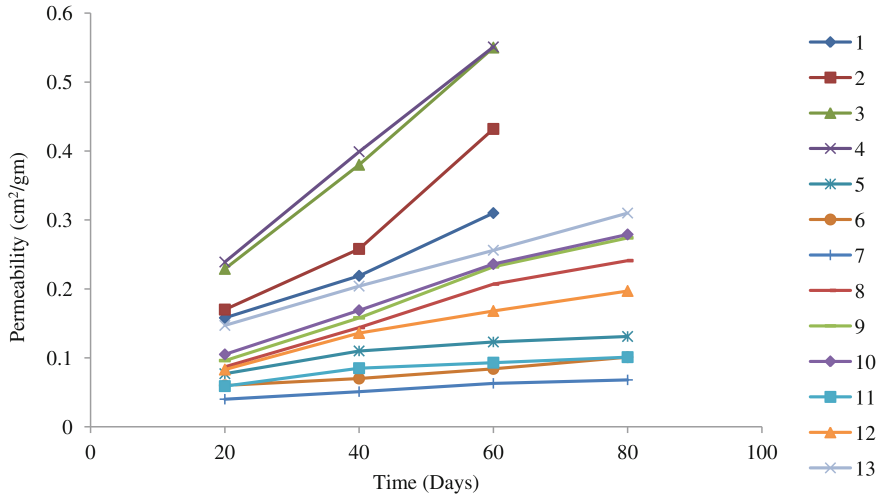


Fig. 5.26 Permeability of RAC (1–4: Matias et al. (2013) (0% RCA, 100% RCA, RAC100 with SP1, RAC100 with SP2, respectively); 5–7: Zhu et al. 2013 (0% RCA, Saline 100 and 200 g/cm^2, respectively); 8–10: Zega and Di Maio (2011) (RCA0%, 20%, 30%, respectively); 11–13: Evangelista and de Brito (2010) (0% RCA, 30% RCA and 100% RCA, respectively))

Li (2008) reported in his review that Xiao et al. (2004) had examined the influence of the replacement level of NA by RCA in concrete on the resistance against chloride ion penetration. It was found that the resistance against chloride ion penetration becomes inferior when the percentage of RCA increased. However, based on ASTM C 1202, the resistance against chloride ion penetration of concrete is classified as low when it was made with less than 50% RCA, and it began to change into medium when the RCA was more than 50%.

Otsuki et al. (2003) investigated the influence of the method of mixing with different water–binder ratio on the chloride penetration of RAC. Two methods of mixing, namely the normal mixing method and double mixing methods, were adopted. The results reported by the authors are presented in Fig. 5.27. In normal mixing, it was shown that the chloride penetration of concrete prepared with both natural and recycled aggregates increased with the increase in water–binder ratio. Further, it was found that the depth of chloride penetration of RAC was slightly higher than that of normal concrete for a given water–binder ratio. This was due to the presence of old ITZ and adhered mortar in RA, which creates the RAC more permeable than concrete with natural aggregate. In double mixing method, the whole water is divided into two parts. First, the coarse and fine aggregates were mixed for 30 s and then added the first part of the water to the mix and continued the mixing for 30 s. Then, cement was added, and mixing was continued for 60 s by machine and then mixing for 60 s by hand. After that, the second part of the water was added to mix for 30 s and continued the mixing for 90 s. This method was developed to cover the RA with mortar of lower water–binder ratio than the

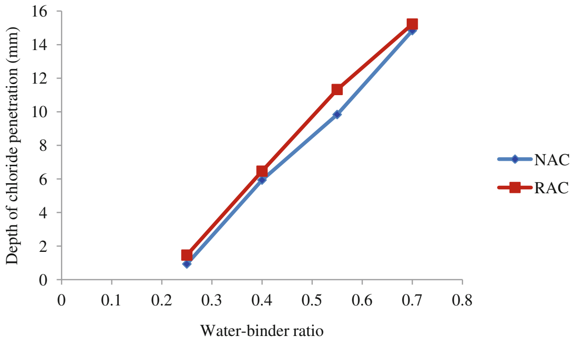


Fig. 5.27 Chloride depth of penetration of concretes for different water–binder ratios (Otsuki et al. 2003)

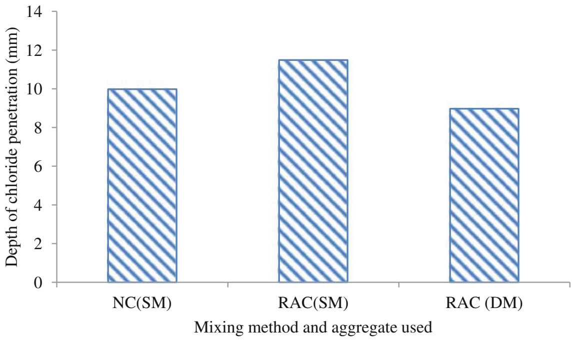


Fig. 5.28 Chloride penetration depth of concretes using single and double mixing methods (Otsuki et al. 2003)

rest of the mortar matrix. In this process, the ITZ becomes denser due to the prevention of crystal growth during hydration, as the water near the aggregate was lower. This was further confirmed by the authors with the results of microhardness, i.e., the Vickers microhardness of new ITZ based on double mixing method was more than that of single mixing method. Due to these improvements in new ITZ, it was found a reduction in depth of chloride penetration of RAC with water–binder ratio of 0.55 using double mixing method (Fig. 5.28).

Rao et al. (2017) performed the chloride penetration of RAC with different proportions of RCA based on the immersion procedure recommended by Otsuki et al. (1992). The test results of chloride penetration of both normal concrete and recycled aggregate concrete made with different percentages of recycled coarse aggregate obtained from three different Sources are presented in Fig. 5.29. It reveals that the depth of chloride penetration increases as the coarse aggregate content increases in all the Sources of mixes. The chloride penetration in RAC prepared with 25, 50, 100% RCA obtained from Source 1 is 4.5, 9.5, 11.4%; from Source 2 is 9.3, 11.8, 13.9% and from Source 3 is 2.5, 7.9, 10.2%, respectively, which are higher than those of the corresponding normal concrete. In addition, it was observed that 25% RCA does not influence much on depth of chloride penetration. The increase in chloride penetration depth in RAC was due to the more

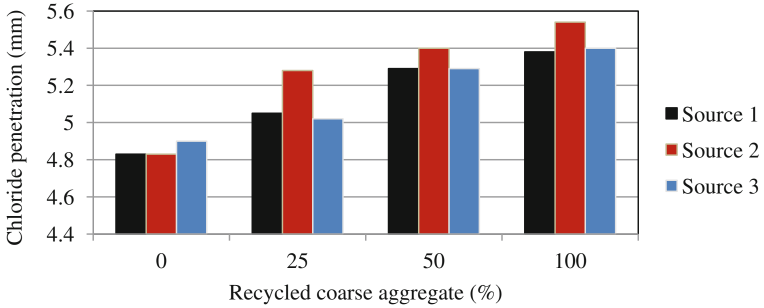


Fig. 5.29 Chloride penetration in both normal concrete and RAC made with RCA obtained from all the three Sources (Rao et al. 2017)

permeable nature of recycled coarse aggregate by the presence of old porous mortar and old ITZ and the presence of more microcracks in recycled coarse aggregate. As it was reported in their earlier research, the total volume of voids in case of RAC with 25–100% RCA obtained from all the three Sources is approximately 11–20% higher than that of corresponding normal concrete.

Poon et al. (2006) studied the influence of method curing on the resistance against chloride penetration of RAC. The authors considered two methods of curing, namely normal water curing and steam curing. Authors considered two Series of mixes: Series I mixes were prepared with a w/c ratio of 0.55, and in Series II mixes, 0.45 w/c ratio was adopted. In each series 0, 20, 50 and 100% recycled coarse aggregate was used. The resistance against chloride ion penetration in both the series of mixes at 28 days and 90 days curing reported by the authors is presented in Fig. 5.30.

It reveals that regardless of the method of curing, resistance against chloride ion penetration of concrete increases with decrease in w/c ratio. Due to the reduction in w/c ratio, the total volume of the pores inside the concrete gets reduced and hence the concrete becomes more impermeable, thereby the resistance against chloride ion penetration increased. Further, it reveals that with the increase in curing period from 28 to 90 days, the resistance increased. Mindess et al. (2003) reported that the thickness of C–S–H and the growth of calcium hydroxide (C–H) within the capillary pores increased with the increase in curing age, thus forming the impermeable regions, which enhance the resistance against chloride ion penetration. It was found that the steam curing shows better resistance against chloride ion penetration than the wet curing. This better performance might be due to the development of convoluted interconnecting network of capillary pores in the steam curing, which results non-uniform hydration products (C–S–H). Also, with an increase in the percentage of recycled aggregate content, the chloride ion penetration resistance marginally reduced.

Kou et al. (2008) examined the influence of different w/c ratio (0.55, 0.50, 0.45, and 0.40) and 25% fly ash as an addition of cement on the chloride ion permeability of RAC mixes prepared with different percentages of RCA. The results found by

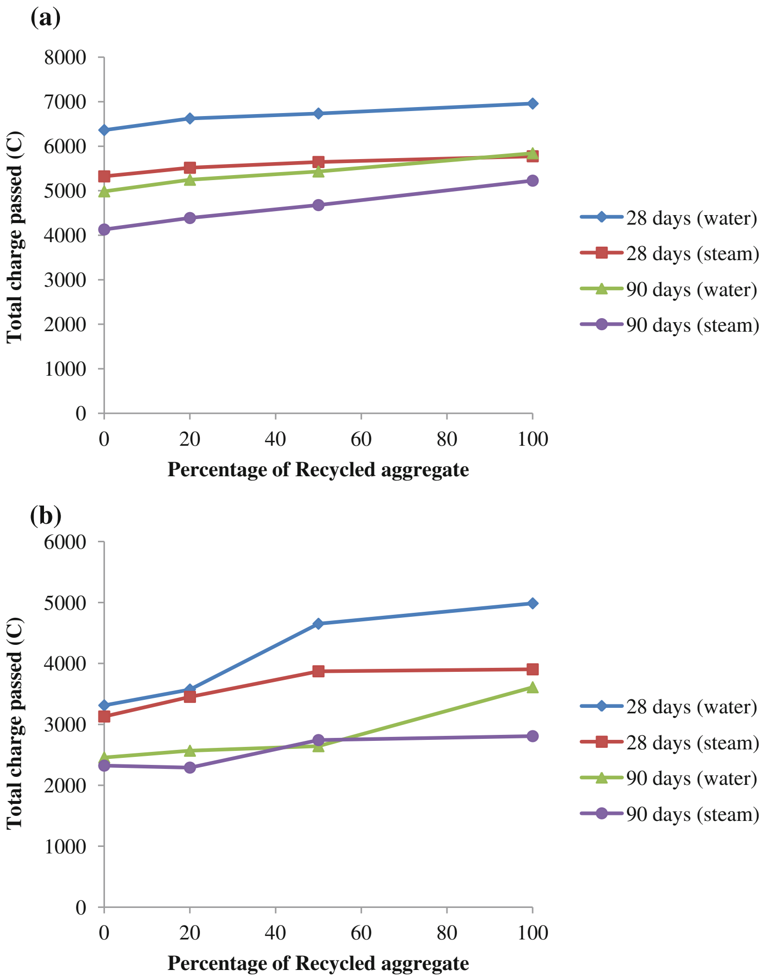


Fig. 5.30 Resistance to chloride ion penetration in (**a**) Series I mixes (w/c 0.55) and (**b**) Series II mixes (w/c 0.45) (Poon et al. 2006)

the authors are presented in Fig. 5.31. It was found that with the increase in the percentage of RCA from 0 to 100%, the chloride ion penetration increased little, as the recycled aggregates were more porous due to the attached old cement mortar. Further, it was noticed that the reduction in w/c ratio from 0.55 to 0.40 resulted a remarkable increase in chloride ion penetration resistance. When the w/c reduced from 0.55 to 0.40 in RAC with 100% RCA, 53% reduction in total charge passed was observed at 28 days. Furthermore, the resistance against chloride ion penetration was increased remarkably with the addition of fly ash. According to Leng et al. (2000), (i) use of fly ash refined the size of pore and shape of pores in concrete, (ii) as fly ash hydrated, development of a large amount of hydration

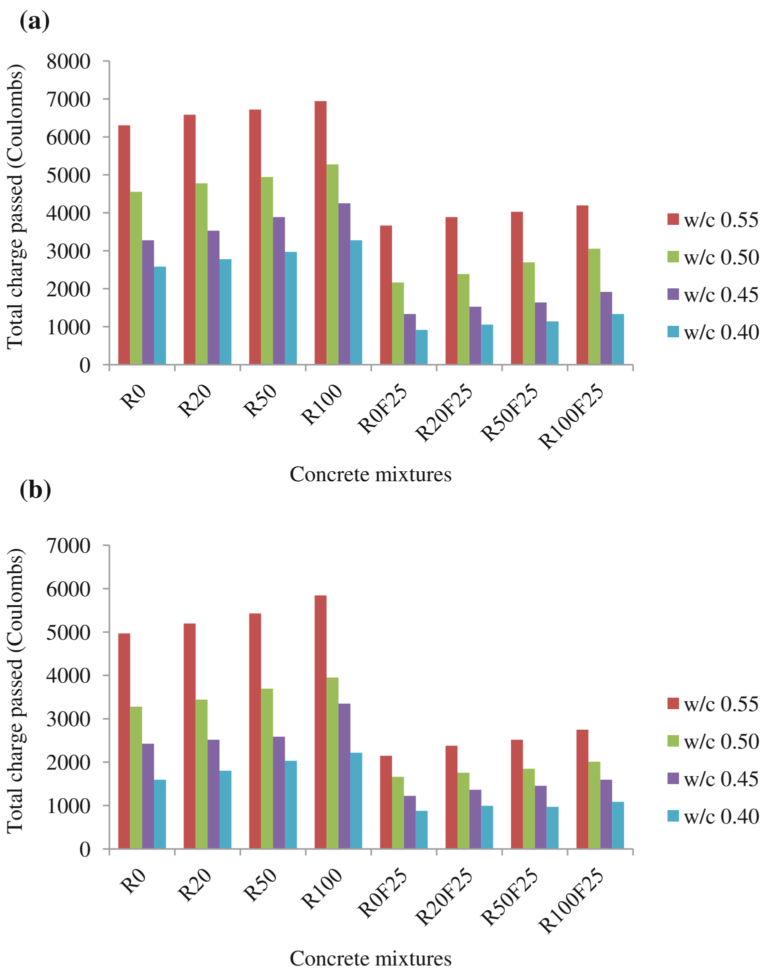


Fig. 5.31 Chloride penetrability of concrete mixes (**a**) at 28 days and (**b**) at 90 days (Kou et al. 2008)

products (C–S–H) absorbed large chloride ions and chunk the conduits to ingress chloride ions, and (iii) fly ash is used as addition of cement causes to decrease the water–binder ratio of the concrete mix, thus the concrete with addition of fly ash enhances the chloride ion penetration resistance. It was reported that the total charge passed was lowered by 81% in RAC with 100% RCA, when the w/c ratio decreased from 0.55 to 0.4 along with 25% addition of fly. As the w/c ratio decreased, the total volume of pores reduced within a concrete, thereby the concrete became more impermeable. Therefore, the combined effect of the addition of fly ash and lower w/c ratio would yield the better performance of concrete against the chloride ion penetration resistance.

Kou et al. (2011) investigated the influence of different mineral admixtures on the chloride ion penetration resistance of RAC mixes prepared with different proportions of natural and recycled coarse aggregates. Four types of mineral admixtures such as 10% silica fume (SF), 15% metakaolin (MK), 35% fly ash (FA), and 55% GGBS by weight of cement were considered. The results of the investigation are presented in Fig. 5.32. It was found that with increase in the percentage of RCA, the total charge passed was increased. However, the addition of mineral admixtures enhanced the resistance against the chloride ion penetration in both normal and recycled aggregate concretes. The highest enhancement occurred due to 10% SF followed by 15% MK, 35% FA, and 55% GGBS. This enhancement could be explained by the improvement in impermeability of concrete and enhancement in chloride binding capacity of SF, MK, and/or FA. The addition of fly ash and blast furnace slag was found to be more effective to enhance the resistance of chloride ion penetration in RAC (Hua and Song (2007) and Du et al. 2006). Yigiter et al. (2007) reported that the GGBS with blended cement was very effective in preventing the ingress of chloride in concrete. The presence of fly ash in concrete considerably increases the resistance against chloride ion penetration (Bilodeau and Malhotra 1993).

Figure 5.33 shows the results reported by Somna et al. (2012), the influence of GFA and GBA as a partial replacement of cement by weight on the depth of chloride penetration in RAC. The samples were immersed in 3% sodium chloride solution at 6, 12, and 18 months. It was found that the chloride penetration depth of RAC was more than that of control concrete. The chloride depth of penetration of RAC at 6, 12, and 18 months were 45, 70, and 75 mm, respectively, compared to 32, 52, and 60 mm in normal concrete. Similar results were reported in the literature (Kou et al. 2007; Ann et al. 2008). Ann et al. (2008) observed that the resistance

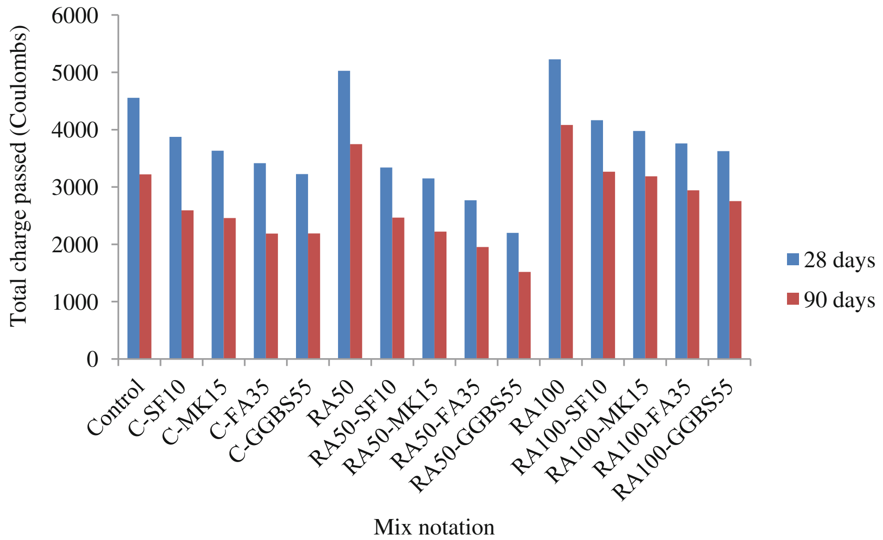


Fig. 5.32 Total charge passed in coulombs in concrete mixes (Kou et al. 2011)

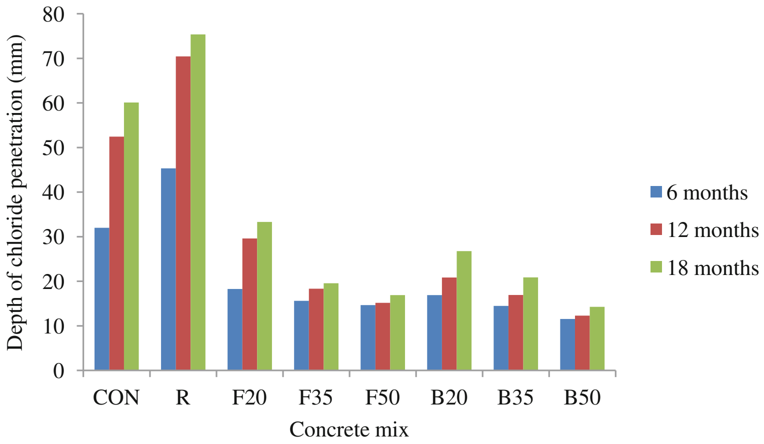


Fig. 5.33 Depth of chloride penetration of concretes immersed in 3% sodium chloride solution at 6, 12, and 18 months (Somna et al. 2012)

against chloride ion permeability and corrosion of RAC were improved with the addition of 30% PFA and 65% GGBS pozzolanic materials. The pozzolanic reaction improves the concrete properties, compatibility, a lowering segregation, and densifying the concrete pore structure, thereby resistance against external aggressive ions (Schiessl and Breit 1996). When natural coarse aggregates are fully replaced with recycled coarse aggregates, increase the porosity and cracks in RA, which provides the vent to ingress chloride ions easily into the recycled aggregate concrete (Gomez-Sobron 2003; Etxeberria et al. 2006). It was also found that the partial replacement of cement by GFA in RAC mixes could considerably improve the chloride ion penetration resistance of RAC when compared to control concrete and RAC without GFA.

Kou and Poon (2015) reported the results of the chloride ion penetration resistance of two RAC mixes (w/c 0.5 and 0.35) prepared with RCA obtained from different strengths ranges from 30 to 100 MPa of parent concrete and are presented in Fig. 5.34.

It was shown that the chloride ion penetration of concrete mixes increased with the inclusion of RCA obtained from different strengths of parent concretes when compared to normal concrete. It was shown further that the resistance against chloride ion penetration of concrete prepared with RA obtained from lower strength parent concrete was lower than that of RAC prepared with RA derived from higher strength parent concrete. As the RA derived from 30 MPa strength parent concrete had higher water absorption capacity, the corresponding concrete mixture shown the highest total charge passed in coulombs. Figure 5.35 shows the chloride ion resistance of concrete prepared with different percentages of RCA reported by different investigators in the literature. It shows that the chloride ion penetration of RAC is higher than that of normal concrete and the difference is in the order of 10–50%.

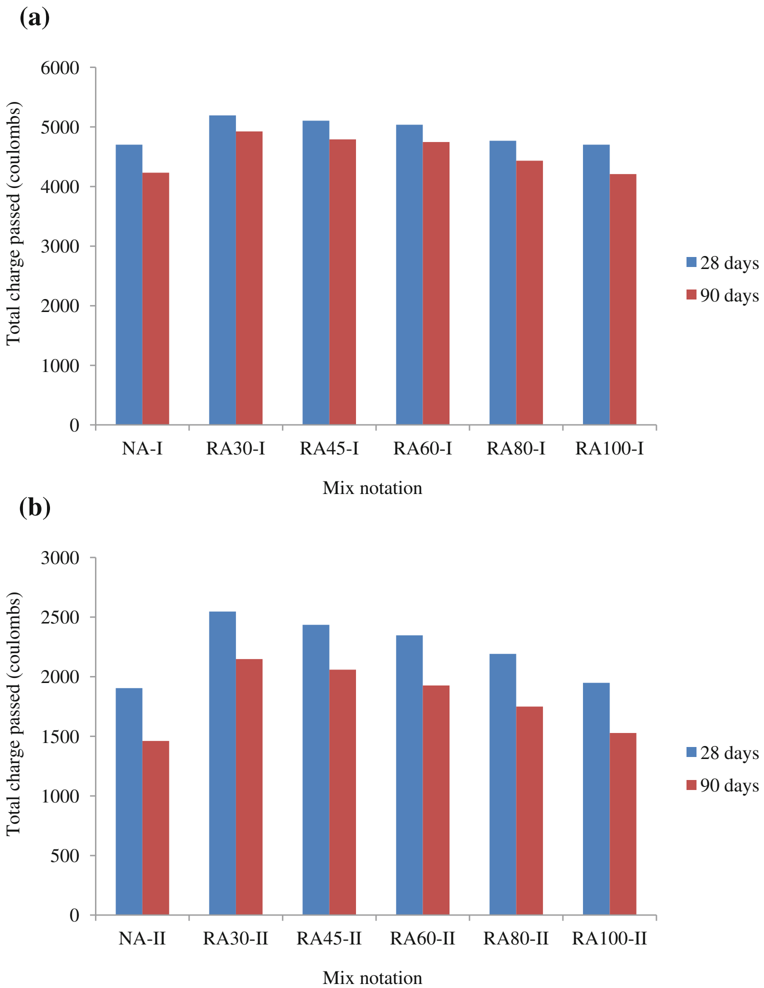


Fig. 5.34 Chloride ion penetration of concrete mixtures (**a**) in Series I (w/c 0.5) and (**b**) in Series II (w/c 0.35) (Kou and Poon 2015) Note: RA30, RA45, RA60, RA80, and RA100: RA obtained from 30, 45, 60, 80, and 100 MPa compressive strength of parent concretes, respectively

5.4.3 Carbonation Depth

Katz (2003) studied the depth of carbonation of RAC prepared with RCA obtained from partially hydrated old concrete (3, 7, 28 days crushing age). The authors considered two types of cements, namely OPC and white Portland cement (WPC). The depth of carbonation was measured at top, bottom, and sides of both normal concrete and RAC at 3 and 7 days (Table 5.12). It was found that the recycled aggregate concrete exhibited larger depth of carbonation of 1.3–2.5 times that of

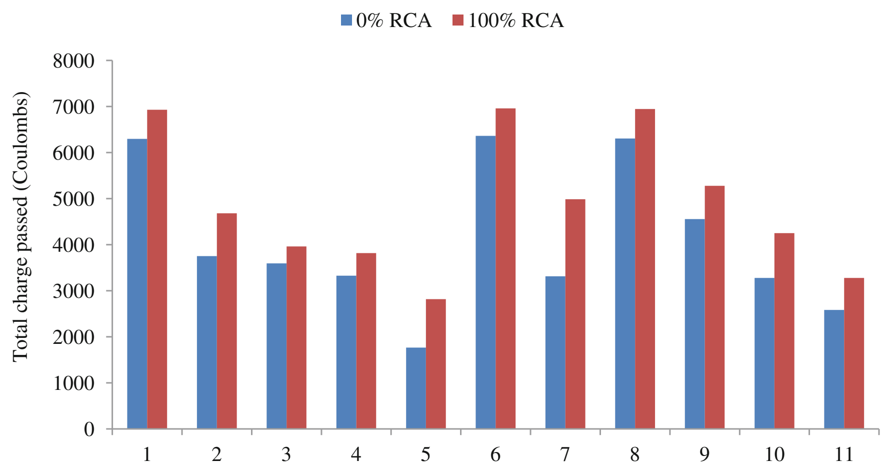


Fig. 5.35 Resistance against chloride penetration (1–3: Kou and Poon 2013 (0, 25 and 55%FA); 4–5: Zhu et al. 2013 (Silane 100 and 200 g/cm^2); 6–7: Poon et al. 2006 (w/c 0.55 and 0.45); 8–11 Kou et al. 2008 (w/c 0.55, 0.5, 0.45, and 0.4, respectively))

Table 5.12 Depth of carbonation (mm) of controlled and recycled aggregate concrete at 3 and 7 days testing (Katz 2003)

Measuring location	Duration of test (days)	WPC concrete				OPC concrete			
		Controlled concrete	Crushing age (days)			Controlled concrete	Crushing age (days)		
			28	3	1		28	3	1
Top	3	6.3	10.2	9.2	8.9	8.8	13.2	14.4	13.2
	7	7.4	13.3	12	12.8	13.8	17	17.9	17.7
Bottom	3	4.5	8.4	7.9	9.1	6.6	11.7	12.2	11.5
	7	5.9	10.1	9.7	12.3	10.8	17	18.2	14.7
Sides	3	6.2	9.9	8.9	9.9	10.9	13.8	14.2	12.7
	7	7.3	11.9	10.5	13.1	12.8	16.3	17	17.1

normal concrete at both 3 and 7 days testing and RAC with OPC had shown higher values than WPC recycled aggregate concrete. Further, it was reported that the RAC made with WPC and aggregates crushed at 3 days had shown the lower depth of carbonation, whereas the effect of the crushing age of RA on RAC prepared with OPC was not clear.

Levy and Helene (2004) investigated the durability performance of RAC made with different proportions of recycled coarse and fine aggregates. The coarse recycled concrete aggregate (CRCA) and fine recycled concrete aggregates (FRCA) were obtained from 6-month-old concrete structure whose compressive strength was 25 MPa. Similarly, the coarse recycled masonry aggregate (CRMA) and fine recycled masonry aggregates (FRMA) were obtained from one-year-old clay brick masonry wall covered with cement mortar. A total 13 mixes were considered for the investigation of the durability performance of concrete. The results of the carbonation depth of various concrete families reported by the authors are shown in Table 5.13 and Fig. 5.36. It reveals that the depth of carbonation decreased with the increased percentage of recycled aggregate up to 50%. Further, it revealed that the depth of carbonation of RAC with 100% RA obtained either from concrete or masonry for all strength levels was even lower than the controlled concrete. The recycled aggregate concrete requires larger cement content to achieve the same strength compared to the controlled concrete with natural aggregate, which resulted the higher alkaline reserve act by shielding the concrete surface against carbonation mechanism. In addition, the recycled aggregate obtained from either old concrete or masonry adhered with old mortar with cement and calcium hydroxide particles, which also result in the increase in alkaline reserve in recycled concretes. The alkaline reserve not only beneficial for the carbonation mechanism but also increase the service life of the structure (Clifton 1993), because the reinforcement embedded in concrete inherently protected against corrosion by passivation of reinforcement surface due to high alkalinity of the concrete, which means increase in the initiation period (Tuutti 1982).

The depth of carbonation of RAC increased with the increase in the percentage of RCA (Wu and Song 2006a, b; Sun et al. 2006). It was reported that the depth of carbonation of RAC was 62% more than the reference concrete when RAC contains 60% RCA. This was mainly due to higher amount of RCA in RAC. However, this can be enhanced by the addition of slag, steel slag, or fly ash in RAC (Sun et al. 2006).

Kou and Poon (2012) investigated the influence of fly ash on the durability of RAC made with different proportions of RCA. The authors considered two series of mixes: Series I mixes with a w/c ratio of 0.55 and Series II mixes with a w/c ratio of 0.42. In each series 0, 25, 35% fly ash by weight of cement was used. The carbonation depth of concrete mixes of both Series I and II reported by the authors is shown in Fig. 5.37. It was observed that with the increase in percentage of recycled aggregate content, the depth of carbonation increased. This was expected as the capillary absorption and chloride penetration have shown the same trend in their studies. Evangelista and de Brito (2010) reported a similar result with the fine aggregate. In their studies, it was reported that the depth of carbonation increased with the increase in fine recycled aggregate replacement ratio. Further, the carbonation depth of concrete mixes increased with the use of fly ash as an addition as well as partial replacement of cement.

The depth of carbonation of normal concrete and recycled aggregate concrete prepared with 100% RCA reported by different researchers in the literature is presented in Fig. 5.38. It can be seen from the figure that the inclusion of 100% RCA increases the depth of carbonation of concrete. However, this can be compensated by the addition of mineral admixtures in recycled aggregate concrete mixes.

Table 5.13 Cement content and carbonation depth of all concrete families (Levey and Helene 2004)

Aggregate	% of replacement	Cement content (kg/m^3)			Carbonation depth, CO_2 (mm)		
		fc_{28} = 20 MPa	fc_{28} = 30 MPa	fc_{28} = 40 MPa	fc_{28} = 20 MPa	fc_{28} = 30 MPa	fc_{28} = 40 MPa
Natural	0	179	291	397	9.6	6	3.5
CRCA	20	269	341	407	5.5	3.9	2.7
	50	231	329	422	5	3.4	2.3
	100	190	293	392	7.7	5.2	3.4
Natural	0	179	291	397	9.6	6	3.5
CRMA	20	200	333	476	7	4.2	2.2
	50	279	417	569	7.6	3.4	0.4
	100	326	522	852	4.3	0.2	0
Natural	0	179	291	397	9.6	6	3.5
FRCA	20	239	325	404	6	4.6	3.6
	50	216	330	445	6.5	4.6	3.3
	100	266	366	461	6.7	4.9	3.6
Natural	0	179	291	397	9.6	6	3.5
FRMA	20	220	329	434	6.4	4.3	2.8
	50	191	300	407	5.8	4	2.7
	100	217	332	455	9	5.8	3.5

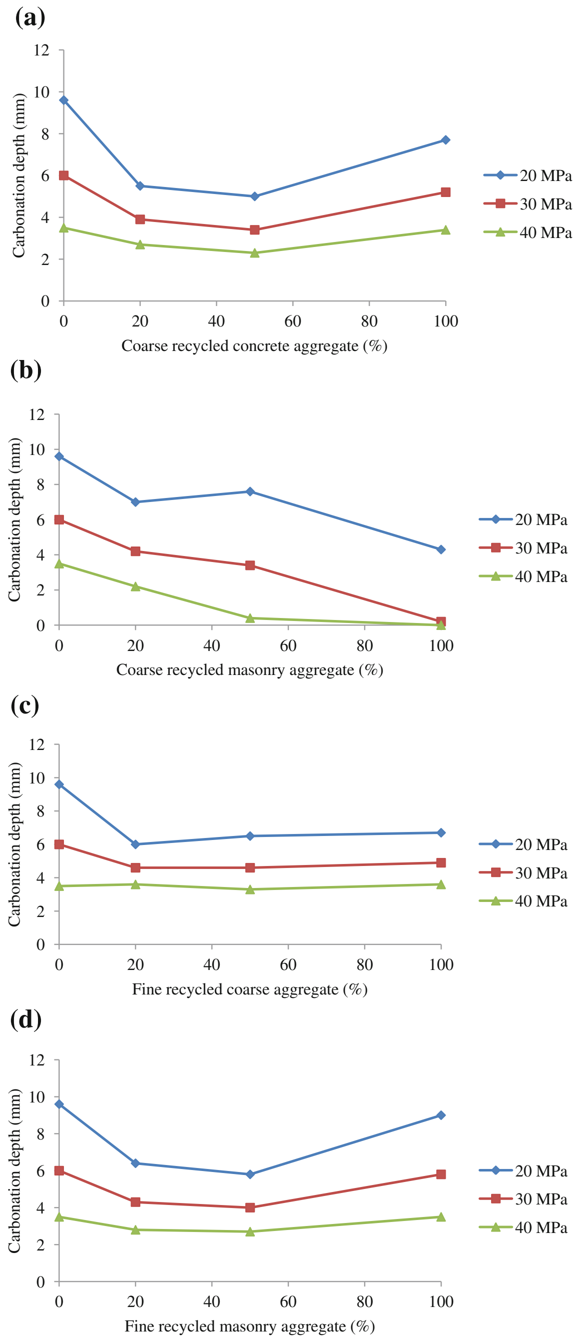


Fig. 5.36 Carbonation of different concrete families (**a**) with CRCA, (**b**) with CRMA, (**c**) with FRCA and (**d**) with FRMA (Levey and Helene 2004)

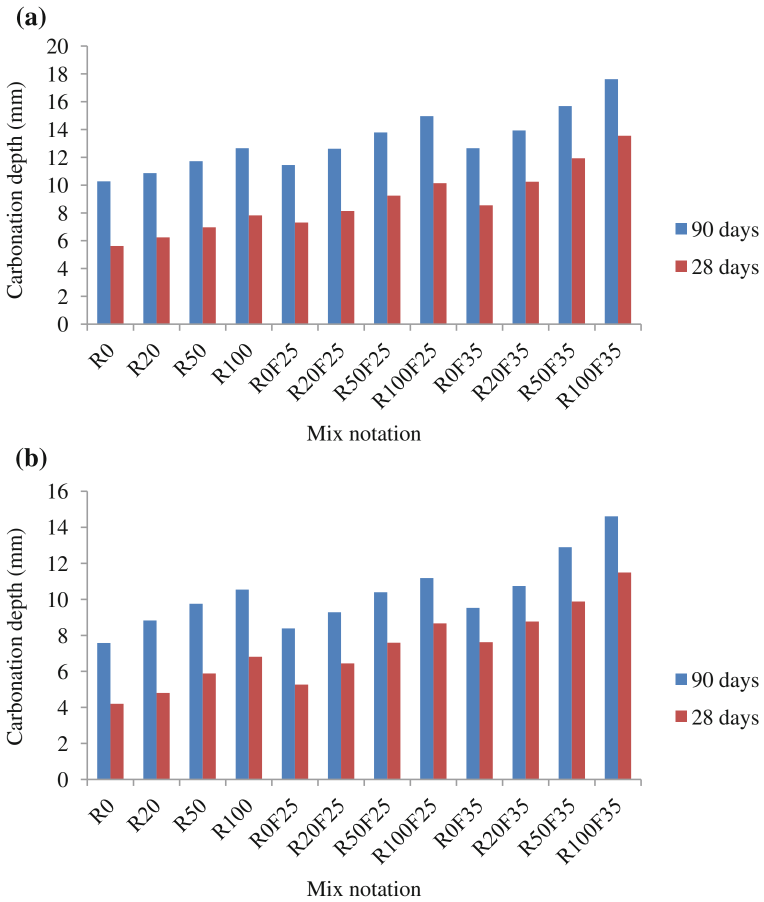


Fig. 5.37 Carbonation depth of concrete mixes in (**a**) Series I and (**b**) Series II (Kou and Poon 2012)

5.5 Summary

The long-term properties such as shrinkage and creep and the durability properties like permeability, chloride penetration, and carbonation depth of recycled aggregate concrete are discussed. The influence of different factors such as RA percentage, quality of recycled aggregate, mineral admixtures, source concrete, crushing method and age of crushing, method of curing is described. Based on these discussions, the following important aspects are observed.

- The factors such as the amount of recycled aggregate, quality of recycled aggregates, curing conditions, method of crushing procedure, method of mixing, use of mineral admixtures mainly influence the long-term and durability performance of recycled aggregate concrete.

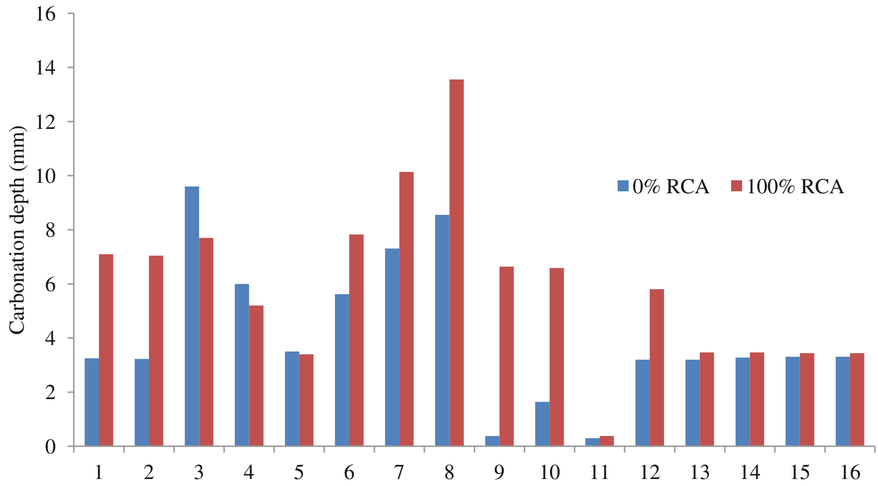


Fig. 5.38 Depth of carbonation at 28 days reported by 1 and 2: Zhu et al. (2013) (Silane 100 g/m^2); 3–5: Levey and Helene (2004) (20 MPa, 30 MPa, and 40 MPa, respectively); 6–8: Kou and Poon (2012) (0% FA, 25% FA, and 35% FA, respectively); 9–11: Sim and Park (2011) (0% FA, 15% and 30% FA, respectively); 12: Evangelista and de Brito (2010); 13–16: Kou and Poon (2013) (0% FA, 25% FA, 35% FA, and 55% FA, respectively)

- The drying shrinkage of recycled aggregate concrete increases with the increase in substitution of recycled aggregate, and it seems to have linear development with the increase in substitution level of RA due the increased volume of total cement paste by the contribution of adhered old cement mortar in recycled aggregate. Nevertheless, 20–30% substitution of RA did not show any significant difference in the shrinkage behavior of RAC and normal concrete. But, the inclusion of recycled fine aggregate greatly affects the shrinkage behavior of RAC.
- The drying shrinkage of RAC depends on the quality of the source concrete from which the RCA derived. The water absorption is a good indication of the amount of adhered cement paste in RCA, and the larger water absorption resulted by the attached old cement mortar in RCA increases the drying shrinkage. The recycled aggregate concrete prepared with RCA obtained from higher strength parent concrete resulted in lower shrinkage strains.
- The role of w/c ratio on the shrinkage behavior of RAC was not clear as some of the researchers found that lower the w/c ratio lower the shrinkage of RAC in contrary to the results of Tam et al. (2015) that the medium w/c ratio produces the lowest drying shrinkage in RAC.
- The effect of mineral admixtures particularly the fly ash and silica fume in controlling the shrinkage of RAC is clearly evident. The improvement in shrinkage resistance of RAC was due to its smaller particle size and pozzolanic reaction, which results in the exclusion of water on the surface of aggregate in non-coated aggregate–cement system, denser microstructure, and rich interfacial transition zone.

- The adoption of high-quality recycled coarse aggregate in concrete may enhance the shrinkage resistance to the level of conventional concrete. This may be achieved by adopting additional crushing stages in recycling procedure of RA, which decreases the amount of more deformable characteristic of old attached mortar and thus efficiently restrains shrinkage of concrete.
- The samples cured in dry environment had shown more harmful effect on the shrinkage resistance of recycled aggregate concrete. Less loss of water due to evaporation if the samples remain cured in environment with high relative humidity, results an equivalent or slightly more shrinkage of RAC than normal concrete.
- Like shrinkage, the creep of RAC also increased with the increase in amount of recycled aggregate, and particularly, this increase is worse when fine recycled aggregate was used due to the presence of a large amount of adhered mortar in smaller size aggregate. The creep of RAC with 100% RCA is 50–60% higher than that of concrete with natural aggregate.
- The effect of w/c ratio and aggregate-to-cement ratio did not show a clear trend on the variation of creep of RAC. The addition of fly ash by weight of cement can effectively lower the creep strain of RAC prepared with recycled coarse aggregate due to the development of long-term strength by the pozzolanic reaction of fly ash.
- Recycled aggregate concrete is found to be more sensitive to curing type than concrete with natural aggregate with respect to creep, and the specific creep of RAC can be minimized with the increase in duration of curing.
- The existing models such as ACI, Eurocode, and Bazant Baweja B3 were found to be sufficiently accurate to estimate the creep of both natural and recycled aggregate up to a creep coefficient of less than 2, and probably, these models not considered the properties of aggregates in the process. An empirical method proposed by Lye et al. (2016) based on the published data in the literature can be used in conjunction with any code of practice for estimating the creep of RAC for a given strength and percentage of RA.
- The durability performance of recycled aggregate concrete is inferior to the normal concrete. The meager performance of RAC in durability is mainly related to the poor quality of RA because of the presence of a large number of pores, cracks, and fissures present inside the RA, thus making it more prone to the permeation.
- TSMA significantly improves the durability performance of recycled aggregate concrete as the initial half of the water added in the first stage of mixing was able to form a thin layer of cement slurry on the RCA surface, which will try to penetrate and fill up the pores and cracks in the old attached mortar; hence, the ITZ becomes stronger when compared to SMA.
- Resistance against chloride ion penetration of concrete increases with decrease in w/c ratio, as the decrease in w/c ratio reduces the total volume of the pores inside the concrete, and hence, the concrete becomes more impermeable. Further, the resistance against chloride ion penetration can be increased by increasing the curing period from 28 days to 90 days.

- In recycled aggregate concrete also the fly-ash addition shown to be very effective in increasing the chloride ion penetration and carbonation resistance. It can be advantageous in increasing the service life of structures made with RAC as it increases the initiation period for corrosion of reinforcement. The combined effect of the addition of fly ash and lower w/c ratio would yield the better performance of concrete against the durability performance of RAC.

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Chapter 6
Microstructure of Recycled Aggregate Concrete

6.1 Introduction

Concrete is a composite material which consists of primarily aggregate fragments dispersed in a binding medium (cement matrix). Macrostructure is the gross structure of a material that is visible to the human eye. From the macroscopic examination, the two phases of the concrete can be differentiated easily: cement matrix and aggregates of different shapes and size. Microstructure is the subtle structure of a material that can be resolved with the help of a microscope (Mehta and Monterio 2006). As the concrete is a composite material, the complexities of the microstructure of concrete are evident. It becomes obvious that the two phases of the microstructure are neither homogeneously distributed with respect to each other, nor are they themselves homogeneous. For example, at some places, the hydrated cement paste mass appears to be as dense as the aggregates, while in other places it is highly porous. In the presence of aggregates, the microstructure of hydrated cement paste in the vicinity of large size aggregate is different from the microstructure of the bulk paste. In fact, many aspects of concrete behavior under stress can be explained only when the aggregate–cement paste interface is treated as a third phase of the concrete microstructure (Mehta and Monterio 2006). Thus, the microstructure of concrete has three phases: bulk hydrated cement paste phase, aggregate phase, and the interfacial transition zone (ITZ), which is generally 10–50 μm thick around the aggregate and is generally weaker than the other two phases (bulk cement paste and aggregate) between aggregate particles and cement mortar. Therefore, the ITZ greatly influences the mechanical and durability characteristics of concrete. As the two features of the microstructure of concrete, namely ITZ and hydrated cement paste, are subjected to change with time, environmental humidity, and temperature, the microstructure of concrete is not an intrinsic property of the material.

The interfacial transition zone (ITZ) between the aggregate and the cement mortar matrix is the most important interface in concrete. A fundamental study of this ITZ gives a more insight into the understanding of the concrete characteristics. In general,

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the ITZ is the weakest link of the chain and is treated as the strength-limiting phase in concrete (Mehta and Monterio 2006). Therefore, the concrete fails at a considerably lower stress level than the strength of either of the two main components (mortar matrix and aggregate) due to the presence of ITZ. In this composite material (concrete), the ITZ acts as a bridge between the two components, i.e., the coarse aggregate and cement mortar matrix. Even when the individual components are of high stiffness, the stiffness of the composite (concrete) is reduced because of the broken bridges (i.e., voids and microcracks in the ITZ), which do not permit stress transfer. Therefore, the properties of ITZ, especially the porosity and microcracks, have great influence on the stiffness or the elastic modulus of concrete (Mehta and Monterio 2006). In particular, the study of the interface is more important in recycled aggregate concrete, as the recycled aggregate concrete has more interfaces than normal concrete. That is the interfacial transition zone between the original aggregate and the adhered mortar (old ITZ), and another interface between the adhered mortar and new mortar matrix (new ITZ) in recycled aggregate concrete is shown in Fig. 6.1. In contrast, the normal concrete has only one ITZ, i.e., interface between the aggregate and mortar, as can be seen in Fig. 6.2.

Fig. 6.1 The Interfacial transition zones in RAC

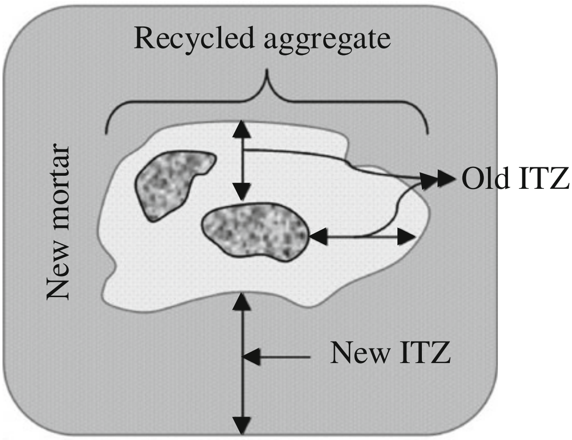
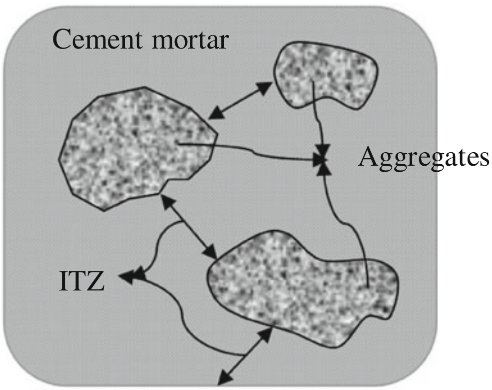


Fig. 6.2 ITZ in normal concrete



It is generally accepted that the cement paste which is adhered to the recycled aggregate is one of the major factors that influence the performance of recycled aggregate concrete. The quality of the mortar and interfaces, as well as the mortar components of the original concrete, thus influences the properties of concrete. The present chapter discusses the characteristics of ITZ in terms of hydration compounds, anhydrous cement, porosity and microhardness, and their influence on the strength of concrete. Further, it discusses the influence of the binder, quality, and quantity of adhered mortar on interfacial transition zone.

Chakradhara Rao (2010) conducted microscopic examination on RAC made with RCA obtained from different sources of RCA. To carry out the microscopic examination, a total of five 100 mm diameter × 200 mm height cylinders each from normal concrete made with type 1 cement (M-RAC0), RAC made with RCA obtained from Sources 1 and 2 (MM-RAC100, MK-RAC100), normal concrete made with type 2 cement (M-RAC0) and RAC made with Source 3 RCA (MV-RAC100) were cast and cured for 28 days along with the samples prepared for the rest of the studies. Both type 1 and type 2 cements belong to OPC 43 grade; however, there is a little difference in their physical, chemical and mechanical properties. In the above mixes, the first letter represents the mix, second letter indicates the Source of RCA (details of Sources are already discussed in Chap. 4), RAC indicates the recycled aggregate concrete, and the number represents the percentage of RCA in the mix.

6.2 Sample Preparation for Microscopic Study

Specimen preparation is very important for identifying the features in scanning electron microscope (SEM). Identification of suitable concrete for microscopic analysis is important due to the complex heterogeneity of concrete. Concrete surfaces with large areas of paste are better suited against those with a lot of aggregate to obtain maximum information. After 28 days of curing, 10–12 mm thick slices were cut from a 100 mm diameter × 200 mm height cylinder at different heights using a precision diamond saw and kerosene as lubricant. From each slice again, approximately 10 × 10 mm rectangular sections were cut. The specimens were then dried in desiccators for more than 3 days. The dried specimens were then vacuum impregnated with a low viscosity epoxy coded as Epoxil-43 and hardener as Epoxil-MH43 in a 3:1 ratio (shown in Fig. 6.3) and allowed to harden at room temperature for 1–6 h.

The importance of the impregnation of the specimens with epoxy resin is to protect the specimen and prevent its damage during subsequent stages of grinding and polishing. In addition, the epoxy resin fills the voids and enhances contrast between the pores, hydration compounds, and anhydrous cement. The impregnated specimens are then carefully grounded and polished. The impregnated specimens are then coarse polished on #500 and #1200 grit paper to remove epoxy from the surface of the specimen. This surface is then subjected to fine polishing on

Fig. 6.3 Test setup for specimen impregnation with low viscosity epoxy resin

automated polishing machine using series of successively finer grade of diamond pastes: 9, 3, 1 and 0.25 μm. Petroleum is used as lubricant during polishing. After every stage of polishing, the specimens are checked under an optical microscope for the quality of polishing achieved. Polishing is then advanced to the next stage and continued till desired level of polishing achieved. Normally, each specimen is polished for 3–4 h to obtain a good surface. The polished specimens are then cleaned in an ultrasonic bath and dried in vacuum to remove any remaining lubricant from the surface. The specimens are then coated with a thin layer of carbon to prevent charging during backscatter scanning electron (BSE) imaging.

6.3 Scanning Electron Microscope (SEM)

The JEOL-JSM-6490 is a high-performance scanning electron microscope with a high resolution of 3.0 nm and is coupled with an energy dispersive X-ray spectrometer (EDS) which is used to analyze the samples in the study. This facilitates the qualitative analysis of the major elements on the surface.

6.4 Image Acquisition

The selection of areas within a given sample, resolution, magnification and the number of images required to acquire are important aspects to be considered during the analysis. To measure the capillary porosity, 800× magnification was appropriate (Sahu et al. 2004). In this study, almost all the backscatter scanning electron (BSE) images are acquired with 1000× magnification at 512 × 512 pixel resolution. Accordingly, each pixel is approximately 0.3 μm in each direction. The basic aim of this study is to characterize the microstructural features (porosity, hydration compounds, and anhydrous cement) of the ITZ. Therefore, each image is selected in such a way that it covers a small part of the aggregate in addition to the mortar matrix. Similarly, a series of images are acquired randomly along the length of the ITZ. Three samples are considered for each concrete mix, and a total of 20 images are acquired from all the three samples randomly.

6.5 Image Analysis

All the BSE images are analyzed quantitatively by using earth resources data analysis system ERDAS 8.5 image processing software. Depending upon the gray level, intensity 256 classes of shades from black to white are available in the BSE images. Generally, the lowest level, recorded as gray level 0, is fully black and the highest level, recorded as gray level 255, is fully white. In general, darker areas on the image is considered as areas of lower gray level and conversely of the brighter areas as areas of higher gray levels (Diamond 2004). One can easily identify the residual cement from the other components, as the residual cement is at the bright end (fully white) of the gray scale. Similarly, the pores are usually filled with epoxy resin in specimen preparation can accurately be separated from other components, as they are at the dark end of the gray scale. The separation of calcium hydroxide (CH) from the calcium silicate hydrate (C–S–H) is slightly uncertain, as CH is slightly brighter than C–S–H in the gray scale (Diamond 2001). In the present study, based on the gray-level histogram of the BSE images, the threshold values are decided for various components. Nevertheless, in some cases the peak for CH is not clearly visible. One of the histograms of such cases is shown in Fig. 6.4. This may be partly due to the resolution of the BSE technique and partly due to the physical nature of the CH boundary.

The technique adopted in the work of Patel (Scrivener 2004), shown in Fig. 6.5 is used in the present study to fix the arbitrary threshold value for CH.

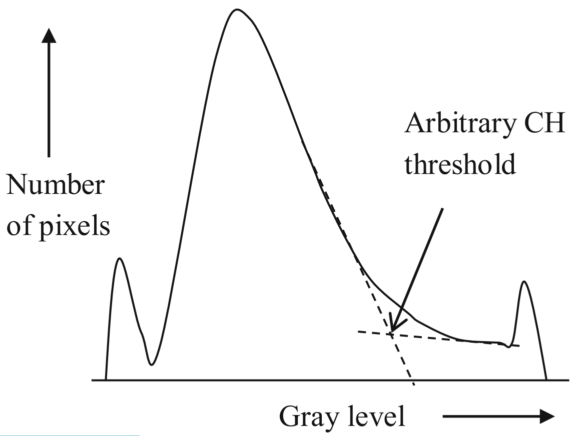


Fig. 6.4 Typical gray-level histogram (from the left, the peaks correspond to pores, C–S–H and anhydrous cement (UH). There is no defined peak for the CH as discussed in the text)

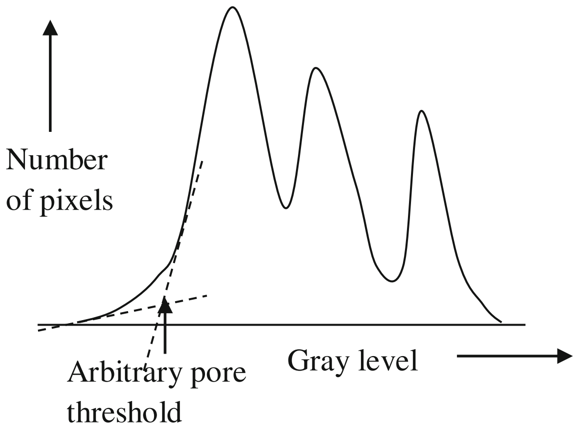


Fig. 6.5 Typical gray-level histogram of hardened cement paste (from the left, the peaks correspond to C–S–H, CH and UH. There is no discrete peak for porosity) (Scrivener 2004)

After finalizing the threshold of each component, the pixels corresponding to each component are separated using binary segmentation. The binary segmentation of some of the images for various components is presented in Figs. 6.6, 6.7, 6.8, 6.9, and 6.10. The percentage area of each component is calculated by dividing the number of pixels of each component by the total number of pixels in the image filed. It is important to mention that the pixels corresponding to aggregate particles are excluded and the percentage area of each component is calculated based on the area of the paste, rather than the area of concrete.

6.6 Vickers Microhardness (HV)

A UHL VMHT microhardness tester (VMH-001) is used to measure the microhardness of ITZ of both normal concrete and recycled aggregate concrete made with all the three Sources of RCA. It has motorized selection of test force which offers full control via touch panel display, and it has the automatic loading procedure. It is interfaced with a computer, which facilitates the easy and accurate measurements of hardness. A typical hardness measurement is shown in Fig. 6.11.

6.7 Characteristics of Interfacial Transition Zone (ITZ)

Interfacial transition zone (ITZ) is the region of cement paste around the aggregate particles, which is perturbed by the presence of aggregate (Scrivener et al. 2004). Due to the large differences between the sizes of aggregates and cement grains, each aggregate particle acts as a mini “wall”, which interrupts the packing of the cement grains. This gives a zone, closest to the aggregate, and contains predominantly

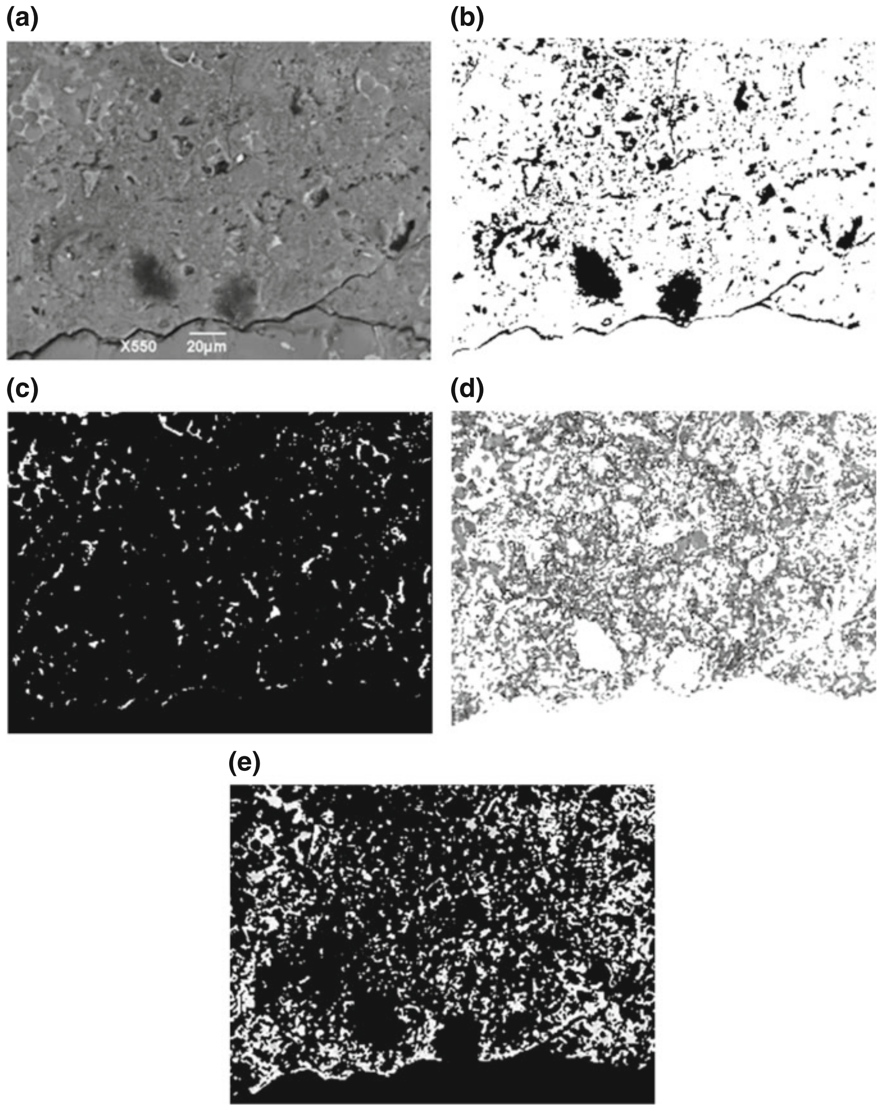


Fig. 6.6 Binary segmentation of BSE images in M-RAC0. (**a**) Gray image (**b**) binary image of pores (**c**) binary image of residual cement (**d**) binary image of C–S–H and (**e**) binary image of CH (Chakradhara Rao 2010)

smaller grains of cement particles and has a significantly higher porosity, while larger cement grains are found to be farther away from aggregate. Due to the deficit of cement grains in ITZ, there is effectively higher w/c ratio initially in this region. Therefore, for a given overall w/c ratio, the w/c ratio in the "bulk" cement paste is comparatively less. In addition, there is a relative movement of paste and aggregate

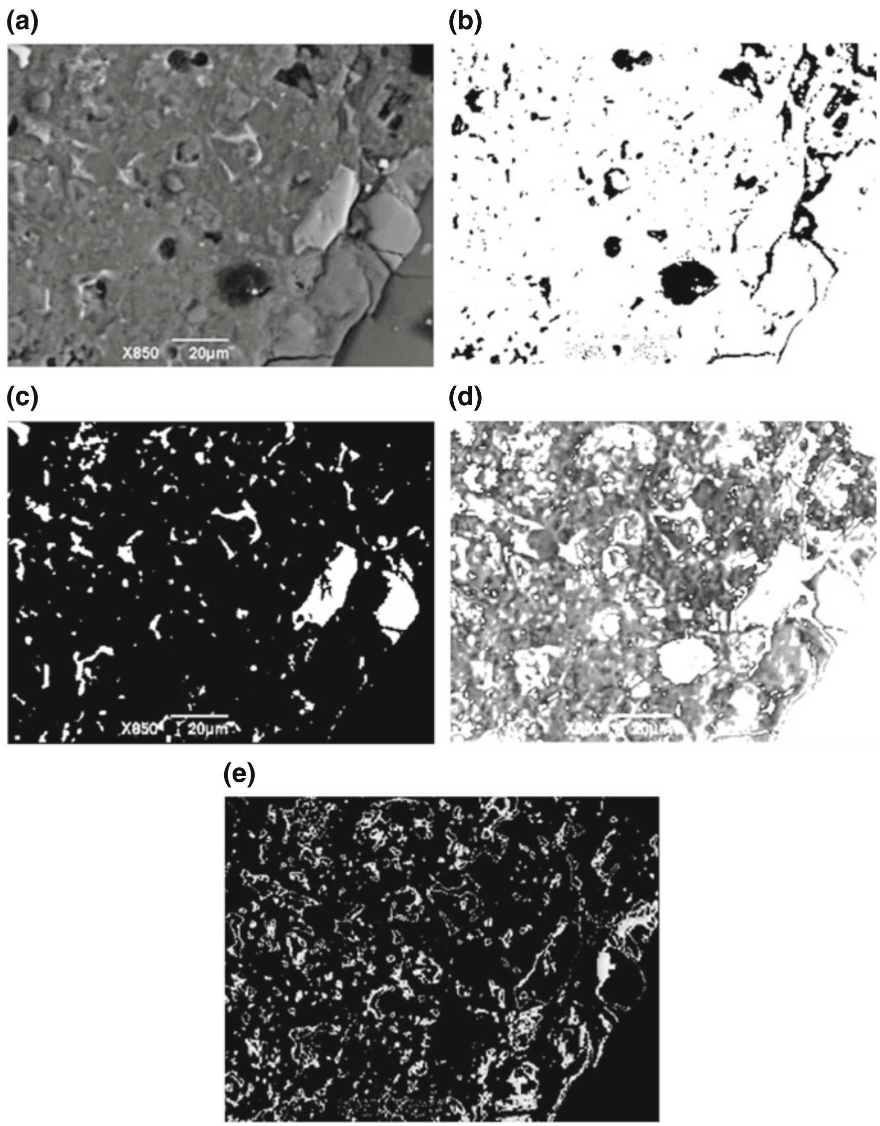


Fig. 6.7 Binary segmentation of BSE images in M-RAC0. (**a**) Gray image (**b**) binary image of pores (**c**) binary image of residual cement (**d**) binary image of C–S–H and (**e**) binary image of CH (Chakradhara Rao 2010)

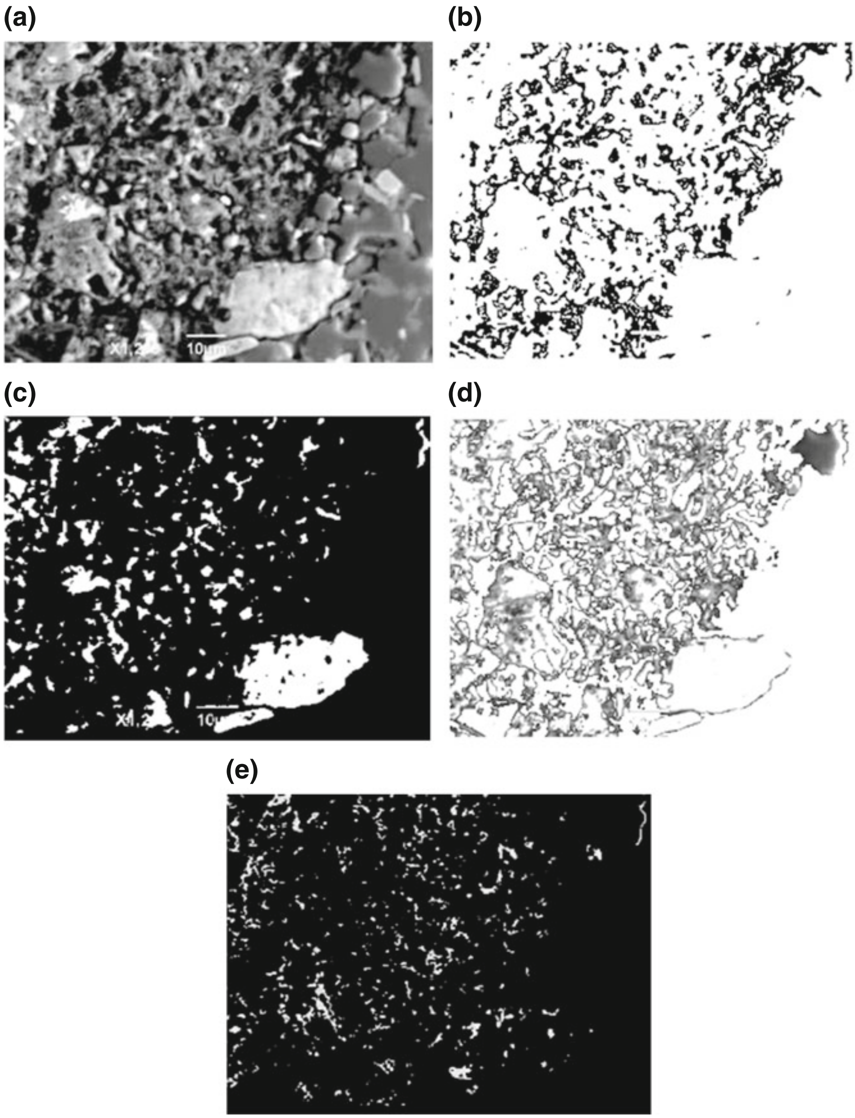


Fig. 6.8 Binary segmentation of BSE image in MM-RAC100. (**a**) Gray image (**b**) binary image of pores (**c**) binary image of residual cement (**d**) binary image of C–S–H, and (**e**) binary image of CH (Chakradhara Rao 2010)

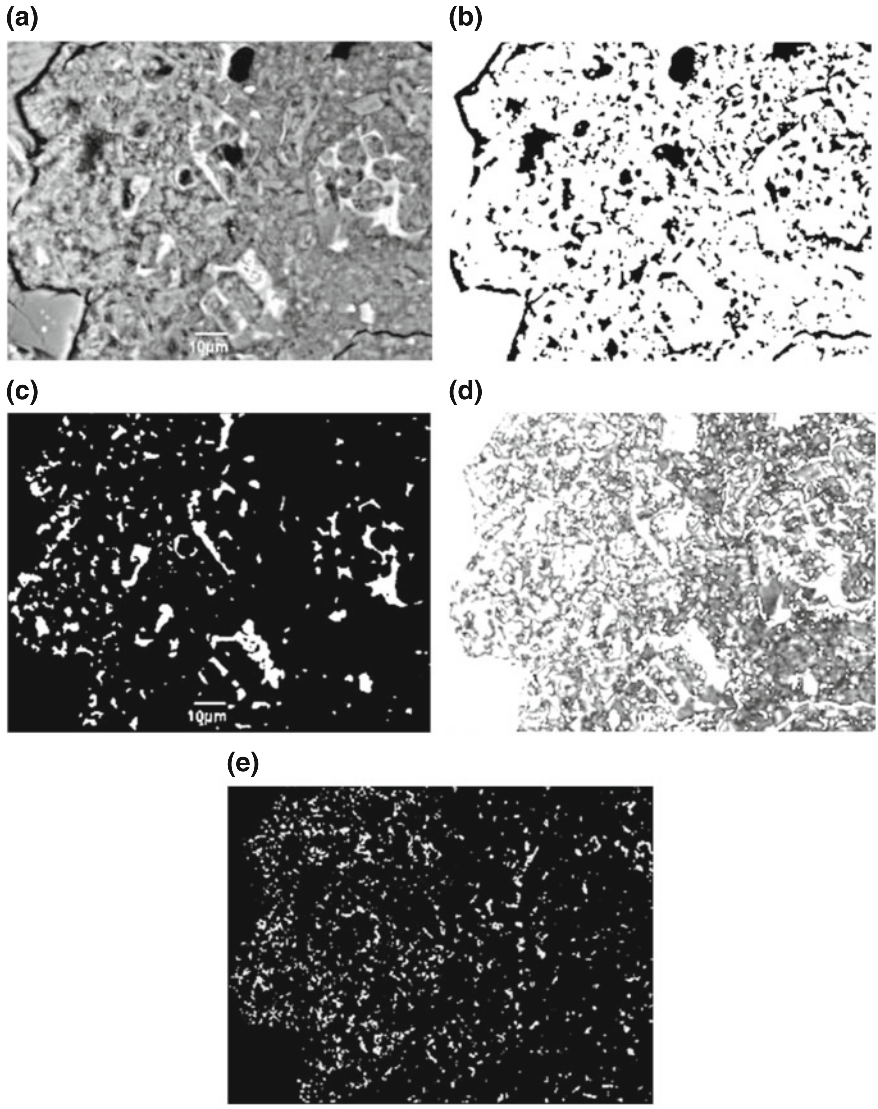


Fig. 6.9 Binary segmentation of BSE image in MK-RAC100. (**a**) Gray image (**b**) binary image of pores (**c**) binary image of residual cement (**d**) binary image of C–S–H, and (**e**) binary image of CH (Chakradhara Rao 2010)

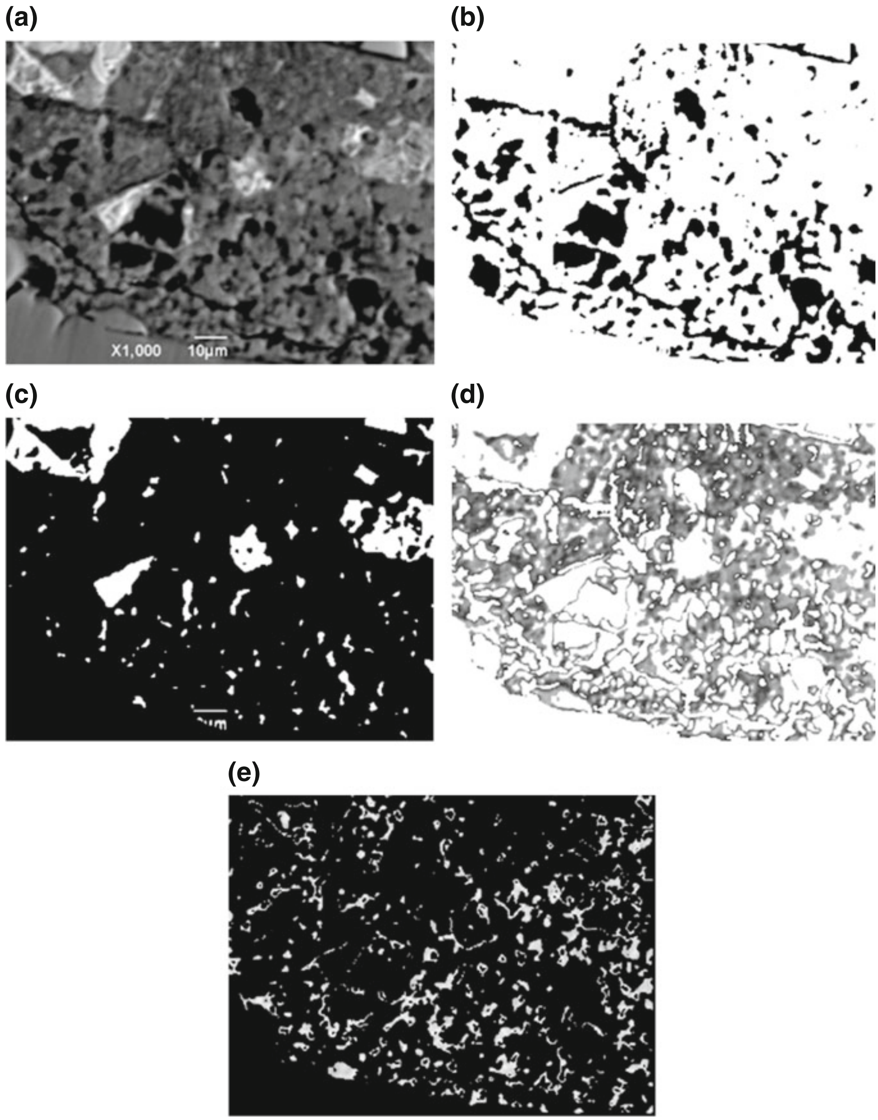
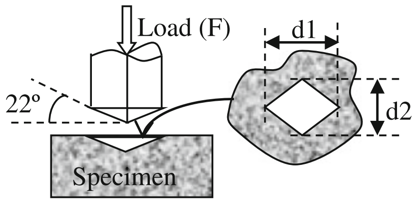


Fig. 6.10 Binary segmentation of BSE image in MV-RAC100. (**a**) Gray image (**b**) binary image of pores (**c**) binary image of residual cement (**d**) binary image of C–S–H and (**e**) binary image of CH (Chakradhara Rao 2010)

Fig. 6.11 Measurement of Vickers microhardness

Vickers Indenter details



Vickers hardness (HV)
$= 1.854F/((d1 + d2)/2)^2$
(Hardness unit HV 0.01, load 0.01kgf)
d1 and d2 are in mm

particles during mixing. Therefore, the high degree of heterogeneity and relative movement of particles gives large variations in the microstructure of concrete. In the subsequent sections, the important features of ITZ such as hydration compounds, anhydrous cement, and porosity are discussed.

6.7.1 Hydration Compounds, Anhydrous Cement, and Porosity

A large number of BSE images were analyzed using image processing techniques from three samples of each mix (M-RAC0, MM-RAC100, MK-RAC100, M-RAC0, and MV-RAC100) and the percentage area of individual hydration compounds, anhydrous cement, and porosity details are presented in Table A.1 in Appendix A. The mean area percentages of hydration compounds, anhydrous cement, and porosity of both normal concretes and recycled aggregate concretes made with recycled aggregate obtained from all the three Sources are presented in Table 6.1.

Table 6.1 Mean area percentages of porosity and hydration compounds of cement at ITZ (Chakradhara Rao 2010)

Source of RCA	Mix designation	Mean areas of porosity and hydration compounds (%)			
		Porosity	C–S–H + others	CH	UH
Source 1	M-RAC 0	15.22	56.25	18.96	9.57
	MM-RAC 100	20.28	47.94	20.87	10.91
Source 2	M-RAC 0	15.22	56.25	18.96	9.57
	MK-RAC 100	21.02	45.31	21.37	12.72
Source 3	M-RAC 0	16.79	55.41	16.90	10.90
	MV-RAC 100	21.00	49.80	17.67	11.53

It is ascertained that the cement is not fully hydrated in both normal and recycled aggregate concretes after 28 days of curing. This can be observed from the percentage area of remnant unhydrated cement grains (UH) in all the concrete mixes. There are approximately 9.5–11% remnant unhydrated cement grains in case of normal concretes and 11–13% in recycled aggregate concretes. It is also observed that the area percentage of total hydration compounds (calcium hydrate silicate (C–S–H) gel plus other hydrates and CH) is almost same in all the recycled aggregate concretes made with RCA obtained from all the three Sources. Nevertheless, the mean percentage area of total hydration compounds in RAC made with all the three Sources of RCA are less than those of normal concrete. The distribution of individual hydration compounds in normal concrete and recycled aggregate concretes made with recycled coarse aggregate obtained from all the three Sources is presented in Figs. 6.12, 6.13, and 6.14. The C–S–H gel is one of the major hydration

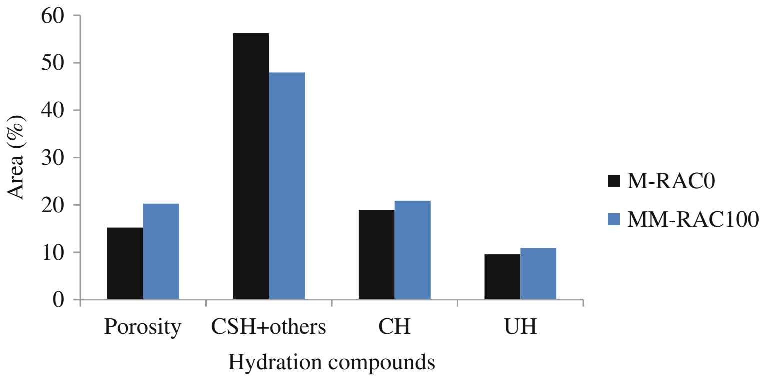


Fig. 6.12 Percentage area of porosity and hydration compounds in ITZ and in bulk paste in both normal concrete and RAC made with Source 1 RCA (Chakradhara Rao 2010)

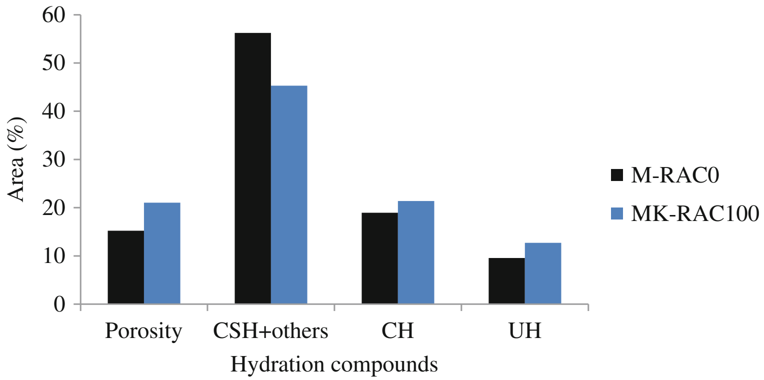


Fig. 6.13 Percentage area of porosity and hydration compounds in ITZ and in bulk paste in both normal concrete and RAC made with Source 2 RCA (Chakradhara Rao 2010)

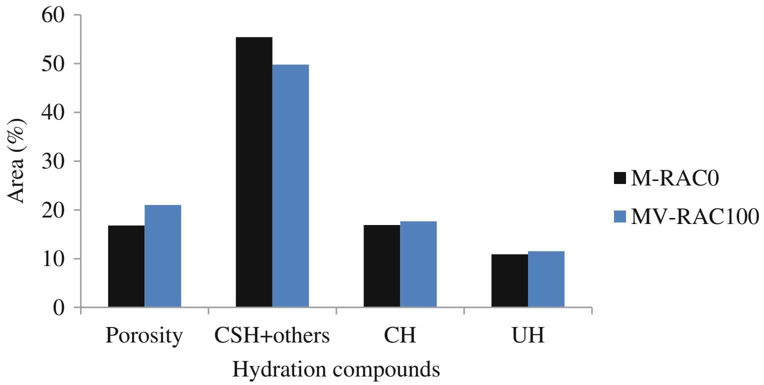


Fig. 6.14 Percentage area of porosity and hydration compounds in ITZ and in bulk paste in both normal concrete and RAC made with Source 3 RCA (Chakradhara Rao 2010)

compounds which contribute to the strength of concrete. It is a form of gel which binds the cement mortar and aggregate and enhances the density of ITZ (Tasong et al. 1998 and 1999). Generally in the initial stages of hydration, the C–S–H gets deposited directly around the cement grains, and in contrast, the calcium hydroxide (CH) mainly gets deposited in the open pores (Scrivener et al. 2004).

The percentage area of gel depends mainly on the ratio of the presence of calcium and silica in the cement and the aggregate type. The percentage area of C–S–H, CH, and UH in RAC made with RCA obtained from the Sources 1 is 47.94, 20.87, and 10.91, respectively, against to those of 56.25, 18.96, and 9.57 in corresponding normal concrete. The percentage area of C–S–H, CH, and UH in RAC made with RCA obtained from Source 2 is 45.31, 21.37, 12.72, respectively, against to those of 56.25, 18.96, 9.57 in corresponding normal concrete. Similarly, the percentage areas of C–S–H, CH, and UH in RAC made with Source 3 RCA are 49.80, 17.67, 11.53 against 55.41, 16.90, 10.9 in the corresponding normal concrete. These percentages of hydration compounds indicate that the ITZ in case of recycled aggregate concrete made with RCA obtained from all the three Sources is less dense when compared to ITZ in normal concrete. This attributes to the high absorption capacity of old mortar adhered to recycled aggregates in RAC.

Porosity is the volume which has not been filled by the cement grains or by the hydration products. The resolution in backscatter SEM limits the measurement of pore sizes. In the present study, all images are captured at 512 × 512 pixels; each pixel is approximately 0.3 μm in each direction which covers an area of 0.09 μm^2. Therefore, the minimum pore size that can be measured is 0.3 μm, and these are generally called capillary pores. The capillary porosity mainly depends on the rate of hydration and w/c ratio. The capillary porosity in matured concrete with a w/c ratio of 0.65 is four times higher than that in concrete with w/c ratio of 0.4 (Sahu et al. 2004). It is observed from Table 6.1 and Figs. 6.12, 6.13, and 6.14, the porosity in recycled aggregate concrete made with RCA obtained from all the three Sources are more than those of corresponding normal concretes. The area of porosity in normal concrete is in the range of 15.22–16.7%. Whereas, the porosity in RAC made with RCA obtained from Sources 1, 2, and 3 are 20.28, 21.02, and 21%, respectively. Albeit there are some chemical reactions expected between the remnant cement particles in recycled aggregates and new cement mortar would create some interfacial bonding effects, the results indicates that the ITZ in recycled aggregate concrete is loose and porous than normal concrete ITZ. This may be due to the presence of old mortar in recycled aggregates, which absorbs more water during the initial stages of mixing and leads to the higher porosity. The high porosity and water absorption capacity of recycled aggregates made from normal strength concrete coupled with its low initial water content rendered the aggregate to take up a larger amount of water during the initial stages of mixing and hence the open and loose ITZ in RAC (Poon et al. 2004).

6.7.2 Distribution of Hydration Compounds, Anhydrous Cement, and Porosity Across the Width of ITZ

As discussed earlier, due to “wall” effect, there is a deficiency of cement grains near the aggregate surface zone. Therefore, there is a substantially higher porosity than the bulk paste. Also, there is a change in the degree of hydration compounds in this zone compared to the bulk paste due to the mobility of hydration compounds. This zone is extended about 30 μm from the aggregate surface (Diamond 2001). As there is a change in the quantities of hydration compounds, anhydrous cement, porosity across the width of ITZ and bulk paste, this section describes how these components distribution differ from ITZ to the bulk paste. To analyze these distributions, each BSE image is segmented into series of strips of each 10 μm wide parallel and along the ITZ. Based on the intensity of gray level, each gray-level image strip is converted into binary image to calculate the quantities of hydration compounds, anhydrous cement grains, and porosity. The original gray-level image, the segmentation and the binary images of various compounds are presented in Figs. 6.15 and 6.16. For each mix, three BSE images are considered and the average results are reported in Table 6.2 (percentage area of each component for each image is presented in Tables A.2–A.6 in Appendix A). The distribution of these components along the length and across the width of ITZ for both normal concrete and recycled aggregate concrete made with RCA obtained from all the three Sources are presented in Figs. 6.17, 6.18, 6.19, 6.20, and 6.21. It is clearly observed that even after 28 days of curing, the cement is not hydrated fully in both normal concrete and recycled aggregate concrete made with all the three Sources of RCA. The gradients of percentage area of anhydrous cement clearly exhibit the wall effect near the aggregate face, i.e., the deficiency of cement grains. The gradients of the percentage area of residual cement grains starting from the aggregate surface

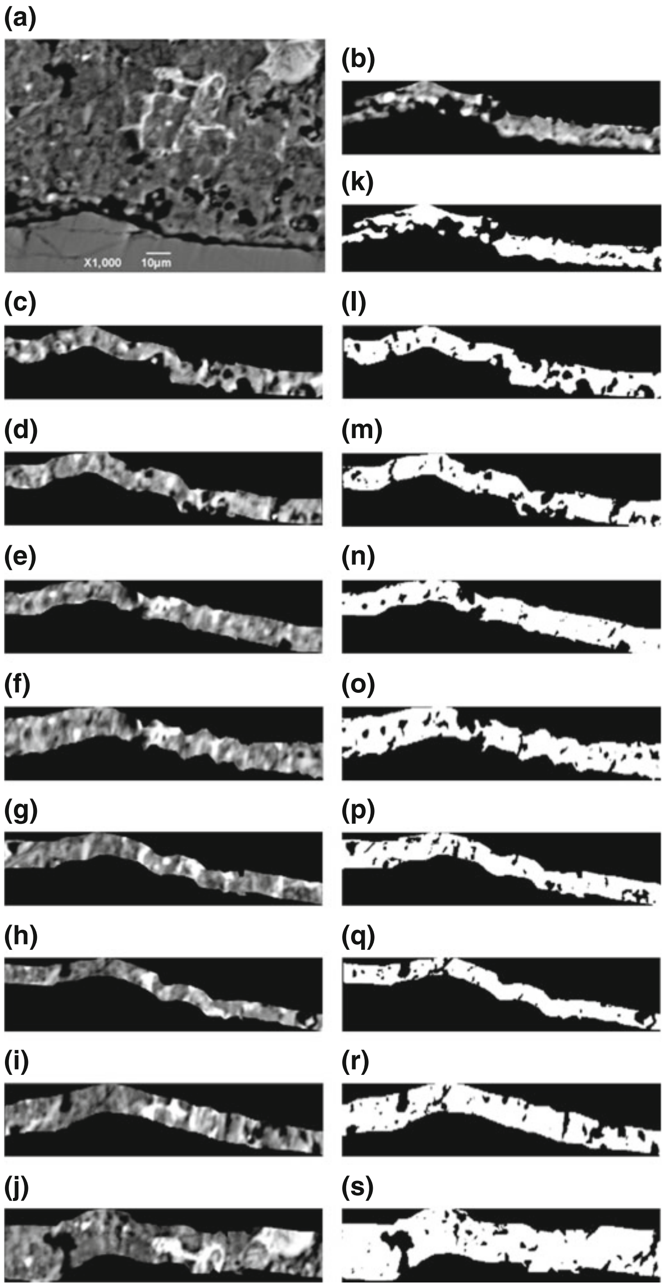


Fig. 6.15 Binary segmented images showing the distribution of pores across the ITZ from aggregate surface (**a**) Gray image (**b–j**) are gray images of 10–80 µm distance from aggregate of each 10 µm width (**k–s**) the pore binary images corresponding to (**b–j**) gray images (Chakradhara Rao 2010)

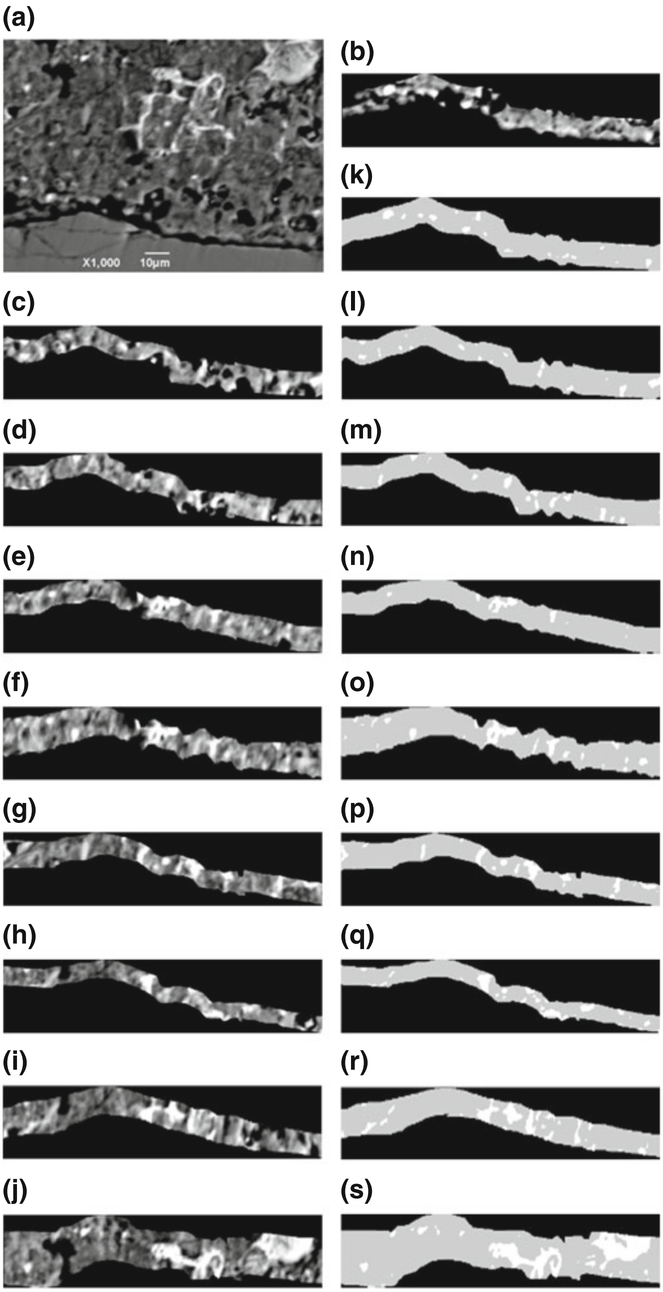


Fig. 6.16 Binary segmented images showing the distribution of residual cement across the ITZ from aggregate face. (**a**) Gray image (**b–j**) are gray images of 10–80 µm distance from aggregate of each 10 µm width (**k**–**s**) the residual cement binary images corresponding to (**b–j**) gray images (Chakradhara Rao 2010)

Table 6.2 Average percentage area of microstructural constituents of ITZ (Chakradhara Rao 2010)

Source of RCA	Mix designation	Constituent (area percentage)	Distance from aggregate interface (μm)								
			10	20	30	40	50	60	70	80	100
Source 1	M-RAC0	Porosity	21.63	17.70	17.57	16.78	17.71	17.35	16.03	13.82	13.29
		C–S–H + others	45.81	52.84	53.12	55.84	52.77	55.35	56.86	61.45	62.41
		CH	30.23	25.66	24.13	22.13	23.19	20.23	19.46	15.66	13.92
		UH	2.34	3.80	5.18	5.25	6.33	7.07	7.66	9.08	10.38
	MM-RAC100	Porosity	26.34	21.96	20.77	19.80	19.79	19.77	18.34	18.58	19.73
		C–S–H + others	44.93	49.03	51.83	53.54	53.55	53.95	56.38	55.31	55.72
		CH	25.40	23.51	20.66	17.81	17.20	16.50	14.64	15.44	13.82
		UH	3.33	5.50	6.73	8.85	9.46	9.79	10.64	10.67	10.72
Source 2	MK-RAC100	Porosity	26.74	25.41	18.99	19.47	18.02	17.63	16.87	16.61	16.36
		C–S–H + others	44.46	47.09	52.49	54.04	53.45	54.26	56.78	57.67	57.23
		CH	21.99	20.91	20.93	18.19	20.07	19.25	15.88	14.45	14.74
		UH	6.80	6.58	7.59	8.30	8.45	8.86	10.47	11.27	11.67
Source 3	M-RAC0	Porosity	22.59	18.87	17.99	13.73	14.16	14.65	13.30	13.30	15.20
		C–S–H + others	52.47	55.90	58.98	62.85	62.19	60.85	62.79	61.23	59.72
		CH	21.11	20.23	17.29	17.48	16.37	16.79	14.94	15.75	15.49
		UH	3.83	5.00	5.74	5.94	7.28	7.71	8.97	9.71	9.59
	MV-RAC100	Porosity	28.69	24.44	21.54	19.87	18.66	18.39	18.39	16.71	17.67
		C–S–H + others	42.44	47.24	51.70	52.97	54.74	54.82	56.05	58.11	57.72
		CH	21.85	21.07	19.24	19.01	16.48	16.18	14.93	14.38	14.62
		UH	7.02	7.26	7.52	8.15	10.12	10.60	10.62	10.80	9.99

indicate that the residual cement is increasing progressively as the bulk paste approached in both normal concrete and recycled aggregate concrete made with all the three Sources of RCA (also can be seen in Fig. 6.16).

From Fig. 6.17, it is ascertained that the percentage area of residual cement in the first 10–20 μm distance from the aggregate surface is approximately 1/3 of that of the corresponding bulk paste in normal concrete. Whereas, in RAC made with RCA obtained from all the three Sources, the percentage area of anhydrous cement in this zone is approximately 1/2 of that of the corresponding bulk cement pastes (Figs. 6.18, 6.19, 6.21). Beyond 50 μm distance from the aggregate surface, the percentage area of residual cement is almost same in both normal concrete and recycled aggregate concrete. This indicates that the percentage area of UH in the first 10–20 μm zone in RAC is more than that of normal concrete. Similar results are reported in the literature for normal concrete made with dolomite sand and coarse aggregate at 3 days curing (Diamond and Huang 2001). This may be

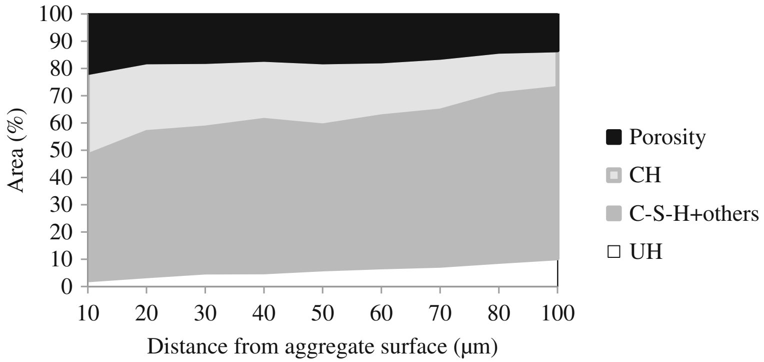


Fig. 6.17 Average distribution of microstructural constituents at the interfacial transition zone (ITZ) in normal concrete (Sources 1 and 2) (Chakradhara Rao 2010)

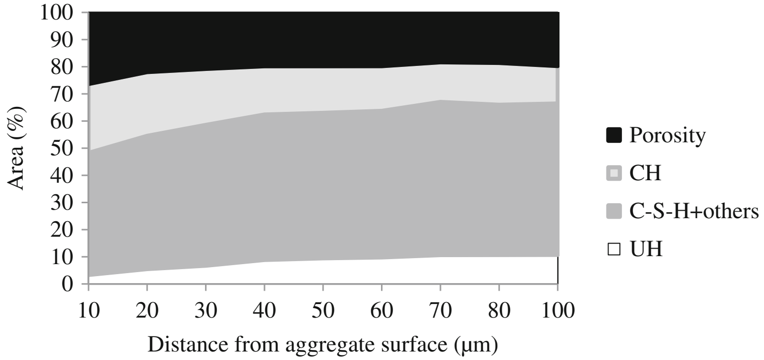


Fig. 6.18 Average distribution of microstructural constituents at the interfacial transition zone (ITZ) in MM-RAC100 (Source 1) (Chakradhara Rao 2010)

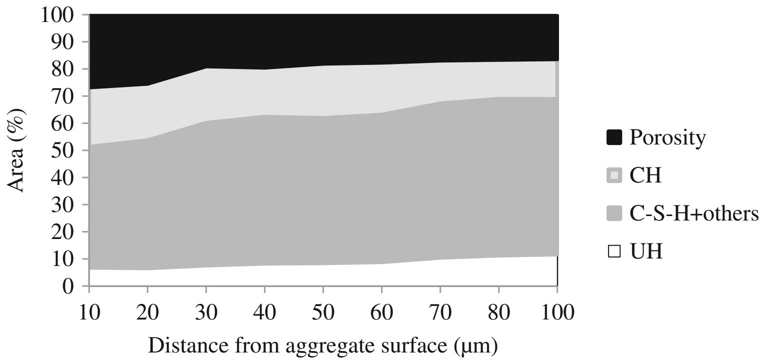


Fig. 6.19 Average distribution of microstructural constituents at the interfacial transition zone (ITZ) in MK-RAC100 (Source 2) (Chakradhara Rao 2010)

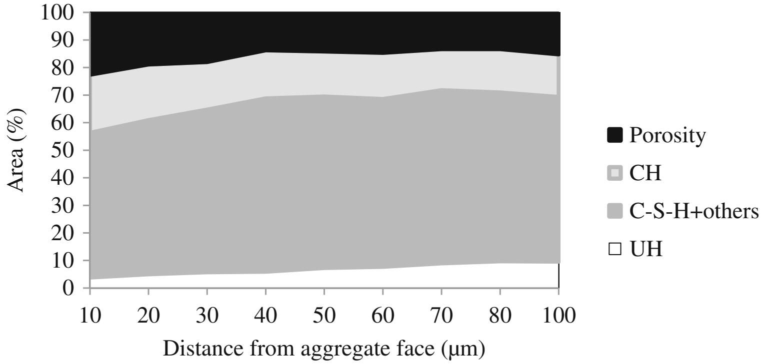


Fig. 6.20 Average distribution of microstructural constituents at the interfacial transition zone (ITZ) in normal concrete (Source 3) (Chakradhara Rao 2010)

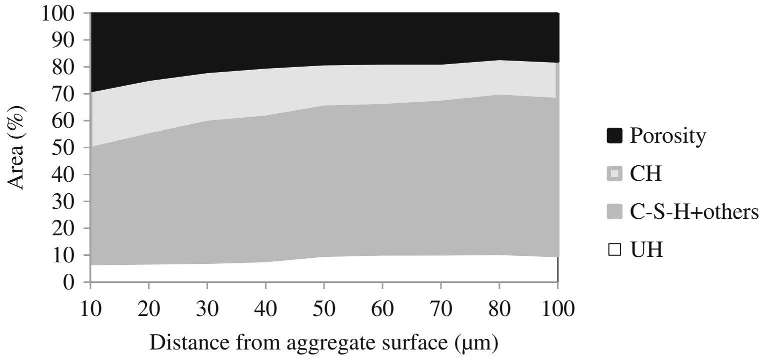


Fig. 6.21 Average distribution of microstructural constituents at the interfacial transition zone (ITZ) in MV-RAC100 (Source 3) (Chakradhara Rao 2010)

attributed to the adherence of old cement mortar to RCA in RAC. Due to high absorption capacity of old cement mortar which is adhered to RCA, the net available water for hydration of new cement grains in this zone is less when compared to normal concrete. The percentage area of UH in the first 10–40 µm zone is around 8.15–8.85% in RAC made with RCA obtained from all the three Sources of RCA against a value of 5.25–5.94% in corresponding normal concrete. With respect to the gradient of anhydrous cement, the effective width of ITZ may be approximately 40–50 µm. Scrievener and Pratt (1996) ascertained from the gradient of anhydrous cement of a one year cured concrete the effective width of interfacial transition zone is at least 50 µm confirming the long-range effect of the aggregate "wall" on the packing of cement grains. Diamond and Huang (2001) found that the effective width of ITZ was 50 µm or more.

Calcium hydroxide (CH) and calcium silicate hydrates (C–S–H) are the major hydration compounds of the cement. The C–S–H directly gets deposited around the cement grains and calcium hydroxide directly gets deposited in pores. The gradients of the percentage areas of C–S–H and CH in normal concrete and recycled aggregate concretes are depicted in Figs. 6.17, 6.18, 6.19, 6.20, and 6.21. It is observed from the figures that, the C–S–H increases as the distance increased from the aggregate surface to the bulk paste in both normal and recycled aggregate concretes. In contrast, starting from the aggregate surface, the CH progressively decreased as the bulk paste approached. In the first 0–40 µm distances from the aggregate surface, the percentage area of CH in normal concrete is relatively higher than that of RAC made with RCA obtained from all the three Sources. This indicates that there is relatively more deposits of calcium hydroxide in the zone nearby aggregate in case of normal concrete compared to recycled aggregate concretes. Similarly, the figures clearly show that there is a relatively higher deposit of C–S–H in case of normal concrete compared to RAC. These deposits of hydration compounds improve the ITZ density. Therefore, the ITZ of normal concrete is relatively dense than those of RAC. This may be mainly due to the presence of old mortar in RCA, which consumes more water during the initial period of mixing leading to less water for hydration at the ITZ in case of RAC. Beyond 20 µm distance from the aggregate surface to the bulk paste, the percentage area of C–S–H is almost identical in both normal concretes and recycled aggregate concretes made with all the three Sources of RCA. Similar results are reported in the literature for 3-day old and 100-day old well-mixed concretes made with dolomite aggregate, in which the authors found that the area percentage of C–S–H was almost identical throughout the concrete (Diamond and Huang 2001). It indicate that within the aureole the quantity of C–S–H formed from the limited content of cement locally available is supplemented by C–S–H derived from the cement rich areas outside of the aureole.

As there is a large difference between the sizes of aggregate particles and cement grains, each aggregate particle acts as a mini "wall" which interrupts the cement grains packing, resulting in the wall effect (Scrivener et al. 2004). As a result of this, the preponderance of small size cement grains at near the aggregate surface has a significantly higher porosity, whereas larger particles of cement grains being further out. In other words, due to this wall effect near the aggregate surface (around

15 μm), less cement grains are present in the fresh state. This is equivalent to a higher w/c ratio. In a typical concrete, some 20–30% of the cement paste lies within 15 μm of aggregate. Therefore, a higher w/c ratio in this zone means that the w/c ratio of the bulk paste is reduced. For a concrete with an overall w/c ratio of 0.4, the w/c ratio of the bulk paste is only around 0.35 (Scrivener et al. 2004). The distribution of the porosity in both normal concrete and recycled aggregate concretes made with recycled coarse aggregates obtained from all the three Sources is presented in Figs. 6.17, 6.18, 6.19, 6.20, and 6.21. It indicates that the percentage area of porosity decreased progressively with the increased distance from the aggregate surface to the bulk paste in all the concretes. In addition, it is observed that a relatively high-percentage area of porosity lies within the first 20 μm distance from the aggregate surface compared to the bulk paste region in both normal and recycled aggregate concretes. The percentage area of porosity in the first 10 μm zone from aggregate in RAC made with RCA obtained from the Sources 1 and 2 is 26.34 and 26.74, respectively, against 21.63% in the corresponding normal concrete. Similarly, the percentage area of porosity in RAC made with Source 3 RCA is 28.69 compared to a value of 22.59 in the corresponding normal concrete. Scrivener et al. (2004) have reported similar results for normal concrete in the literature, in which the authors ascertained that the percentage volume of porosity adjacent to the interface was 40% more than that in the bulk at the time of mixing. After one day, the porosity was reduced to only 10–20% and the slope is less steep. In addition, the authors ascertained that the area percentage of porosity at the ITZ and in bulk paste reduced almost equally at larger period of curing and the results are reported in Fig. 6.22. These results indicate that the ITZ in RAC is less dense and more porous than those of the corresponding normal concretes. In recycled aggregate concrete, though there may be some additional hydration compounds developed due to the reactions between old cement mortar adhered to RCA and new cement matrix, still the percentage area of porosity is higher.

As the recycled aggregates absorb more water, the net water available for hydration of cement is reduced at the initial stages of mixing, which leads to higher

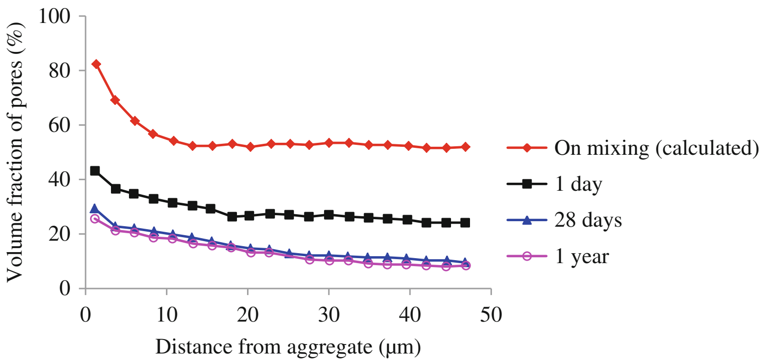


Fig. 6.22 Average porosity in ITZ at various ages (Scrivener et al. 2004)

porosity. The gradients of the percentage area of porosity of RAC made with all the Sources of RCA are relatively steeper than those of corresponding normal concretes after 20 µm distance from the aggregate. This indicates that the effective width of ITZ is more in case of RAC than normal concrete. As discussed earlier, the total aggregate–cement ratio is less in case of RAC mixes compared to normal concrete mixes due to lower specific gravity of RCA, these further allows the rearrangement of cement grains over greater distances resulting in wider interfacial zones. Low water–cement ratios and high aggregate–cement ratios give dense packing of cement grains at the aggregate and minimize the width of interfacial transition zone (Scrivener and Pratt 1996).

6.7.3 *Effect of Aggregate*

Gradet and Ollivier ascertained that the mineralogy of the aggregate influences the degree of orientation of calcium hydroxide (CH) in the interfacial transition zone (Scrivener and Pratt 1996). Whereas, Crumbie (1994) found that the influence of type of aggregate on microstructural gradients analyzed by image analysis was relatively less in real concretes. The EDS analysis of both natural aggregate and recycled coarse aggregates obtained from all the three Sources is presented in Figs. 6.23, 6.24, 6.25, and 6.26, respectively.

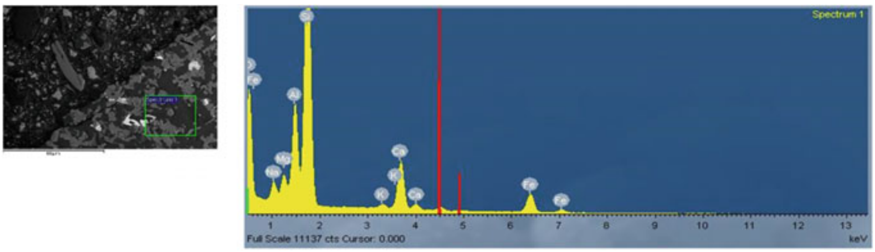


Fig. 6.23 BSE image with EDS analysis of natural coarse aggregate (Chakradhara Rao 2010)

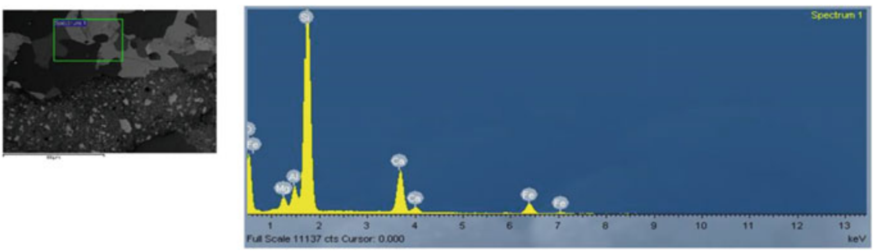


Fig. 6.24 BSE image with EDS analysis of recycled coarse aggregate obtained from Source 1 (Chakradhara Rao 2010)

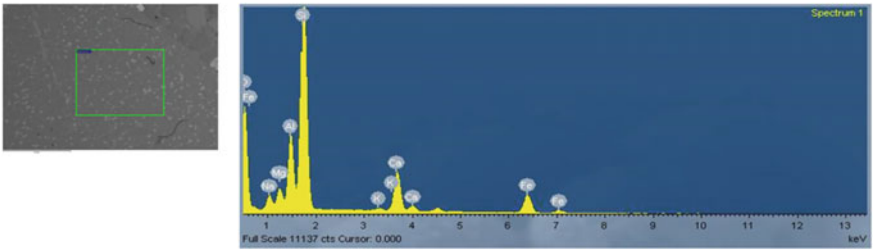


Fig. 6.25 BSE image with EDS analysis of recycled coarse aggregate obtained from Source 2 (Chakradhara Rao 2010)

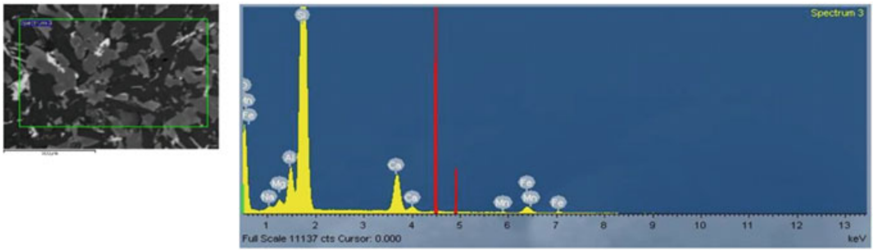


Fig. 6.26 BSE image with EDS analysis of recycled coarse aggregate obtained from Source 3 (Chakradhara Rao 2010)

It is observed that all the aggregates belong to the group of feldspar mineral with the predominant element Si and other elements of Al, Ca, Na, and Fe. Even though the basic mineralogy of both natural and recycled aggregates is similar, the surface texture of recycled aggregate is rough and porous due to the adherence of old mortar which may affect the width of ITZ. The rougher surface attracts the more number of smaller size cement particles, and there may be some additional hydration products due to the reactions between old and new cement mortars which may further improve the ITZ. However, in the present study, the gradients of ITZ indicate that this effect is not significant, as the porosity is higher in case of RAC than normal concrete.

6.8 Compressive Strength–Porosity Relationship

The compressive strength of both normal concrete and recycled aggregate concrete made with RCA obtained from all the three different demolished structures of different sources are discussed in detial in Chap. 4. The porosity measured in the ITZ and bulk paste of various concretes using image processing techniques are discussed in the previous sections. For better understanding, the compressive

Table 6.3 Compressive strength and porosity of both normal concrete and recycled aggregate concretes (Chakradhara Rao 2010)

Mix designation	Compressive strength (f_{ck}) in MPa	Fraction of porosity
M-RAC0	43.08	0.2028
MM-RAC100	40.08	0.2102
MK-RAC100	45.5	0.21
M-RAC0	55.25	0.1679
MV-RAC100	49.45	0.1522

strength at 28 days and porosity of both normal concrete and recycled aggregate concrete made with all the three Sources of RCA are presented in Table 6.3.

Strength of concrete is influenced by the volume of all voids in concrete (Neville 2006). A Power's model which relates the strength and porosity of bulk cement paste in normal concrete presented in Eq. 6.1 is used in the present study.

$$f_{ck} = f_{c,0}(1 - p)^n \tag{6.1}$$

where p is the porosity

f_{ck} is the strength of concrete with porosity p
$f_{c,o}$ is the strength at zero porosity and
n is a coefficient

The relationship between compressive strength (f_{ck}) and porosity (p) is obtained by plotting the compressive strength (f_{ck}) versus (1–p) as shown in Fig. 6.27. A power best fit is assumed and the obtained best-fit equation is expressed as

$$f_{ck} = 98.79 \times (1 - p)^{3.598} \qquad (\mathrm{R} = 0.98) \tag{6.2}$$

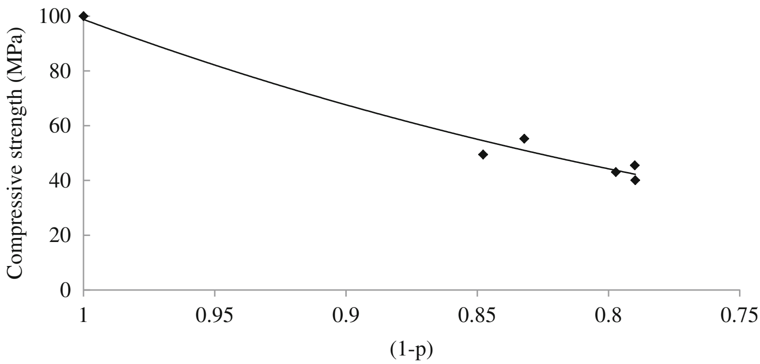


Fig. 6.27 Relationship between compressive strength and porosity (Chakradhara Rao 2010)

This shows that the Power's model given for normal concrete very well matches with the experimental results obtained for both normal concrete and recycled aggregate concretes made with all the Sources of RCA. The intrinsic strength of concrete (corresponding to zero porosity) is 98.79 which is close to the theoretical value of 100.

6.9 Microhardness of ITZ

The microhardness testing has been reported as a means of characterizing the properties of ITZ relative to the bulk and as a means of determining the width of ITZ. The usefulness of this method is its ability to determine the response to load of a volume element that is considerably smaller than the ITZ (Igarashi et al. 1996). The Vickers microhardness test is conducted on the samples on which the BSE images are acquired using scanning electron microscopy. It was reported that in recycled aggregate concrete, there are two ITZs; one is the old ITZ, i.e., ITZ, between original aggregate and the adhesive mortar, i.e., old cement mortar, and the other is between old cement mortar and new cement mortar (new ITZ) (Otsuki et al. 2003). As the new ITZ, i.e., ITZ, between the old cement mortar and new cement mortar could not be identified in the images, the microhardness test is conducted only at old ITZ, i.e., ITZ, between original aggregate and old cement mortar in the present study in case of RAC. The Vickers microhardness is measured at 14 points within the distance of 205 µm from the aggregate surface. The measurements are taken randomly at a constant load of 10 gf with 10 s time. The test is conducted on three samples for each mix, and the average results are reported in Table 6.4. Here the microhardness symbol HV 0.01 means the test load is of 10 gf. The microhardness indentations on various samples are shown in Fig. 6.28.

The distribution of Vickers microhardness across the ITZ for both normal concrete and recycled aggregate concrete made with all the three Sources of RCA is presented in Fig. 6.29.

From the results, it is ascertained that the Vickers microhardness increased with the increase in distance from the aggregate surface in both normal concrete and recycled aggregate concrete made with all the three Sources of RCA. In addition, it is observed that the Vickers microhardness up to 40–55 µm distance from the aggregate surface is not varying much in both normal concrete and recycled aggregate concrete. However, beyond these distances, the Vickers microhardness is progressively increasing up to around 150 µm distances, and thereafter, almost it is constant. Lower value of Vickers microhardness indicates the presence of more microcracks and micropores which is an indication of higher porosity. This variation defines the width of ITZ. The width of ITZ in both normal concrete and recycled aggregate concretes is within the range of 40–55 µm. Similar results are obtained in SEM examinations; the porosity near the aggregate surface (10–50 µm) is much higher than that of bulk paste in all the concretes. The variation in Vickers microhardness of RAC is almost the same irrespective of the Source of RCA.

Table 6.4 Average Vickers microhardness of both normal concrete and recycled aggregate concrete made with RCA obtained from all the three Sources (Chakradhara Rao 2010)

Distance from aggregate surface (μm)	Mix designation			
	M-RAC0	MM-RAC100	MK-RAC100	MV-RAC100
	Vickers microhardness (HV 0.01)			
10	64	45.67	43.00	40.50
25	70	47.00	42.00	43.50
40	72	47.50	49.67	43.00
55	70	58.67	43.00	45.00
70	76	53.50	59.00	50.00
85	85	72.00	52.00	53.33
100	75	62.00	83.50	62.00
115	85	63.00	64.00	53.50
130	76	64.00	74.00	60.50
145	100	62.50	69.00	63.00
160	105	57.50	71.00	67.00
175	109	60.00	70.00	64.00
190	112	62.00	70.00	62.75
205	113	61.00	74.00	63.00

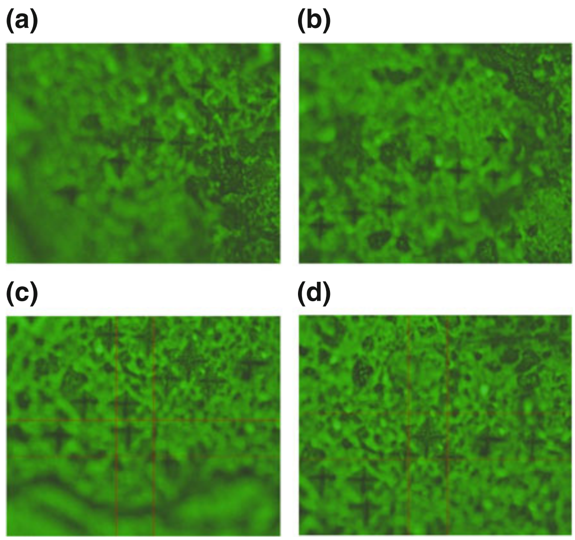


Fig. 6.28 Microhardness indentations of ITZ (**a**–**b**) normal concrete and (**c**–**d**) MM-RAC100 (Chakradhara Rao 2010)

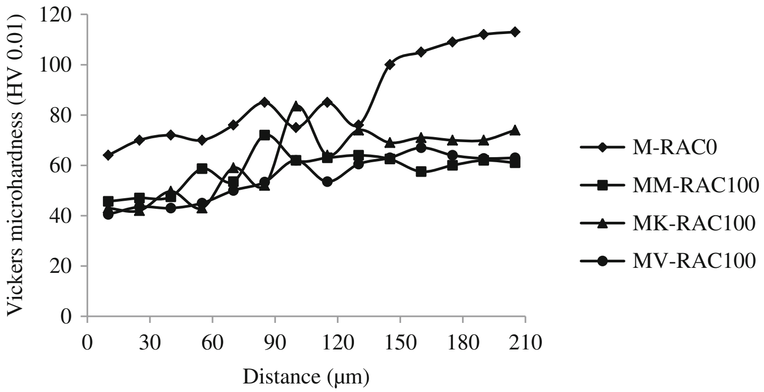


Fig. 6.29 Gradients of Vickers microhardness of both normal concrete and RAC made with all the three Sources of RCA (Chakradhara Rao 2010)

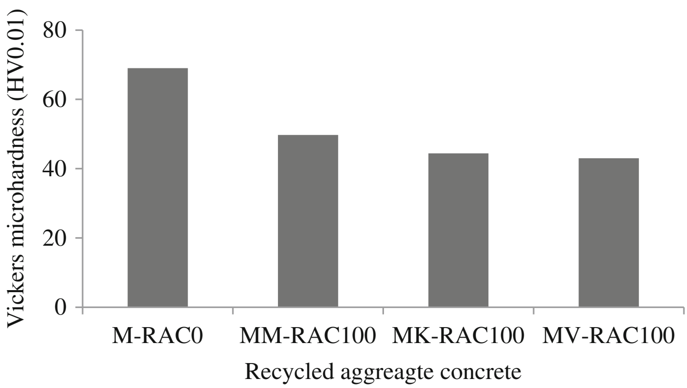


Fig. 6.30 Microhardness of ITZ of both normal concrete and recycled aggregate concretes (Chakradhara Rao 2010)

However, the Vickers microhardness of normal concrete is higher than that of recycled aggregate concretes made with all the Sources of RCA across the ITZ. This may be due to the presence of old mortar adhered to RCA in RAC, which absorbs more water at the initial stages of mixing and leads to the lesser hydration compounds and more porosity.

The ITZ Vickers microhardness is defined as the average value of Vickers microhardness measured within 10–50 μm distance from the aggregate surface (Otsuki et al. 2003). As there is no significant change observed within 10–55 μm distance from the aggregate surface, the width of ITZ is defined as the average Vickers microhardness within 10–55 μm distances from the aggregate surface in the present study. Here microhardness symbol HV 0.01 means the test load is of 10 gf. The Vickers microhardness of ITZ in both normal concrete and recycled aggregate concrete made with RCA obtained from all the three Sources of RCA is presented in Fig. 6.30. It can be seen that the Vickers microhardness of RAC made with RCA is less than that of normal concrete. The Vickers microhardness of RAC made with RCA obtained from the Sources 1, 2, and 3 is 49.7, 44.4, and 43, respectively, against a value of 69 in normal concrete. Lower values of Vickers microhardness indicate the higher values of porosity and softness of cement paste.

6.10 Compressive Strength–ITZ Microhardness Relationship

The characteristics of ITZ play a major role on the mechanical properties of concrete. The Vickers microhardness is also an indication of the quality of ITZ. Hence, the compressive strength and ITZ Vickers microhardness are related in the plot shown in Fig. 6.31.

The results reported by different researchers for different w/c ratios are also presented. It can be seen that the compressive strength decreases with the decrease in ITZ Vickers microhardness. Lower microhardness indicates higher porosity and hence lowers the strength. Similar trends have been reported in the literature (Otsuki et al. 2003; Ryu 2002).

Ryu (2002) had conducted the experiments on the strength of old and new ITZ in RAC at w/c ratio of 0.55 and 0.25. It was observed that at higher w/c ratio (0.55), the Vickers microhardness of old ITZ is higher than the Vickers microhardness of new ITZ, and at lower w/c ratio (0.25), the Vickers microhardness of new ITZ is higher than the old ITZ. That is, at lower w/c ratio, the old ITZ governs the failure. The lower value of microhardness indicates the higher values of porosity and softness of cement paste. That is, the old ITZ is loose and more porous at lower w/c ratio. Therefore, at lower w/c ratio, the strength of RAC depends on the quality of RCA, as the strength of old ITZ weaker than the strength of new ITZ and at higher w/c ratio, the strength does not depend on the quality of RCA, as the old ITZ is stronger than the new ITZ. Therefore, at higher w/c ratio, the strength of RAC was same as that of normal concrete.

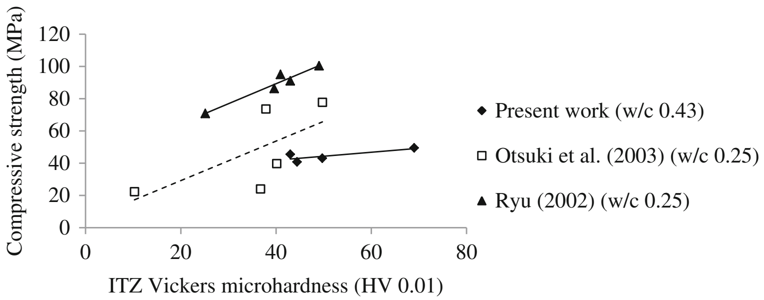


Fig. 6.31 Relationship between compressive strength and ITZ Vickers microhardness (Chakradhara Rao 2010)

The overall performance of the concrete may depend on the quality of RCA i.e. the strength of original concrete from which the recycled aggregate derived and the crushing technology. The concrete produced with recycled coarse aggregate may not differ much with normal concrete with respect to the mechanical characteristics. However, the RCA may influence the durability and long-term properties of concrete such as creep and shrinkage, as more microcracks and high porosity exist in RAC due to the adherence of porous cement paste.

6.11 Influence of Binder on ITZ

Li et al. (2001) studied the influence of different binders, namely pure cement paste binder (C-binder), expansive binder (E-binder), and polymer modified and fly-ash binder (F-binder) on interfacial transition zone (ITZ) between new and old concretes. The microstructure of the transition zone between old concrete (three months) and a new concrete of same mix proportion with the above binders was examined by using H-1030 SEM with EDS analyzer. The mix proportion of the binders is shown in Table 6.5.

The interfacial transition zone was highly dense and uniform and no ettringite or calcium hydroxide was found in the transition zone (Fig. 6.32a) when E-binder was used. This was due to the formation of additional calcium silicate hydrate (C–S–H), when the calcium hydroxide produced by the hydration of cement reacts with the amorphous silica of fly ash in F-binder, fills the pores and the pozzolanic reaction decreases the calcium hydroxide content. Further, a large number of globular particles of fly ash fill the weak spaces of ITZ, which make it highly dense and uniform. In E-binder, the hydration products of U-type expansive agent are the mainly ettringite crystals, which creates more number of microcracks in the

Table 6.5 Mix proportion of binders (Li et al. 2001)

Binder type	Cement	Water	Sand	Fly ash	U-type expansive agent	Supe plasticiser dosage[a]
C-binder	1	0.4	–	–	–	0.5
F-binder	0.75	0.4	1	0.25	–	1.5
E-binder	0.9	0.4	–	–	0.1	0.5
Polymer (YJ-302)	One of the main components of the polymer-modified binder (YJ-302) was emulsified epoxy resin					

[a]Dosage given as percentage of total binder content by mass

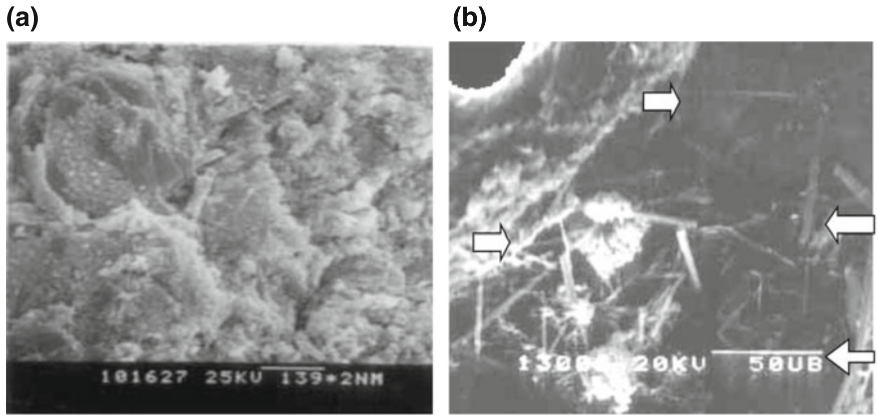


Fig. 6.32 Transition zone with (**a**) F-binder and (**b**) E-binder (Li et al. 2001)

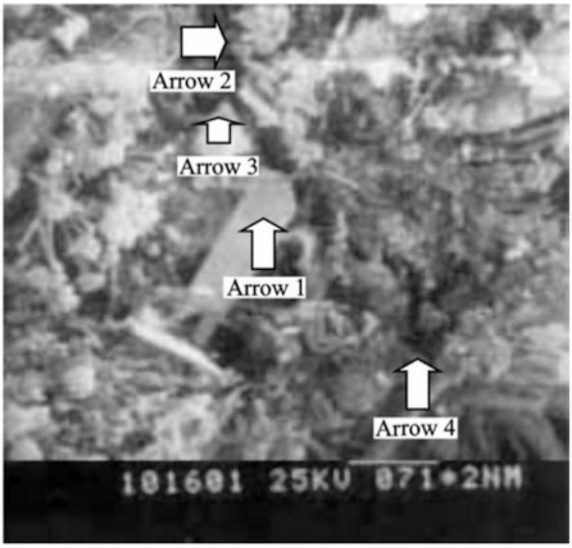


Fig. 6.33 Transition zone with C-binder (Li et al. 2001)

interfacial transition zone (Fig. 6.32b). A large number of microcracks and crystals in the ITZ and interface debond between new to old concrete were found due to drying shrinkage of cement in C-binder (Fig. 6.33). When polymer binder was used near the old concrete surface, only the polymer film was found (Fig. 6.34).

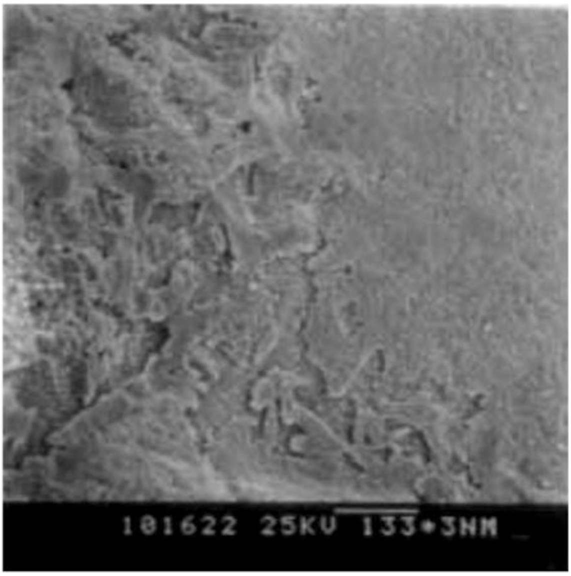


Fig. 6.34 Cross section of interface with polymer binder (Li et al. 2001)

6.12 Influence of the Water–Binder Ratio, Quality and Quantity of Adhesion Mortar

The influence of water–cement ratio (0.25, 0.40, 0.55, 0.70), strength of adhesive mortars, and quantity of adhesive mortars of recycled coarse aggregate (Table 6.6) on the interfacial transition zones of recycled aggregate concrete and natural aggregate concrete are presented in Figs. 6.35, 6.36, 6.37, 6.38, 6.39 and 6.40 (Otsuki et al. 2003). The microhardness was considered as the criterion for characterizing the ITZ. It can be seen from Figs. 6.35 and 6.36 that the Vickers microhardness increased with the decrease in water–binder ratio in both natural

Table 6.6 Details of natural and recycled coarse aggregates (Otsuki et al. 2003)

Symbol	Aggregate type	Adhesive mortar		Specific gravity	Water absorption (%)	Fineness Modulus
		Strength (MPa)	Quantity (%)			
VC	Normal aggregate	–	–	2.66	0.69	6.73
A2	Recycled aggregate	68.5	39.7	2.47	3.58	6.50
B1		51.7	20.8	2.54	2.68	6.45
B2		51.7	44.6	2.44	4.50	6.69
B3		51.7	50.8	2.41	5.13	6.62
C2		32.3	35.0	2.45	4.36	6.46
Original aggregate in recycled aggregate				2.64	0.84	6.68

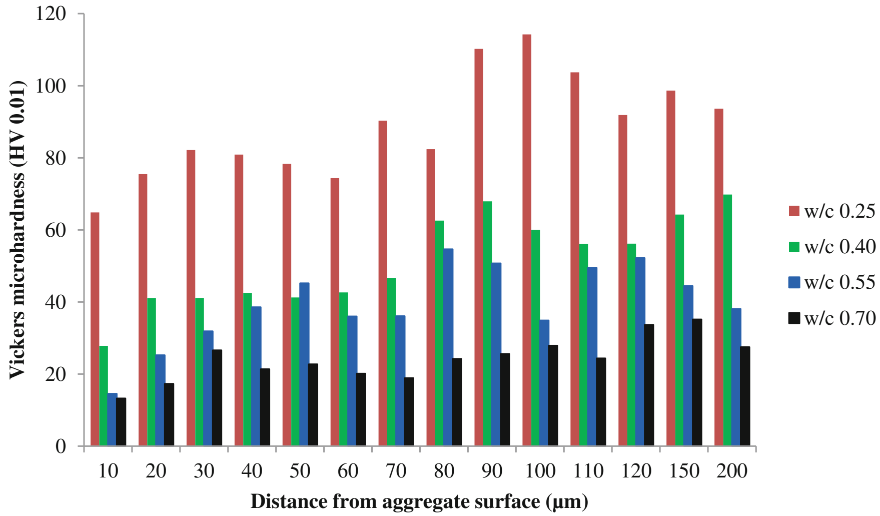


Fig. 6.35 Vickers microhardness distribution at new ITZ of recycled aggregate concrete B2 (Otsuki et al. 2003)

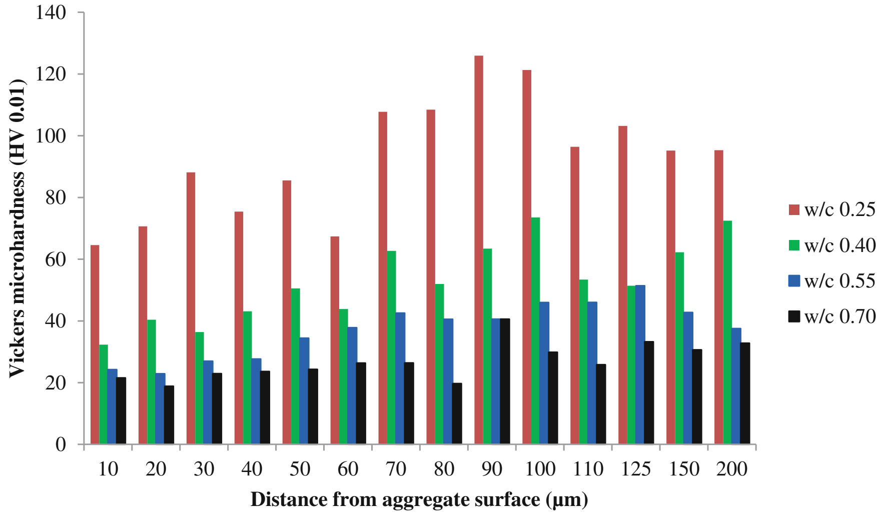


Fig. 6.36 Vickers microhardness distribution at new ITZ of normal aggregate concrete VC (Otsuki et al. 2003)

aggregate concrete and recycled aggregate concrete. Figure 6.37 shows that the Vickers microhardness of old ITZ increased with the increased quality of adhesive mortar, whereas the Vickers microhardness of old ITZ does not change with the quantities of mortar (Fig. 6.38). That means, the characteristics of ITZ depend on

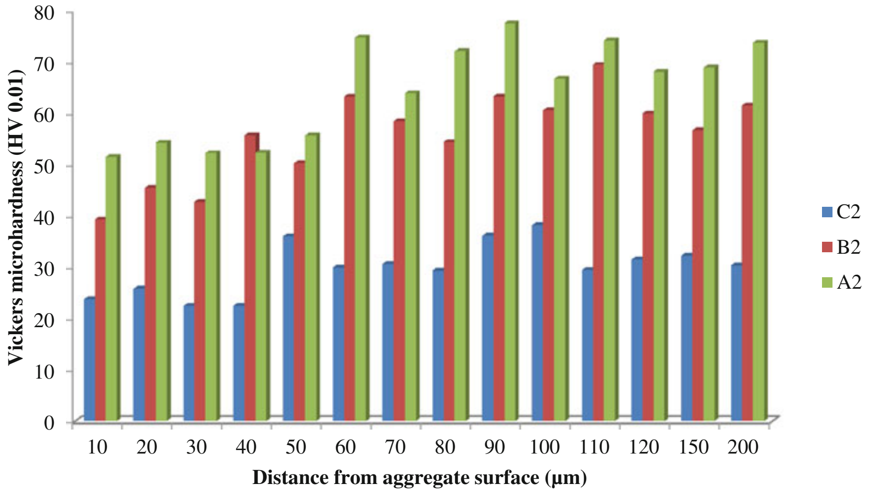


Fig. 6.37 Vickers microhardness distribution at old ITZ of recycled aggregate with different adhesive mortar strengths (*Source* Otsuki et al. 2003)

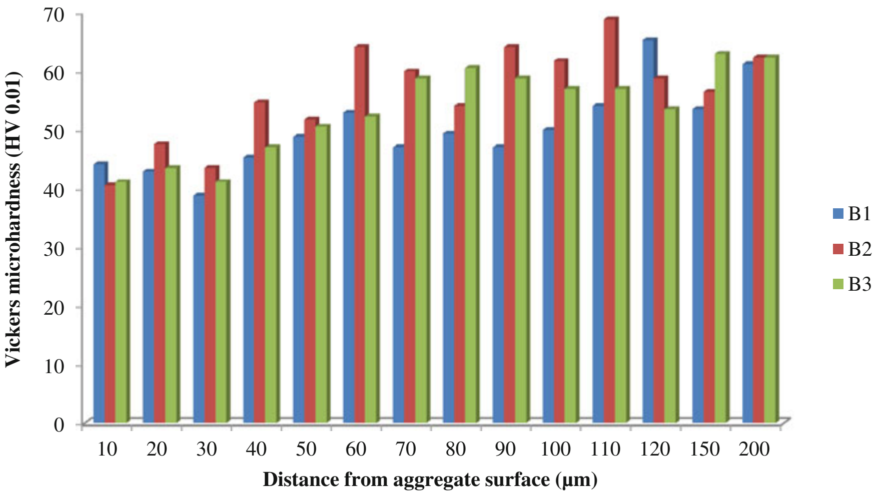


Fig. 6.38 Vickers microhardness distribution at old ITZ of recycled aggregate with different adhesive mortar quantities (*Source* Otsuki et al. 2003)

the quality of mortar surrounding the aggregate and not on the adhered mortar quantity. Figure 6.39 and 6.40 show the Vickers microhardness of old and new ITZ for 0.25 and 0.55 water–binder ratios. It can be seen that at lower water ratio, the new ITZ is stronger than old ITZ, whereas at higher water–binder ratio, the old ITZ

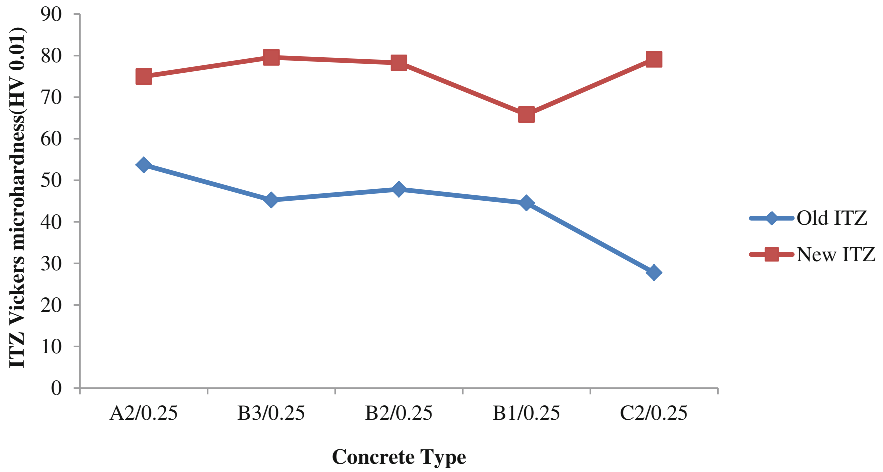


Fig. 6.39 New and old ITZs Vickers Microhardness (w/b 0.25) (*Source* Otsuki et al. 2003)

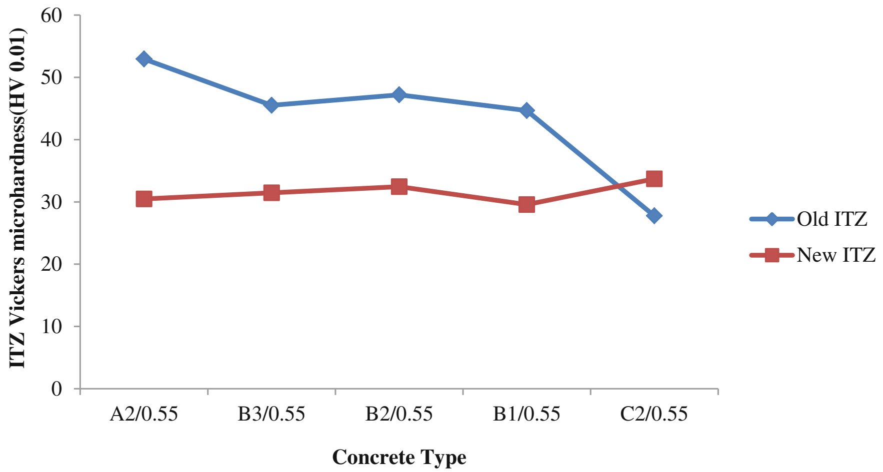


Fig. 6.40 New and old ITZs Vickers microhardness (W/C 0.55) (*Source*: Otsuki et al. 2003)

becomes stronger except in case RAC with C2 type (having lowest strength of adhesive mortar among all types of RA). At lower w/c ratio, the strength of RAC depends on the quality of RCA, and as the strength of old ITZ weaker than the new ITZ and at higher w/c ratio, the strength does not depend on the quality of RCA, as the old ITZ stronger than the new ITZ. Therefore, at higher w/c ratios, the strength of RAC was same as that of normal concrete.

6.13 Influence of Strength of Source Concrete

Influence of natural aggregate and RCA derived from the high performance concrete (HPC) and normal strength concrete (NC) on microstructure of RAC were examined by Poon et al. (2004) are presented in Figs. 6.41, 6.42, and 6.43. It can be seen from Fig. 6.41 that the granite aggregate and cement interface was relatively loose and the interfacial transition zone thickness varied along the aggregate surface. The interfacial transition zone between RCA obtained from normal strength concrete and cement matrix is shown in Fig. 6.42. It was observed that the width of ITZ was approximately 30–60 μm which consists mainly of loose particles (Fig. a). Further, at higher magnification, it can be seen that the ITZ was porous with high porosity (Fig. b). A relatively dense ITZ in RAC made with RCA obtained from HPC was observed (Fig. 6.43). The poor ITZ in RAC with normal strength concrete

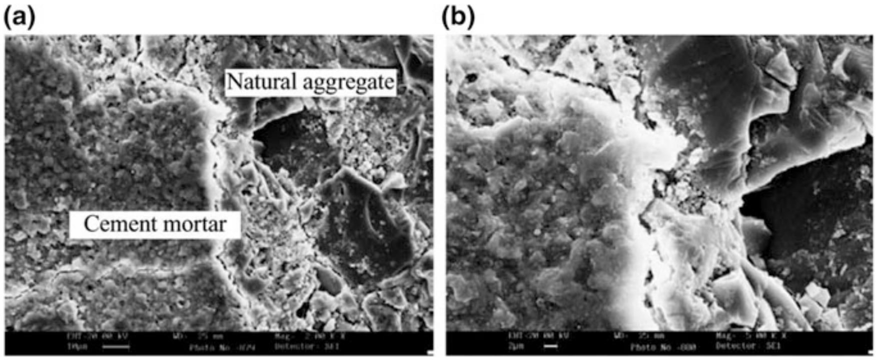


Fig. 6.41 Microstructure of concrete prepared with natural crushed granite (Poon et al. 2004)

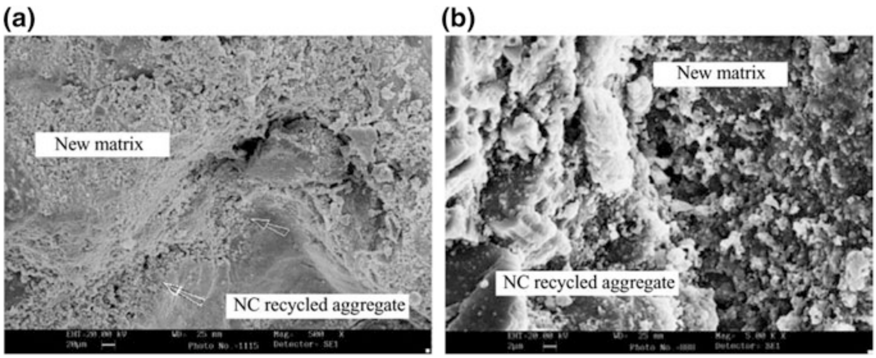


Fig. 6.42 Microstructure of concrete prepared with recycled NC (Poon et al. 2004)

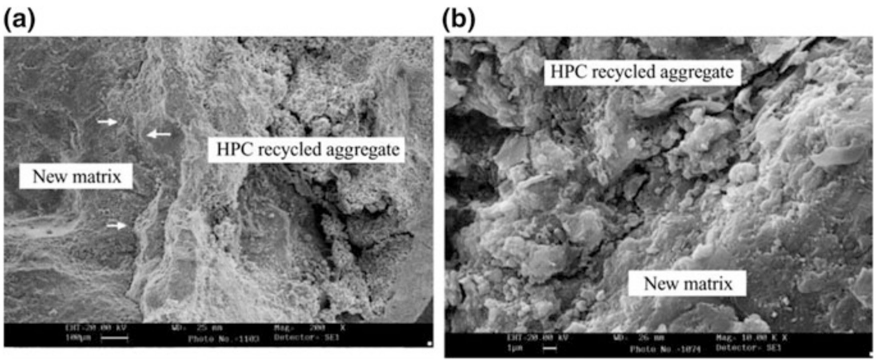


Fig. 6.43 Microstructure of concrete prepared with recycled HPC (Poon et al. 2004)

aggregate attributes to the higher porosity and absorption capacity of recycled aggregate. The ITZ in recycled aggregate concrete is considered to be an important factor in governing the compressive strength development in RAC.

6.14 Influence of Treatment of RCA on ITZ

The quality of ITZ between recycled aggregate and the new cement mortar can be improved by impregnating the recycled aggregate in silica fume solution (Katz 2004). By impregnating the recycled aggregate in silica fume solution, the cracked and loose layer of the recycled aggregate could be filled with silica fume particles. During the hardening of concrete, due to this filler effect, the ITZ improves. In addition, the pozzolanic reaction between the portlandite and the silica fume strengthen the feeble structure of the recycled aggregate to form an improved zone, which extends from the natural aggregate through the residues of the old cement paste into the new cement matrix.

6.15 Summary

The detailed experimental results of the characteristics of interfacial transition zone in both normal concrete and recycled aggregate concrete made with 100% recycled coarse aggregate obtained from three different Sources have been presented. The detailed specimen preparation for microscopic examination, image acquisition, and image analysis techniques along with the brief description of scanning electron microscope and Vickers microhardness instruments is discussed. The important features of ITZ, viz: anhydrous cement, hydration compounds, and porosity are discussed. The strength—porosity and microhardness—porosity relationships are discussed. Further, the effect of water–cement ratio, strength of adhesive mortars and quantity of adhesive mortars, strength of source concrete and impregnation of RCA in silica fume on the interfacial transition zones were discussed. Based on these discussions, the following key points are noted down.

- The ITZ in RAC is less dense than the concrete with natural aggregates. Possibly, the adhered mortar to the aggregates in RCA consumes more water in the initial stages, thereby making the surrounding more porous.
- The gradients of anhydrous cement and hydration compounds indicate that the width of ITZ is around 40 μm in RAC with different percentages of RCA investigated. However, the porosity gradients indicate that the width of ITZ in RAC is more than the width of ITZ in normal concrete. In addition to the high absorption capacity of RCA stated above, the total aggregate–cement ratio also influences the width of ITZ.
- The Vickers microhardness of ITZ in RAC is lower than that of normal concrete. Lower values of microhardness indicate the presence of more microcracks and micropores, which are indication of higher porosity. Possibly, this makes the concrete less dense and lower stiffness.
- The compressive strength of concrete decreases with the decrease in microhardness and increase in porosity of ITZ.
- The type of binder influences the characteristics of ITZ. Among the three binders, namely pure cement paste binder (C-binder), expansive binder (E-binder), and polymer modified and fly-ash binder (F-binder), the ITZ was highly dense and uniform when F-binder was used.
- The characteristic of ITZ depends on the quality of mortar surrounding the aggregate and not on the adhered mortar quantity.
- At lower w/c ratio, the strength of RAC depends on the quality of RCA, as the strength of old ITZ weaker than the new ITZ, and at higher w/c ratio, the strength does not depend on the quality of RCA, as the old ITZ stronger than the new ITZ.
- The strength of source concrete influences the ITZ. ITZ was relatively dense in RAC made with RCA obtained from HPC than that made with RCA from normal strength concrete.

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Chapter 7
Structural Behavior of RAC

7.1 Introduction

Mechanical properties, long-term, durability aspects, and microstructure of recycled aggregate concrete were discussed in earlier chapters. In practice, there are many incidents in which the structures undergo impact loading, such as during an explosion, transportation structures subjected to vehicle crash impact, impact of ice load on marine and offshore structures, accidental falling loads on structural elements, protective structures under projectile or aircraft impact. The behavior of concrete beams subjected to impact load is different compared to the behavior under quasi-static loading. Due to short duration of loading, the strain rate of material is significantly higher than that under quasi-static loading conditions. Also, the failure behavior may be different from those under quasi-static loading conditions. To understand the behavior of RAC beams under impact load, a drop weight impact test was conducted by the authors on RAC beams. The behavior of beams made of recycled aggregate concrete prepared with different amount of recycled coarse aggregate under low-velocity impact is discussed in detail in this chapter. Further, the investigations made by different researchers on the flexural, shear, and axial behavior of reinforced concrete beams made with recycled aggregate are highlighted in this chapter.

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7.2 Impact Behavior

Many researchers have studied the impact behavior of plain, composite, and reinforced concrete beams subjected to low-velocity impact with natural coarse aggregates. Bentur et al. (1986) studied the behavior of plain and conventionally reinforced concrete beams under low-velocity drop weight impact load. The authors reported that in both the specimens, the peak load occurred within 1 ms after contact. In addition, it was reported that the energy estimated from the instrumented tup load did not agree with the calculated total energy. The authors also developed a method to calculate the bending load for failure of specimens under single impact. Banthia et al. (1987) investigated the impact behavior of normal strength, high-strength, and fiber-reinforced concrete beams subjected to low-velocity impact. It was reported that both normal and high-strength concretes are strain rate sensitive and the prediction of its behavior under impact load is not possible on the basis of static testing. The authors also reported that high-strength concrete had higher impact strength and more brittle than normal concrete. In addition, it was reported that the fiber-reinforced concrete is better than plain concrete in dynamic conditions due to its ductility and increased impact resistance. Wang et al. (1996) examined the influence of different types and volumes of fiber on the impact behavior of fiber-reinforced concrete (FRC) beams subjected to repeated drop of impacts. It was reported that the fracture energy of FRC with 0.5% steel fibers was more than that of polypropylene fibers. In addition, it was reported that the two different failure mechanisms exist in FRC with hooked steel fibers: One is a fiber-breaking failure mechanism which occurred when the fiber volume is below a critical value, and the second is a fiber “pull-out” mechanism which occurred when the fiber volume is above the critical value. Tang and Saadatmanesh (2003) investigated the behavior of conventionally reinforced beams strengthened with different types of composite laminates subjected to impact loads. From the experimental results, the authors concluded that the impact resistance of concrete beams significantly improved and the deflection and crack width reduced with the composite laminates. In addition, the authors concluded that the gain in strength depends on the type, thickness, weight, and material properties of the composite laminate. May et al. (2006) studied the influence of shape of impactor and type of interface on the behavior of reinforced concrete beams subjected to high-mass low-velocity drop weight impact loads. Two types of steel impactors were used: (1) spherical surface with a radius of 125 mm and a profile diameter of 90 mm and (2) a flat surface with a profile diameter of 100 mm. Similarly, two types of contact interfaces are used: One is a 12 mm thick plywood pad placed between the impactor and the beam at the impact zone, and the second is a direct contact of impactor with the beam. The authors also carried out a numerical study for the same. The authors have found good agreement between experimental and numerical results.

Rao et al. (2011) have conducted a detailed investigation on behavior of recycled aggregate concrete under drop weight impact load. In this study Ordinary Portland

Cement (OPC) of 43 Grade conforming to Bureau of Indian Standard Specifications (BIS) (1959, 1970, 1982, 1989, 1999) (IS: 8112-1989) with a specific gravity of 3.14 is used in this study. The locally available sand conforming to grading Zone II (IS: 383-1970) is used in both normal and recycled aggregate concretes. The natural coarse aggregates obtained from the locally available quarries with maximum size of 20 mm and satisfying the grading requirements of BIS (IS: 383-1970) are used in both normal and recycled aggregate concretes. The recycled coarse aggregate was obtained from old demolished RCC culvert near Kharagpur. Four concrete mixes, namely M-RAC0, M-RAC25, M-RAC50, and M-RAC100, are prepared using different proportions of recycled coarse aggregates and natural coarse aggregates. In the term M-RAC0, the letter M stands for the Mix, RAC represents the recycled aggregate concrete, and the number represents the percentage of recycled coarse aggregate. All concrete mixes are designed for M25 grade of concrete in accordance with the Bureau of Indian Standards (BIS) (IS: 10262-1982). In all the mixes, the free water–cement (w/c) ratio was kept constant at 0.43 and slump was maintained in the range of 50–60 mm by adding Sika Viscocrete R-550(l) superplasticizer. The details of the mixture proportioning are presented in Table 7.1.

The mechanical properties of RAC are conducted on standard test specimens of 100 mm cubes and 150 mm diameter × 300 mm height cylinders in accordance with BIS (IS: 516-1959; IS: 5816-1999) and American Society of Testing and Materials (ASTM C 469-02 (2002)), and the test results are presented in Table 7.2. Three recycled aggregate concrete beam specimens each for 0, 25, 50, and 100% recycled coarse aggregates are prepared. A total of 12 beam specimens of size 1.15 × 0.10 × 0.15 m are prepared for drop weight impact load test.

Table 7.1 Details of mixture proportions (kg per cubic meter of concrete) (Rao et al. 2011)

Mix designation	RCA (%)	Cement (kg)	Natural aggregates		RCA (kg)	Superplasticizer[a]
			Fine Aggregate (kg)	CA (kg)		
M-RAC0	0	401	574	1261	0	0.05
M-RAC25	25			930.75	310.25	0.05
M-RAC50	50			602	602	0.175
M-RAC100	100			0	1128	0.225

[a]Percentage by weight of cement

Table 7.2 Mechanical properties of RAC (Rao et al. 2011)

Mix designation	RCA (%)	Compressive strength (MPa)	Indirect tensile strength (MPa)	Modulus of elasticity (MPa)	Density (kg/m^3)
M-RAC0	0	49.45 (28.77[a])	2.67	3.120×10^4	2415.64
M-RAC25	25	45.75 (27.9[a])	2.30	2.675×10^4	2349.45
M-RAC50	50	42.5 (35.33[a])	2.19	2.671×10^4	2257.96
M-RAC100	100	40.8 (31.5[a])	2.05	2.640×10^4	2148.10

[a]Compressive strength at 7 days

7.2.1 Instrumented Drop Hammer Impact Test Setup and Devices

The in-house built drop hammer test setup shown in Fig. 7.1 is used in the present study to investigate the impact behavior of recycled aggregate concrete beams with different amount of recycled coarse aggregates.

The test setup consists of a fiber-reinforced plastic (FRP) guide tube which facilitates the aligned movement of drop weight hammers of different diameters required to be used for a particular test. The FRP guide tube was firmly fixed to the vertical posts, which are rigidly connected to the rigid steel frame, to avoid eccentricity. In the present study, a steel impactor of 50 mm diameter and 5 kg mass is suspended through the FRP guide tube passes over the frictionless pulley to induce the impact on beams. The impactor may be dropped from a maximum height of 1.5 m. To get a point contact between the impactor and the specimen, a spherical steel ball having a diameter of 15 mm was welded to the impactor at the bottom.

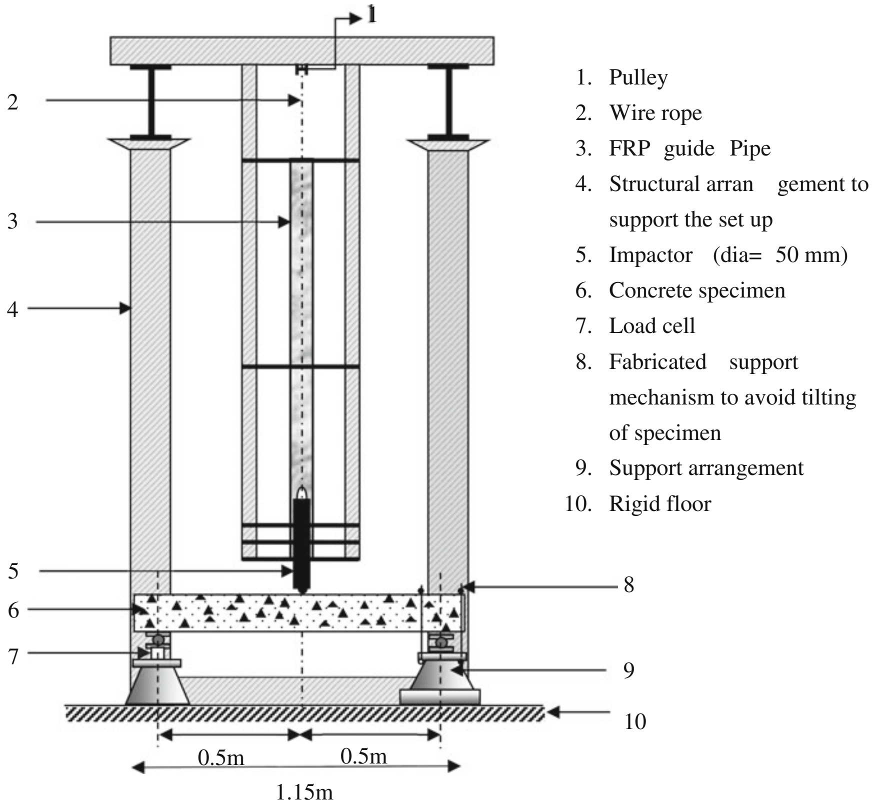

Fig. 7.1 Schematic diagram of drop hammer setup (Rao et al. 2011)

Fig. 7.2 Support arrangement (Rao et al. 2011)

A simply supported arrangement is made as shown in Fig. 7.1 for supporting the beam specimens. To prevent the slippage of the beam specimen after each impact, a bracing arrangement was made with steel angle sections as shown in Fig. 7.2.

Spectrum analyzer and accelerometers Vibration measurement system is used for measuring the acceleration, velocity, and displacement histories at different locations on the bottom surface and along the length of the beam. It is a six-channel pulse fast Fourier transform (FFT) and the constant percentage band (CPB) analyzer. In order to measure the acceleration, velocity, and displacement of the beams, four accelerometers were fixed at the bottom surface of the beams. The positions of the accelerometers (A1 to A4) are shown in Fig. 7.3. Accelerometers are DeltaTron type 4507 with a range of $\pm 700\ \mathrm{ms}^{-2}$ and working on the

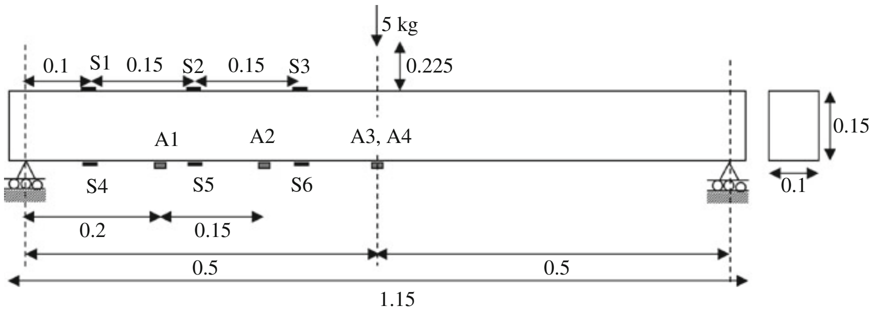

Fig. 7.3 Position of accelerometers and strain gauges on RAC beam (Rao et al. 2011). *Note*: All dimensions are in meters

piezoelectric principle. The accelerometers are connected to the six-channel pulse fast Fourier transform (FFT) analyzer. The acceleration data are captured at interval of 244 μs (Bruel & Kjaer Pulse user manual).

Data acquisition system (DAQ) and strain gauges An eight-channel NI SCXI 1000 chassis with 1520 universal strain module data acquisition system is used to record the strain data (NI SCXI 1520 user manual). The sampling can be recorded simultaneously at a minimum acquisition rate of 10^5 samples per second. BKCT-30 electrical strain gauges with ±350 Ω resistance (gauge length 30 mm and gauge factor 2.00 ± 0.02) are used to measure the strains at different locations along the length of the beams. A total of six strain gauges (S1–S6), three each on both top and bottom surfaces of the beams, were mounted, and their locations are shown in Fig. 7.3.

Load Cell A load cell (200 kg capacity) of cylindrical type is used to measure the support reaction. It has four inbuilt strain gauges. All strain gauges and load cell are connected to a DAQ, an eight-channel NI SCXI 1000 chassis with 1520 universal strain module. The strain data are acquired at a sampling rate of 10^5 records/s. The strain data acquisition is started simultaneously with the release of hammer from the required height.

7.2.2 *Impact Test Results*

The accelerometers and strain gauges were mounted on the completely dried beam specimens. The beams were then positioned on the supports with an effective span of 1.0 m. A weight of mass 50 N (5 kg) was set to drop from a height of 0.225 m above the top of the beam specimen and then released. This gives a tip velocity of 2.101 m/s at the time of impact. The hammer is dropped repeatedly from the same height till the failure occurred. During each impact, the acceleration, displacement, strain, and support reaction histories are recorded using suitable devices and the results are presented herein.

7.2.2.1 Accelerations

As discussed earlier due to the constraint of the acquisition rate in the instrument, the accelerations were acquired at 244 μs interval which is too small. This may lead to the missing of actual peak values of accelerations. However, the authors are basically interested to know the behavior of recycled aggregate concrete in contrast to the normal concrete with respect to accelerations, displacements, etc., under impact load. The variation of acceleration at midspan of the beam with recycled coarse aggregate having different percentages of RCA during the first impact is presented in Fig. 7.4.

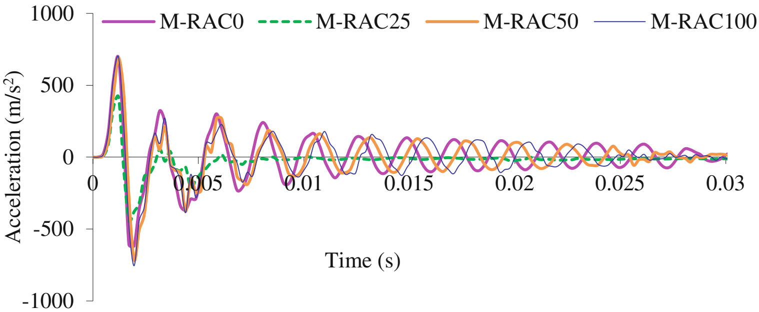

Fig. 7.4 Accelerations at midspan during the first drop of impact (Rao et al. 2011)

It is observed that in the first cycle of wave propagation the magnitude of acceleration is more in case of RAC with higher percentage of recycled coarse aggregate (50 and 100%) when compared to normal concrete. The peak value of acceleration in RAC with 100, 50, and 25% RCA is 756, 715, and 440 m/s^2, respectively, compared to an acceleration of 686 m/s^2 in normal concrete (M-RAC0). This indicates the acceleration depends on the mass (density) of the material: The higher the density, the lower is the acceleration. As discussed in previous section, the density of recycled coarse aggregates is 15% less than that of natural aggregates due to lower density of old mortar adhered to those and hence the density of RAC with 100% RCA is around 9 and 12% lower than the RAC with 25% RCA and normal concrete, respectively. After the first cycle of wave propagation, in the subsequent cycles of wave propagation the peak values of acceleration of RAC with 100% RCA are less than those of RAC with 25 and 50% RCA and normal concrete. This indicates that the frequency of recycled aggregate concrete beams with higher amount of recycled coarse aggregate is more than that of normal concrete. That is, the period of vibration is less in case of RAC.

The acceleration histories of normal and recycled aggregate concrete beams with 100, 50, and 25% recycled coarse aggregates at midspan for repeated drops of same height are presented in Figs. 7.5, 7.6, 7.7, and 7.8, respectively.

It is found that in both normal and recycled aggregate concrete beams the period of vibration decreases with the increase in drop number. This indicates the softening of the beam with the increase in drop number. That is, the stiffness of the beams reduces with the increase in drop number. It is also observed that the first peak values of accelerations increase with the increase in drop numbers in both normal and recycled aggregate concrete beams. In addition, it is observed that in normal concrete the acceleration diminished immediately after the peak and disappeared completely after 5 ms during the last drop when compared to the previous drops. This indicates that the beam failed immediately after impact. The same is observed physically at the time of testing. Whereas, in case of RAC with all percentages of recycled coarse aggregates, the frequency of vibration is more within the first 10 ms and thereafter the acceleration disappears completely during the last drop impact

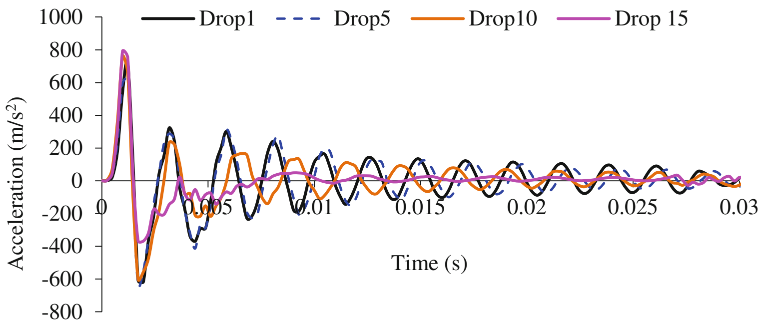

Fig. 7.5 Accelerations at midspan for repeated drops in normal concrete beam (M-RAC0)

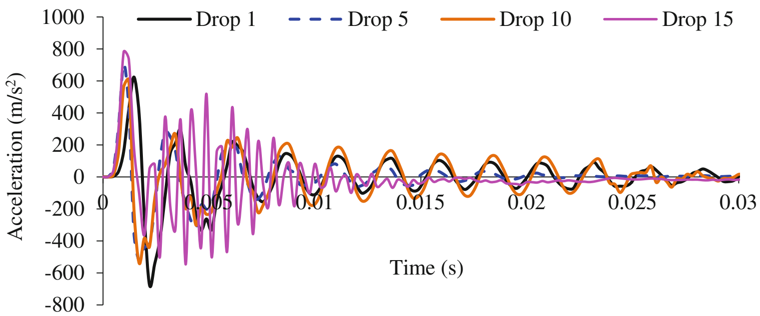

Fig. 7.6 Accelerations at midspan for repeated drop impacts in M-RAC100 beam (Rao et al. 2011)

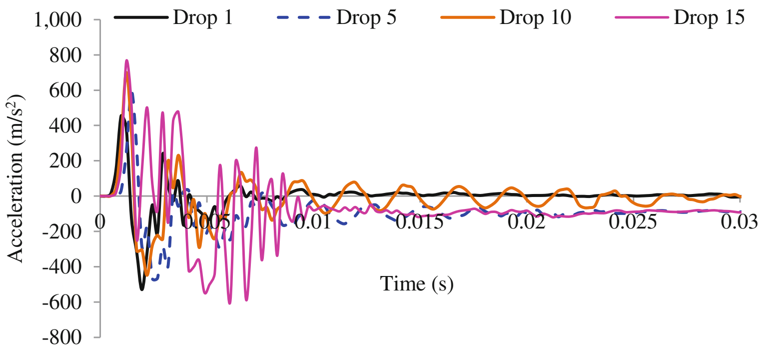

Fig. 7.7 Accelerations at midspan for repeated drop impacts in M-RAC50 beam (Rao et al. 2011)

compared to the previous drops. This indicates the reduction in stiffness of the material after certain number of drops. The recycled aggregate concrete is more vulnerable for stiffness reduction due to the presence of weaker interfaces and microcracks.

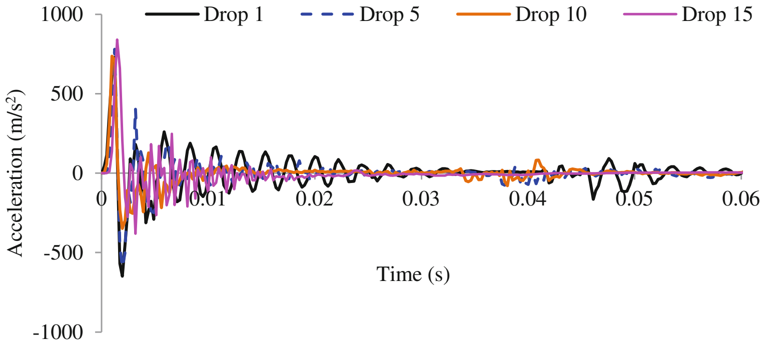

Fig. 7.8 Accelerations at midspan for repeated drop impacts in M-RAC25 beam (Rao et al. 2011)

Table 7.3 Average accelerations along the length (half span) during first and last drops of impact (Rao et al. 2011)

Mix designation	First drop			Last drop		
	500 mm	350 mm	200 mm	500 mm	350 mm	200 mm
M-RAC0 (3)	686	626	554	792	786	627
M-RAC25 (3)	568	548	341	731	657	543
M-RAC50 (3)	706	637	445	813	793	420
M-RAC100 (3)	733	707	462	814	832	576

Note The number within the parenthesis is the number of specimens tested

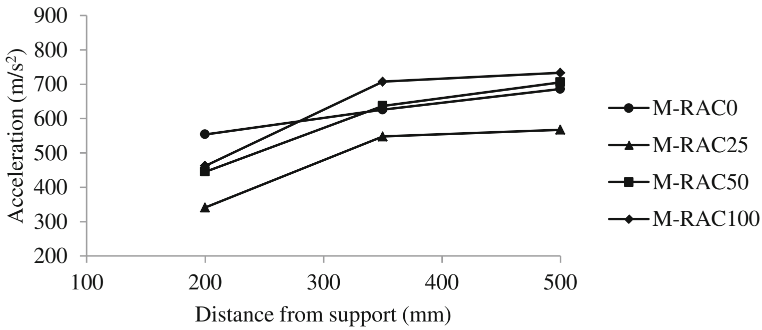

Fig. 7.9 Acceleration variation along length (half span of beam) during the first drop impact (Rao et al. 2011)

The variation in acceleration along the length (for half span of beam) for first and last drops in both normal and recycled aggregate concrete beams with all percentages of recycled coarse aggregate is presented in Table 7.3 and Figs. 7.9 and 7.10, respectively.

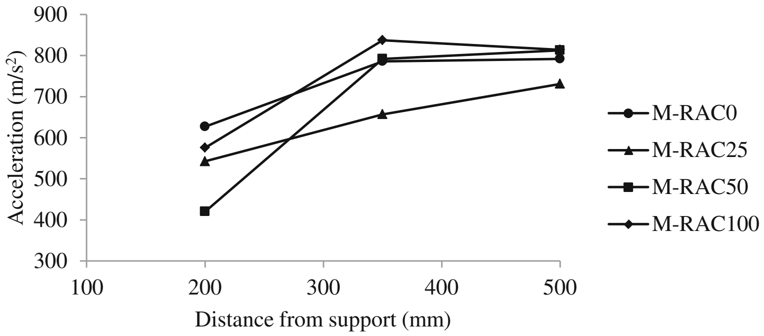

Fig. 7.10 Acceleration variation along length (half span of beam) during the last drop impact (Rao et al. 2011)

It is observed that the variation is similar in both normal and recycled aggregate concrete beams except in case of RAC with 100% recycled coarse aggregate. The acceleration decreases as the wave propagates toward the support. In case of RAC with 100% recycled coarse aggregate, the acceleration is more at 350 mm distance from support compared to 500 mm (middle) during the last drop of impact. This represents the crack initiated at a location away from the center (impact point). Figures 7.11, 7.12, 7.13, and 7.14 represent the variation in acceleration along half span of the beam for repeated drop impacts of same height for both normal and RAC beams with all percentages of recycled coarse aggregate. Each presented value is an average of three beams. It is observed that the magnitude of acceleration increases with the increase in number of drops at locations 350 mm and 500 mm from the support along the half span of the beam irrespective of recycled coarse aggregate percentage.

It is also observed that in case of RAC with 100% recycled coarse aggregate, the magnitude of acceleration at 350 mm distance from the support is more than that of acceleration at midspan after nine drops. The average accelerations at 350 mm

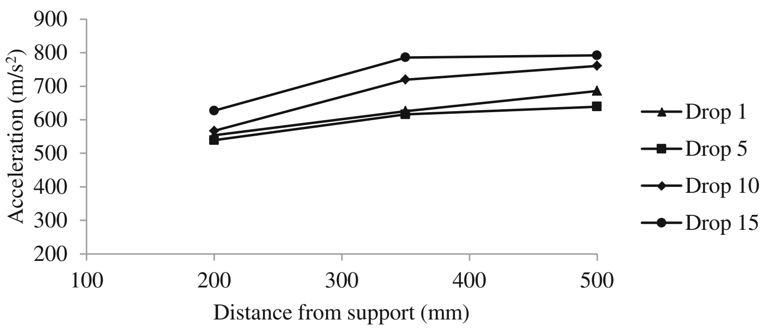

Fig. 7.11 Acceleration variation along length (half span of beam) for repeated drop impacts in normal concrete beam (Rao et al. 2011)

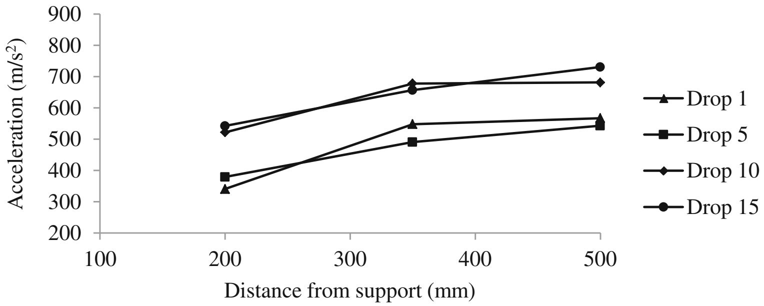

Fig. 7.12 Acceleration variation along length (half span of beam) for repeated drop impacts in M-RAC25 (Rao et al. 2011)

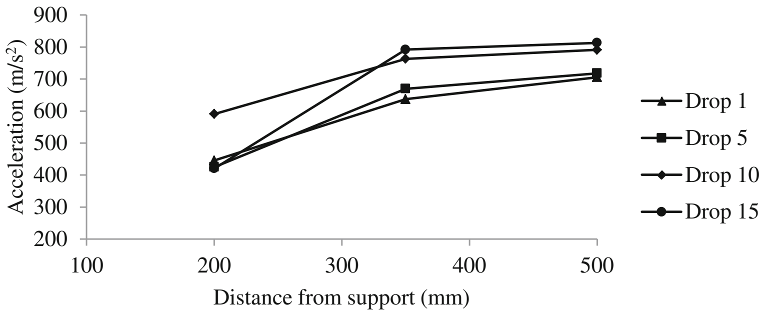

Fig. 7.13 Acceleration variation along length (half span of beam) for repeated drop impacts in M-RAC50 (Rao et al. 2011)

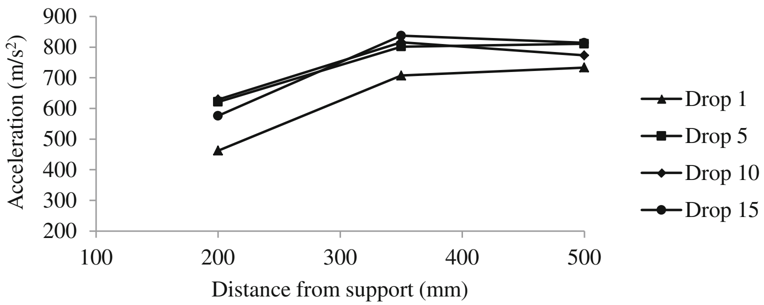

Fig. 7.14 Acceleration variation along length (half span of beam) for repeated drop impacts in M-RAC100 (Rao et al. 2011)

location are 837 and 814 m/s^2 during 10 and 15 drops of impacts, respectively, when compared to an acceleration of 816 and 773 m/s^2 at middle of the beam (500 mm location). This may be indicated that the failure initiated at a location away from the impact point instead of below the impact point.

7.2.2.2 Displacement

As discussed in previous section, the actual displacement peak values may be missed due to the small acquisition rate. However, the main focus of the experiment was to establish a comparative study between the recycled aggregate concrete and normal concrete. The displacement histories are measured from the acceleration histories at the middle of the beam on the lower side during each impact as discussed earlier. The maximum displacement histories among all repeated drop impacts for both normal and recycled aggregate concrete beams are presented in Fig. 7.15.

It is observed that the variation in displacement with time is similar for both normal and recycled aggregate concrete beams. The displacement mainly depends on the stiffness of the beam. It is observed that the maximum displacement is increased with the increase in percentage of recycled coarse aggregate. This is obvious that the stiffness of recycled aggregate concrete beams is lower than that of normal concrete beam. As discussed in Table 7.2, the modulus of elasticity of RAC with 100% RCA is 15.4% lower than that of normal concrete. It was observed under a uniaxial static loading that the strain is more at a given stress (load) in case of recycled aggregate concrete than that of normal concrete due to the presence of microcracks, weaker interfaces between recycled aggregate and old and new mortars and lower modulus of elasticity of recycled coarse aggregate. In addition, it is observed that there is an initial positive displacement occurred due to inertia force. The maximum displacement in recycled aggregate concrete at 25, 50, and 100% recycled coarse aggregates is 1.32, 1.63, and 1.795 mm corresponding to a displacement of 1.077 mm in normal concrete.

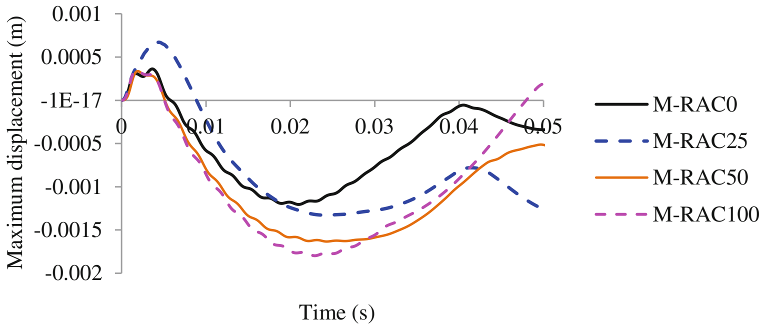

Fig. 7.15 Maximum displacement among all drops

7.2.2.3 Strains

The strain gauges are mounted on the top and bottom surfaces of half span of the beams to study the longitudinal distribution of strains. Three strain gauges each are mounted on top and bottom faces of the beam, and their positions are shown in Fig. 7.3. As the beams undergo impact, the nature of strains at a point oscillates from tension to compression or from compression to tension. Table 7.4 and Fig. 7.16 show the distribution of strains on both top and bottom faces of half span of the beam during the first drop of impact in both normal and recycled aggregate concrete beams. Each presented value in the graph is the average of three beams. In Fig. 7.16, the dotted line represents the tensile strains and solid line represents the compression strains.

It is observed that at all positions of strain gauges the strains are increased with the increase in percentage of recycled coarse aggregate. It is also observed that the strains on compression face are varying more linearly along the half span of the beams compared to those on tension face in both normal and recycled aggregate concrete beams. This indicates that the flexural crack may affect the tensile strains. In addition, in both normal and recycled aggregate concrete beams the magnitudes

Table 7.4 Average peak strains during the first drop of impact along the half span of beams

Location	Average peak strains			
Location (mm)	M-RAC 0	M-RAC 25	M-RAC 50	M-RAC 100
Top 400	−0.00016	−0.00016	−0.00017	−00000.00018
Top 250	−9.1E−05	−0.00011	−0.00011	−0.00013
Top 100	−7.3E−05	−3.2E−05	−4.9E−05	−7.9E−05
Bottom 400	0.00017	0.00017	0.000182	0.000208
Bottom 250	0.000113	0.000115	0.000136	0.000148
Bottom 100	6.8E−05	0.000058	6.5E−05	0.000104

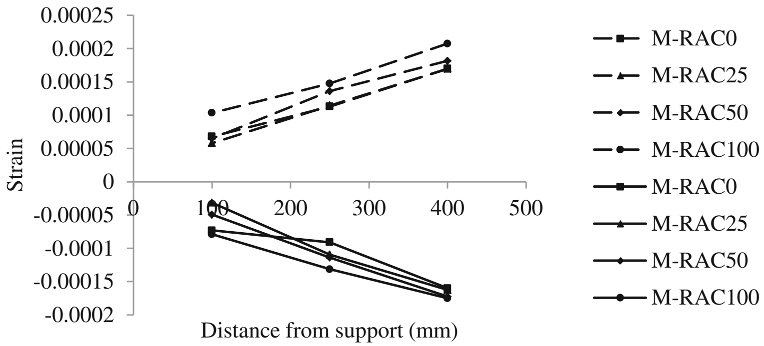

Fig. 7.16 Longitudinal distribution of strains for half span of the beam during the first drop impact

of strains on tension face at all positions of strain gauges are more than those on compression face. The magnitude of both compression and tensile strains at all locations is almost same in case of normal concrete and recycled aggregate concrete made with 25% RCA beams, whereas the magnitude of tensile and compression strains is much deviating in case of RAC with 50% RCA and 100% RCA from the normal concrete beams. The maximum tensile and compression strains in RAC with 50% RCA and 100% RCA are 0.000182, −0.00017 and 0.000208, −0.00018, respectively, compared to 0.00017, −0.00016 in normal concrete during the first drop of impact. This may be due to existence of microcracks in recycled coarse aggregate and weaker interfaces between aggregate and old mortar and new mortar.

The measured strains at different points along the half span of the beams in both normal and recycled aggregate concretes for repeated drops of impact are plotted in Figs. 7.17, 7.18, 7.19 and 7.20, respectively. The strains become erratic and cannot be measured properly once the specimen is cracked and fails in subsequent blows. The plots indicate the strain values prior to cracking only. The number of blows required to cause failure of the specimens varies depending on the percentage of

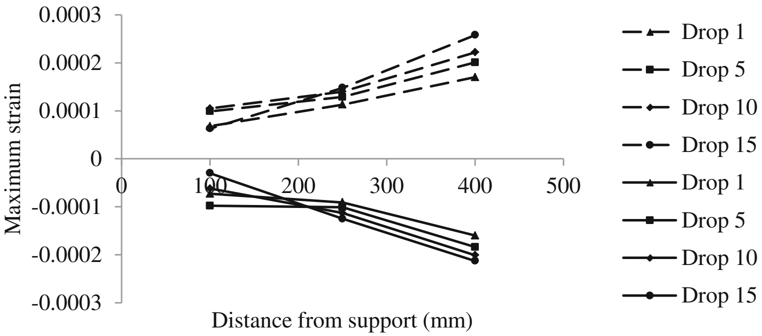

Fig. 7.17 Longitudinal distribution of strains for half span in normal concrete for repeated drops

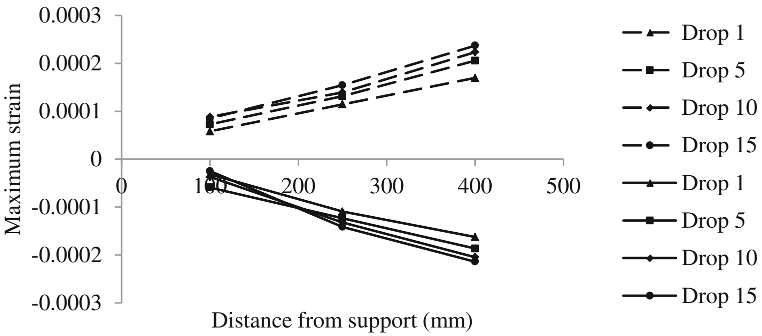

Fig. 7.18 Longitudinal distribution of strains for half span in M-RAC25 concrete for repeated drops

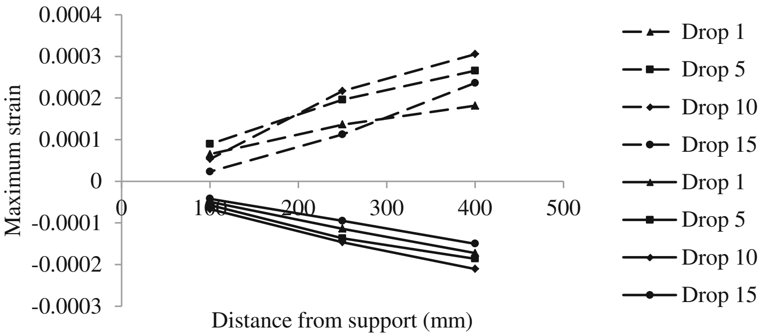

Fig. 7.19 Longitudinal distribution of strains for half span in M-RAC50 concrete for repeated drops

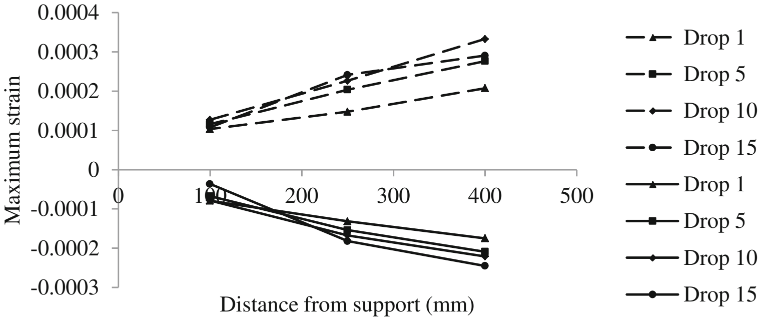

Fig. 7.20 Longitudinal distribution of strains for half span in M-RAC100 concrete for repeated drops

RCA. It is observed from the figures that the magnitude of strains on tension side is more than those on compression side at all positions of strain gauges in both normal and recycled aggregate concrete beams. It is also observed that both tension and compression strains are increased with the increase in drop number and the increment is more uniform with the increase in drop number in case of normal and RAC with 25% recycled coarse aggregate compared to RAC with higher percentage of recycled coarse aggregate (50 and 100%). In addition, nearer to impact point, the strain values on tension side have more deviation from the first drop to the subsequent drops. This indicates that the flexural crack influences the tensile strains and softening the beam with the repeated impacts. This may be due to the existence of microcracks and weaker interfaces between recycled aggregate and old and new cement mortars which may propagate during the repeated impacts. The maximum compression strains in RAC with 25, 50, and 100% RCA are −0.000214, −0.000.211, and −0.000250 and in those the maximum tensile strains are 0.000237, 0.000306, and 0.000333, respectively, compared to −0.000213, 0.000256 compression and tensile strains in normal concrete. This shows that 25% RCA on recycled aggregate concrete does not have significant influence on both maximum tension and compression strains during the repeated drops of impact.

7.2.2.4 Support Reactions

As discussed earlier, one of the supports on which the beam supported is arranged to measure the support reaction using a 200 kg load cell. The load cell is connected to the DAQ (NI SCXI 1000 chassis with 1520 strain module), and the data are recorded at 1/10000 s intervals in terms of strain. The load cell is calibrated with same DAQ under static loading, and then the strain output is converted into support reaction using the calibration chart. Figure 7.21 shows the support reaction histories for both normal and recycled aggregate concrete beams during the first drop of impact. The enlarged view of the peak values of support reaction is presented in Fig. 7.22.

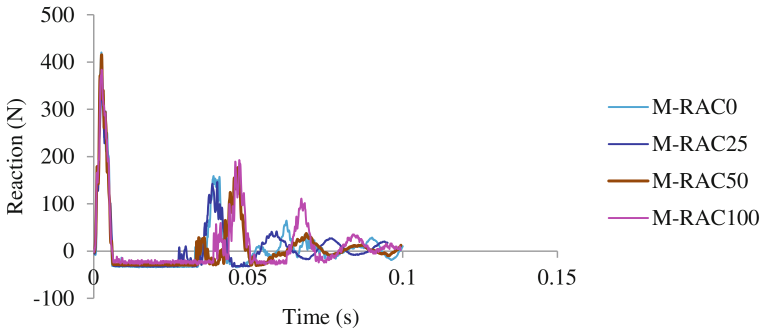

Fig. 7.21 Support reaction in both normal and recycled aggregate concretes during the first drop impact

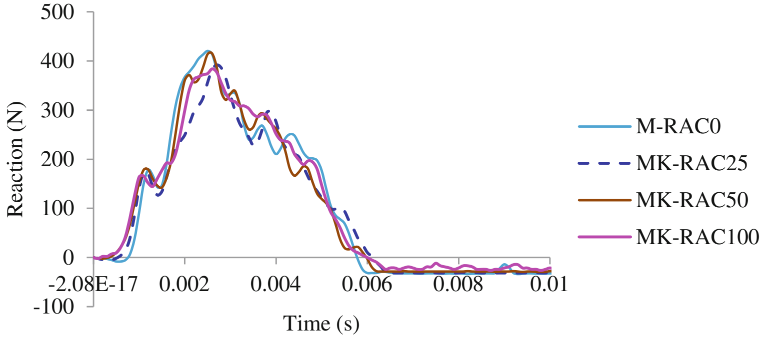

Fig. 7.22 Enlarged view of peak support reactions in normal and recycled aggregate concretes during the first drop impact

The reaction obtained from the load cell is the combined effect of direct impact caused by impactor and the inertia force produced by the vibration of the member. The reaction developed before the peak value in figures is due to the effect of direct impact induced by impactor and after the peak it is due to inertia force caused by vibration of the beam. It is observed that during the first drop of impact, the magnitude of reaction in case of normal concrete is more than that of recycled aggregate concrete at all percentages of recycled coarse aggregate. This indicates that the impact resistance depends on the stiffness of the member. It is also observed from the figures that the time taken to travel the longitudinal stress wave from center to the support is less in case of normal concrete compared to RAC with 100% recycled coarse aggregate. This may be due to the presence of weaker interfaces between recycled aggregate and old and new mortars, high porous nature of recycled coarse aggregates. In addition, the first small peaks observed in Fig. 7.22 may be due to the slipping of plates from the load cell at the time of impact. In Fig. 7.21, there are some new peaks, which are due to second drop of the hammer after the hammer bounced back. The variation in support reaction from the first drop of impact to the failure drop for both normal and recycled aggregate concretes is presented in Fig. 7.23.

It is observed that in both normal and recycled aggregate concrete beams, the support reaction marginally increases with the increase in drop number up to a certain number of drops and thereafter it reduces with further increase in drop number. It was observed during experimentation that the beam fails immediately on impact once the crack is initiated. However, possibly as the tested beam specimens are stronger in comparison with the impactor, the beam did not show any sign of distress even up to 10th blow and that is the reason there is increase in the reaction values. In addition, it is observed that the maximum reaction of concrete with natural aggregate is more than that of RAC with all percentages of RCA. This indicates that the stiffness of the member influences the impact and inertia forces: The larger the stiffness, higher the impact and inertia forces.

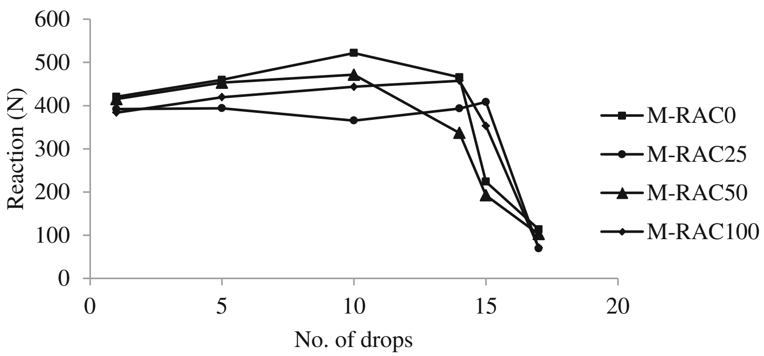

Fig. 7.23 Variation in support reaction with number of drops

Table 7.5 Maximum average values of support reaction

Mix	% of RCA	Maximum average support reaction (N)
M-RAC0	0	521.60
M-RAC25	25	492.13
M-RAC50	50	499.93
M-RAC100	100	469.22

Table 7.5 shows the average maximum values of support reaction among all the drops for both normal and recycled aggregate concrete beams. It is observed that the average maximum value of support reaction decreases with the increase in percentage of recycled coarse aggregate.

At 25, 50, and 100% recycled coarse aggregates, the average support reactions are 492, 499, and 469 N, respectively, compared to 521 N in case of normal concrete. The reduction in support reaction is in the order of 4–10% in case of RAC with 25–100% RCA compared to normal concrete. This may be due to lower stiffness of RAC. As discussed in the previous sections, the compressive strength and modulus of elasticity of RAC with 100% RCA are 15.4% lower than that of normal concrete and the impact value of recycled coarse aggregate is also less than that of natural aggregates.

7.2.2.5 Failure Pattern

The crack pattern and failure surfaces of beam specimens for both normal and recycled aggregate concretes are presented in Figs. 7.24 and 7.25, respectively. It is observed that both normal concrete and RAC made with all percentages of RCA are failed between 15 and 17 drops. In addition, it is observed that the cracks initiated at the bottom surface, vertically at or near the impact point in both normal and recycled aggregate concrete beams except in RAC with 100% recycled coarse aggregate. All beams are failed in the immediate next one or two drops after the initiation of crack. This may be due to brittleness of the material. In case of RAC with 100% recycled coarse aggregate, the crack initiated away from the impact point. Figure 7.25 shows the fractured surfaces of both normal and recycled aggregate concrete specimens. It shows that the failure path or surface is more tortuous in case of normal concrete and RAC with 25% recycled coarse aggregate.

This indicates that the failure is through the interface between aggregate and cement mortar and this is common in case of normal concrete as it is the weakest portion in normal concrete. In case of recycled aggregate concrete with higher percentage of recycled coarse aggregate (50 and 100%), the fractured surface is more even. This indicates that the failure is through the aggregate in addition to the weaker interfaces between aggregate and old and new mortars. A similar failure pattern is observed in case of split tensile test.

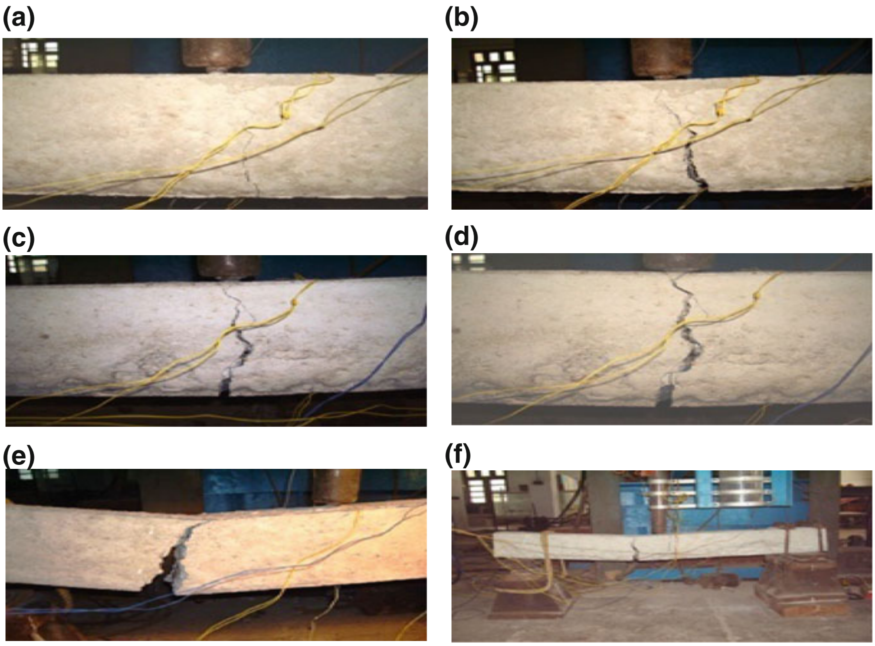

Fig. 7.24 Crack pattern of both normal and recycled aggregate concrete beams **a** and **b** normal concrete, **c** M-RAC25, **d** M-RAC50, **e** and **f** M-RAC100

7.3 Flexural Behavior of RAC

Ajdukiewicz and Kliszczewicz (2007) conducted a series of RCC beams made with six types of recycled aggregates obtained from crushed prefabricated concrete elements on flexural behavior under static loading. The concrete mixes of low, medium, and high strength were considered. Further two types of reinforcement were considered to obtain different shapes of failure, i.e., flexure and shear. All beams were tested under simply supported condition, and two equal loads were applied at 5 kN increment. It was found that the load-carrying capacity of beams with recycled aggregate concrete was 3.5% lower than those made with normal concrete in case of flexural failure and the capacity was bit greater when beams failed in shear. In addition, the deformations were large in beams made with recycled concrete compared to those made with normal concrete; however, this effect was controlled by the presence of reinforcement. It was further observed that at probable service load the deflections of beams made with RAC were approximately 18–100% greater than those made with natural concrete. At failure, the crack pattern and failure shapes were more or less similar to all the beams including

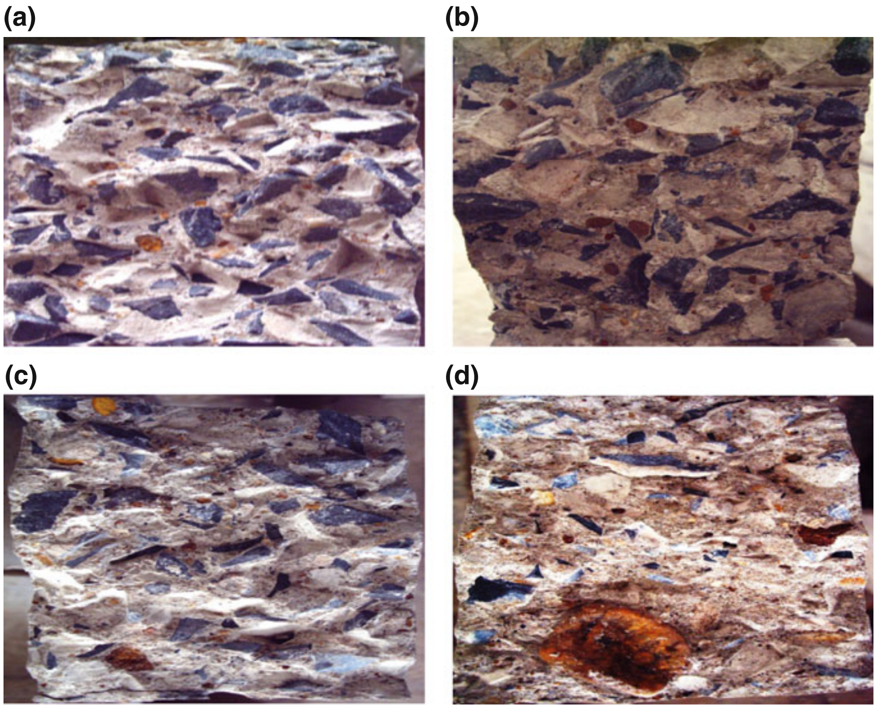

Fig. 7.25 Failure surfaces of **a** M-RAC0, **b** M-RAC25, **c** M-RAC50 and **d** M-RAC100

beams with normal concrete. In RAC beams, the first crack appeared at one step of loading ahead than in normal concrete beams. Initial cracks were developed along the stirrups in all the beams and as the load advanced, many more branched cracks of lesser width were found. This ensured the proper bond between reinforcement and concrete, and it was continued till the end of failure.

Sato et al. (2007) conducted studies on the flexural behavior of reinforced concrete beams made with recycled aggregate under static and sustained loadings. In their study, two types of recycled coarse aggregate and two types of recycled fine aggregate were obtained from normal concrete with w/c ratio 0.45 and 0.6, respectively. Another kind of recycled coarse and fine aggregate was obtained from real site, i.e., beams, columns, and slabs of old reinforced concrete (RC) buildings which were constructed during 1961 and 1967. A total of 37 RC beams were prepared to investigate the behavior of RC beams with the following factors, viz., w/c ratio of normal concrete from which the RCA obtained, usage of recycled coarse and/or fine aggregate, curing condition, w/c of recycled aggregate concrete and tension reinforcement ratio. The authors agreed with Ajdukiewicz and

Kliszczewicz (2007), that for a given w/c ratio, the deflections were large in case of beams with recycled concrete when compared to normal concrete. Further, not much significant difference in crack spacing was observed between recycled and normal concrete RC beams. In case of dry condition, the crack spacing was smaller when compared to wet condition. Further, it was found that the width of cracks in RC beams made with recycled concrete was larger than those made with normal concrete. However, these values were within the limits of Japan Society of Civil Engineers (JSCE) code. The authors also reported that the ultimate moment-carrying capacity of RC beams made with RAC was almost same as that of those made with normal concrete for a given w/c ratio and yielding of steel prior to the compression failure.

Arezoumandi et al. (2015) conducted a full-scale testing on flexural strength of reinforced concrete beams made with 100% recycled coarse aggregate as well as with conventional aggregate. In their study, two different longitudinal reinforcement ratios (0.47 and 0.64%) with shear reinforcement to avoid shear failure were considered. All beams are of rectangular section of size 300 mm × 460 mm with a span of 2.1 m (c/c distance of supports) of simply supported condition and subjected to a four point bending load. It was observed that the behavior of RC beams with conventional concrete and recycled aggregate concrete is similar with respect to crack morphology and crack progression. However, the cracks were closer in case of beams with recycled concrete compared to those made with conventional concrete. The failure was initiated by yielding of tension reinforcement in both the conventional concrete and recycled aggregate concrete RC beams. The load vs deflection of RC beams with both conventional concrete and recycled concrete is presented in Fig. 7.26. It was observed from the figure that all beams behaved as linearly elastic until the first flexural crack appeared, i.e., point A. With further increment in the load, it was observed that the tension steel yielded (point B). Upon further increment in the load, the concrete crushed in the compression zone and the beams were failed finally. It can also be seen from the figure that the cracking moment of beams with RCA was lower when compared to those made with conventional concrete. This may be attributed to the presence of two interfacial transition zones (ITZs) in beams with RCA (ITZ between original aggregate and adhered mortar and ITZ between adhered mortar and new mortar) as compared to only one interfacial transition zone, i.e., natural aggregate and mortar in conventional concrete beams. Figure 7.27 shows the load vs strain in longitudinal reinforcement in both conventional and recycled aggregate concrete beams. It can be seen that all the reinforcement gets yielded. Further, it can be seen that the stiffness of the beams with recycled concrete was lower than those made with conventional concrete. It could be ascribed to the lower modulus of elasticity of RAC compared with the conventional concrete. The crack pattern under flexural failure in both the RC beams made with conventional concrete and recycled concrete is shown in Fig. 7.28.

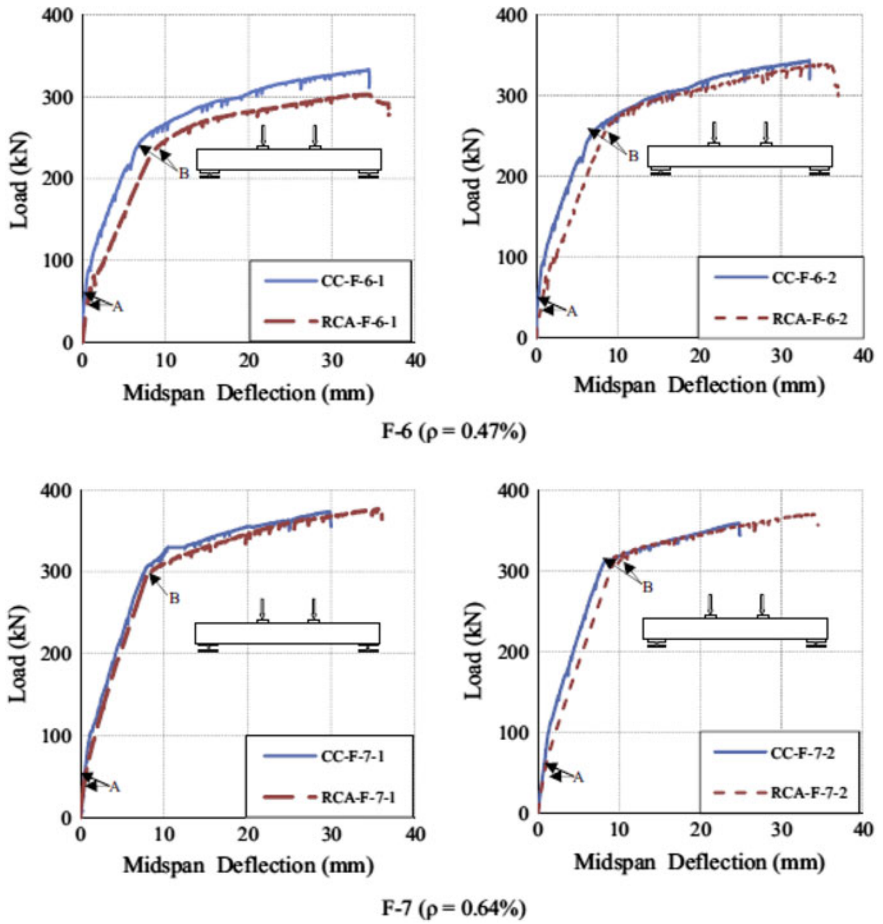

Fig. 7.26 Load versus deflection at midpoint for different longitudinal reinforcement ratios (F-6 means flexural beam with 2–19 mm diameters bars and F-7 means flexural beam with 2–22 mm diameter bears; 1, 2 represents beam1 and 2) (*Source* Arezoumandi et al. 2015)

Initially, the flexural cracks were developed in the maximum moment region and as the load increased the additional flexural cracks were developed in the region between supports and load points. With further increase in the load, most of the flexural cracks developed vertically and later on inclined flexure-shear cracks began to appear.

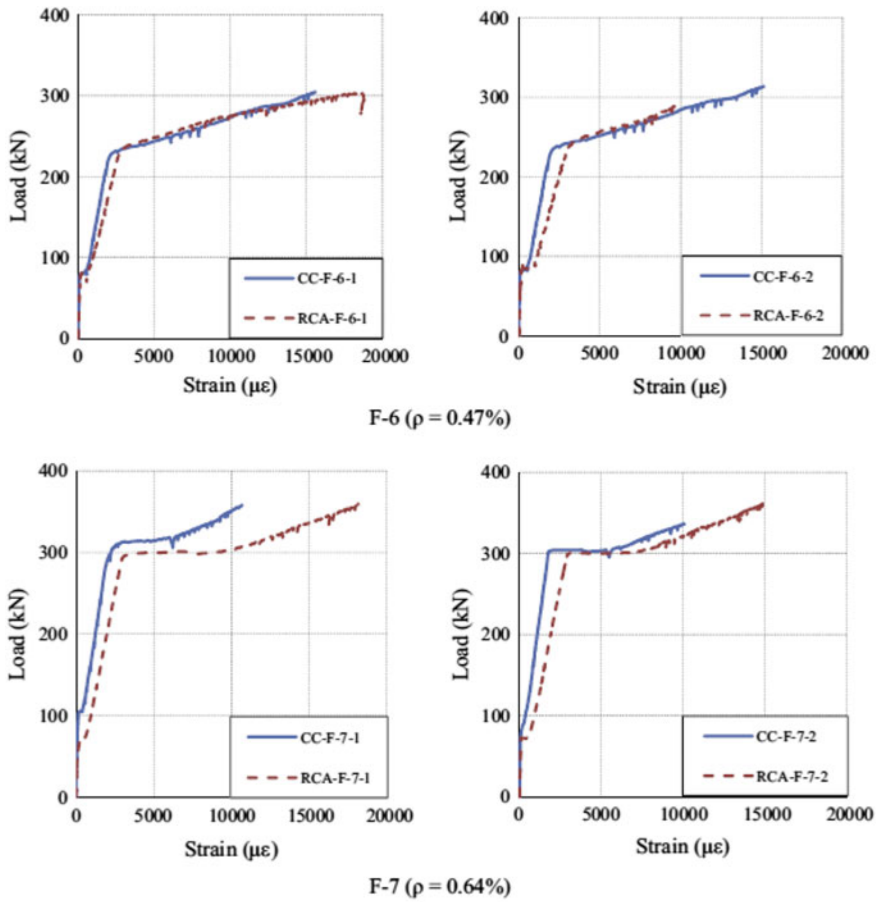

Fig. 7.27 Load versus strain in steel in both conventional concrete beams and recycled concrete beams with different percentage of longitudinal reinforcement (*Source* Arezoumandi et al. 2015)

7.4 Shear Behavior of RAC

Etxeberria et al. (2007) conducted investigations on shear behavior of recycled aggregate concrete without and with shear reinforcement. A total of twelve beam specimens with same compressive strength and same amount of longitudinal reinforcement, four concrete mixes of different percentages of recycled coarse aggregate (0, 25, 50, and 100%), and three different amounts of web reinforcements were considered in their investigation. All beams are simply supported with a span of 2.6 m and the cross sections of 200 × 350 mm and subjected to a two-point load (symmetric) system with shear span to depth ratio of 3.3. It was observed that in case of beams without shear reinforcement, the cracks were developed initially flexural and progressed at 45° (approximately) inclined toward the midsection and with

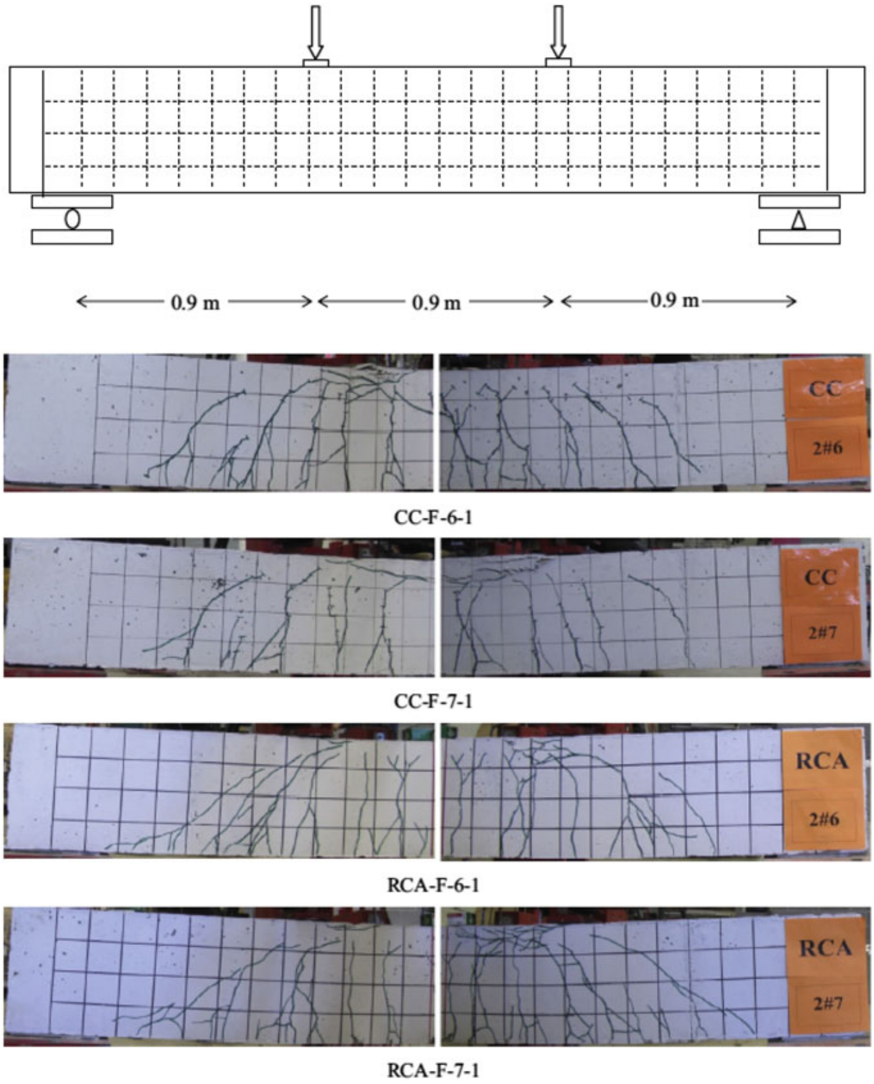

Fig. 7.28 Crack pattern of both conventional and recycled concrete beams at flexural failure (*Source* Arezoumandi et al. 2015)

an increase in the load a single inclined crack appeared joining the load point to the tip of the main initial crack, and the beam failed suddenly. In contrast, a more ductile response was observed in case of beams with shear reinforcements. After the development of initial shear crack, the shear reinforcement (stirrups) started functioning and thereafter series of shear cracks developed. It was further found that the shear strength of reinforced concrete beams without shear reinforcement made with recycled concrete depends on the content of RCA and at 25% RCA the effect was practically not significant. The cracking load was decreased with the recycled aggregate content in beams without shear reinforcement. This was probably due to the fact that the crack occurred at the weakest zone, i.e., at the interface between aggregate and old adhered cement mortar. In case of reinforced concrete beams with shear reinforcement, it was found that the effect of RCA content on ultimate shear strength was less significant; due to fact that, the compressive strength of RAC made with RCA was same as that of normal concrete by the addition of extra cement and low w/c ratio. It was finally concluded that 25% RCA can be used as a structural material, provided that all measures related to dosage, compressive strength, and durability aspects were adopted. Gonzalez-Fonteboa and Martinez-Abella (2007) studied the shear strength of RAC beams made with 50% RCA with different amounts of transverse reinforcement. The authors found that there was little difference in behavior of RAC beams in terms of deflections and ultimate load when compared to beams with normal concrete. In addition, the authors observed premature cracking and notable splitting cracks along the tension reinforcement in case of RAC beam. Gonzalez-Fonteboa et al. (2009) concluded that by the addition of silica fume, the notable splitting cracks which were observed along the longitudinal reinforcement in their previous study were palliated (Gonzalez-Fonteboa and Martinez-Abella 2007).

7.5 Other Structural Aspects

You-Fu and Lin-Hai (2006) examined the behavior of steel tubular columns filled with recycled aggregate concrete for different shapes of columns and for different load eccentricity ratios. It was concluded that both normal and recycled aggregate concrete-filled steel tubular columns behave in a similar manner and all were buckling failures. The load-carrying capacity of recycled aggregate concrete-filled steel tubular (RACFST) columns was slightly lower than that of normal concrete-filled steel tubular (CFST) columns. At 25% and 50% RCA, the load-carrying capacity of normal CFST columns had 1.7–9.1% and 1.4–13.5% higher than RACFST columns in circular and square shapes, respectively. This low strength of RACFST columns attributes to the lower strength of RAC than that of normal concrete. In a study, Xiao et al. (2006) discussed the seismic response of reinforced frames made with recycled aggregate concrete. It was reported that under low-frequency lateral loading, the failure pattern of frames is similar to all percentage of RCA and they failed at the end of the beams first and then at the columns' bottom. The ultimate load capacities of RAC frames were lower than that of normal concrete frames. However, this reduction was less than that of mechanical properties of RAC made with RCA. Finally, the authors concluded that frames made with properly mix designed RAC were suitable to resist an earthquake according to Chinese Standard GB 5011-2001.

7.6 Summary

The experimental results of behavior of recycled aggregate concrete beams under drop weight impact load are presented in this chapter. The behavior of RAC beams in terms of accelerations, displacement, strains, and support reaction histories under repeated drops of impact is discussed. The failure pattern of the recycled aggregate concrete beams under impact load is also highlighted. The behavior of reinforced concrete beams made with recycled aggregate concrete under flexure and shear reported by different researchers is discussed. The other structural aspects of the recycled aggregates are also described. Based on these studies, the following highlights are observed.

- At a given impact energy (energy imparted by the hammer per blow), the accelerations and midspan displacements of RAC beams are more than those of concrete with natural aggregates.
- The recycled aggregate concrete beams are more sensitive to the high strain rate of loading. Both tensile and compressive strains of RAC with high percentage of RCA (50 and 100) are significantly higher than those of normal concrete. However, inclusion of lower percentage (<25%) of RCA does not have much influence on strains during the repeated drops.
- Larger quantity of RCA (more than 25%) reduces the impact resistance of concrete. However, the lower percentage of RCA (<25%) in concrete has no significant influence on impact resistance.
- The load-carrying capacity of RAC beams and columns was lower than those of normal concrete beams when they are subjected to static flexural and compression loading. In addition, the deformations are larger in case of RAC than normal concrete. Nevertheless, the failure phenomenon was slower in case of RAC columns, as a result of greater ductility.
- With reference to shear strength of RAC beams with transverse shear reinforcement, 25% RCA does not show any significant effect on the strength and failure behavior.
- The modes of failure of recycled aggregate concrete-filled steel tubular columns (RACFST) and normal aggregate concrete-filled steel tubular columns (NACFST) are similar. However, ultimate load-carrying capacity of RACFST was little lower than that of NACFST.
- The frames made with properly mix designed RAC were suitable to resist an earthquake force according to the Chinese standards.

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Chapter 8
Quality Improvement Techniques

8.1 Introduction

In Chap. 3, the physical and mechanical characteristics of recycled aggregates are discussed. The influence of characteristics of recycled aggregate, amount of RA, strength of source concrete, method of mixing, method of curing, addition of mineral admixtures, etc., on the mechanical properties, shrinkage, creep and durability performance of RAC have been discussed in detail. Further, the microstructure of RAC and the behavior of RAC under various types of loads have been presented in Chaps. 6 and 7, respectively. It was found that the recycled aggregates have lot of potential benefits in the applications of concrete but at the same, there is some negative effect of recycled aggregates particularly the density, porosity, water absorption, and ITZs. One of the reasons for the negative effects of RA is the adherence of the old cement mortar on the surface of RA. Taking all the aspects into consideration, the past investigators have suggested various techniques for the improvement of the quality of recycled aggregates and recycled aggregate concrete. The detailed procedures of mechanical treatment, thermal treatment, acid treatment and their influence on the recycled aggregate properties and recycled aggregate concrete are discussed. The influence of the impregnation techniques on the properties of RAC is also discussed. Furthermore, various new methods of mixing procedures and their beneficial effects on properties of RAC are discussed.

8.2 Quality Improvement Techniques for Recycled Aggregate

Recycled aggregates are the natural aggregates adhered with old cement mortar derived from the construction and demolition waste/rejected concrete elements. The adhered old cement mortar is loose and porous in nature which makes the recycled

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aggregates inferior quality. The volume of the adhered mortar depends on the size of the aggregate and strength of the source concrete from which it derived, and generally, it varies from 25 to 60% according to its size (Corinaldesi 2010). Around 20% of the cement paste adhered on the surface of the aggregates for aggregate size 20–30 mm (Nassar and Soroushian 2012; Rahal 2007). It is important to note that there are more number of interfacial transition zones between old and new cement mortars which may play a vital role in the internal microstructure of concrete (Despotovic 2016). Therefore, either by removal of adhered mortar or enhance the quality of it can facilitate the applications of RCA in concrete. The attached old cement mortar from the surface of RA can be removed by three methods such as (i) mechanical treatment, (ii) thermal treatment, and (iii) chemical treatment. Similarly, strengthening of the adhered mortar can be done by using various impregnation techniques of RA.

8.2.1 Mechanical Treatment

The attached mortar from the aggregate can be separated using crushing and ball milling. It is a modest and widespread treatment which has lot of variations. However, the recycled aggregate could be damaged during the mechanical grinding (Despotovic 2016). Using the autogenous cleaning (Pepe et al. 2014) process the fine materials attached to the surface of recycled aggregate can be reduced. In this process, RCAs are placed in a rotating mill drum and collide against each other while removing pieces of attached mortar. The mill drum, 30 cm in diameter and 50 cm in depth (Fig. 8.1), was filled up to 33% with raw recycled aggregates, and the drum was rotated at a rate of 60 rotations per minute. After the autogenous cleaning process, aggregates were cleaned with water and subsequently dried to remove all the produced remaining fines and impurities.

Another simple instrument that can be used in this mechanical treatment is the Los Angeles abrasion test machine (Babu et al. 2014). The machine was loaded with 10 kg of recycled aggregate (20–10 mm and 10–5 mm each 5 kg) and 10 number of steel balls approximately 40 mm diameter and each weighing 390–445 g. The machine was rotated at 33 rpm with different number of total rotations. After completion of total rotations in each batch, the materials were discharged and washed with water properly till the water after washing was clean. The washed material was air-dried and stacked before the use.

Pepe et al. (2014) investigated the efficiency of the mechanical treatment in removal of the adhered cement mortar from the aggregate surface. It was assessed in terms of bulk density and water absorption (Table 8.1). It was found a significant decrease in water absorption of RCA. After 15 min duration of cleaning, the water absorption of RCA was reduced by 50 and 20% for aggregate size 19–9.5 mm and 9.5–4.75 mm, respectively. This effect was less with the shorter duration of cleaning. Similarly, the bulk density results of RCA further confirm the efficiency of this treatment (Table 8.1). Further the effectiveness of autogenous cleaning of

Fig. 8.1 Mill drum and autogenous cleaning process (Pepe et al. 2014)

Table 8.1 Water absorption and bulk density of RCA by mechanical and thermal treatment (Pepe et al. 2014)

Property of aggregate	Size of aggregate (mm)	Natural aggregate	RCA (without treatment)	Treated RCA by mechanical treatment	
				10 min	15 min
Water absorption (%)	19–9.5	1.28	4.94	4.01	4.09
	9.5–4.75	3.39	11.94	6.06	5.56
Bulk density (kg/m^3)	19–9.5	2634	2268	2358	2328
	9.5–4.75	2464	1946	2220	2261

RCA was also assessed by conducting tests on compressive strength, modulus of elasticity, and tensile strengths of recycled aggregate concrete mixes. It was found that the 28 days compressive strength of RAC made with RCA without treatment was 20% lower than reference concrete. However, the beneficial effect of autogenous cleaning was clearly emerged as the reduction in compressive strength was only 8.9% when it was made with treated RCA. Similarly, the tensile strength of RAC with untreated and treated RCA was 13% and 4%, respectively lower than reference concrete, whereas the results of modulus of elasticity of RAC did not show any difference between untreated and treated RCA.

Table 8.2 Properties of aggregates after mechanical treatment (Babu et al. 2014)

Property of aggregate	Natural aggregate	RCA (without treatment)	RCA after mechanical treatment by Los Angeles abrasion machine		
			200 revolutions	500 revolutions	700 revolutions
Specific gravity	2.64	2.6	2.61	2.62	2.63
Water absorption (%)	0.42	4.16	1.82	1.47	1.39
Bulk density (kg/m^3)	1611	1580	1585	1596	1601
Elongation index (%)	13.50	23.15	18.69	14.17	13.69
Flakiness index (%)	7.5	15.65	13.16	11.66	11.62
Fineness modulus	6.46	6.60	6.56	6.46	6.42
Los Angeles abrasion value (%)	14.71	32.17	20.88	15.22	15.05
Impact value (%)	13.53	23.84	17.45	14.86	14.06
Crushing value (%)	27.99	32.14	29.52	28.19	28.05

Babu et al. (2014) found a significant improvement in the physical and mechanical properties of recycled coarse aggregates after mechanical treatment (Table 8.2).

8.2.2 *Thermal Treatment*

In this treatment, the recycled aggregates were several cycles of soaking in water and heating. This method was proposed by several authors (De Juan and Gutiérrez 2009; Fathifazl et al. 2009). The authors reported that this method produces the precise results for evaluating the adhered mortar content in RCAs. The procedure of estimation of adhered mortar is as follows.

After removal of all kind of impurities like plastics, wood, bricks, asphalt, a known sample of recycled aggregate of mass (m_1) immersed in water for 2 h, so that the adhered mortar gets fully saturated. The saturated aggregates are then subjected to a constant temperature of 500 °C for 2 h in a muffle furnace. Then immediately the recycled aggregates are immersed in cold water (at room temperature 20 °C). The sudden heating and cooling cause thermal stresses and cracks in the old attached mortar so that it can easily be debonded and unleashed from the natural aggregate. Further, it is necessary to remove either by hand or rubber hammer if any mortar remains attached on the surface of the aggregates. When the whole mortar has been removed, the aggregates are dried at 100 °C maximum to obtain a constant mass (m_2).

Table 8.3 Properties of RCA before and after treatment (Al-Bayati et al. 2016)

Property	Bulk relative density	Apparent specific gravity	Bulk density (SSD)	Absorption (%)	Adhered mortar loss (%)
RCA untreated	2.295	2.622	2.425	5.91	3.02
RCA heat at 250 °C	2.309	2.648	2.437	5.54	3.28
RCA heat at 350 °C	2.334	2.667	2.458	5.35	3.98
RCA heat at 500 °C	2.254	2.623	2.394	6.25	9.76
RCA heat at 750 °C	2.302	2.652	2.434	6.73	41.02

$$\text{The adhered mortar (AM) content as AM (\%)} = \frac{(m_1 - m_2)}{m_1} \times 100$$

It was reported that finer fraction of aggregate was adhered with larger quantity of old mortar than coarse fraction. The amount of adhered mortar for 4/8 mm was found to be 33–55% and for 8/16 mm, and it was 23–44%. The authors established the relationships between the adhered mortar content and other properties of aggregates, and it was reported that these relationships were useful for establishing the RA requirement for different applications.

Al-Bayati et al. (2016) reported the results of RCA treated at different temperatures as shown in Table 8.3. It reveals that the heat treatment efficiently removed the large amount of adhered mortar from the RCA, which assists in improving the density and water absorption of RCA. A similar results were reported in the literature (Sui and Mueller 2012; De Juan and Gutiérrez 2009). It was found that RCA treated at 350 °C results larger specific gravity and lesser water absorption with an approximate reduction of 9.5%, whereas at 500 and 750 °C the results were contrast with others. This indicates that the mechanical behavior of recycled aggregate concrete between 400 and 600 °C largely affected when RCA exposed to thermal expansion and consequent internal stress. Vieira et al. (2011) reported that the cement matrix was subjected to severe microcracking between 600 and 800 °C due to decarbonation of RCA. Therefore, RCA suffers largely from degradation, and when it was exposed to high temperature, there is a breakdown and loss of mass of concrete particles (Wong et al. 2007; Gupta et al. 2012).

The combination of heat treatment followed by short mechanical treatment significantly reduces the water absorption of RCA with the increase in temperature (Fig. 8.2). When the temperature reached to 350–450 °C (approximately), there was a breakdown in water absorption and then sudden increase at temperature ranges between 500 and 750 °C (approximately). This was confirmed by other researchers who found that when RCA exposed to high temperature results a degradation, mass loss, breakdown, and microcracking (Wong et al. 2007; Gupta et al. 2012).

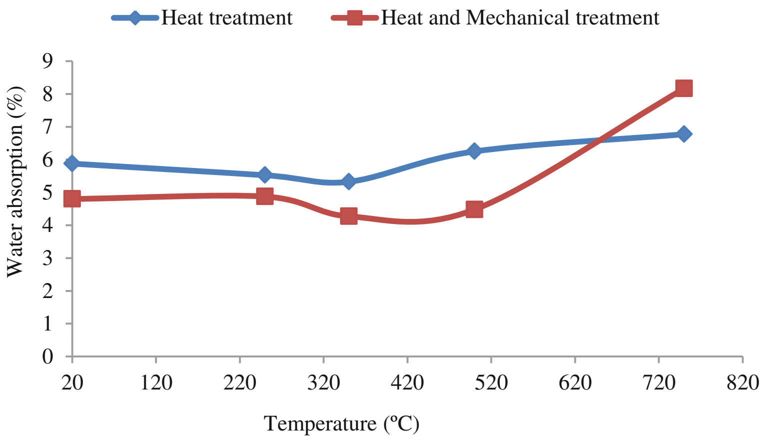

Fig. 8.2 Water absorption behavior by heat treatment (Al-Bayati et al. 2016)

8.2.3 Chemical Treatment (Tam et al. 2007a, b)

This method is based on the dissolution of adhered cement paste when recycled aggregate immersed in an acidic environment. In this method, first soak the recycled aggregate in an acidic environment at around 20 °C for 24 h and then watering with distilled water to remove the acidic solvents afterward (Fig. 8.3). The acidic solvents of low concentration acids like hydrochloric acid (HCl), sulfuric acid (H_2SO_4), phosphoric acid (H_3PO_4) can be used. Limestone aggregate cannot be used as the acid attacks the aggregates (De Juan and Gutiérrez 2009).

Determination of adhered mortar loss (Ismail and Ramli 2013)

$$\text{The loss of adhered mortar}(\%) = \frac{M1 - M2}{M1} \times 100$$

where

M1 Mass of oven-dried sample of RCA before immersion in acid
M2 Mass of sample of RCA after acid treatment

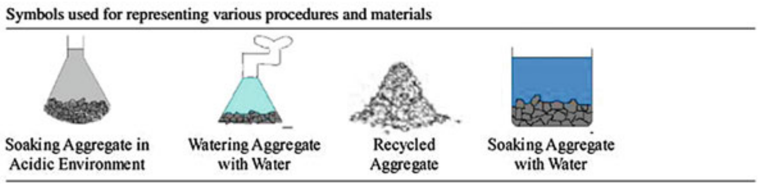

Fig. 8.3 Chemical treatment of RA (Tam et al. 2007a, b)

Table 8.4 Properties of RCA before and after treatment (Tam et al. 2007a, b)

Property	Size of aggregate (mm)	Before presoaking treatment	After presoaking treatment		
			HCl	H_2SO_4	H_3PO_4
Water absorption (%)	20	1.65	1.45	1.48	1.53
	10	2.63	2.31	2.37	2.41
Chloride content (%)	20	0.0016	0.0025	0.0001	0.0001
	10	0.0012	0.0056	0.0001	0.0001
Sulfate content (%)	20	0.0025	0.0076	0.1090	0.0110
	10	0.0025	0.0082	0.1040	0.0109
Value of pH	20	10.46	9.07	8.95	8.55
	10	11.63	9.34	9.35	9.33

Tam et al. (2007a, b) examined the efficiency of presoaking of RCA in three different acids, namely HCl, H_2SO_4, and H_3PO_4 with 0.1 M concentration on the properties of RCA and concrete. The authors considered 5, 10, 15, 20, 25, and 30% RCA of 10–20 mm size in each case. The efficiency of this technique was also assessed by estimating the mechanical properties of RAC. The results of RCA before and after treatment are presented in Table 8.4.

It reveals that after the pre-treatments, the water absorption was significantly reduced and the reduction was in the order of 7.27–12.17%. This shows that the presoaking of RCA in different acids can effectively remove the large portion of adhered cement paste from the RA surface, which helps to improve the weak link between new cement mortar and RA. Even though there was a slight rise in the chloride and sulfate contents after pre-treatment, they were still within the limits as per the respective standards 0.05 and 0.1% (Buildings Department 2005). Further the pH values of pre-treated RCA were slightly lowered down but still they were within the alkaline group (pH above 8.5). It was reported that after pre-treatment the results show a little hostile effect to RCA. Further noticeable improvements in mechanical properties of RAC prepared with RCA after pre-treatment were reported. Specimen preparation, surface characteristics of RCA, chemical bonding, and degree of bleeding influence the quality of ITZ. The ITZ behavior of aggregate with new cement mortar could be enhanced with the pre-treated recycled aggregate when compared to normal approach (Figs. 8.4, 8.5, 8.6, and 8.7). The concrete strength also increases as the bond strength of paste–aggregate increases (Mindess et al. 2003). Thus, the weak link of RA leading to its restrictive low-grade applications of concrete could be removed.

Al-Bayati et al. (2016) reported the results of presoaking of RCA in HCL (37%) and $C_2H_4O_2$ (99.7%) with low concentration of 0.1 M for 24 h at room temperature (Table 8.5). The authors also investigated the combined effect of acid treatment followed by short mechanical treatment on the properties of RCA.

It was found that RCA treated by HCL has more effect than by acetic acid with a reduction of 4.9% in water absorption. The resistance to freezing and thawing was

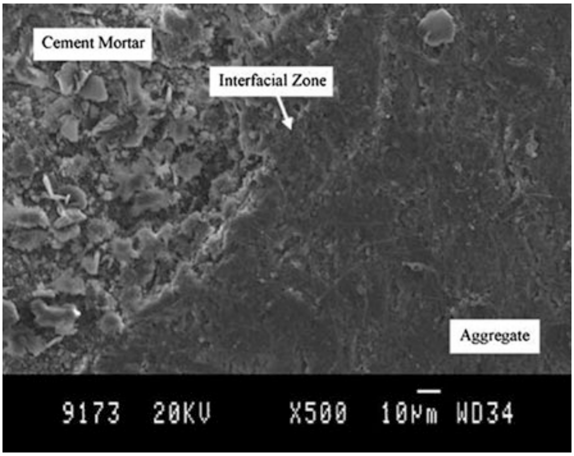

Fig. 8.4 ITZ after treatment with HCl (Tam et al. 2007a, b)

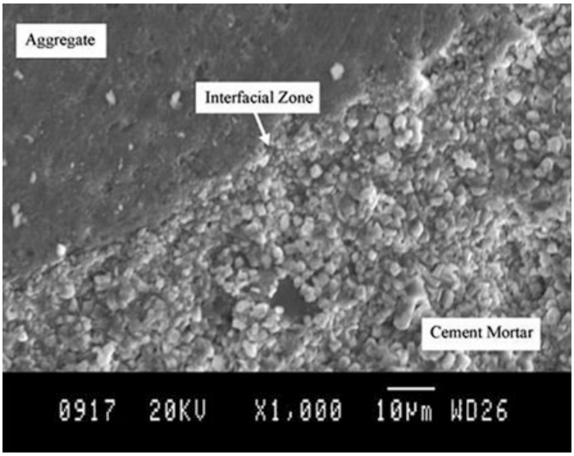

Fig. 8.5 ITZ after treatment with H_2SO_4 (Tam et al. 2007a, b)

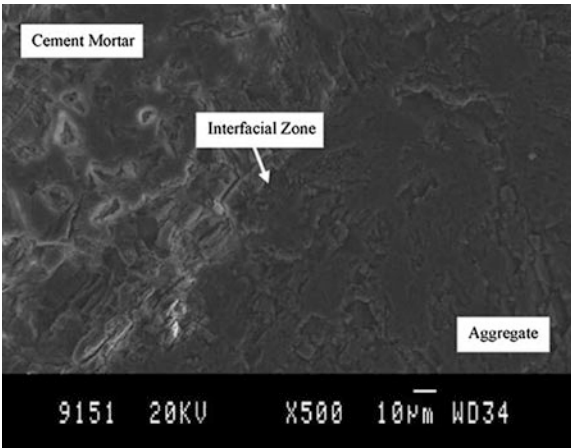

Fig. 8.6 ITZ after treatment with H_3PO_4 (Tam et al. 2007a, b)

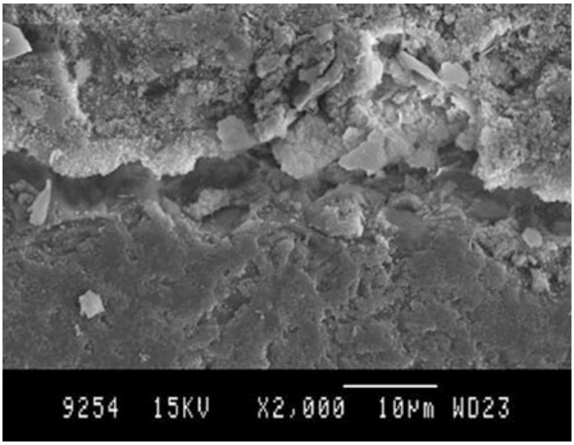

Fig. 8.7 ITZ for normal mixing method (Tam et al. 2007a, b)

Table 8.5 Properties of untreated and treated recycled aggregates (Al-Bayati et al. 2016)

Property	Bulk relative density	Apparent specific gravity	Bulk density (SSD)	Absorption (%)	Adhered mortar loss (%)
RCA untreated	2.295	2.638	2.425	5.91	3.02
RCA soaked in $C_2H_4O_2$	2.299	2.651	2.432	5.79	3.92
RCA soaked in HCl	2.305	2.651	2.435	5.66	4.56

improved significantly when RCA treated with acetic acid than HCl. It was observed that the resistance of freezing and thawing was lowered by 16% and 7%, respectively, when RCA was treated with acetic acid and HCl. It was also found that there was a poor influence on porosity with the acid treatments and only 4% and 2% reduction in porosity was observed with HCl and acetic acid treatments, respectively. Furthermore, the combined effect of weak acid treatment followed by short mechanical treatment reduces the water absorption efficiently by 20.6%. Based on the characterization images of SEM, it was concluded that weak acid was more efficient than the strong acid to reduce the influence of acid attacks on RCA surface.

Ismail and Ramli (2013) assessed the influence of different concentrations (0.1, 0.5, 0.8 M) of HCl and different treatment durations (1, 3, 7 days) on the properties of RCA in accordance with the procedure suggested by Tam et al. (2007a, b). It was reported that a linear correlation exists between the adhered mortar content and acid concentration, which indicates that the adhered mortar loss was increased significantly with the increase in acid concentration (Table 8.6). After treatment, the

Table 8.6 Mortar loss (%) of RCA with different molarity of acid and age of treatment (Ismail and Ramli 2013)

Description	Aggregate size (mm)	Treated with different molarity of acid and ages of treatment								
		0.1 M HCl			0.5 M HCl			0.8 M HCl		
		1 day	3 days	7 days	1 day	3 days	7 days	1 day	3 days	7 days
Mortar loss (%)	20	0.56	0.41	0.40	2.87	2.36	2.40	5.11	4.89	3.61
	10	0.80	0.66	0.86	3.18	2.81	2.82	4.73	4.28	4.65

reduction in mortar content with 0.8 M, 0.5 M, and 0.1 M HCl concentrations was 4–5, 3, and 1%, respectively, whereas the difference observed in percentage of adhered mortar loss with respect to the age of treatment was very minimal. Further, based on SEM analysis, the surfaces of the untreated RCA were more porous and are shielded with some amount of loose cement paste and other impurities which were connected loosely to the aggregate because of the crushing process (Fig. 8.8). Figures 8.9, 8.10, and 8.11 reveal that the loose particles on the surface of RCA were significantly decreased and even made the aggregate surface more uniform and clean when they were treated with different acid molarities. However, the RCA surface treated with HCl of 0.8 M shows that the surface was covered with little brittle and fragile particles several micron small when compared to the 0.5 M and 0.1 M HCl treated aggregate surfaces. This may be due to higher molarity of acid. The acid not only removed the loose mortar pieces from the surface of RCA but also probably eroded the bulk mortar, which was relatively more porous and weaker than the original RA.

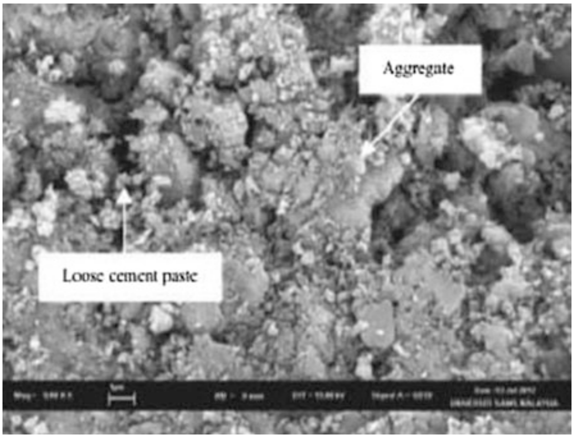

Fig. 8.8 Untreated RCA (Ismail and Ramli 2013)

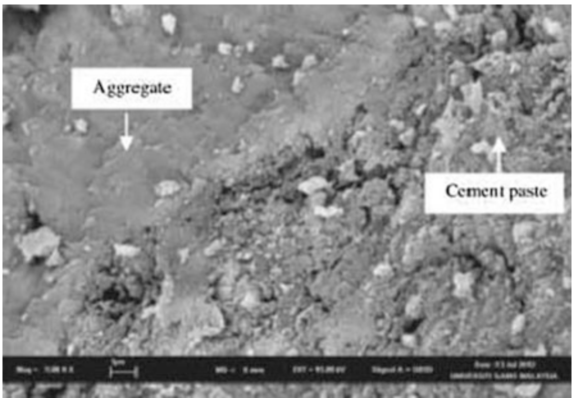

Fig. 8.9 Treated RCA with HCl of 0.1 M (Ismail and Ramli 2013)

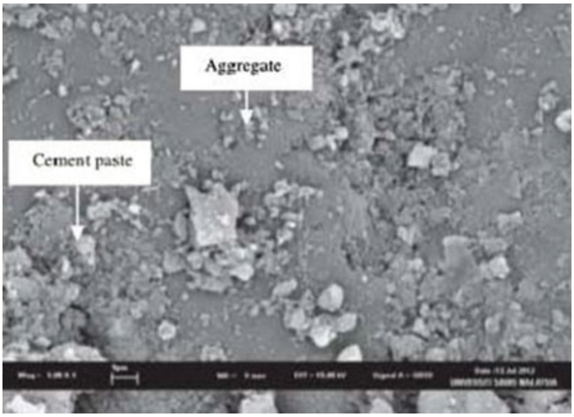

Fig. 8.10 Treated RCA with HCl of 0.5 M (Ismail and Ramli 2013)

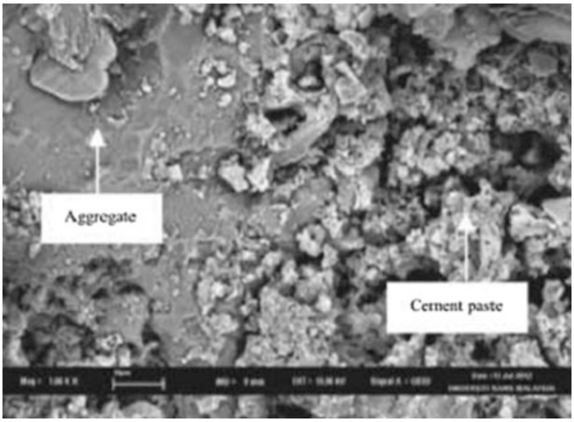

Fig. 8.11 Treated RCA with HCl of 0.8 M (Ismail and Ramli 2013)

The physical and mechanical properties of RCA without treatment were weaker than those of natural aggregate. However, after treatment with different concentrations of acid immersion significant improvements were observed in density, water absorption, and mechanical properties of RCA (Table 8.7). Further, it was observed that higher increase in density in 10 mm aggregate than 20 mm with varying concentrations of acid. Smaller size aggregates tend to be adhered with larger amount of adhered mortar than larger size aggregates (De Juan and Gutiérrez 2009; Padmini et al. 2009). The water absorption of RCA was reduced by 1–28% with different concentrations of acid, and particularly, the reduction was more with higher concentrations of acid (0.5 and 0.8 M). After treatment, the mechanical strengths of RCA were significantly improved, as the majority of the loose mortar and particles removed from the aggregate surface. Further, the results of chloride and sulfate contents of RCA agreed with Tam et al. (2007a, b) and remain within the limits of the respective standards. The efficiency of acid treatment technique was

Table 8.7 Properties of RCA with different molarity of acid and age of treatment (Ismail and Ramli 2013)

Property of aggregate	Aggregate size (mm)	Natural granite	Untreated RCA	Treated with different molarity of acid and ages of treatment								
				0.1 M HCl			0.5 M HCl			0.8 M HCl		
				1 day	3 days	7 days	1 day	3 days	7 days	1 day	3 days	7 days
Particle density (OD) Mg/m^3	20	2.60	2.33	0.56	0.41	0.40	2.87	2.36	2.40	5.11	4.89	3.61
	10	2.58	2.23	0.80	0.66	0.86	3.18	2.81	2.82	4.73	4.28	4.65
Water absorption (%)	20	0.6	4.44	3.99	3.58	4.63	3.67	3.48	3.75	3.83	3.51	3.95
	10	0.7	5.58	5.50	4.77	5.33	4.82	4.48	4.66	4.65	3.94	5.01
Agg. Crushing value (%)	14	24.32	29.15	28.02	27.39	28.86	28.73	27.95	28.68	28.34	28.14	28.8
Agg. Impact value (%)	14	13.98	21.78	19.59	20.27	20.90	18.97	19.56	20.37	19.08	21.37	20.71
pH value	Mixed		12.56	12.68	12.60	13.89	13.12	12.86	12.69	12.61	12.52	12.78
Chloride content (%)	Mixed		0.002	0.002	0.002	0.003	0.005	0.006	0.012	0.018	0.016	0.016
Sulfate content (%)	Mixed		0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001

also assessed by estimating the slump, density, and compressive strength with different percentages of RCA content. It was found that the slump and density of RAC prepared with treated RCA were not much improved when compared to the concrete prepared with untreated RCA. However, the compressive strength of RAC mixes with treated RCA was considerably improved at all test ages when compared to the mixes with untreated RCA. Most of the concrete mixes prepared with treated RCA have achieved their target strength, i.e., 50 MPa and even more at 28 days. Further, it was found that the RCA treated with 0.1 and 0.5 M HCl concentrations was substantially improved the compressive strength of RAC when compared to the mixes prepared with RCA treated with 0.8 M HCl. The soaking period of RCA in acid for not more than three days was sufficient for treatment.

8.2.4 Three-Step Method (Gao et al. 2013)

This method was divided into three phases: coarse crushing of the concrete, thermal treatment of the crushed concrete to remove the paste from the aggregate surface, and chemical attack of the remaining adhered paste with salicylic acid (Fig. 8.12).

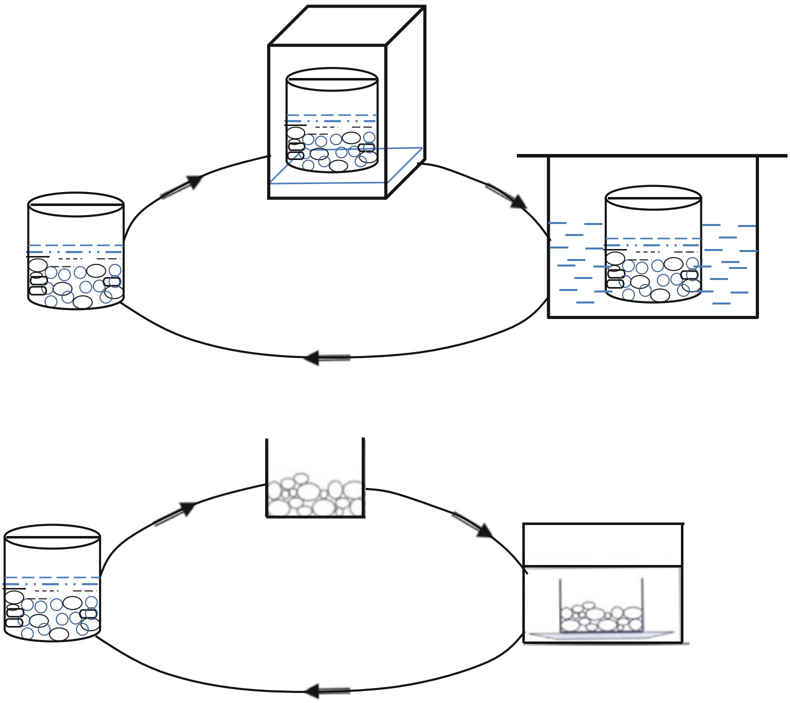

Fig. 8.12 Three-step method: soundness test and liquid nitrogen—microwave heating cycles (Gao et al. 2013)

Two variants were tested for the thermal treatment: a soundness test (ST) consisting in apply cycles of freezing (−17 °C) and heating (+60 °C) of the sample immersed in a 26% Na_2SO_4 solution, and liquid nitrogen—microwave heating cycles (LNMC). These two methods showed a similar efficiency, i.e., a direct recovery rate of 84% of clean aggregates of the size class 4/20 mm (52% recovered compared to 62% of 4/20 mm aggregates initially present in the concrete). The soundness test was kept in the final method due to its easier application in the laboratory. The chemical treatment of the remaining aggregates covered by cement paste by means of salicylic acid successfully dissolved the paste, with an efficiency of around 67–69%. Only thin layers of paste remained on 31–33% of final aggregates (size classes 0/1 and 1/4 mm). The overall efficiency of the three-step method, evaluated by comparing the amounts of recovered aggregates and natural aggregates, reached 90–92% on quartzite and siliceous limestone aggregates, respectively.

8.3 Impregnation Techniques

In previous sections, various techniques suggested by some of the researchers for removal of the adhered cement mortar from the recycled aggregate surface have been discussed. A few researchers suggested impregnation techniques as an alternative to improve the quality of recycled aggregate and hence recycled aggregate concrete. Katz (2004) attempted to improve the properties of recycled coarse aggregate and the compressive strength of RAC by (i) impregnation of recycled aggregate in a silica fume solution that is intended to add a thin layer of silica fume particles over the RA surface and (ii) an ultrasonic cleaning of recycled aggregate in order to remove the loose particles and strengthen the bond between RA and new cement paste. Low (A), medium (B), and high (C) strength concretes were considered for investigation.

8.3.1 Silica Fume Impregnation

- Prepare a solution of 1 kg of raw silica fume (loss on ignition 3.4%, SiO_2 90.4% and specific surface area (BET) 25.8 m^2/g) and 10 “l” of water by mixing small batches of solution in a mixer and add superplasticizer (1% by weight of silica fume) to confirm the proper diffusion of silica fume particles.
- Before soaking the recycled aggregate in silica fume solution for 24 h, the RA should be dried in an oven for 48 h and then cool it to room temperature and measure its weight.
- For suitable penetration of silica fume particles into the surface of RA, the saturated aggregates should then be dried again in an oven for 24 h and weighed the dried aggregates, so that the amount of silica fume that is impregnated into the aggregate can be estimated.

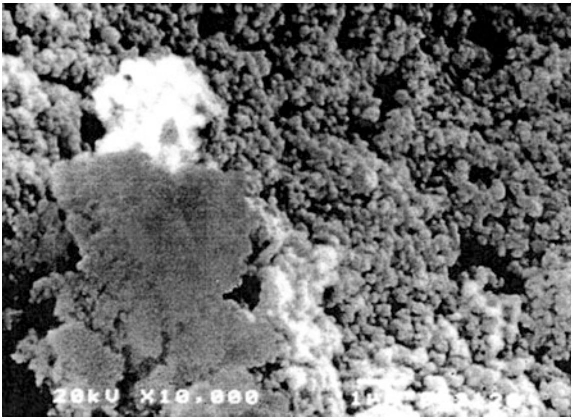

Fig. 8.13 SEM micrograph of the RA surface after silica fume impregnation (concrete C) (Katz 2004)

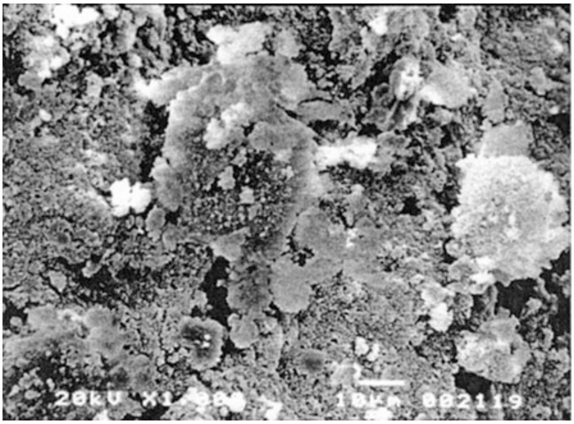

Fig. 8.14 SEM micrograph of untreated RA surface (concrete C) (Katz 2004)

The image of the SEM (Fig. 8.13) of RCA after impregnation in silica fume shows the surface of aggregate was covered with a layer of silica fume particles and a very little amount of crumbs. For comparison, the SEM image of untreated RA is presented in Fig. 8.14.

8.3.2 Ultrasonic Cleaning

In this process, the ultrasonic (US) bath was filled with huge amount of water and the recycled aggregates were immersed in it and cleaned for 10 min. After that the water was replaced with new water, and the recycled aggregates were cleaned for another 10 min. This process was repeated till the clean water was obtained. The SEM image of RA after this treatment is presented in Fig. 8.15. It was noticed during the cleaning process that RA obtained from high-grade concrete required

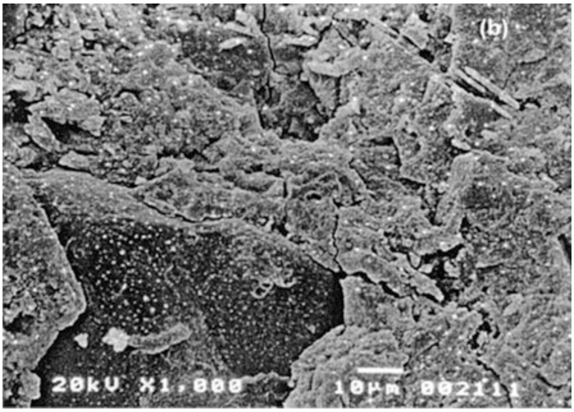

Fig. 8.15 SEM micrograph of RCA surface after cleaned in an ultrasonic bath (concrete C) (Katz 2004)

Table 8.8 Compressive strength of concrete at tested ages (Katz 2004)

Mix type	7 days		28 days	
	Strength (MPa)	Change (%)	Strength (MPa)	Change (%)
SF-RA/(Ref)	34.4/(27.2)	26	47.8/(42.3)	+13
SF-RB/(Ref)	33.5/(27.3)	23	49.3/(42.8)	+15
SF-RC/(Ref)	37.6/(28.4)	33	51.3/(44.3)	+16
SF-natural/(Ref)	24.2/(31.0)	−22	39.5/(45.7)	−13
US-RB/(Ref)	27.9/(24.2)	15	41.2/(38.7)	+7
US-RC/(Ref)	24.4/(23.8)	3	40.8/(38.2)	+7

RA, *RB*, *RC* Low, medium and high strength concretes

less number of cleaning cycles to get the clear water compared to the RA from lower grade concrete.

It was found that the silica fume treatment gives an increase of 23–33% and 15% in compressive strength of RAC at 7 and 28 days, respectively, against an increase of 3 and 7% with ultrasonic cleaning of RCA (Table 8.8). This shows that there was a significant improvement in compressive strength at early age when compared to later age with the silica fume impregnation technique.

This was due to filler effect of silica fume which improves the interface between recycled aggregate and new cement matrix. That is during the impregnation process, the particles of silica fume penetrate into the cracked and loose layer of RA and during the concrete hardening, this layer improves the ITZ through filler effect. Further, the pozzolanic reaction between the silica fume and $Ca(OH)_2$ produces secondary C–S–H gel which in turn to form an improved zone and penetrates from the RCA through the residue of the old cement paste into the new cement matrix. The effect of silica fume treatment was more strong at the early age strength due to

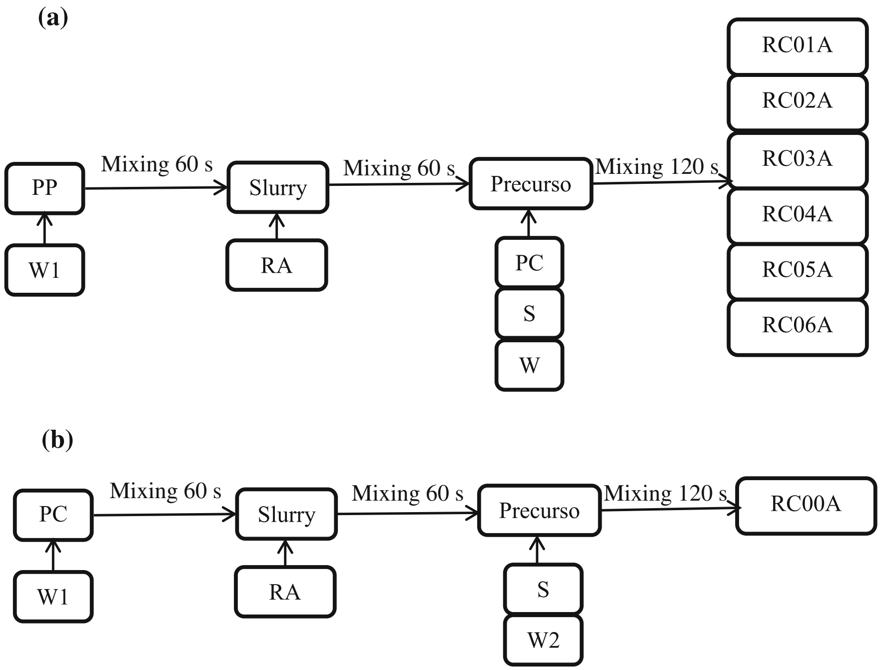

Fig. 8.16 Mixing procedure of **a** SEPP and **b** SEPC (Li et al. 2009)

the filler effect of silica fume which was dominant than the pozzolanic reaction. As the age progress, the cement matrix gets strengthened; these effects are weaker leading to a lesser influence on strength.

Li et al. (2009) suggested a new technique, i.e., coating with pozzolanic materials such as fly ash (FA), silica fume (SF), and blast furnace slag (BFS) and their combination for improving the quality of recycled aggregate concrete. A total 500 kg/m^3 of binder and 220 kg/m^3 of water were adopted in each mix. Two groups of mixes: Group A mixes were made with stone enveloped by pozzolanic powder (SEPP) approach and Group B mixes were prepared by stone enveloped with Portland cement (SEPC) approach. The detailed procedure adopted in these approaches is presented in Fig. 8.16.

(i) Divided the whole mixing into two stages. In the first stage, part of the total mixing water (W1) was mixed with pozzolanic powder (PP) for one minute to produce slurry and then the RCA was added to the slurry and mixed for another one minute so that the surface of the RCA was coated.
(ii) In the second stage, the rest of the water (W2), fine aggregate, and cement were added and mixed for about three minutes.

In Group A, W1:PP of 0.3 was adopted in mixes prepared with FA, BFS, and combination of FA and BFS and in mixes prepared with SF and combination of SF and other pozzolanic materials (FA, BFS), the W1:PP were 0.65, 0.5, and 0.45, respectively, adopted. For comparison, the normal mixing method (NMA) and stone enveloped with Portland cement (SEPC) were used. NMA was adopted in Group B mixes. The water-to-binder ratio was kept constant at 0.44 in all Group B mixes.

It was found from Fig. 8.17 that the workability was improved greatly either with SEPP or SEPC when compared to NMA. It also reveals that the pozzolanic materials superior to Portland cement for coating of RA, as the slump of the mixes with SEPP technique was more than that of SEPC. Furthermore, it demonstrates that the desired workability of the RAC can be attained with the pozzolanic powder coating technique. The advantage of this technique was neither major change in production process of RAC nor additional increase in cost. The improvement in workability was attributed to the thin coating film made from pozzolanic powder which prevents the water absorption of RCA during initial stages of fresh mixes. In addition not much difference was observed in slump loss between NMA and these mixing techniques.

The results of concrete in hardened state show that the compressive and flexural strengths of RAC were improved with this new technique compared to normal mixing approach (Table 8.9). Further, RCA coated with pozzolanic materials have shown significant improvement in compressive strength and flexural strength than those coated with Portland cement. In addition, the combination of silica fume and fly ash further enhances the strengths of RAC which is primarily attributable to the higher packing density. The microstructure of ITZ of RAC by SEPP (combination of SF and FA) was denser (Fig. 8.18) than that of by NMA (Fig. 8.19). It can be seen from Fig. 8.19, a crack of 30–40 μm length perpendicular to ITZ and a large amount of $Ca(OH)_2$ crystals at the ITZ in RC04B, whereas a uniform C–S–H gel in

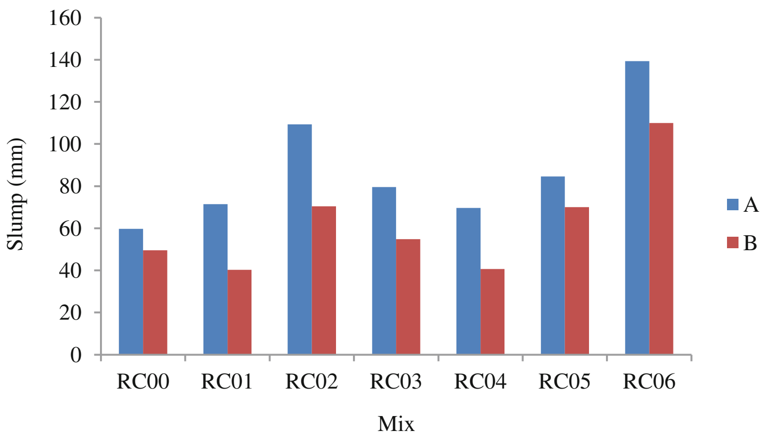

Fig. 8.17 Slump of various mixes (Li et al. 2009)

Table 8.9 Compressive strength and flexural strength of RAC mixes (Li et al. 2009)

Mix	Compressive strength (MPa)		Flexural strength (MPa)	
	7 days	28 days	7 days	28 days
RC00A	32.9	43.6	4.36	5.16
RC00B	34.5	36.2	4.44	4.97
RC01A	40	45.8	4.53	5.84
RC01B	23.9	31.5	4.42	5.73
RC02A	37.5	43.8	4.61	5.73
RC02B	24.8	32.7	4.05	4.67
RC03A	42	46.2	5.8	7.23
RC03B	25.3	35.5	4.68	5.69
RC04A	31.5	47.6	5.92	7.31
RC04B	38.1	43.6	4.75	5.6
RC05A	34.8	47.2	5.84	7.27
RC05B	26.7	43.5	4.54	5.72
RC06A	32.8	42	5.78	7.15
RC06B	28.5	38.3	4.61	5.82

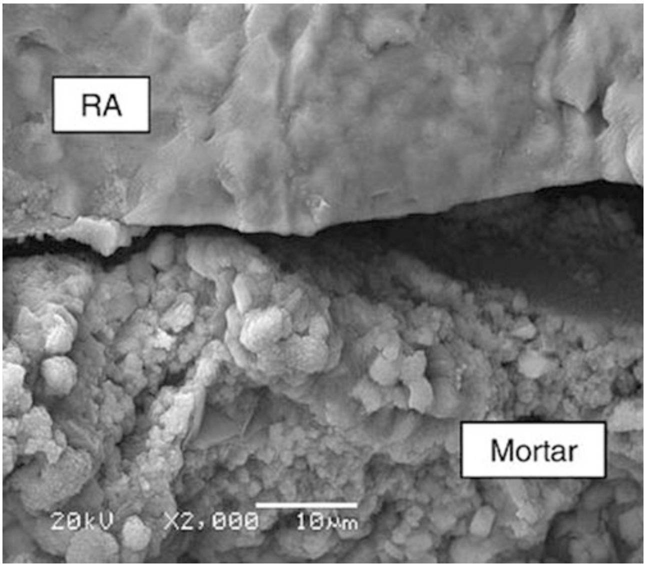

Fig. 8.18 SEM of ITZ of RC04A by SEPP (coated by combination of SF and FA) (Li et al. 2009)

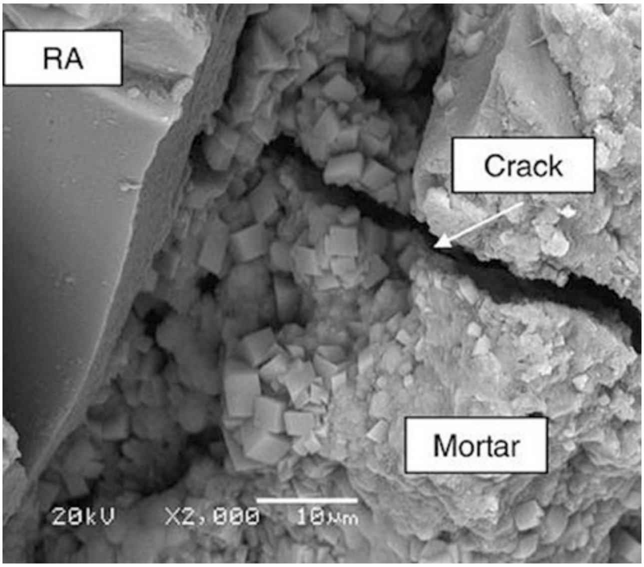

Fig. 8.19 SEM of ITZ of RC04B by NMA (coated by combination of SF and FA) (Li et al. 2009)

ITZ of RC04A mix (Fig. 8.18). The high absorption capacity of RA leads to the bleeding of water from mortar (inner bleeding), and after evaporation, the bleeding path results the cracks in ITZ, whereas in SEPP technique, a thin film coated over the surface of RA by the pozzolanic powder which acts as a barrier to the inner bleeding of water, hence the improved workability and denser ITZ.

8.3.3 Polymer Emulsion Impregnation

Spaeth and Tegguer (2013) used different silicon-based polymers in which the recycled coarse aggregates were soaked to improve the quality of recycled coarse aggregates. The silicon additives are emulsions composed of alkyl alkoxy silanes (silane), polydiorganosiloxanes (siloxane) or both of them. Different polymer solutions (Table 8.10) were prepared with different concentrations to find the optimal concentration and combination of polymer-based treatment to enhance the properties of recycled aggregates.

Before the treatment, the recycled aggregates were saturated in water for 48 h and then dried in an oven at 105 ± 5 °C for 24 h. After drying, the recycled aggregates were treated with two types of polymer impregnation processes:

Process 1 (Simple impregnation): The aggregate samples were impregnated by each polymer solution for 5 min, then dried at room temperature maintained at 20 °C and about 50% relative humidity (RH) for 24 h, then in ventilated oven at a temperature of 50 ± 5 °C until the difference in mass is less than 0.1% during 24 h.

Table 8.10 Set of polymer-based treatments (Spaeth and Tegguer 2013)

Treatment acronyms	Names of product	Composition	Concentration gradient	
			C_{min} (%)	C_{max} (%)
P1	Sodium silicate solution	Sodium silicate	7	30
P2	BS 2 Wacker siloxane/ silane emulsion	Octyl/methyl methoxy co-oligomeric siloxane/silane	5	30
P3	IE 4 Dow Corning silane emulsion	Octyl triethoxy silane	5	40
P4	BS 3 Wacker siloxane/ silane emulsion	Siloxane/Propyl trimethoxy silane	5	50
P5	BS 4 Wacker siloxane/ silane emulsion	Siloxane/Propyl triethoxy silane	5	60
P6	BS 5 Wacker siloxane/ silane emulsion	Siloxane/alkylalkoxysilane	5	40

Process 2 (Double impregnation and heat treatment): The aggregate samples were impregnated by soluble sodium silicate for 3 min followed by drying at room temperature maintained at 20 °C for 20 h and 50% relative humidity (RH), then the samples were again impregnated in each polymer solution (different siloxane/silane emulsions) for 5 min followed by drying during 24 h in a room maintained at 20 °C and in ventilated oven at a temperature of 50 ± 5 °C until the difference in mass is less than 0.1%.

The water absorption coefficient (%) of treated recycled aggregates with different polymers (min and maximum concentrations) in simple impregnation is presented in Fig. 8.20. It was found that the water absorption coefficient of 1.8% was the lowest when RA treated with min 5% P3 against 4.5% for untreated recycled aggregate. Similarly with higher concentrations (C_{max}), the lowest water absorption coefficient occurred when RCA treated with 40% P3 or 40% P4 and it was found to be 0.5% and 0.9%, respectively, when compared to untreated RCA and these values were very close to the natural aggregates. When RCA was impregnated in polymer solutions, the polymeric particles from it were spread and diffuse into the pores of RCA, and during polymerization, polymeric film deposit was developed and transform into hydrophobic resinous which permit to reduce the water intake.

In case of double impregnation technique, the combination of P1 with P2 and P6 had shown the lowest water absorption coefficients of 0.7% and 1.1%, respectively, against 2% in simple impregnation (Fig. 8.21). During the first impregnation, sodium silicate partially fills the pores and then silicate film attached on RCA pore network. During the second impregnation process, siloxane-based polymer particles spears into RCA and then the particles were changed into silanol. The silanol group reacts with silicates group in P1 and forms organic silicate chains. Therefore, the polymer film obtained by the combination of two polymers significantly reduces water absorption.

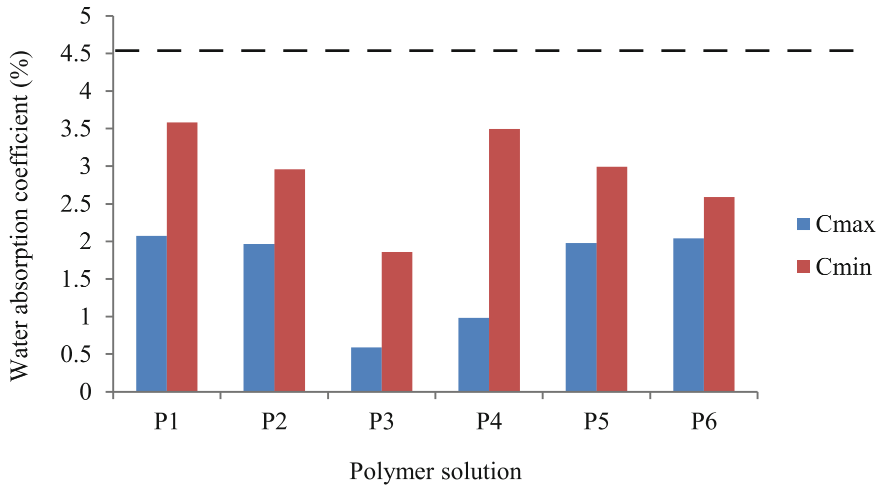

Fig. 8.20 Water absorption coefficient versus polymer solution (Spaeth and Tegguer 2013)

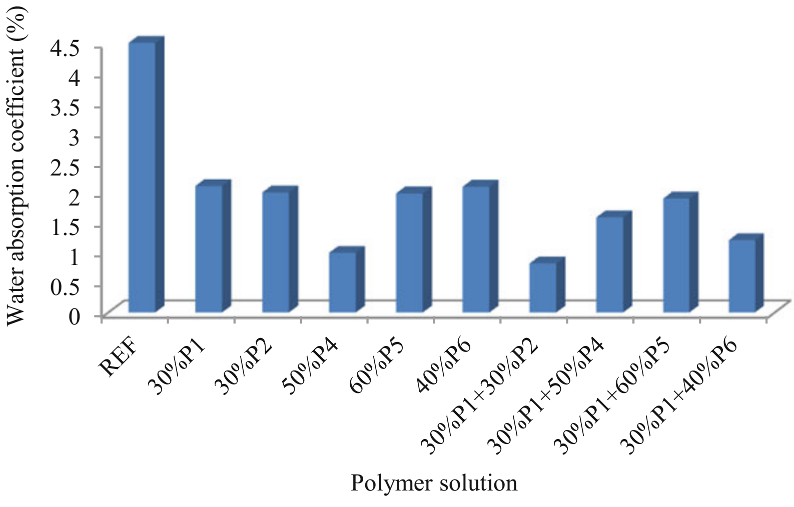

Fig. 8.21 Effect of impregnation treatments on water absorption coefficient of RCA (Spaeth and Tegguer 2013)

8.3.4 Surface Treatment with Nanomaterials

Zhang et al. (2016) have attempted to provide an effective and economical modification technique for RAC by soaking the recycled aggregate in two different nanoslurries. Two strengthening slurries were prepared by incorporating different nanomaterials to treat the surface of RA. The first slurry mainly consists of

Table 8.11 Proportions of two nanoslurries (Zhang et al. 2016)

Parameters	nSi + nCa slurry	Cement + nSi slurry
Water	90	50
Cement	0	95
Nano-SiO_2 dispersant	5	5
Nano-$CaCO_3$ slurry	5	0
Superplasticizer	0	1.5

Note All values are in kg

nano-SiO_2 (nSi) and nano-$CaCO_3$ (nCa), which is designated as nSi + nCa. Due to the high cost of nanomaterials, the second slurry was prepared with less dosage of nanosilica and more quantity of cement, which is labeled as cement + nSi. The proportions of components of these two nanoslurries are presented in Table 8.11. First the strengthening slurries with good dispersion were prepared by mixing the corresponding strengthening material with water for 120 s. Then, the recycled aggregates (RA) were soaked in the corresponding strengthening slurry for 45 min. The RA was then removed from the bath, and the redundant nanoslurry attached to the RA was removed by using the screen. At last, the recycled aggregates were dried at a temperature of 20 ± 2 °C and a relative humidity of 64 ± 5% for at least 3 days.

Natural aggregate (NA) of limestone (5–25 mm) and recycled aggregates were purchased from a manufacturer in Shanghai, China, and then crushed through a jaw crusher and sieved. The untreated NA and RA were labeled as N_0 and R_0 and the treated recycled aggregates with two slurries, i.e., nSi + nCa and Cement + nSi were labeled as R_1 and R_2, respectively. The treated and untreated RA samples are shown in Fig. 8.22.

After surface treatment, it was found that a coating was formed on the surfaces of R_1, R_2 which was due to the precipitation of the particles present in the two nanoslurries. Using four types of aggregates, viz. N_0, R_0, R_1, R_2, a total of four groups of concrete mixes (NAC_0, RAC_0, RAC_1, RAC_2) were prepared. All the mixes were prepared with a target compressive strength of 30 MPa. Nanoindentation tests were conducted on old and new ITZs to determine the mechanical properties of concrete at microlevel. Further, the meso-mechanical properties of concrete with treated and untreated RA and aggregate properties were investigated to assess the effectiveness of this technique. The width of old ITZ and new ITZ developed at 3 and 28 days in RAC prepared with untreated and treated RA reported by the authors is presented in Figs. 8.23 and 8.24, respectively. On the basis of width of ITZ, it was reported that no significant difference in old ITZs among three groups of RACs as some of the unhydrated cement particles remains in the old ITZ and nanomaterials could not penetrate into the old ITZs. Further, not much development in width of old ITZ in all three groups of RAC over the time, i.e., from 3 days to 28 days, whereas a significant development was observed in

Fig. 8.22 Untreated (1) R_0 and treated RA samples (2) R_1 (3) R_2 (Zhang et al. 2016)

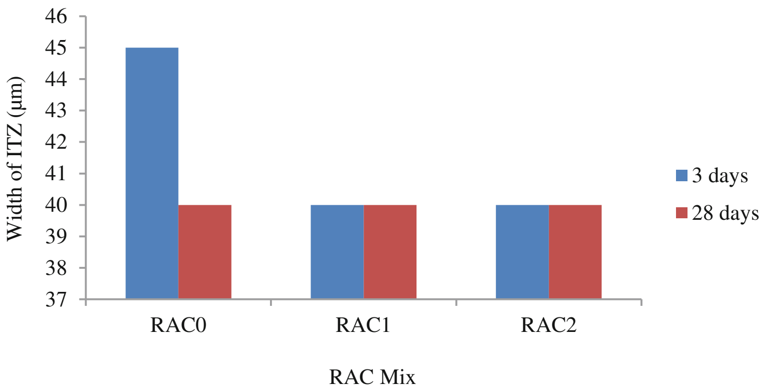

Fig. 8.23 Variation of width of old ITZ in different groups of RAC mixes (Zhang et al. 2016)

width of new ITZs in all the three groups of RACs from 3 days age to 28 days age, as the width of these ITZs significantly decreased during this period of time.

The modulus development within the new ITZs of all three groups of RACs in relation to the distance away from old mortar surface at 3 and 28 days reported by the authors is presented in Fig. 8.25. The elastic modulus of new ITZ in RAC_0 at 3 and 28 days did not show a clear trend whether it increased or decreased away from the old mortar surface, whereas the elastic modulus of new ITZs in RAC_1 and RAC_2 clearly shown that it decreased with the increase in distance from the old mortar's surface. The enhancement in new ITZs of RAC_1 and RAC_2 probably due

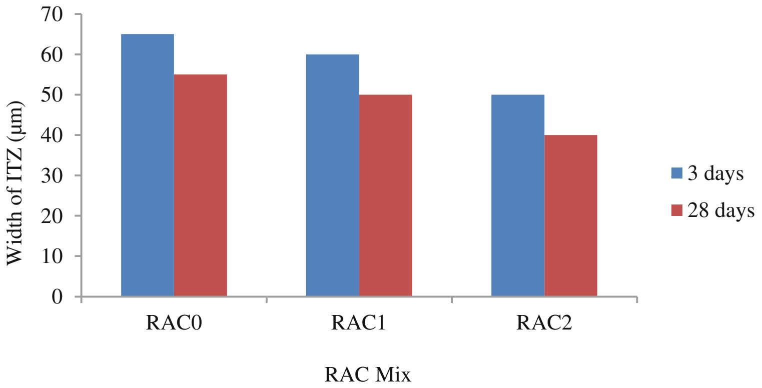

Fig. 8.24 Variation of width of new ITZ in different groups of RAC mixes (Zhang et al. 2016)

to proper dissolving and penetration of nanomaterials or cement particles in a thin layer coated around RA_1 and RA_2, respectively. However, the elastic modulus of these new ITZs far away from the old mortar's surface was not significantly improved as the nanomaterials or cement particles might not penetrate deeply.

The coarse aggregate properties before and after treatment using nanoslurry reported by the authors is shown in Table 8.12. It was reported that no significant difference in apparent density of RA was observed between treated and untreated RA, whereas a significant improvement in resistance of RA against crushing value and water absorption of RA after treatment with both the nanoslurries was reported. This may be due to the surface strengthening of RA_1 and RA_2 by the penetration of nanoparticles.

8.4 New Mixing Techniques

In Sects. 8.2 and 8.3, various quality improvement techniques of recycled aggregates recommended by different researchers have been discussed. In the subsequent sections, the new mixing techniques suggested by some of the researchers to enhance the properties of recycled aggregate concrete are discussed.

8.4.1 Two-Stage Mixing Approach

Tam et al. (2005) proposed a new method of mixing called two-stage mixing approach (TSMA), in which the whole mixing was separated into two parts and proportionately separates the required water into two parts which are added to the mixes at different timings, while the normal mixing approach (NMA) all the

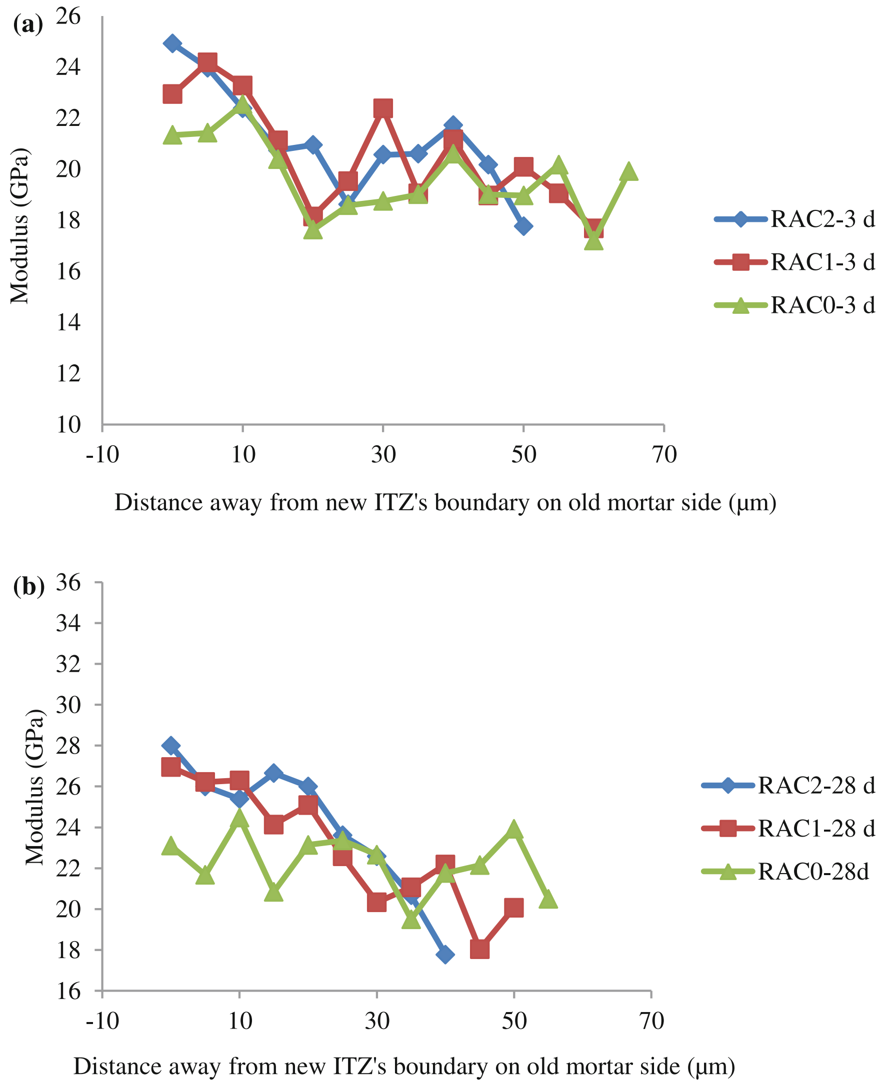

Fig. 8.25 Modulus development within new ITZ of three groups of RACs, in relation to distance away from old mortar's surface at age of **a** 3 days and **b** 28 days (Zhang et al. 2016)

Table 8.12 Coarse aggregate properties before and after treatment (Zhang et al. 2016)

Type of aggregate	Apparent density (kg/m^3)	Crushing value (%)	Water absorption (% by weight)
NA	2763	6.5	1.5
RA_0	2570	10.4	6.4
RA_1	2570	9.4	3.6
RA_2	2581	9.2	4.4

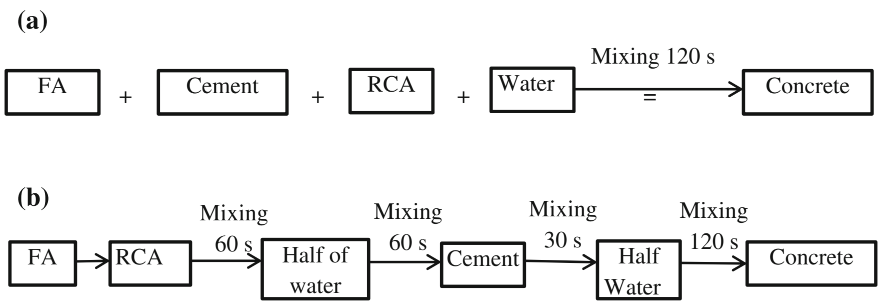

Fig. 8.26 Mixing procedures of **a** NMA and **b** TSMA (Tam et al. 2005)

Table 8.13 Compressive strength and percentage improvement in different proportions of RA using NMA and TSMA (Tam et al. 2005)

RCA (%)	Normal mixing approach (days)				Two-stage mixing approach (days)				Improvement percent (days)			
	7	14	28	56	7	14	28	56	7	14	28	56
0	43.87	53.01	55.72	67.6	45	54	56	68	2.59	1.86	0.51	0.59
10	50.29	54.53	58.98	74.6	54	61.4	64.5	79.2	7.41	12.62	9.41	6.18
15	45.14	51.72	56.26	70.19	49.6	55.7	61.3	72.4	9.83	7.67	8.88	3.15
20	42.21	51.92	53.68	68.84	45.1	56.6	65.1	72	6.96	9.02	21.19	4.64
25	51.09	52.62	52.31	67.23	53	57.1	63.1	77.7	3.82	8.44	20.64	15.61
30	45.49	54.58	58.07	72.78	54.8	60.6	66.2	77.5	20.46	11.05	13.94	6.44

ingredients of concrete and mix them. The procedures of NMA and TSMA explained in detail as schematically in Fig. 8.26.

The compressive strength of RAC prepared with different proportions of RA was investigated by using NMA and TSMA, and the results are presented in Table 8.13. It was found that a significant improvement in compressive strength of RAC at all proportions of RA in TSMA when compared to NMA. In TSMA, during the first stage of mixing, the use of half of the required water for mixing leads to the formation of a thin layer of cement slurry on the surface of RCA which permeates into the porous old cement mortar, filling up the old cracks and voids. In the second stage of mixing, the remaining water was added to complete the cement hydration process. This gives the access for gelatin the recycled aggregate with the cement slurry, providing a stronger ITZ by filling up the pores and cracks within the recycled aggregate. In SEM observations, it was found that the cracks and pores in RA get filled up after adopting TSMA (Fig. 8.27), whereas the similar cracks remain unfilled in RA in NMA (Fig. 8.28).

It was concluded the compressive strength can be enhanced in TSMA by developing the stronger interfacial transition zones in RAC. This was confirmed by the SEM images of old and new ITZs in TSMA and NMA (Figs. 8.29 and 8.30).

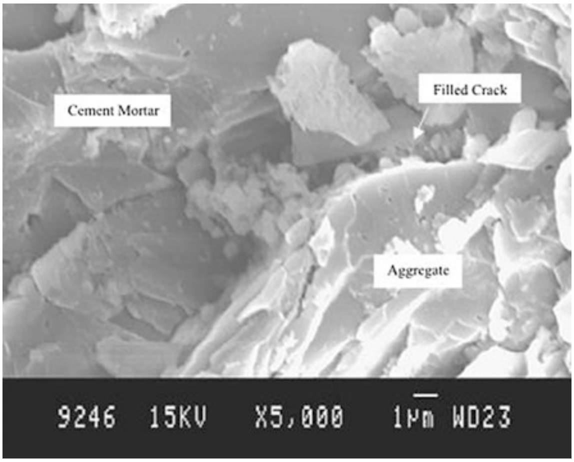

Fig. 8.27 Filled crack in RA using TSMA (Tam et al. 2005)

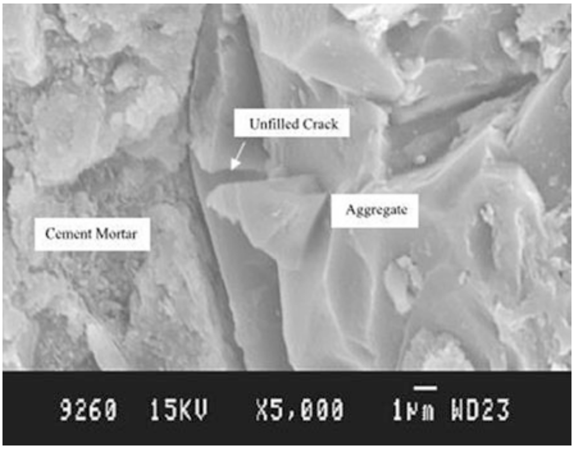

Fig. 8.28 Unfilled crack in RA using NMA (Tam et al. 2005)

Tam et al. (2007a, b) have further extended this technique to explore the possibility of the substitution of RA level ranging from 0 to 100% and compare their performance with traditional method of mixing. Based on the improvement in compressive strength, tensile strength, and modulus of elasticity and using regression analysis, it was found that 25–40% of RCA was most suitable in concrete by using TSMA with improvements of 12–18% at 56 days and 7 days curing and 50–70% of RCA also gives some improvement but less significant compared to 25–40% replacement. Similar improvements were observed in flexural strength and static modulus of elasticity.

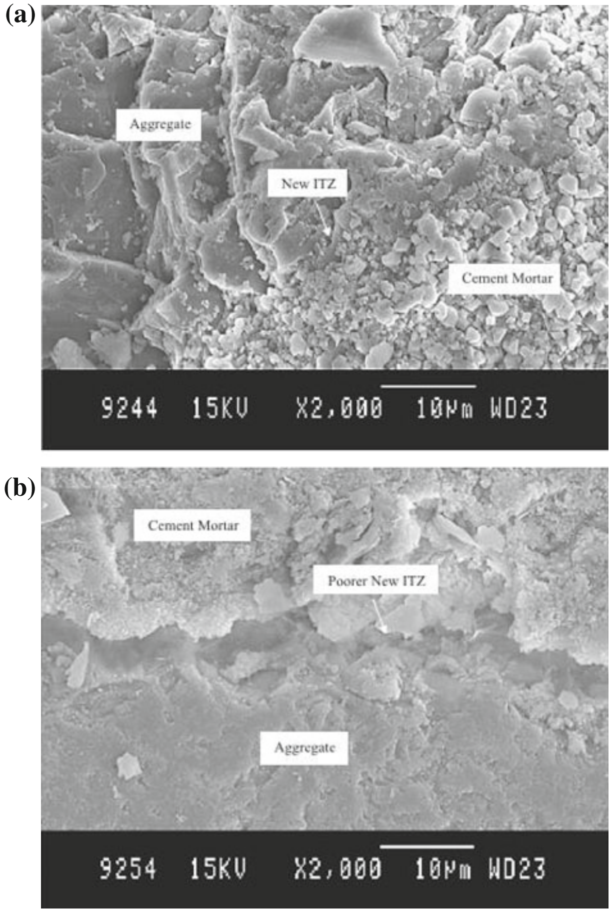

Fig. 8.29 **a** New ITZ in TSMA and **b** Poorer new ITZ in NMA (Tam et al. 2005)

8.4.2 Diversifying Two-Stage Mixing Approach

Tam and Tam (2008) have modified the TSMA developed by Tam et al. (2005) by adding silica fume, and silica fume and cement in the premixing stage are called diversifying two-stage mixing approach$_{\text{(silica fume)}}$ (TSMA$_{s}$) and diversifying two-stage mixing approach$_{\text{(silica fume and cement)}}$ (TSMA$_{sc}$) respectively.

Two-stage mixing approach $_{\text{(silica fume)}}$ (TSMA$_{s}$) In this method, silica fume was added by replacing 2% of the required cement into certain percentages of RA in the pre-mix stage. The remaining natural aggregate, fine aggregate, the remaining cement, and water were then added during the second mixing stage.

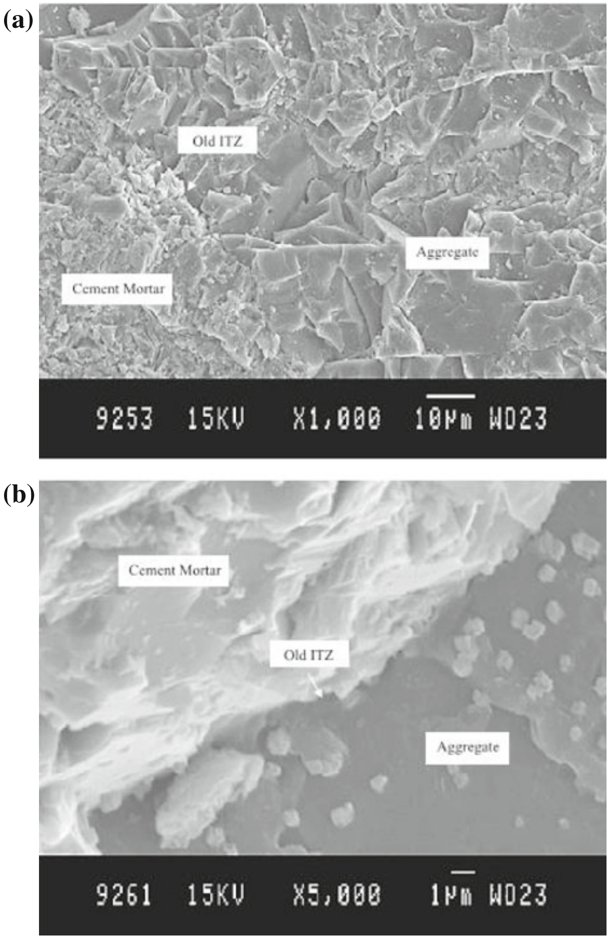

Fig. 8.30 **a** Old ITZ in TSMA and **b** Old ITZ in NMA (Tam et al. 2005)

Two-stage mixing approach$_{\textbf{(silica fume and cement)}}$ (TSMA$_{\textbf{sc}}$) This method following the similar procedure as adopted in TSMA$_{s,}$ cement was added proportionately to the percentage of RA used in addition to the silica fume in the pre-mix stage.

It was found from the test results of compressive strength, split tensile strength, modulus of elasticity and density, the modified two-staging approaches, i.e., TSMA$_s$ and TSMA$_{sc}$ can achieve a higher improvement when compared to TSMA and further TSMA$_{sc}$ had shown better improvement in the properties of RAC than TSMA$_s$. Hence, it was concluded that TSMAs and TSMA$_{sc}$ can be effective alternative methodologies for the improvement of the quality of RAC.

8.4.3 Self-healing

Elhakam et al. (2012) investigated the efficiency of this technique in enhancing the properties of recycled aggregate concrete. Self-healing process was achieved by immersing the recycled aggregates in water for 30 days. This period gives good chance to the unhydrated cement particles to react again with water to enhance the properties of concrete particles. The efficiency of this process to enhance the mechanical properties of hardened concrete was evident from Table 8.14.

The effect was more significant with the increase in age of concrete. The increase in compressive strength of RAC with 25 and 75% RCA and 250 kg/m^3 cement content at 56 days were 15.7 and 36.6%, respectively, as compared to concrete without self-healing. Similarly with 400 kg/m^3, the compressive strength of RAC with 25 and 75% RCA at 56 days, the increase was 9.4 and 24.3%, respectively, than concrete without self-healing. The beneficial effect of self-healing process was also clear on split tensile strength, bond strength, and porosity of RAC from the table. These improvements may be due to further hydration of the remaining unhydrated cement particles.

8.5 Summary

Various methods of enhancing the characteristics of recycled coarse aggregate and recycled aggregate concrete developed by different researchers are discussed. Further some of the new mixing techniques for the improvement of the quality of recycled aggregate concrete suggested by different researchers are also discussed. Based on these techniques, the following conclusions are drawn.

- All the five methods such as mechanical treatment, thermal treatment, acid treatment, three-step method, and ultrasonic cleaning were found to be efficient techniques to remove the adhered cement mortar from the surface of RA.
- A thermal treatment effectively removes the attached mortar from RA thereby significant improvement in properties when the temperature ranging from 350 to 450 °C. However, when RCA exposed to higher temperatures, yields degradation, mass loss, breakdown, and microcracking.
- The presoaking of RCA in different acids can effectively remove the large portion of adhered cement paste from the RA surface, which helps to improve the weak link between new cement mortar and RA. Nevertheless, the acid treatment slightly increases the chlorides and sulfates contents and lowers the pH but they were all within the limits specified by respective standards.
- A linear correlation exists between the adhered mortar content and acid concentration, which indicates that the adhered mortar loss was increased significantly with the increase in acid concentration. But higher concentrations yields the RA more porous and weaker than original RA as the acid not only remove the loose particles of old mortar but also eroded the bulk mortar.

Table 8.14 Properties of concrete (Elhakam et al. 2012)

Property	Cement content (kg/m^3)	Replacement ratio (%)	RAC without healing process			RAC after healing process			Improvement percent		
			7 days	28 days	56 days	7 days	28 days	56 days	7 days	28 days	56 days
Compressive strength (MPa)	250	25	21	26.7	28.1	22	25.3	32.5	4.8	5.2	15.7
		75	20	21.5	22.7	18.5	24	31	7.5	11.6	36.6
	400	25	32.2	38.3	41.7	31.9	39	45.6	0.9	1.8	9.4
		75	27.2	34.9	35.4	27.4	38	44	0.7	8.9	24.3
Tensile strength (MPa)	250	25	1.65	2.18	2.35	1.96	2.62	3.12	18.8	20.2	32.8
		75	0.98	1.41	1.78	1.27	2.12	2.39	29.6	50.4	34.3
	400	25	2.98	3.24	3.83	3.43	3.53	4.04	15.2	9.1	5.62
		75	1.59	2.53	2.82	2	2.9	3.06	25.8	14.6	8.5
Bond strength (MPa)	250	25	–	10	–	–	10.7	–	–	7	
		75	–	9.9	–	–	10.3	–	–	4	
	400	25	–	11.2	–	–	12.2	–	–	8.9	
		75	–	11.8	–	–	11.8	–	–	0	
Porosity (%)	250	25	–	–	15.1	–	–	14.5	–		4
		75	–	–	16.2	–	–	14.8	–		8.6
	400	25	–	–	13.15	–	–	12.7	–		3.4
		75	–	–	14.77	–	–	12.9	–		12.7

- Weak acids were more effective than strong acids to reduce the influence of acid attacks on RA surfaces.
- The combined effect of treatments: (i) weak acid treatment followed by mechanical treatment and (ii) thermal treatment followed by short mechanical treatment were more effective in enhancing the quality of RA than the treatment with each individual technique.
- The impregnation of RA in pozzolanic materials was found to be more effective in strengthening the ITZs in RAC due to their filler effect and the pozzolanic reaction between the silica fume and $Ca(OH)_2$ produce secondary C–S–H gel and hence significant improvement in compressive strength of RAC.
- SEPP was a new mixing technique without major changes in the production process and no additional increase in cost provides a thin film coated over the surface of RA by the pozzolanic powder which acts as a barrier to the inner bleeding of water and hence better workability and denser ITZ in RAC.
- Two-stage mixing approach and modified two-stage mixing approaches were other new mixing techniques which were also improve the properties of RAC considerably.

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Appendix A
Additional Results of Microstructure of Concrete

A.1 Introduction

Microstructure analysis was carried out on approximately 18–19 images from three samples of each mix. The average percentage area of residual cement, hydration compounds, and porosity of each mix is presented in Chapter 6. Also, the average percentage areas of the above features obtained from three images of each mix across the width of ITZ are discussed. The percentage areas of each component obtained from each image are presented in this Appendix Tables A.1, A.2, A.3, A.4, A.5, and A.6.

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Table A.1 Percentage areas of porosity, hydration compounds, and anhydrous cement

Hydration constituents	Image number																
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
M-RAC0																		
Pores	19.53		18.89		16.47	17.95	15.94	13.18	15.24	10.67	15.27	9.70	4.86	15.32	16.90	22.46	17.03	12.50
15.51	16.58																	
CSH	49.10		50.98		51.62	54.94	56.98	63.04	59.90	65.66	63.28	68.68	73.44	47.59	46.22	40.00	54.01	59.81
56.26	50.99																	
CH	22.14		22.64		18.11	15.42	14.04	20.04	21.20	20.24	19.13	18.14	19.14	26.25	17.94	12.43	15.36	26.99
14.96	17.12																	
UH	9.23		7.48		13.80	11.69	13.05	3.73	3.65	3.43	2.32	3.48	2.55	10.84	18.94	25.11	13.60	0.70
13.27	15.31																	
MM-RAC100																		
Pores	22.95		16.48		18.50	17.21	19.41	20.70	21.92	23.79	20.91	16.40	20.71	22.91	21.56	18.30	24.52	19.89
23.05	15.76																	
CSH	24.77		53.42		53.54	51.90	51.14	47.45	51.12	41.30	53.15	55.90	51.59	38.07	51.24	50.75	43.09	47.21
44.52	52.73																	
CH	29.79		20.94		22.34	21.26	13.63	20.14	10.37	19.43	24.55	23.46	22.73	33.23	15.65	19.83	22.32	19.46
16.63	19.94																	
UH	22.49		9.16		5.61	9.63	15.82	11.71	16.59	15.48	1.40	4.24	4.98	5.80	11.55	11.12	10.07	13.43
15.79	11.57																	
MK-RAC100																		
Pores	19.40		19.29		20.07	22.90	23.12	20.57	17.94	18.83	21.25	19.94	18.05	20.19	22.59	18.82	18.34	18.51
22.17	28.90																	
CSH	51.32		49.93		50.71	48.46	39.50	52.55	52.69	39.98	35.46	50.58	52.69	50.11	49.40	42.82	53.08	15.14
42.63	38.43																	
CH	15.25		20.67		14.11	13.68	24.01	16.19	19.01	29.03	28.80	16.19	17.15	16.65	12.61	31.26	21.26	51.77
20.91	16.07																	
UH	14.03		10.10		15.11	14.95	13.37	10.68	10.36	12.16	14.48	13.30	12.11	13.05	15.39	7.10	7.32	14.57

(continued)

Table A.1 (continued)

Hydration constituents	Image number																	
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
14.30	16.60																	
M-RAC0																		
Pores	10.19		17.70		13.33	22.43	19.99	15.87	15.61	17.54	18.45							
CSH	58.59		53.66		63.29	47.79	52.36	59.84	48.80	56.79	57.57							
CH	24.02		17.49		16.93	14.95	13.96	16.53	22.04	13.53	12.63							
UH	7.21		11.15		6.45	14.83	13.69	7.76	13.55	12.14	11.35							
MV-RAC100																		
Pores	20.86		19.45		21.77	21.03	18.25	27.14	22.28	22.44	22.03	20.95	20.40	20.19	21.06	20.21	20.39	18.88
19.54	22.01																	
CSH	45.92		54.49		50.17	53.87	51.33	41.18	51.16	47.59	52.67	56.05	56.84	52.88	25.20	51.61	55.91	51.36
52.68	47.65																	
CH	24.68		18.03		17.64	14.48	15.29	17.59	9.55	19.21	14.44	12.97	5.92	16.26	40.59	18.62	10.63	21.65
16.36	21.85																	
UH	8.53		8.04		10.41	10.63	15.12	14.09	17.01	10.77	10.87	10.03	16.84	10.67	13.16	9.56	13.07	8.11
11.42	8.50																	

Table A.2 Percentage areas of porosity, hydration compounds, and porosity across the ITZ in M-RAC0

Constituent (area percentage)	Distance from aggregate surface (μm)								
	10	20	30	40	50	60	70	80	100
Image 1									
Porosity	23.41	17.25	13.44	13.65	16.95	16.57	15.09	15.46	13.20
C–S–H	42.35	51.04	59.13	62.28	52.75	57.13	54.85	50.74	60.12
CH	32.14	30.45	24.51	18.93	25.76	21.90	21.67	24.17	17.60
UH	2.10	1.26	2.91	5.14	4.54	4.40	8.39	9.63	9.07
Image 2									
Porosity	24.04	19.81	21.29	20.38	20.51	19.32	19.38	14.82	16.71
C–S–H	43.40	47.52	43.91	46.39	47.88	49.05	48.89	61.23	57.38
CH	29.20	29.39	27.11	27.18	26.22	23.65	25.38	16.42	16.07
UH	3.36	3.27	7.69	6.05	5.40	7.98	6.35	7.53	9.84
Image 3									
Porosity	18.04	16.46	17.03	13.49	15.77	16.11	13.49	12.53	11.15
C-S-H	52.68	53.45	52.01	56.79	52.24	54.22	60.28	65.88	65.92
CH	27.57	23.25	24.14	24.20	24.35	21.80	19.23	13.31	14.11
UH	1.70	6.85	6.83	5.52	7.64	7.87	7.00	8.27	8.82
Image 4									
Porosity	22.17	18.39	19.91	19.56	21.59	19.15	17.66	16.63	14.89
C–S–H	43.20	57.62	56.03	55.76	52.27	57.42	60.21	63.39	62.88
CH	31.93	18.94	21.41	19.92	19.33	13.88	13.92	9.35	8.63
UH	2.70	5.06	2.66	4.76	6.81	9.55	8.21	10.63	13.60
Image 5									
Porosity	20.48	16.57	16.16	16.82	13.74	15.58	14.51	9.65	10.48
C–S–H	47.41	54.59	54.52	58.00	58.71	58.95	60.06	66.00	65.76
CH	30.29	26.28	23.51	20.42	20.30	19.92	17.08	15.03	13.16
UH	1.82	2.56	5.82	4.76	7.25	5.55	8.35	9.33	10.59

Table A.3 Percentage areas of porosity, hydration compounds, and porosity across the ITZ in MM-RAC100

Constituent (area percentage)	Distance from aggregate surface (μm)								
	10	20	30	40	50	60	70	80	100
Image 1									
Porosity	26.10	24.54	24.92	23.81	23.71	24.59	23.90	23.84	23.46
C–S–H	47.87	48.21	47.98	49.41	48.36	48.06	48.33	47.15	50.48
CH	23.46	21.84	18.59	16.76	16.52	15.69	13.33	14.50	13.02
UH	2.57	5.42	8.51	10.02	11.41	11.66	14.43	14.51	13.04
Image 2									
Porosity	26.68	24.79	21.27	20.35	19.76	19.77	15.16	14.32	19.18
C–S–H	42.47	44.53	49.84	53.44	53.95	54.60	60.55	60.05	57.59
CH	25.89	24.92	21.40	17.59	17.25	15.61	14.32	16.14	12.36
UH	4.96	5.76	7.49	8.62	9.04	10.03	9.97	9.48	10.87
Image 3									
Porosity	26.23	16.54	16.13	15.24	15.91	14.94	15.96	17.58	16.55
C-S-H	44.45	54.37	57.67	57.76	58.34	59.18	60.26	58.72	59.10
CH	26.85	23.78	22.00	19.08	17.83	18.19	16.26	15.66	16.08
UH	2.47	5.31	4.20	7.92	7.92	7.69	7.52	8.04	8.26

Table A.4 Percentage areas of porosity, hydration compounds, and porosity across the ITZ in MK-RAC100

Constituent (area percentage)	Distance from aggregate surface (μm)								
	10	20	30	40	50	60	70	80	100
Image 1									
Porosity	28.32	24.34	19.48	21.05	19.65	19.42	18.44	17.35	17.14
C–S–H	39.32	45.63	48.39	51.72	52.53	51.03	56.55	58.91	60.79
CH	25.55	24.06	25.80	20.47	20.31	20.75	14.94	12.84	12.40
UH	6.82	5.97	6.33	6.77	7.50	8.80	10.06	10.90	9.67
Image 2									
Porosity	26.17	25.18	19.63	19.93	16.47	14.90	14.19	13.96	14.35
C–S–H	45.25	47.25	53.09	52.11	53.55	57.50	60.71	61.98	58.60
CH	22.42	20.62	19.23	18.46	20.94	17.86	14.31	12.34	12.23
UH	6.16	6.95	8.05	9.49	9.05	9.74	10.79	11.72	14.81
Image 3									
Porosity	25.74	26.72	17.86	17.43	17.96	18.58	17.98	18.51	17.59
C-S-H	48.82	48.40	56.00	58.29	54.28	54.26	53.08	52.14	52.29
CH	18.01	18.05	17.75	15.64	18.97	19.13	18.39	18.16	19.59
UH	7.43	6.84	8.39	8.64	8.79	8.03	10.55	11.19	10.53

Table A.5 Percentage areas of porosity, hydration compounds, and porosity across the ITZ in M-RAC0 (*Source* 3)

Constituent (area percentage)	Distance from aggregate surface (μm)								
	10	20	30	40	50	60	70	80	100
Image 1									
Porosity	23.27	18.72	18.89	16.57	18.74	17.70	14.96	15.09	16.12
C–S–H	53.01	58.26	57.26	61.76	58.51	60.19	61.42	59.92	58.65
CH	20.01	18.70	18.06	17.17	16.47	16.44	16.52	16.16	15.12
UH	3.71	4.32	5.79	4.49	6.29	5.68	7.10	8.82	10.11
Image 2									
Porosity	22.14	18.81	17.54	12.31	11.97	13.24	12.47	12.32	14.94
C–S–H	51.97	54.33	59.84	63.39	64.58	61.67	63.48	61.43	61.06
CH	21.56	20.85	16.91	17.63	16.47	17.10	14.15	15.44	15.89
UH	4.33	6.02	5.71	6.67	6.98	7.99	9.90	10.82	8.12
Image 3									
Porosity	22.35	19.08	17.54	12.31	11.77	13.03	12.47	12.50	14.54
C-S-H	52.44	55.10	59.84	63.39	63.47	60.70	63.48	62.33	59.44
CH	21.76	21.15	16.91	17.63	16.18	16.82	14.15	15.66	15.47
UH	3.45	4.68	5.71	6.67	8.58	9.45	9.90	9.50	10.55

Table A.6 Percentage areas of porosity, hydration compounds, and porosity across the ITZ in MV-RAC100

Constituent (area percentage)	Distance from aggregate surface (μm)								
	10	20	30	40	50	60	70	80	100
Image 1									
Porosity	28.23	25.21	19.68	19.68	18.44	16.01	16.31	15.99	16.68
C–S–H	42.27	45.86	55.80	55.80	56.72	56.49	57.84	58.93	57.60
CH	22.76	21.91	19.55	19.55	16.31	18.76	16.84	14.59	15.15
UH	6.74	7.02	4.97	4.97	8.53	8.74	9.00	10.49	10.56
Image 2									
Porosity	28.16	25.18	21.93	18.78	17.90	18.43	18.40	15.31	16.04
C–S–H	42.38	46.80	48.20	51.16	54.68	54.45	57.06	63.95	63.39
CH	22.04	21.28	21.38	21.02	17.24	15.49	13.00	11.92	11.81
UH	7.42	6.74	8.49	9.04	10.18	11.63	11.53	8.83	8.75
Image 3									
Porosity	29.70	22.92	23.01	21.15	19.65	20.74	20.46	18.84	20.28
C-S-H	42.66	49.04	51.09	51.95	52.82	53.52	53.25	51.45	52.17
CH	20.75	20.00	16.78	16.47	15.88	14.31	14.95	16.63	16.88
UH	6.89	8.03	9.12	10.43	11.66	11.43	11.34	13.09	10.66