Chapter 5
Use of Construction and Demolition Waste as Aggregate: Properties of Concrete

5.1 Introduction

Studies on recycling of construction and demolition waste (CDW) as aggregate have been conducted for a long time. Nowadays, this material is considered as aggregate to produce several types of concrete. To use CDW as aggregate in concrete, it is necessary to process the waste to remove impurities and to comply with size grading requirements. The production process and properties of various CDW aggregates have been presented in Chap. 3. In this chapter, the fresh and hardened properties of concrete containing various types of CDW aggregates are discussed from existing literature data.

"CDW" is used here to indicate waste generated during construction and demolition activities. In several references RA is meant to represent recycled aggregate, which contains impurities like brick, wood, ceramics, asphalt and other materials, and RCA for aggregate represents waste generated from crushing concrete or that contains very small amounts of impurities (<1 %). A similar nomenclature will also be used in this chapter.

5.2 Fresh Concrete Properties

As the properties of CDW aggregate are different from those of conventional aggregate, the use of CDW aggregate in concrete substantially changes various properties of the concrete mix. In this section, fresh properties such as workability, density and air-content will be presented from various references.

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5.2.1 Workability

The workability of fresh concrete is a very important property, which controls various other fresh and hardened-state properties of concrete such as density, air-content and strength. The workability of concrete depends on various properties of its constituents. The workability performance of concrete containing CDW aggregate was studied extensively since various properties of CDW aggregate, which controls the workability of concrete, do not match those of natural aggregate (NA). The workability of concrete is determined by various methods; the most versatile one of which is slump.

The slump of concrete containing any type of CDW aggregate should be lower than that of conventional concrete due to the higher water absorption capacity of CDW aggregate than that of natural aggregate. The surface texture and angularity of CDW aggregate have also considerable influence on the workability performance of concrete (Buyle-Buddin and Zaharieva 2002).

Topçu (1997) observed 75 mm of slump of concrete containing coarse RCA generated from crushed concrete, while that of the equivalent conventional concrete was 100 mm. A lower workability of concrete containing CDW aggregate than of concrete containing natural aggregate was reported in several earlier publications and therefore additional amount of water was added to control the workability (Topçu 1997; Rasheeduzzafar and Khan 1984; Mukai and Kikuchi 1978; Buck 1977; Frondistou-Yannas 1977; Malhotra 1978; Hansen and Narud 1983).

Due to the higher water absorption capacity and consequent porosity of CDW aggregate by comparison with natural aggregate, in several studies, a water pre-saturation of the CDW aggregate was adopted before adding it to the mix. However, depending on the pre-saturation technique, the slump varies greatly. Table 5.1 shows some typical literature data on the slump performance of concrete containing pre-saturated CDW aggregate along with the water absorption capacity.

Poon et al. (2009) studied the slump performance of concrete mixes containing coarse granite aggregate as well as various amounts of coarse recycled concrete aggregate (RCA) at three different moisture states: air-dried (AD), oven dried at 105 °C for 24 h and then cooled down to room temperature prior to mixing (OD), and saturated surface dried (SSD). The water and aggregate contents were adjusted to keep the design proportions similar in all concrete mixes.

Due to the higher water absorption capacity of RCA at OD and AD states, higher amounts of water need to be added to the mix containing this aggregate. The initial slump (0 min) was measured immediately after mixing and then slump values were measured in 15 min intervals. The changes in slump of the mixes containing various types of aggregates are presented in Fig. 5.1. Their results can be summarised as follows: (1) the initial slump of the mix containing OD aggregate was higher than that of the other two types of aggregate; (2) the initial slump of mix containing OD and AD aggregates increased with their contents in the mix; (3) the higher slump of the mix containing OD aggregate or with higher

Table 5.1 Slump of concrete containing pre-saturated recycled aggregate

Reference	Type of aggregate (replacement amount)	w/c value	Slump (mm)	Water absorption capacity (%)	Pre-saturation method
Sagoe-Crentsil et al. 2001	Coarse basalt	0.76	90	1.0	Pre-saturated for 10 min before mixing
	Coarse recycled concrete (100 %)	0.73	75	5.6	
		0.74	95		
		0.70[a]	80		
Yang et al. 2011	Natural aggregate		33	1.4	Pre-soaked before mixing
	Coarse recycled concrete (100 %)		24	4.2 (RCA)	
	Coarse recycled concrete (80 %) + coarse crushed clay brick (20 %)		20	10.2 (CCB)	
	Coarse recycled concrete (50 %) + coarse crushed clay brick (50 %)		10		
Vieira et al. 2011	Natural aggregate	0.43	89 ± 2.8	1.0	10 min pre-saturation in water before mixing
	Recycled coarse aggregate (20 %, v/v)	0.44	91 ± 6	6.7	
	Recycled coarse aggregate (50 %, v/v)	0.46	88 ± 7.4		
	Recycled coarse aggregate (100 %, v/v)	0.49	82 ± 4		
Etxeberria et al. 2007a	Natural aggregate	0.50	80–100	0.88	Moistened by sprinkler system and covered by plastic for 1 day
	Recycled aggregate	0.40–0.50	80–100	4.44	
Gonzalez-Fonteboa et al. 2011	Natural aggregate	0.65	120	2.25 (average)	Pre-soaked in water for 10 min before use
	Coarse recycled aggregate (20 %, v/v)	0.66[b]	110	5.01	
	Coarse recycled aggregate (20 %, v/v)	0.68[b]	110		
	Coarse recycled aggregate (20 %, v/v)	0.68[b]	110		

[a] Higher cement content than the other mixes; [b] Estimated w/c

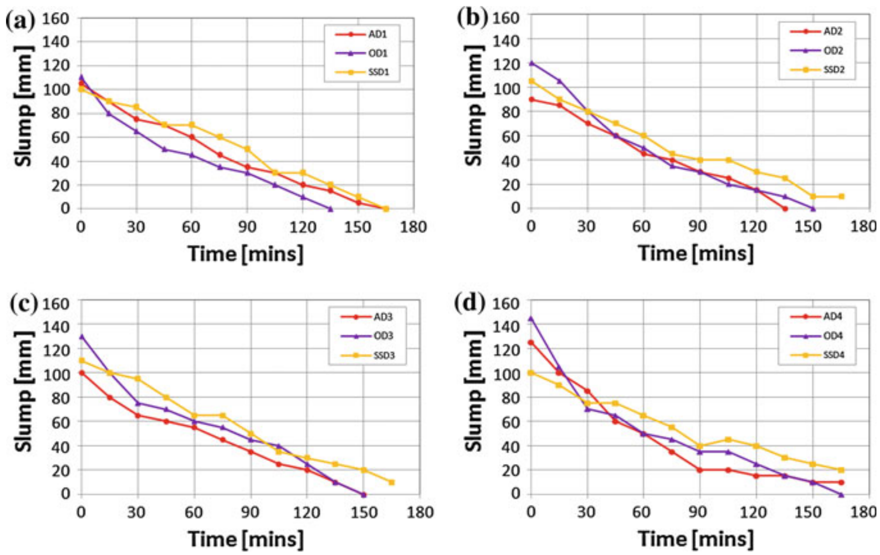


Fig. 5.1 Changes in slump of concrete containing various types of aggregate (Poon et al. 2009) **a** 100 % crushed granite, **b** 80 % crushed granite + 20 % recycled aggregate, **c** 50 % crushed granite + 50 % recycled aggregate, **d** 100 % recycled aggregate

content of OD and AD aggregates was due to the increase in free water in the mix; (4) the initial slump of the mix containing RCA in SSD state was almost constant for all replacement levels of natural aggregate; (5) the slump loss was faster and slower for mixes containing OD and SSD aggregates, respectively.

Poon et al. (2009, 2004a) also reported that the replacement of natural coarse aggregate by RCA in concrete prolonged the slump loss time. The slump value and the rate of slump loss with time of mixes containing RCA respectively increased and decreased with the addition of fly ash (Poon et al. 2004a). On the other hand, Thangchirapat et al. (2008) observed faster water loss in mixes containing coarse RCA aggregate than in the mix containing natural aggregate, when water was added to reach the required slump range of 50–100 mm. Sagoe-Crentsil et al. (2001) did not observe any difficulty in achieving the desired workability and subsequent compaction of concrete mixes containing RCA even though the amount of water absorbed by RCA was higher than for basalt aggregate.

Several mixing methodologies were developed to control the workability-related problems of concrete containing CDW aggregate. They can be summarised as: (1) increasing the amount of added water according to the water demand of concrete mixes containing dry CDW aggregate; (2) pre-soaking CDW aggregate in water for 10–20 min or for 24 h before use; (3) increasing the moisture content of CDW aggregate up to 70–80 % of total water absorption capacity for 24 h before followed by covering with plastic to control water loss due to evaporation; (4) increasing the super-plasticizer amount; (5) increasing cement content in the concrete composition.

Evangelista and de Brito (2007) adopted two different techniques to mix concrete, where fine natural aggregate was replaced by fine RCA. In the first technique, fine aggregates (recycled as well as natural) were mixed with water (2/3 of the required mixing water, plus the estimated absorbed water by fine aggregate) for 10 min before adding the other constituents. They increased the mixing time to 20 min without changing the remaining procedure in the second technique. However, they suggested the use of a superplasticizer to overcome the huge water demand of the mixes containing high amounts of RCA. Their results are presented in Table 5.2. Gonzalez-Fonteboa et al. (2011) pre-soaked the RA for 10 min before their use in concrete preparation to reach around 70 % of the RA's water absorption capacity. Padmini et al. (2009) also reported that the 10 min' water absorption value of CDW aggregate satisfied the desirable workability performance of concrete containing this aggregate. Etxeberria et al. (2007a) used partially saturated CDW aggregate to prepare concrete. The authors recommended 80 % humidity of the total water absorption capacity and therefore the aggregate was moistened by a sprinkler system 1 day before the preparation of concrete. The wet aggregate was then covered with a plastic sheet in order to maintain a high humidity level.

On the other hand, Gonzalez-Fonteboa and Martinez-Abella (2008) increased the cement content by 6.2 % in concrete prepared by replacing 50 % (by volume) of coarse natural aggregate with coarse recycled concrete aggregate keeping the slump similar to that of the control concrete. The average slump of 10 different concrete mixes containing recycled aggregate was 76 mm versus the average slump of 73 mm of control mixes. The water to cement ratio of both types of concrete remained constant at 0.55 (including the moisture content of both aggregates) and in both types of concrete around 1.2 % of superplasticizer was used. Limbachiya et al. (2000) changed the water to cement ratio by changing the water content, the cement content or both to obtain a mixes containing various amounts of coarse CDW aggregate with adequate fresh concrete properties. Etxeberria et al. (2007a) added higher amounts of superplasticizer to mixes containing CDW aggregate than to the control mix to guarantee constant slump and water to cement ratio.

The water to cement ratio and therefore the slump of fresh concrete is also dependent on the composition of CDW aggregate used to replace natural aggregate. Normally, the mortar and crushed brick contained in CDW aggregate absorb

Table 5.2 Effective and actual water to cement ratio of concrete containing recycled aggregate (Evangelista and de Brito 2007)

Constituents	Conventional concrete	Amount of recycled fine aggregate (%)				
		10	20	30	50	100
Cement (kg)	380	380	380	380	380	380
Water (kg)	155.8	160.6	165.4	170.2	175.6	180.9
Actual w/c	0.41	0.42	0.44	0.45	0.46	0.48
Effective w/c	0.41	0.42	0.43	0.44	0.45	0.45
Superplasticizer (g)	4.9	4.9	4.9	4.9	4.9	4.9

high amounts of water and therefore their content in the CDW aggregate control the slump of the resulting mix. Table 5.3 shows a typical water to cement ratio for concrete mixes with 80 ± 10 mm slump and containing coarse CDW aggregate from crushed concrete as well as a mixture of masonry brick and cement mortar (Gomes and de Brito 2009). The water absorption capacity of aggregate generated from crushed brick and mortar was considerably higher than that of the aggregate generated from crushed concrete and therefore the water to cement ratios of concrete mixes containing the former type of aggregate was also higher than that of the concrete mixes contacting the latter type of aggregate.

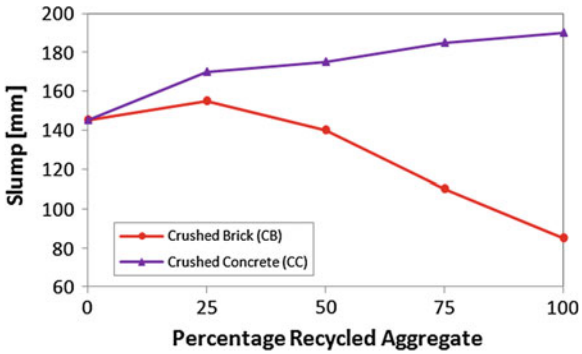
Yang et al. (2011) observed a reduction of about 27 % in slump due to the addition of pre-saturated recycled coarse concrete aggregate (RCA) as complete replacement of natural coarse aggregate (NCA). This reduction increased to about 40 % when the NCA was replaced by a mixture of 20 % recycled coarse crushed brick (CCB) and 80 % RCA. The percentage reduction in slump further rose to 70 %, when the NCA was replaced by a mixture of 50 % CCB and 50 % RCA. The reduction in slump was due to: (1) the presence of porous adhered mortar in RCA; (2) the generation of fine particles due to the relatively weak particles in RCA and particularly in CCB; (3) the decrease in density in the mix when 100 % of high-density natural coarse aggregate was replaced by low-density RCA and CCB particles. The fine RCA contains higher amounts of attached mortar than the coarse RCA and therefore a higher amount of water is needed to reach workability similar to that of the concrete mix containing coarse RCA (Buyle-Buddin and Zaharieva 2002).

Khatib (2005) reported an increase in slump due to the addition of crushed concrete (CC) as fine aggregate without using any type of admixture at free water to cement ratio of 0.5. In the case of fine aggregate generated from crushed brick (CB), the slump of the mixes increased up to a replacement ratio of 25 % by

Table 5.3 Variation of water to cement ratios of concrete mix prepared by different types of recycled aggregate considering water absorption capacity of recycled aggregate (Gomes and de Brito 2009)

Type of aggregate	Replacement ratio (%, v/v)	Water absorption capacity (%)	w/c	Effective w/c
Natural aggregate	0	2.29	0.430	0.43
Coarse recycled aggregate from concrete (1)	12.5	8.49	0.436	
	25		0.442	
	50		0.453	
	100		0.476	
Coarse recycled aggregate from mortar and bricks (2)	6.25	16.34	0.442	
	12.5		0.454	
	25		0.477	
	50		0.524	
Mixtures of (1 + 2)	12.5 (1) + 6.25 (2)		0.448	
	25 (1) + 12.5 (2)		0.465	
	50 (1) + 25 (2)		0.500	

Fig. 5.2 Slump of concrete containing two types of CDW aggregate (Khatib 2005)



weight of sand and then gradually decreased and, at 100 % replacement level, the slump of concrete mix was about 80 mm in comparison to 145 mm of the control mix. These results are presented in Fig. 5.2.

Li et al. (2009) observed higher slump in mixes containing coarse RCA coated with pozzolanic powder than in mixes containing coarse RCA prepared by conventional methods. The poor workability of the mix containing RCA due to the presence of attached mortar was significantly improved due to the surface coating of RCA by pozzolanic powder, and therefore reduced the amount of water absorbed by RCA.

Katz (2003) observed similar slump and comparable contents of free water in conventional concrete and mixes containing coarse recycled aggregate (RCA) from crushed concrete cured for 1, 7 and 28 days. However, due to insufficient content of fine aggregate in RCA, an extra amount of fine natural aggregate was added to the mix containing RCA to maintain workability and cohesivity. Kou et al. (2011a) reported that the addition of silica fume (SF) and metakaolin (MK) can reduce the slump of concrete mix containing RCA, which otherwise increases due to the addition of extra water to compensate the increased water requirement of RCA present in the mix. On the other hand, fly ash and granulated blast furnace slag addition to similar concrete containing RCA can further increase the slump. Tu et al. (2006) reported that the workability loss of high-performance concrete (HPC) containing RCA was much higher than that of conventional HPC. The RCA with higher water absorption capacity had a slight influence on the workability at the initial stages of mixing of HPC; however, the workability of the HPC mix deteriorated after 1 h of mixing as the added water amount was insufficient to satisfy the water demand of the mix.

5.2.2 Density

Fresh concrete density is the mass of fresh, normally compacted concrete, including its remaining voids per unit volume. This property depends on several others such as aggregate and cement types, water content and void content. The density of fresh concrete also controls various hardened-state properties of concrete. For example, given the same quantity of cement and aggregate, lower fresh concrete density indicates lower strength because the density decreases as the water and voids content increases.

The density of fresh concrete containing CDW aggregate is slightly lower than that of the mix containing natural aggregate since the density of CDW aggregate is lower than that of natural aggregate. The presence of lower density residual cement mortar particles attached to the aggregate is the main factor for lowering density of concrete due to the addition of CDW aggregate (Hansen and Narud 1983; Gonzalez-Fonteboa et al. 2011). Table 5.4 shows some typical values of the density of various concrete mixes containing natural as well as CDW aggregate along with the density of the aggregate.

Katz (2003) observed no significant difference in densities of concrete mixes containing RCA prepared from old concrete of three different ages (1, 7 and 28 days), which suggested that the amount of adhered mortar content in the various concrete aggregates was similar regardless of their crushing age (Table 5.4). Soutsos et al. (2011) observed marginally lower wet density in concrete containing coarse RCA than in concrete containing coarse limestone aggregate due to volumetric rather than weight-based substitution of coarse aggregate. Lopez-Gayarre et al. (2009) reported that the variations of properties of RCA have little effect on the resulting density of concrete. They observed a reduction of about 5 % in the density of concrete mix when all natural coarse aggregate was replaced by RCA. They used the analysis of variance (ANOVA) method to study the effects of various parameters of aggregate on the density behaviour, which is presented in Table 5.5.

5.2.3 Air-Content

The presence of a certain amount of air bubbles trapped during concrete mixing has several beneficial effects for fresh and hardened concrete properties. In fresh concrete, an air-content of around 3 % can reduce the water demand of concrete and make the mix stickier, which helps to reduce segregation and bleeding. However, if the air-content is higher than the specified amount, the increased stickiness makes concrete finishing more difficult. Thus the air-content of a concrete mix is an important property and a few studies are available on the behaviour of air-content of fresh concrete due to the incorporation of CDW aggregate. Table 5.6 shows the air-content of different concrete mixes containing natural as well as CDW aggregates of various types.

In some of the studies, it was reported that the addition of CDW aggregate to concrete increases the air-content (Katz 2003; Lopez-Gayarre et al. 2009). Lopez-Gayarre et al. (2009) observed an increase in the air-content of concrete with the RCA aggregate content, which was visible above 50 % replacement of aggregate

Table 5.4 Density of concrete mix containing pre-saturated recycled aggregate

Reference	Type of aggregate (replacement amount)	Density (kg/m^3)	Density/specific gravity[a] of aggregate (kg/m^3)
Sagoe-Crentsil et al. 2001	Coarse basalt	2,466	2,890 (bulk)
	Coarse recycled (100 %)	2,335 2,321[b] 2,335[c]	2,394 (bulk)
Gomez-Soberon (2002)	Natural (coarse + fine)	2,130	2593.3 (average)
	Recycled concrete (coarse + fine)	2,090	2236.7 (average)
Vieira et al. 2011	Natural	2413.5	2,600 (coarse)
	Recycled concrete (coarse, 20 %, v/v)	2392.3	2,400
	Recycled concrete (coarse, 50 %, v/v)	2355.0	
	Recycled concrete (coarse, 100 %, v/v)	2299.8	
Etxeberria et al. 2007a	Natural	2,420	2,670
	Recycled concrete (coarse, 25 %, v/v)	2,400	2,430
	Recycled concrete (coarse, 50 %, v/v)	2,390	
	Recycled concrete (coarse, 100 %, v/v)	2,340	
Gonzalez-Fonteboa et al. (2011[d] (MatStruc)	Natural	2,340 and 2,360	2,725
	Recycled concrete (coarse, 25 %, v/v)	2,320 and 2,330	2,400
	Recycled concrete (coarse, 50 %, v/v)	2,300 and 2,310	
	Recycled concrete (coarse, 100 %, v/v)	2,270 and 2,270	
Katz 2003	Natural	2,463	–
	Recycled concrete (coarse + fine)	2,175 (1-day[e]) 2,145 (7-day[e]) 2,156 (28-day[e])	2.23–2.59[a] 2.25–2.60[a] 2.23–2.55[a]
Buyle-Budin and Zaharieva 2002	Natural	2,360–2,410	2,600 (fine); 2,680 (coarse
	Recycled concrete (coarse + fine)	2,195–2,220	2,160 (fine); 2,250 (coarse)
Topçu and Sengel 2004	Natural	~2386	2,700
	Recycled concrete (coarse, 50 %, v/v)	2,301 (50 %)	2,470
	Recycled concrete (coarse, 100 %, v/v)	2,251	

[a] Specific gravity; [b] Binder is slag cement; [c] 5 % more cement was used; [d] w/c ratios: 0.65 and 0.50; [e] Crushing age of old concrete

Table 5.5 Influence of various parameters of RCA on concrete density (Lopez-Gayarre et al. 2009)

Parameters	Levels	Density		Parameters	Levels	Density	
		Influence	Variation (%)			Influence	Variation (%)
Type of aggregate	OV	XXX		Declassified content (%)	0		
	MA		1		5		
Replacement ratio (%)	0	XXX			10		
	20		−1.2	Base concrete (MPa)	35		
	50		−2.8		25		
	100		−5.2	Targeted slump (cm)	8	X	<1
Type of sieve curves	CF	X	−1		13		
	CC			Replacement criteria	SR		
	D				CR		<0.5

XXX Very influential; *X* Slightly influential; *OV* and *NA* RCA with different properties (*OV* has higher adhered mortar and declassified contents, los Angeles coefficient and water absorption capacity than *NA*); *CF* Fine sieve continuous curve; *CC* Coarse sieve continuous curve; *D* Discontinuous curve; *SR* Coarse natural aggregate replaced by identical volume of RCA; *CR* Fine natural aggregate replaced by identical volume of declassified content in RCA and natural coarse aggregate replaced by coarse fraction present in RCA

Table 5.6 Air-content of concrete mix containing pre-saturated recycled aggregate

Reference	Type of aggregate (replacement ratio)	Air-content (%)	Comment
Sagoe-Crentsil et al. (2001)	Coarse basalt	2.4	
	Coarse recycled (100 %)	2.4	
		1.8	slag cement as binder
		2.3	5 % more cement
Katz (2003)	Normal	1.3	White Portland cement as binder
	Recycled concrete (coarse + fine)	5.4 (1 day) 4.1 (7 day) 5.0 (28 day)	
	Recycled concrete (coarse + fine)	4.8 (1 day) 5.4 (7 day) 5.6 (28 day)	Ordinary Portland cement as binder
Rustom et al. (2007)	Normal	1.0–2.0	Ordinary Portland cement as binder
	Recycled concrete	1.8–3.3	

(Fig. 5.3). Their results also indicated lower air-content in the mix containing better quality RCA aggregate (MA) than that of inferior one (OV). The quality of the two aggregates used in this investigation is presented in Table 5.7.

Katz (2003) observed higher air-content in concrete containing RCA aggregate than in concrete containing natural aggregate, when the air-content was measured by a gravimetric method. The air-content of the aggregate was also measured in the method. However, the air-content of concrete containing RCA was not affected by the crushing age of the original hardened concrete. On the other hand,

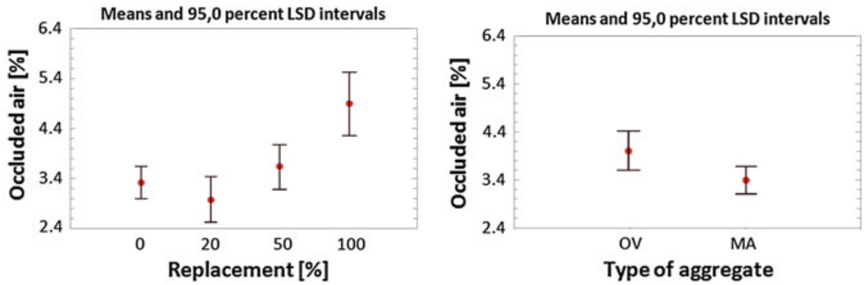


Fig. 5.3 Effect of replacement ratio (%) and type of aggregate on the air-content of concrete containing CDW aggregate

Table 5.7 Properties of two types of RCA aggregate (Lopez-Gayarre et al. 2009)

Aggregate name	Size (mm)	Dry density (kg/m^3)	24 h water absorption (%)	Los Angeles coefficient (%)	Attached mortar content (%)	Declassified content (%)
OV	4–20	2,200	5.0	37.2	34.2	2.6
MA	4–20	2,360	3.8	33.1	23.0	1.5

Sagoe-Crentsil et al. (2001) observed no difference in air-content between concrete containing normal aggregate and concrete containing RCA aggregate.

5.2.4 *Bleeding*

Bleeding of concrete is the upward movement of water during settling of concrete mix. It is a particular form of segregation, in which some of the water within concrete comes out of its surface. Sometimes, with this water along comes a certain quantity of cement. A higher bleeding to the surface increases the water to cement ratio and therefore decreases the strength of the concrete surface. The increase in capillary porosity of hardened concrete due to bleeding can also affect the durability performance. However, in some cases, the bleeding water may not come up to the surface and is trapped by flat or flaky pieces of aggregates and also by reinforcement and accumulates below such obstacles. This is known as internal bleeding, which can affect several properties of hardened concrete such as cement paste-aggregate bonding and enhance micro cracks. High bleeding of concrete occurs due to various factors such as high water to cement ratio, low cement content, coarse cement particles and poorly graded aggregates. Few references are available on the bleeding behaviour of concrete containing CDW aggregate.

The old cement paste on the surface of the recycled aggregate can absorb some mixing water. Thus, the total amount of bleeding of concrete decreases as the replacement level of recycled coarse aggregate increases (Kim et al. 1993). Yang et al. (2008) also found a reduction in total bleeding due to the replacement of 100 % coarse NA by coarse RCA and 100 % fine NA by fine RCA. Bleeding of concrete containing fine RCA was lower than that of concrete containing coarse RCA. Total bleeding and rate of bleeding of RCA concrete decreased as the relative water absorption by the aggregate increased.

Poon et al. (2007) reported the bleeding performance of concrete containing coarse air-dried RCA, where 0, 20, 50, 80 and 100 % by mass of natural aggregate was replaced by RCA. The bleeding of concrete was measured in terms of bleeding rate (defined as the amount of water in ml collected per cm^2 surface area of concrete per second during the first 60 min of the test) and bleeding capacity (defined as the total volume of bleeding water collected during the entire course of the experiment and expressed as a fraction of the initial volume of concrete).

Their results showed that adding air-dried RCA to concrete increases its bleeding rate and bleeding capacity and this trend rises as the content of RCA in concrete increases. The use of 100 % RCA increased the bleeding rate and bleeding capacity by 26 and 22 %, respectively. On the other hand, delaying the starting time of the experiment or adding fly ash to cement can reduce the bleeding rate and bleeding capacity of concrete with natural or RCA aggregates. These results are presented in Table 5.8. Poon et al. (2004a) also reported that over-wetting of RCA should be avoided to reduce concrete bleeding, which also had some effect on the hardened concrete properties.

Table 5.8 Bleeding capacity and bleeding rate of concrete due to the addition of RCA (Poon et al. 2007)

Concrete mix	Immediately after mixing		30 min after mixing		120 min after mixing	
	Bleeding capacity, 10^{-3}, ml/ml	Bleeding rate, 10^{-6}, ml/cm^2.s	Bleeding capacity, 10^{-3}, ml/ml	Bleeding rate, 10^{-6}, ml/cm^2.s	Bleeding capacity, 10^{-3}, ml/ml	Bleeding rate, 10^{-6}, ml/cm^2.s
R0	18.8	47.9	13.2	19.6	5.2	9.6
R0F25	16.2	43.5	9.6	17.4	4.6	6.5
R20	19.9	50.9	14.2	20.5	5.4	10.0
R20F25	16.9	45.7	10.4	18.3	4.8	6.7
R50	21.2	53.6	15.2	20.9	5.6	10.4
R50F25	18.1	49.2	10.8	20.5	5.0	7.1
R80	22.6	56.6	16.2	21.3	5.8	10.5
R80F25	19.5	52.3	11.4	21.8	5.1	7.3
R100	23.0	60.1	17.1	22.2	6.1	10.5
R100F25	20.0	54.5	12.0	22.7	5.4	7.8

R0, R20, R50, R80 and R100: concrete mixes containing ordinary Portland cement (OPC) and prepared by replacing 0, 20, 50, 80 and 100 % by mass of natural aggregate by RCA
R0F25, R20F25, R50F25, R80F25 and R100F25: concrete mixes containing a blended cement with 75 % OPC plus 25 % fly ash (FA) and prepared by replacing 0, 20, 50, 80 and 100 % by mass of natural aggregate by RCA

5.3 Hardened Concrete Properties

As the properties of CDW aggregate are significantly different from those of NA, the various hardened properties of conventional concrete change with the addition of CDW aggregate. In this section, the hardened properties of concrete containing various types of CDW aggregate are discussed from information available in various references. As indicated in the introductory section, a similar terminology (RA and RCA) will be used to indicate recycled aggregate with different origin.

5.3.1 Compressive Strength

In this section, the compressive strength (CS) of concrete containing CDW aggregate (both RA and RCA) is highlighted. Results will be analysed in terms of size and composition of CDW aggregate. The relationship with fresh properties is also presented.

Normally, the CS of concrete decreases with the addition of CDW aggregate (Oliveira and Vazquez 1996; Dhir et al. 1999; Topçu and Sengal 2004) and the reduction in strength can reach 40 % (Katz 2003; Chen et al. 2003). In some studies, it was pointed out that the reduction in CS was between 12 and 25 %, when 25–30 % (Corinaldesi 2011; Etxeberria et al. 2007a) or 100 % NA was replaced by CDW aggregate (Li et al. 2009; Rahal 2007; Safiuddin et al. 2011). However, a negligible influence is observed when the coarse or fine recycled aggregate is used to replace up to 30 % of coarse NA (Gomez-Soberon 2002; Li et al. 2009; Limbachiya et al. 2004, 2012; Rao et al. 2011; Yang et al. 2011) or 20 % of fine NA (Dhir et al. 1999), respectively. Some typical results are presented in Table 5.9. An increase in concrete porosity due to the addition of CDW aggregate (due to old mortar content) and weak aggregate-matrix interface bond are the major reasons for the reduction in CS of CDW aggregate concrete (Kwan et al. 2012).

The reduction in CS due to the addition of CDW aggregate can be controlled by changing various factors of the concrete mix such as adjusting the water to cement ratio, changing the mixing procedure, treating the aggregate and using a mineral addition. The information gathered so far on the use of CDW aggregate in concrete shows that modifications in the concrete mixing procedure are the key step to develop a good quality concrete containing any type of CDW aggregate.

Etxeberria et al. (2007a) observed that the 28-day CS of concrete made with 100 % coarse RA (RAC) was 20–25 % lower than that of conventional concrete. A similar trend was reported by other researchers (Gonzalez-Fonteboa and Martinez-Abella 2005; Sani et al. 2005). Sani et al. (2005) observed about 40 % lower 90-day CS in RAC than in conventional concrete due to the incorporation of RA as complete replacement of coarse and fine aggregates. The mix was prepared with the same cement content and water to cement ratio without using any water-reducing

Table 5.9 Reduction in CS (%) due to the addition of CDW aggregate

References	Type of aggregate	Reduction in strength, (substitution level %)	Comment
Oliveira and Vasquez (1996)	Coarse/RCA	10	Varied depending on moisture content in RCA aggregate
Topçu and Sengel (2004)	Coarse/RCA	23.5	C16 type concrete
		33	C20 type concrete
Sani et al. (2005)	Coarse/fine/RA	40	At constant water to cement ratio
Gomez-Soberon (2002)	Coarse/RA/10 min pre-soaked in water	~2, (15) ~ 5, (30) ~11.5, (100)	Substitution level (v/v): 15, 30 and 100 %
Kou and Poon (2008)	Coarse/RCA/SSD[a]	4–6, (20) 13–17, (50) 16–22, (100)	Substitution level (v/v): 20, 50 and 100 %; strength decreasing with increasing mortar content;
Khatib (2005)	Fine/RCA	~24.5–25, (25–75) ~36, (100)	Substitution level (w/w): 25, 50, 70 and 100 %; prepared at free water to cement ratio of 0.5
Yang et al. (2011)	Coarse/RCA	5.7, (100)	Prepared at w/c of 0.47 with a slump of 24 mm

[a] *SSD* Saturated surface dry

admixture. They also observed an inverse relationship between CS of RAC and open porosity of RA. The addition of fly ash to replace a part of fine RA can recover part of the CS and therefore in this case a reduction of about 30 % in CS was observed. Dapena et al. (2011) did not observe any change in 28-day CS of resulting concrete due to 20, 50 and 100 % (by volume) replacement of coarse NA by coarse RA owing to the small amount of impurities in RA (<2 %) and an initial reduction of water to cement ratio of RAC due to higher water absorption capacity of RA. The 5 and 10 % (by volume) of fine (<4 mm) content in RCA did not affect the strength behaviour of concrete containing 20 and 50 % RCA; however, a drop of around 3.6 MPa in CS was observed in concrete containing 100 % RCA. Poon et al. (2009) observed a reduction in compressive strength of RAC containing RA as the only coarse aggregate as the replacement of fine natural aggregate by fine RA increased.

Etxeberria et al. (2007a) reported that concrete prepared by replacing 25 % (v/v) coarse NA by coarse RA can be used as medium strength (30–45 MPa) concrete having similar CS to conventional concrete, when both mixes are prepared with the same cement content and effective water to cement ratio of 0.55. However, a reduction of effective water to cement ratio of 0.52 or an increase of 6 % in cement content was necessary for RAC containing 50 % RA to achieve a CS similar to that of conventional concrete. These values became 0.50 and 8.3 %, respectively, for RCAC containing 100 % RCA. Gonzalez-Fonteboa and Martinez-Abella (2005) also increased the cement content by about 6.2 % without changing the w/c (including moisture content in aggregate before mixing) in RAC where 50 % (by volume) of natural coarse aggregates were replaced to obtain CS and consistency similar to those observed in a conventional concrete. They observed CS about 2.5 and 0.4 % lower and 2 % higher for RAC than for conventional concrete after 7, 28 and 115 days of curing, respectively.

The detrimental factors that control the CS of RAC are the crushing strength and mortar content in RA (Etxeberria et al. 2007a). Gonzalez-Fonteboa and Martinez-Abella (2005) specified three reasons, which controlled the CS performance of RAC. The higher absorption of water by RA than NA reduced the amount of free water in the mix. These decrease up to a certain level could increase the CS; however, an excessive decrease in free water can deteriorate the CS of RAC due to lesser hydration of cement particles and poor workability of the mix than in conventional concrete. The weak bond of cement paste and RA also lowers the CS of RAC. The addition of an extra amount of cement to the mix boosts the CS by improving bond between the cement paste and the RA and improves porosity and consistency.

The strength development of RAC was faster than for conventional concrete after 28 days of curing (Gonzalez-Fonteboa and Martinez-Abella 2005). Etxeberria et al. (2007a) observed gain of about 12–15 % in CS between 7 and 28 days of curing of RAC prepared by replacing 25, 50 and 100 % (by volume) of NA by RA in comparison to around 20 % in conventional concrete.

Compared to the studies on the use of RA in concrete, more information is available on the use of RCA in preparation of concrete. Similarly to RA, the CS of concrete containing RCA (RCAC) is normally lower than that of the corresponding control concrete and increasing the addition of RCA to concrete further lowers it (Table 5.8). However, RCA addition to concrete does not have an adverse effect on its strength development trend.

Frondistou-Yannas (1977) reported that the substitution of natural gravel by RCA and recycled aggregate that contained mortar only led to a lower CS than that of conventional concrete. The failure observed in both types of concrete was by fracture in the aggregate. Eguchi et al. (2007) observed higher reduction in CS of concrete due to the addition of RCA originated from a concrete which consisted of low strength aggregate than that observed for RCA originated from a concrete consisted of high-strength aggregate. Lopez-Gayarre et al. (2009) also observed a strong dependence of CS of RCAC on the quality of RCA, mainly concerning the adhered mortar content. However, they did not observe any effect of increased addition of coarse RCA on the mean CS of RCAC if the water to cement ratio was kept constant and the loss of workability due to addition of RCA was compensated by using chemical admixtures.

Santos et al. (2002) observed a reduction of about 20 % in 7- and 28-day CS of two types of RCAC from the corresponding strength of conventional concrete, when both types of RCAC were prepared with coarse RCA originated from two concrete mixes with different strengths. Gomez-Soberon (2002) observed that the CS of concrete decreased with increasing content of coarse RCA. The 28-day CS of concrete prepared by replacing coarse NA with 15, 30, 60 and 100 % coarse RCA were respectively about 98, 95, 92 and 88 % that of conventional concrete. After 90 days of curing, these values became 99, 94, 91, 89 %, indicating a similar development of the CS of RCA concrete to that of conventional concrete as curing time increased. Topçu and Sengel (2004) observed a systematic reduction in cubic and cylindrical CS of 16 and 20 MPa RCAC with increasing content of coarse RCA. Compared to a control concrete, the reduction in 28-day CS was 33 and 23.5 % for 16 and 20 MPa RCAC when 100 % NA by volume was replaced by RCA.

Rao et al. (2011) observed higher early strength gain (0–7 days) in RCAC than in conventional concrete due to the high water absorption capacity of old mortar present in RCA and the rough texture of RCA that provides improved bond and interlocking characteristics between mortar and RCA. However, they also observed 8 % gain in CS between 28 and 90 days of curing for mixes prepared by replacing 25 % (by volume) of natural coarse aggregate by coarse RCA compared with 12 % gain for conventional concrete and did not observe any gain in CS when the replacement ratio increased to 50 and 100 %. Just like Xiao et al. (2006a), they observed a linear relationship between RCAC's density and CS but with a different slope. Fonseca et al. (2011) also observed about 80 and 95 % of 56-day CS for normal aggregate and RCA mixes after 7 and 28 days of curing, respectively. Safiuddin et al. (2011) observed more than 80 % of 28-day CS after 7-day curing of conventional concrete and concrete containing various ratios of coarse RCA due to the higher cement content than in conventional concrete as well as the higher workability of the concrete mixes.

In contrast with the majority of the observations, some results are available where the addition of coarse RCA to concrete does not have adverse effect on the CS performance. On the contrary, this type of addition can increase the CS of concrete if some modification is done by improving concrete mixing methodology. For example, Sagoe-Crentsil et al. (2001) observed no difference in 28-day and 1-year CS of mixes containing pre-saturated RCA and NA as coarse aggregate (Fig. 5.4). However a 5 % increase in cement content or the use of slag cement in RCA concrete considerably increased the CS, which was particularly more significant when using slag cement in RCA concrete. Domingo-Cabo et al. (2010) observed an increase in CS as the replacement of NA by RCA increased, possibly due to the reduction in effective water to cement ratio on account of the higher

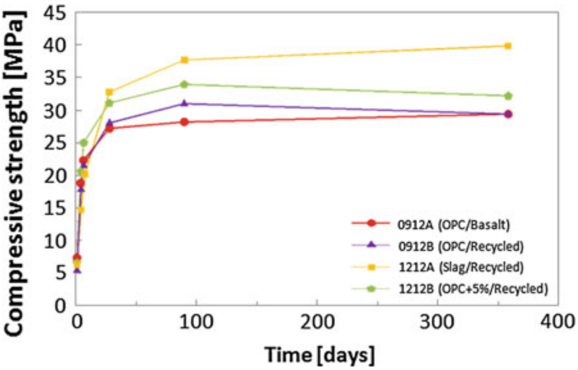


Fig. 5.4 CS of concrete containing RCA (Sagoe-Crentsil et al. 2001)

water absorption capacity of RCA. Evangelista and de Brito (2007) also observed higher CS of concrete incorporating fine RCA, which will be discussed later.

Some studies indicated a relationship between aggregate to cement ratio and the CS of RCAC (Fig. 5.5): decreasing the aggregate to cement ratio is beneficial for the CS of RCAC (Poon and Chan 2007; Poon and Lam 2008). The authors pointed out that the low crushing strength of the aggregate as well as the weak cement paste-aggregate bond are the causes of these results. In fact, the cement content in concrete plays a vital role in the CS (and other mechanical) behaviour of RCAC. A typical example is presented in Fig. 5.6 (Courad et al. 2010).

Similarly to conventional concrete, a linear relationship between water to binder ratio and CS also exists in RCAC (Corinaldesi 2010; Nagataki and Iida 2001). Nagataki and Iida (2001) observed this relationship for RCAC up to a binder to water ratio of 2.5–3.3 depending on the strength of the source concrete from which RCA originates (Fig. 5.7). Kou and Poon (2006) observed a decrease in CS as the content of coarse RCA in concrete increases at water to cement ratios of 0.45 and 0.55. However, the CS of the control concrete and RCAC increased with curing time and decreased with water to cement ratio. However, in a few studies, it was pointed out that the strength characteristics of RCAC were not significantly affected by the quality of RAC at high water to cement ratio, since it was affected only when the water to cement ratio was low (Ryu 2002; Padmini et al. 2002).

Chen et al. (2003) observed much lower differences between the CS of conventional concrete and that of RAC, when the w/c ratio increased to a certain value and they became lowest for w/c values between 0.58 and 0.80 (Fig. 5.8). Katz (2003) observed that the CS of RCAC was comparable to that of reference concrete up to the replacement level of 75 % at a w/c ratio between 0.6 and 0.75.

Padmini et al. (2009) specified several factors that can affect the CS performance of RCAC: to achieve the design CS RCAC required lower w/c ratio and higher cement content than concrete containing granite aggregate; for a targeted mean CS, the actual strength of RCAC increased with the maximum size of RCA used as the maximum size of RCA in concrete decreased and the strength of the source concrete increased.

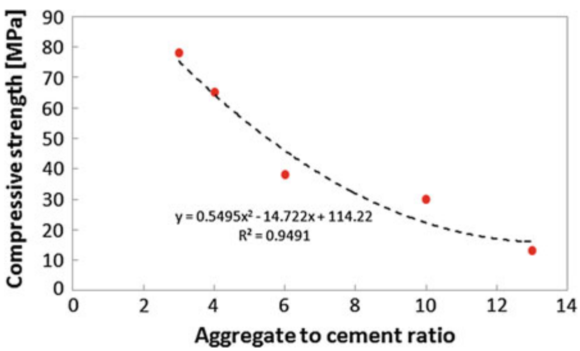


Fig. 5.5 Relationship between aggregate to cement ratio and CS of RCAC (Poon and Lam 2008)

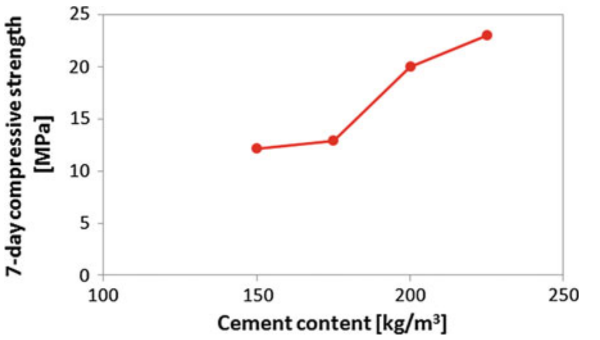


Fig. 5.6 Cement content and CS of roller compacted RCAC relationship (Courad et al. 2010)

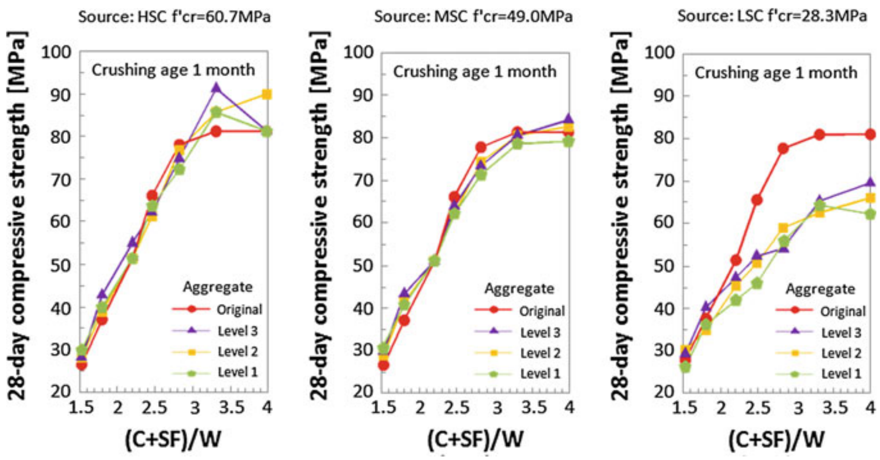
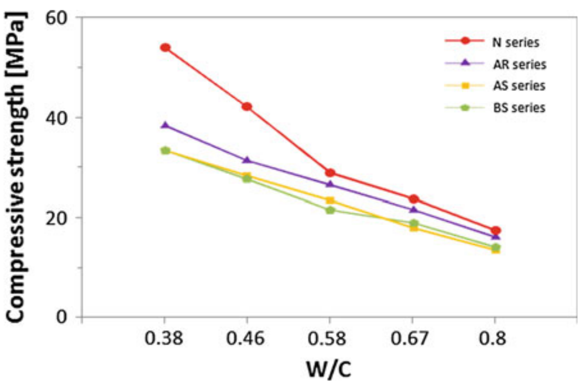


Fig. 5.7 Binder to water ratio *versus* CS (Nagataki and Iida 2001)

Fig. 5.8 Relationship between water to cement ratio and CS of RCAC (Chen et al. 2003)



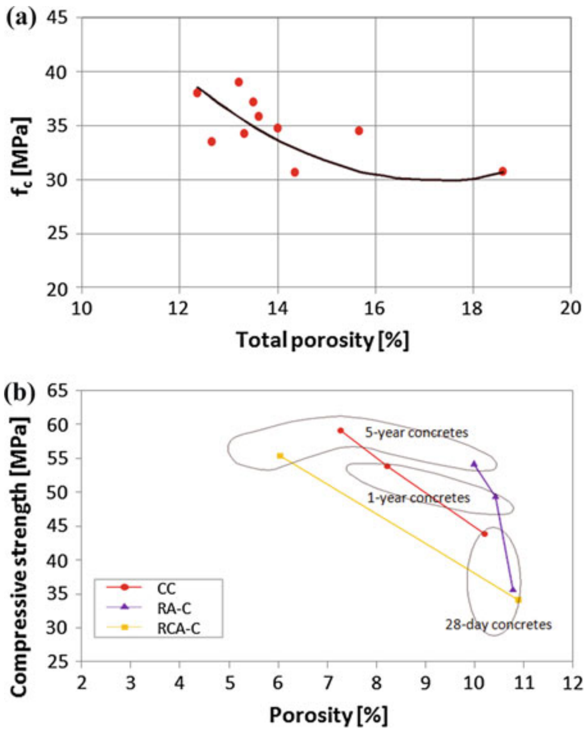


Fig. 5.9 Porosity versus CS of RCAC. **a** Gomez-Soberon (2002), **b** Kou et al. (2011a)

The CS of RCAC decreased as the porosity of concrete containing coarse RCA increased (Fig. 5.9a) (Gomez-Soberon 2002; Kou et al. 2011a). Kou et al. (2011a) observed significant differences in the relationship between the porosity and the CS of RCAC and RAC after 5 years of curing (Fig. 5.9b). The higher improvement of porosity (thus compressive strength) in RCAC than in RAC was due to the filling of pores and therefore improvement in interfacial zone due to continuous hydration of old cement mortar. Park et al. (2005) observed decreasing CS with increasing void ratio of porous concrete where various amounts of coarse NA were replaced by RCA. The CS decreased rapidly above the void ratio of 25 %. In contrast to the above results, Nagataki et al. (2004) observed comparable or even better CS (and splitting tensile strength) of concrete containing coarse high quality RCA than that of conventional concrete even though the RCAC had 20–52 % more permeable voids than conventional concrete.

The moisture content in RCA also controls the CS of RCAC (Oliveira and Vazquez 1996). The CS of RCAC with about 90 % saturated coarse RCA was marginally better than that for dry coarse RCA. The decrease of CS was especially felt when the RCAC contained 100 % saturated coarse RCA (Fig. 5.10). The compressive strength of RCAC containing RCA at three moisture levels as observed in Poon et al. (2004b) study can be rated as: air-dried (AD) > oven dried (OD) > saturated surface dry (SSD). The relatively high w/c ratio in the RCA-

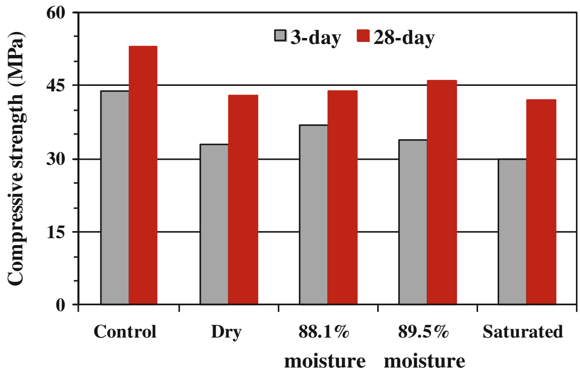


Fig. 5.10 Effect of moisture content in RCA on CS (Oliveira and Vazquez 1996)

cement matrix region due to movement of water from SSD aggregate weakens the cement paste/aggregate bond and lowers the compressive strength. On the other hand, the opposite movement of water from cement matrix to aggregate strengthens the bond and led to higher CS of concrete containing OD RCA.

Concrete containing larger RCA has higher CS than similar concrete containing smaller RCA as the mortar content in larger RCA is generally lower than in smaller RCA (Tavakoli and Soroushian 1996; Hansen and Narud 1983). Corinaldesi (2010) reported that the CS of RCAC containing larger coarse RCA is higher than that for smaller coarse RCA at the same water to cement ratio and at 30 % (by volume) substitution ratio of natural coarse aggregate (Fig. 5.11). Both types of aggregate were generated in same crushing plant at the same crushing period. According to the authors, the coarser RCA came from concrete with a higher CS and hence less friability than the other concrete, which generated the finer RCA. In this study, two classes of concrete (C30/37, C32/42) were prepared from coarse and fine RCA aggregate at the water to cement ratios of 0.5 and 0.4, respectively. Nagataki et al. (2004) observed the dependence of mechanical properties including CS on the size of coarse aggregate of the original concrete as well as on the amount of sand particles present in RCA. The stress concentration at the zone between RCA and mortar in RCAC is lower for smaller coarse aggregate. Low amounts of sand particles in RCA also enhance the CS of RCAC.

A strong relationship between the CS of RCAC and the properties of the source concrete from which RCA originated was reported in various studies. Hansen and Boegh (1985) observed that the 47-day CS of high and medium strength concrete containing RCA generated from high and medium strength concrete were higher than that of the original high and medium strength concrete; however, the CS of low strength concrete containing RCA originated from medium and low strength concrete was lower than that of conventional low strength concrete. Their results are presented in Fig. 5.12.

Tavakoli and Soroushian (1996) reported that the CS of concrete containing RCA depends on several factor such as the strength of the source concrete from which RCA is generated, the mixing procedure, the water to cement ratio and the

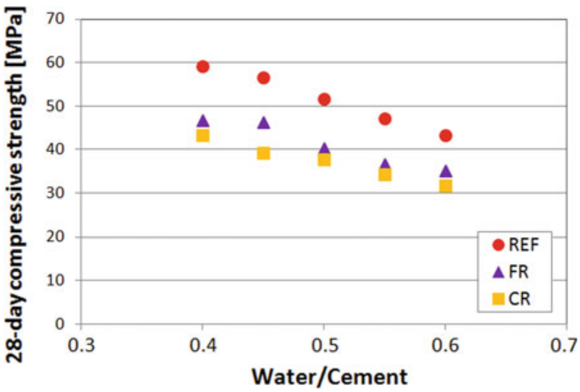


Fig. 5.11 CS of concrete containing coarse and fine coarse recycled aggregates and of reference concrete (Corinaldesi 2010)

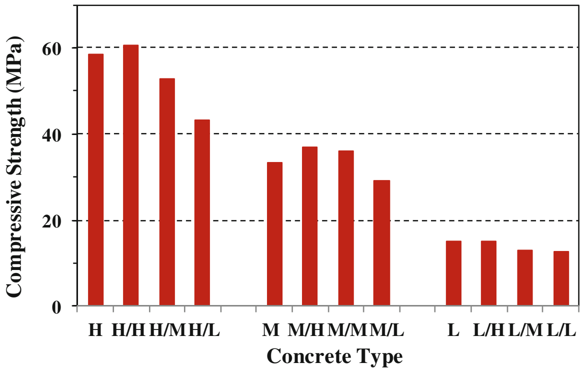


Fig. 5.12 CS of concrete (H, M, L: Original high, medium and low strength concrete; H/H, M/L: high and medium strength concrete containing RCA produced from H and L concrete, respectively, etc.) (Hansen and Boegh 1985)

aggregate size. They observed higher CS in concrete containing RCA than in concrete containing NA provided that the CS of concrete from which RCA was generated that was higher than that of the concrete containing NA used for comparison purposes. Hansen and Narud (1983) observed lower CS in concrete containing RCA than in control concrete, when the CS of control concrete exceeded the CS of concrete from which RCA was generated. The variations of CS of RCAC and control concrete containing NA were due to the bond between cement mortar and coarse aggregate; low bond gave a poor quality RCA and hence lower strength RCAC.

Nagataki and Iida (2001) also observed the dependence of CS on the strength of the source concrete; the CS is higher for RCAC produced from higher CS concrete than for that originated from lower CS concrete. Poon et al. (2004a) reported that the CS of RCAC originated from high CS concrete is higher than the RCAC originated from normal CS concrete due to formation of a denser microstructure in ITZ in the former one than in the latter one and also a higher aggregate strength of RCA originated from the high CS concrete than that of the RCA originated from normal CS concrete. The ITZ of concrete containing RCA originated from high CS concrete was denser than that of natural aggregate concrete too. Although the CS of both RCAC was lower than that of the conventional concrete after 7 and 28-days of curing, the CS of 90-day cured RCAC originated from high CS concrete was comparable to that of the conventional concrete. On the contrary, Santos et al. (2002) observed an insignificant effect of the source concrete's CS on the CS of RCAC. They observed a difference in CS of <1.5 MPa between two types of RCAC containing RCA originated from 56 and 45 MPa concrete mixes (Fig. 5.13).

Katz (2003) observed a moderate effect of the crushing age and strength of the source concrete from which RCA was generated on the CS of RCAC. The concrete made with aggregates crushed at 3 days exhibited higher CS than that made with aggregates generated after 1 and 28 days of curing, when white cement (WC) was used to make the source concrete. On the other hand, the concrete containing RCA made with the ordinary Portland cement (OPC)-based source concrete had slightly higher strength at the crushing age of 1-day and the lowest strength for RCA crushed at 3-day. The strength of the source concrete made with white cement was considerably higher than that of the OPC-based source concrete. The author indicated two factors that seemed to control the CS of the new RCAC: the strength of the source concrete and the presence of un-hydrated cement in the recycled aggregate. Both of the effects were active in the RCA generated from WC-based concrete after crushing age of 3 days and therefore led to the highest strength.

Nagataki and Ida (2001) observes no prominent effect of the crushing level during the preparation of RCA on the CS performance; however, the crushing age had some effect on the CS of RCAC: the crushing age of RCA produced from high (61 MPa) and medium (49 MPa) strength concrete had no effect on the CS of RCAC; but for low strength (28 MPa) RCAC the concrete containing RCA reclaimed after 1 or 2 years exhibited higher CS than the concrete containing RCA reclaimed after 28-days.

Poon and Chan (2007) reported that the presence of contaminants such as crushed brick, tiles, glass and wood in RCA deteriorated the CS of RCAC. However, concrete paving blocks made with crushed brick, tile and glass incorporated RCA at aggregate to cement ratio of 3 met the various standard specifications. Yang et al. (2011) observed 5.3 and 14.9 % drops in 28-day CS of RCAC

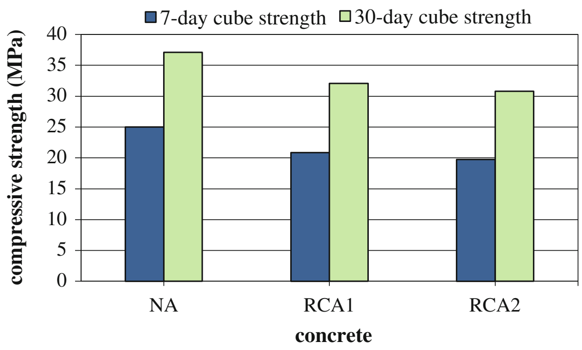


Fig. 5.13 CS of conventional and recycled aggregate concrete (Santos et al. 2002)

due to the replacement of 20 and 50 % by volume of coarse RCA by recycled crushed brick aggregate. Chen et al. (2003) observed little effect of brick and tile content in RCA on the concrete's CS if the ratio was <67 % (Fig. 5.14). The use of unwashed RCA had some effect on the CS of RCAC due to the presence of powdery impurities and other harmful materials; the effect of impurities on the CS of RCAC was observed prominently at lower w/c. Gomes and de Brito (2009) observed an insignificant CS loss due to the simultaneous incorporation of recycled brick and mortar, up to a maximum ratio of 25 % brick-mortar mix and 50 % RCA, or the incorporation of 100 % RCA, by comparison with conventional concrete with natural aggregates only.

A beneficial effect of RCA on the strength performance was observed when the RCAC was subjected to dry curing conditions due to the higher water absorption capacity of RCA. Buyle-Buddin and Zaharieva (2002) observed lower CS by around 9–12 % and 3–6 % for conventional and RCA concrete with 100 % replacement of fine and coarse natural aggregate by same size RCA respectively, when the curing conditions for both concrete were changed from water curing to air curing. The water absorbed by fine RCA gradually made its way to the cement paste and compensated the water loss of cement paste due to air-drying. On the other hand, Rao et al. (2011) observed higher CS for partially moist cured followed by air-cured RCAC than for moist cured similar RCAC (Fig. 5.15). The possible cause was the higher free water content in the old ITZ of moist cured concrete, which weakens the ITZ and therefore lowered the CS. Fonseca et al. (2011) did not observe any significant changes in CS due to variation of curing conditions. In this study, after casting, the concrete mixes were subjected to four curing conditions: (1) laboratory environment; (2) outer environment (atmospheric condition); (3) wet chamber; (4) water immersion.

Razaqpur et al. (2010) evaluated the CS of RCAC, prepared by using a new mixing method (equivalent mortar volume, EMV method). In this method, the amount of residual mortar present in RCA was included in the total amount of mortar present in RCAC. The total amount of mortar in RCAC was equivalent to the mortar amount present in the control concrete. The CS of concrete containing

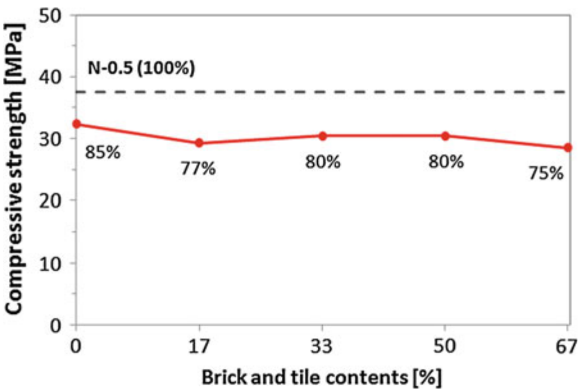


Fig. 5.14 Effect of brick and tile content in RCA on the CS of RCAC (Chen et al. 2003)

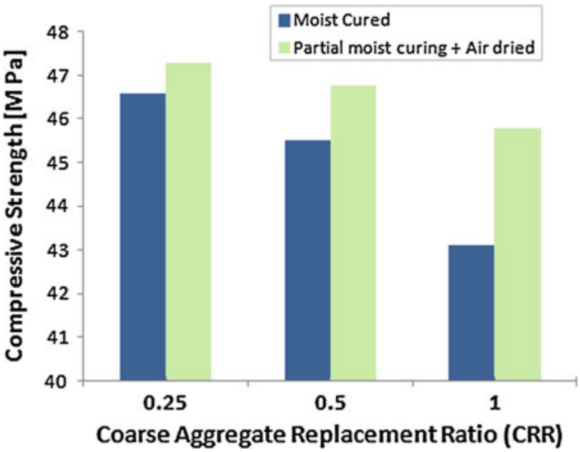


Fig. 5.15 Effect of curing conditions on the CS of RCAC (Rao et al. 2011)

coarse RCA was 10–14 % higher than that of the control concrete when RCA was used to replace partially or fully coarse NA (limestone and gravel). This is due to the addition of a water reducing admixture to the RCA mix, which improved the quality of the mortar more than observed in conventional concrete. One advantage of this method was that the amount of cement necessary to make concrete was about 10 % less than for conventional concrete.

Ferreira et al. (2011) compared the CS of mixes containing coarse RCA using two mixing methods. In the pre-saturation method, the RCA was initially pre-soaked before mixing. In the water compensation method, extra water absorbed by RCA was added during mixing (Fig. 5.16).

The 7- and 28-day CS of RCAC using the pre-saturation method was lower than that using the other method, possibly due to a lower "nailing effect", which results from the penetration of cement paste into the superficial pores of aggregate particles. Before mixing, pre-saturated RCCA exhibited not only a high level of humidity but also water on the surface and within surface pores. This might have impaired the penetration of the cement paste into the pores, leading to a decrease of the "nailing effect" and, consequently, to a weaker ITZ between cement paste and RCCA. However, the differences of CS between the mixes prepared by two methods became insignificant as the RCCA's incorporation ratio increased, which might be a consequence of a higher number of weak zones in concrete.

Using a certain amount of mineral additions such as fly ash (FA), metakaolin (MK), silica fume (SF), rice husk ash or latent hydraulic materials such as ground blast furnace slag (gbfs) into cement have some beneficial effect on the CS performance of RCAC.

Kou and Poon (2006) observed an improvement in 90-day CS of RCAC due to 25 % replacement of OPC by FA; however, the addition of 35 % FA had a negative effect on the CS of RCAC. The gain in CS between 28 and 90 days of RCAC containing 25 and 35 % FA is higher than that observed in conventional concrete. The strength gain of concrete containing RCA between 28 days and

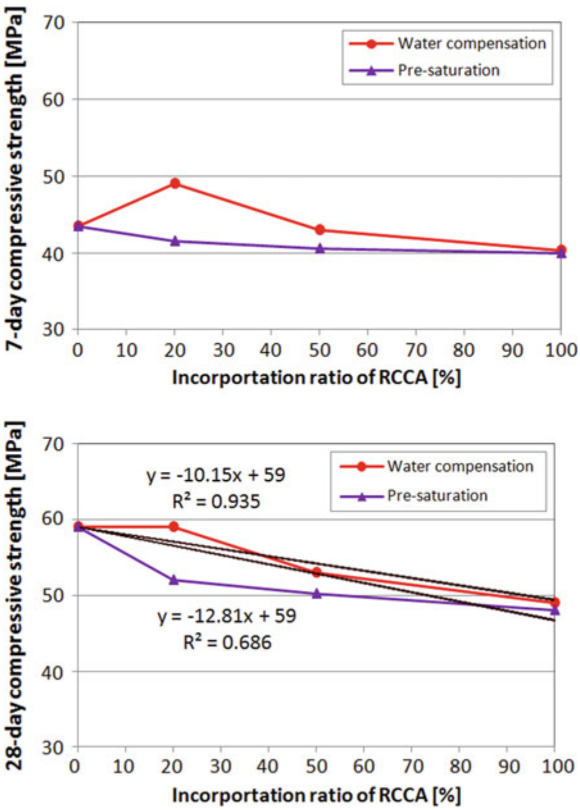


Fig. 5.16 Effect of the mixing procedure on the CS of RCAC (Ferreira et al. 2011)

10 years increased with the FA content (Poon and Kou 2010). Corinaldesi (2009) observed higher CS for RCAC prepared replacing Portland cement with 30 % FA or 15 % silica fume than the RCAC containing OPC when the mixes were moist cured up to 56-days. The strength of RCAC containing silica fume was significantly higher than that of the RCAC containing FA and even better than the conventional concrete at all curing periods. Gonzalez-Fonteboa and Martinez-Abella (2008) also observed a significant improvement of CS of RCAC due to the replacement of 8 % Portland cement by silica fume. Sagoe-Crentsil et al. (2001) observed much higher long-term CS in RCAC with incorporation of slag partially replacing cement than in conventional concrete owing to the higher reactivity of slag than Portland cement at the later stage of hydration. The gain in CS between 7 and 28 days was 12.4 MPa in comparison to 6 MPa for conventional concrete. Razaqpur et al. (2010) observed that the addition of 25 % (w/w) fly ash (FA) into cement reduced the CS of the resulting RCAC due to slow hydration of FA in the concrete matrix. On the other hand, the CS of RCAC containing 35 % gbfs was higher than that of conventional concrete.

Ann et al. (2008) compared the CS of two types of concrete containing RCA as coarse aggregate and made by replacing 30 and 65 % of Portland cement by

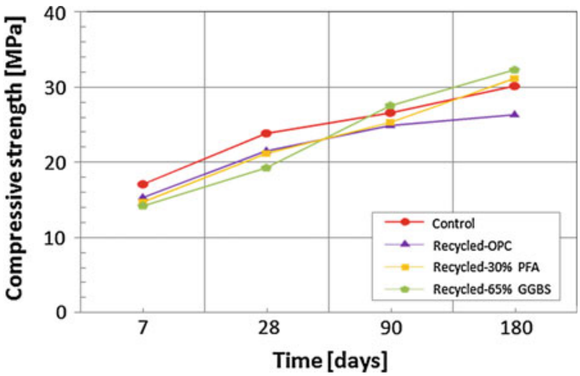


Fig. 5.17 The CS of RCAC due to the replacement of cement by other binder materials (Ann et al. 2008)

pulverized fuel ash (PFA) and gbfs with RCAC and conventional concrete mixes (Fig. 5.17). They observed lower CS for RCA concrete containing PFA and gbfs than for the control and RCAC mixes at the ages of 7 and 28 days, due to the lower hydraulic reactivity of these binders. However, the CS of concrete containing gbfs and PFA overtook the CS of the conventional concrete after 90 and 180 days of curing, respectively.

Kou et al. (2011b) observed a substantial reduction in CS due to the replacement of cement by 55 % gbfs or 35 % FA. However, they observed an improvement in the CS of RCAC with 50 and 100 % (by volume) replacement of coarse NA by RCA due to the replacement of cement by 10 % SF or 15 % MK. The CS of RCAC with SF and MK was similar to that of the conventional concrete for a 50 % replacement ratio of NA by RCA (Fig. 5.18). However, for a 100 % RCA replacement ratio, the 90-day CS of RCAC containing all type of mineral additions was substantially lower than that of the conventional concrete. The gain in CS of RCAC concrete with SF and MK was higher than that of the concrete containing FA and gbfs in the early ages (up to 28 days); on the other hand, later on (28–90 days), the gain in CS was higher for RCAC with FA and gbfs than for RCAC with SF and MK. The improvement of ITZ due to the filling of pores as well as cracks in RCA by the hydration products was the reason for the observed improvement. The contribution of mineral additions to the improvement of CS of RCAC was more than in conventional concrete. Ajdukiewicz and Kliszczzewicz (2002) observed a significant improvement in CS of RCAC due to the addition of SF and superplasticizer. The improvement was more prominent for RCA produced from high-strength concrete aggregates. Thangchirapat et al. (2008) reported that the use of rice husk-bark ash (RHBA) to replace 20 and 35 % by weight of Portland cement in RCAC yielded higher CS than in RCAC prepared without RHBA.

Several other methods were adopted to overcome the negative effect of CDW aggregate on the CS of concrete. Increasing cement content or lowering the water to cement ratio in the mix and improving the concrete mixing process have already been mentioned during the discussion of CS of concrete containing RA and RCA aggregate. Since the major reason of the reduction in CS of RAC and RCAC is the

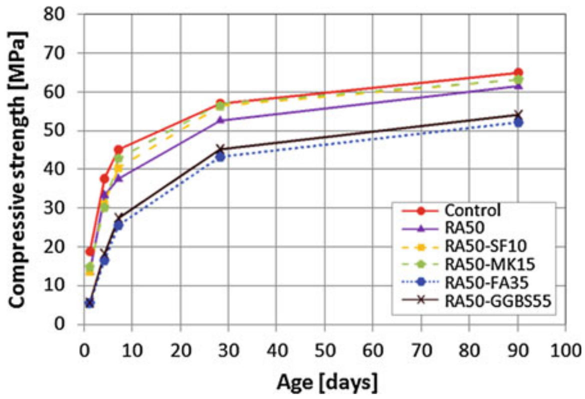


Fig. 5.18 CS of concrete due to the replacement of cement by mineral addition (Kou et al. 2011b)

weaker bond between the recycled aggregate and the cement paste, some studies are also directed at improving the bond strength by using some other materials. Out of these, coating the aggregate's surface with silica fume or nano-silica is most promising one.

Chen et al. (2003) observed a 10–15 % higher CS of concrete containing washed RCA than that of concrete containing unwashed RA at various w/c ratios. The RA was washed to remove the sand content and other impurities like bricks and tiles. Shayan and Xu (2003) observed that the surface treatment of coarse RCA by silica fume slurry can substantially improve the CS of the resulting concrete; however, sodium silicate treatment did not have any beneficial effect. They reported that structural concrete with 50 MPa strength grade could be produced by replacing all the coarse NA with silica fume treated coarse RCA and by replacing a maximum 50 % by weight of fine NA with untreated fine RCA. Katz (2004) also observed an increase of about 23–33 and 15 % in 7- and 28-day CS due to the impregnation of SF on the surface of coarse RCA. The improvement of the microstructure of the ITZ between the cement paste and the RCA aggregate surface and the mechanical performance of RCA can be the major cause for the observed increase in CS. Although it was not as prominent as SF impregnation, ultrasonic treatment to remove unbound particles of RCA also increased by about 7 % the 7- and 28-day CS of RCAC.

Akbarnezhad et al. (2011) observed significantly smaller reduction in the CS of concrete due to the incorporation of RCA obtained after microwave heating (MRCA) than that observed for normal RCA incorporation due to the removal of a part of the adhered mortar content as well as of weak RCA particles (Fig. 5.19). The RCA was heated using microwaves to remove adhered mortar. The differences in 28-day CS between conventional concrete and mixes containing MRCA were negligible up to 40 % replacement of coarse NA. These differences were respectively 10 % and around 30 % for the mixes containing RCA and MRCA as sole coarse aggregate.

Tam et al. (2007) observed up to 21 % improvement in 28-day CS of concrete with 20 % by volume replacement of coarse natural aggregate by RCA using a

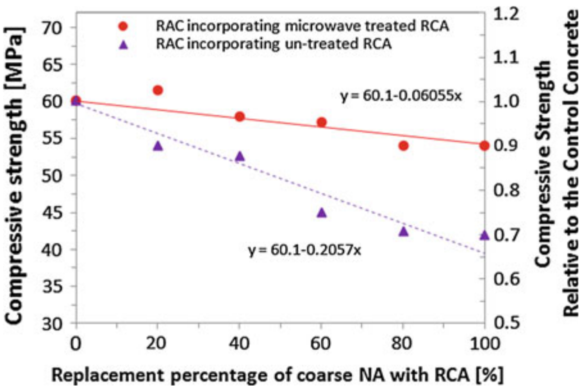


Fig. 5.19 CS of concrete due to the replacement of coarse NA by RCA and MRCA (Akbarnezhad et al. 2011)

two-stage mixing approach. In the first stage, a cement layer is formed by mixing the cement with aggregates and half of the total water content. The cement layer can fill the pores, cracks and voids of the old mortars and improve the bond during the hardening stage. The addition of a small amount of silica fume to the RCA in the pre-mix procedure can improve by about 20 % the 28-day CS of concrete containing 25 % RCA (Tam and Tam 2008). Further improvement of CS could be achieved by adding silica fume and a fraction of cement with RCA during the pre-mix stage (Tam and Tam 2008). The filling of the pores and cracks of the old mortar with silica fume and the improvement of the aggregate—matrix bond associated with the formation of a less porous transition zone and a better interlock between the paste and the aggregate are the reasons for the observed improvement. Mixing the cement with silica fume provides relatively thick and soft coatings of the silica fume slurry and the necessary cement paste surrounding RA in the pre-mix process and therefore further enhances the strength of the ITZ.

Tsujino et al. (2007) observed an increase in the CS of RCA concrete, when the RCA is coated with mineral oil; however, silane coating of the surface of RCA decreased the CS of RCAC. Kou and Poon (2010) observed that the 90-day CS of RCAC prepared by using air-dried 10 % polyvinyl alcohol impregnated RCA was similar to that of the conventional concrete. The observed improvement was due to the various physic-chemical changes such as the filling of pores and cracks of RCA, the improvement of flocculation and coagulation of the cement particles, the improvement in the ITZ section, the reduction of w/c at the paste-aggregate interface and the reduction in bleeding. However, oven drying of PVA impregnated RCA has no effect on the CS performance of concrete.

Kutcharlapati et al. (2011) reported that the treatment of RCA by a colloidal solution of nano-silica can improve the mechanical properties of RCA and RCAC. The cubic CS of nano-silica containing RCAC and conventional concrete at the same age was 22 and 16 MPa, respectively. An improvement of the mix's cohesiveness, a reduction in segregation and bleeding, improvement in pore structure and a densification in ITZ of the RCA-cement paste due to the filler effect and hydration of the nano-silica particles were the major causes for the observed CS improvement due to the nano-silica treatment of RCA.

Topçu and Saridemir (2008) successfully predicted the 3- to 90-day CS of RCAC by applying artificial neural networks and fuzzy logic systems. Lin et al. (2004) used the optimal mix proportioning of RCAC by orthogonal array, ANOVA and significance test with F statistics to prepare a RCA concrete mix with suitable mechanical strength. The optimum RCAC mix observed in this investigation contains RCA and natural river sand as 100 % coarse and fine aggregates, respectively, with water/cement ratio of 0.5 and volume ratio of recycled coarse aggregate in RCAC 42.0 %. The mix should contain no crushed brick and unwashed RCA should be used. The slump and 28-day CS of this optimum mix was 180 mm and 30.17 MPa, respectively.

The use of fine fraction (<4 mm fraction) of CDW aggregate in preparation of concrete is not as thoroughly studied as the coarse fraction. It is believed that the greater water absorption capacity of the fine fraction of CDW waste can jeopardize the use of this fraction as fine aggregate in concrete (Evangelista and de Brito 2007).

Merlet and Pimienta (1993) observed 19–39 % lower CS of concrete made with fine and coarse recycled aggregate than that of a control concrete. Leite (2001), on the other hand, observed increasing and decreasing CS of concrete due to increasing incorporation of fine and coarse recycled aggregate (FRA and CRA, respectively), respectively (Fig. 5.20). The author justifies the results with the stronger bond created between FRA and the matrix, because of the precipitation of cement crystals inside the FRA.

Khatib (2005) observed a reduction in CS with increasing content of fine recycled concrete aggregate (FRCA) in concrete. The particle size of FRCA was <5 mm. The concrete was prepared at free water to cement ratio of 0.5. The reduction in 90-day CS was in the range of 15–27 % when 25–100 % by weight of fine natural aggregate (FNA) was replaced by FRCA. However, 28-day CS of concrete containing 25–75 % and 100 % FCRA were about 25 and 36 % lower than that of the control, respectively. The relative CS of FRCA concrete (FCRAC) in the 28–90-day period increased due to the hydration of un-hydrated cement

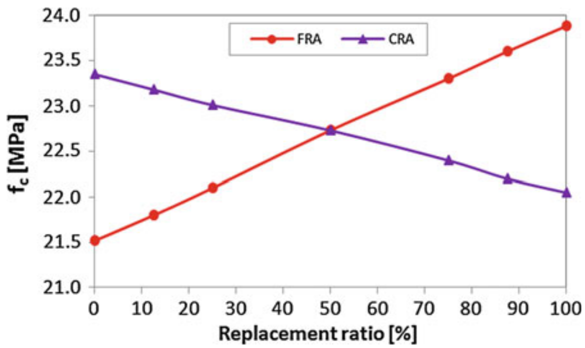


Fig. 5.20 CS of RCAC due to incorporation of fine (FRA) and coarse (CRA) recycled aggregates (Leite 2001)

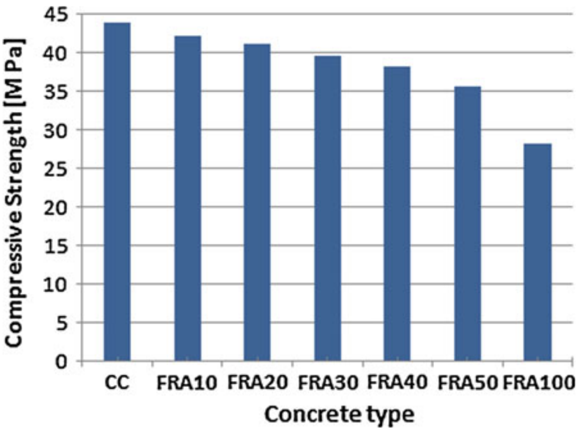


Fig. 5.21 Reduction in CS of concrete due to the incorporation of fine recycled aggregate (Yaprak et al. 2011)

particles of FCRA. Yaprak et al. (2011) also observed a gradual drop in CS as the replacement of FNA by FRCA increased (Fig. 5.21).

Evangelista and de Brito (2007) conducted a comprehensive study on the use of pre-saturated FRCA (<2.36 mm) as a partial or complete replacement of same size FNA in the preparation of structural concrete. The concrete mixing time was maintained as 10 and 20 min for series I and series II concrete, respectively. The results are presented in Table 5.10. The 28-day CS of series I FRCAC was marginally higher (2–5 %) than the conventional concrete, due to the pozzolanic reaction of un-hydrated cement present in FRCA. On the other hand, the CS decreased by about 0.6–7.6 % with respect to the conventional concrete in series II FRCAC since the increased soaking time of FRCA weakened the cement paste-aggregate bond by increasing the w/c in that particular region. The development of CS of conventional concrete almost stabilised after 28 days; however, the CS development of FRCAC continued after 28 days due to the hydration of cement present in FRCA. Zega and Di Miao (2011) also observed a slight decrease in 28- and 84-day CS of concrete due to a 20 and 30 % by volume replacement of fine NA by fine RCA.

Table 5.10 CS of concrete containing fine recycled concrete aggregate (Evangelista and de Brito 2007)

Concrete type	Amount of substitution of FNA by FRCA (%, v/v)	Compressive strength (MPa)	
		Series I	Series II
FNAC	0	59.4	59.3
FRCAC10	10	62.2	59.0
FRCAC20	20	58.4	57.3
FRCAC30	30	61.3	57.1
FRCAC50	50	60.8	58.8
FRCAC100	100	61.0	54.8

Pereira et al. (2012) reported that the compressive strength of concrete containing FRCAC could be improved by using superplasticizer and a lower w/c ratio. The 28-day compressive strengths of conventional concrete and FRCACs containing FRCA as the only fine aggregate with and without two types of superplasticizers were respectively 39.5, 38.6, 45.1 and 63 MPa indicating a significant increase in compressive strength due to the addition of superplasticizer in the mix of RCAC. They also observed much a lower influence of FRCA on CS performance in comparison to the change in w/c ratio. They proposed the following relationship between CS (f_c) and effective w/c ratio $\left(\left(\frac{\mathrm{w}}{\mathrm{c}}\right)\mathrm{ef}\right)$ (correlation coefficient of 0.96) from their experimental results:

$$f_c = \frac{230.3}{(25.9)^{\left(\frac{\mathrm{w}}{\mathrm{c}}\right)\mathrm{ef}}} \times (1 - (-0.077) \cdot W_{24} \cdot r) \tag{5.1}$$

where W_{24} is 24 h water absorption capacity of concrete, r is replacement ratio and numerical values are determined by regression analysis.

Kou and Poon (2009a) prepared two concrete series by replacing 25, 50, 75 and 100 % by weight of fine natural aggregate by fine recycled aggregate (FRA) with particle size below 5 mm. The first and second series of concrete were prepared using the same cement content at constant water to cement ratio (w/c) of 0.53 and a close slump range of 60–80 mm. At same w/c, the CS of concrete decreased with the content of FRA due to higher bleeding as well as poor aggregate-cement paste bond owing to the higher initial free water content. At constant slump, the CS of series II concrete also decreased with the FRA content; however, the deterioration of CS was marginally higher than that observed in series I. According to the authors, this was due to the weaker mechanical properties of FRA and FNA.

By replacing 100 % of fine natural aggregate by fine RCA, Kou and Poon (2009b) prepared a self-compacting concrete, which can yield CS values as high as 64 MPa. The authors concluded that the inclusion of FRCA up to a ratio of 25–50 % does not significantly change the CS of the resulting concrete. Dapena et al. (2011) observed a drop of around 7.3–9.4 % in the CS of concrete due to the replacement of 10 % coarse RCA by FRCA, where natural coarse aggregate was replaced by 20, 50 and 100 % (by volume) of coarse RCA.

Kou and Poon (2008) reported up to 5 years experience of CS of concrete prepared by replacing 0, 20, 50 and 100 % of coarse NA by an equal volume of RA and RCA. They observed lower CS of RCA concrete than that of RA concrete after 28-day but after 5 years the CS was highest for RCA for all substitution ratios. The CS of concrete containing RCA and RA was always lower than that of the conventional concrete; however, the reduction of CS decreases as curing time increases. The 28-day to 5-year gain in the CS of concrete containing RA and RCA was higher than that of the control concrete and it was highest for RCA concrete. The strengthening of the paste-aggregate bond, the healing of cracks in interfacial zone due to the deposition of new hydration products and the reduction in the preferred orientation of $Ca(OH)_2$ crystals were the main causes for the observed improvement of the CS of RCAC after prolonged curing. The gain in CS also increased with the RA and RCA contents in concrete.

5.3.2 Splitting Tensile Strength

Like for CS, the splitting tensile strength (STS) of concrete containing RA or RCA is normally lower than that of conventional concrete and increasing the addition of CDW aggregate into concrete further lowers it. Table 5.11 shows some typical data. The causes for CS reduction are also responsible for STS reduction. According to de Brito and Alves (2008) the lower mechanical properties of the recycled aggregates when compared to the natural ones lead to a fall in the STS of the concrete containing CDW aggregate as the substitution rate increases.

After analysing literature data de Brito and Robles (2010) reported that the replacement ratio of natural aggregate by CDW aggregate had slightly less effect on the reduction in STS than that observed in CS. Table 5.12 shows the differences in 28-day STS and CS of RAC and RCAC from conventional concrete determined from the data presented in various references. A great variation in the reduction in STS due to the incorporation of RCA or RA in concrete was observed, probably due to the variations in experimental parameters and properties of CDW aggregates used in the various studies.

From the results presented in Table 5.12 it can also be concluded that the inclusion of RA or RCA into concrete had little impact on STS or that the CDW aggregate mixes had higher STS than conventional concrete up to a given replacement level. This is probably due to the improvement in aggregate-cement paste bond strength, which induces a higher increase in STS than in CS (Kou and Poon 2008).

Corinaldesi (2009) observed similar STS for RCAC prepared with and without mixing FA into Portland cement. Etxeberria et al. (2007a) observed higher STS of

Table 5.11 28-day splitting tensile strength of concrete containing various amount of CDW aggregate

Reference	Type of aggregate	Tensile strength (MPa)/substitution level (%)
Gomez-Soberon (2002)	RA/fine + coarse	3.7/0; 3.7/15; 3.6/30; 3.4/60; 3.3/100 (v/v)
Gonzalez-Fonteboa and Martinez-Abella (2008)	RCA/coarse	3.15/0; 3.00/50 (v/v)
Kou and Poon (2008)	RCA/coarse	2.43/0; 2.40/20; 2.35/50; 2.26/100 (v/v)

Table 5.12 Reduction in 28-day CS and STS due to addition of CDW aggregate in conventional concrete

Reference	Type of aggregate	Reduction (%)		Replacement amount of similar size aggregate (%)
		Compressive strength	Tensile strength	
Corinaldesi and Moriconi (2009)	RCA/fine + coarse	6.3	6.2	100
Gomez-Soberon (2002)	RA/fine + coarse	10.8	11.5	100
		8.1	8.2	60
		2.7	5.1	30
Gonzalez-Fonteboa et al. (2011)	RCA/coarse	13.2	8.48[a]	20, 50 and 100 %; w/c = 0.50
		25.9	2.41[a]	
		20.4	3.53[a]	
Gonzalez-Fonteboa and Martinez-Abella {2008}	RA/coarse	0.38	4.77	50 (by volume)
		11.6[a]	6.78[a]	50 (by volume) and 8 % silica fume with cement
Katz {2003}	RCA/coarse + fine	23.1	3.03[a]	~100; crushing age: 1-day
		25.4	12.1	Crushing age: 3-day; replacement: ~100 %;
		22.5	6.06	~100; crushing age: 28-day;
Kou et al. (2011a)	RCA/coarse	21.7	9.05	100;
	RA/coarse	18.7	7.00	100;
Mas et al. (2012)	RA/coarse (8/40)	18	14	25; w/c = 0.65
		19	6	50; w/c = 0.65
		21	21	75; w/c = 0.65
	RA/coarse (8/40)	13	11[a]	20; w/c = 0.75
		13	10	40; w/c = 0.75
	RA/coarse (8/20)	26	25	20; w/c = 0.45
		39	34	40; w/c = 0.45
Etxeberria et al. (2007a)	RA/coarse	3.5	19.3[a]	25

[a] Increasing with respect to conventional concrete strength

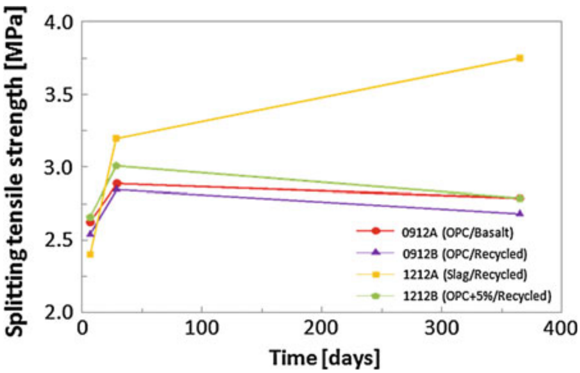


Fig. 5.22 STS of conventional and RCA concrete due to the variation of cement type and cement content (Sagoe-Crentsil et al. 2001)

RAC containing RA as 25 and 50 % replacement of coarse natural aggregate than in conventional concrete but not for the 100 % replacement. In this study, the CS of RAC with 100 % coarse RA was also comparable to that of conventional concrete. The authors point out that the absorption capacity of the adhered mortar present in the partly saturated (humid) recycled aggregate and the effectiveness of the new interfacial transition zone of the recycled aggregate concrete increased the STS.

Sagoe-Crentsil et al. (2001) observed slightly low and significantly high 28-day and 1-year STS, respectively, for RCAC containing OPC and slag cement than for conventional concrete containing OPC (Fig. 5.22). On the other hand, increasing the cement content led to higher STS than for conventional concrete at the early ages, but after 1-year the values became similar.

However, in some studies, a substantial reduction in STS in comparison to that of CS due to the addition of CDW aggregate was also reported. Some typical examples are presented in Table 5.13. Evangelista and de Brito (2007) reported that the un-hydrated cement content in fine recycled aggregate can affect the CS

Table 5.13 Reduction in 28-day CS and STS of concrete containing CDW aggregate in comparison to conventional concrete

Reference	Type of aggregate	Reduction (%)		Replacement ratio of similar size aggregate (%)
		Compressive strength	Tensile strength	
Yang et al. (2011)	RCA/coarse	5.7	13.8	100
Evangelista and de Brito (2007)	RCA/fine	2.7[a]	30.5	100
Rao et al. (2011)	RAC/coarse	7.48	13.9	25
		14.1	18.0	50
		17.5	23.2	100
Gonzales-Fonteboa et al. (2011)	RCA/coarse	10.7	17.2	20, 50 and 100 %; w/c = 0.65
		9.38	14.8	
		10.6	9.96	

[a] Increasing with respect to conventional concrete strength

Table 5.14 STS of concrete with various ratios of CDW aggregate at different curing ages

Reference	Type of aggregate	Substitution level (%, v/v)	Tensile strength (MPa)/day
Gomez-Soberon (2002)	RA/fine + coarse	0	3.6/7d; 3.7/28d; 3.9/90d;
		15	3.3/7d; 3.7/28d; 3.9/90d;
		30	3.3/7d; 3.6/28d; 3.9/90d;
		60	3.2/7d; 3.4/28d; 3.7/90d;
		100	3.5/7d; 3.3/28d; 3.6/90d;
Gonzalez-Fonteboa and Martinez-Abella (2008)	RCA/coarse	0	3.12/7d; 3.15/28d; 3.32/115d;
		50	3.17/7d; 3.00/28d; 3.37/115d;
Kou and Poon (2008)	RCA/coarse	0	2.43/28d; 2.68/90d; 2.83/180d; 2.94/1Y; 3.16/2Y; 3.32/5Y
		20	2.39/28d; 2.56/90d; 2.78/180d; 2.91/1Y; 3.21/2Y; 3.40/5Y
		50	2.34/28d; 2.52/90d; 2.74/180d; 3.04/1Y; 3.28/2Y; 3.52/5Y
		100	2.21/28d; 2.48/90d; 2.76/180d; 3.12/1Y; 3.36/2Y; 3.64/5Y

performance but it does not affect the STS performance and therefore in comparison to CS, a substantial reduction in STS was observed in RCA due to porous nature of the RCA. They observed a lowering in STS as the replacement ratio of fine RCA in concrete increased.

In several studies, it was reported that the STS of concrete containing RCA and RA improved substantially at the latter stages of curing and in some cases, the strength of concrete containing RCA or RA was even better than that of conventional concrete. Table 5.14 presents some typical results of STS of concrete containing natural and CDW aggregates with increasing curing time. Kou and Poon (2008) pointed out that the improvement in the microstructure of the interfacial transition zone (ITZ) and therefore an increase in the bond strength between the new cement paste and the old aggregates might be the factor for the observed improvement of STS.

Kou et al. (2011a) observed a 10 and 7 % lower 28-day STS of concrete containing coarse RA and RAC than that of conventional concrete. However, the STS of both recycled aggregate mixes was higher than that of conventional concrete after 1 and 5 years of curing (Fig. 5.23). The development of STS between 28 days and 5 years for RCAC and RAC were respectively 65 and 56 % compared to 37 % for conventional concrete. The percentage gain in STS from 28 days to 5 years of curing also increased with the RA or RCA contents. An improvement in the microstructure of the ITZ, increasing the bond strength between the new cement paste and the old aggregates after prolonged hydration, and the self-cementing ability of recycled aggregate were the causes for the observed increase in STS of the RCAC at the later stages of curing. The higher improvement in STS than CS observed in this study was due to the improvement in cement paste-aggregate bond strength. In another study, Kou and Poon (2008) also observed

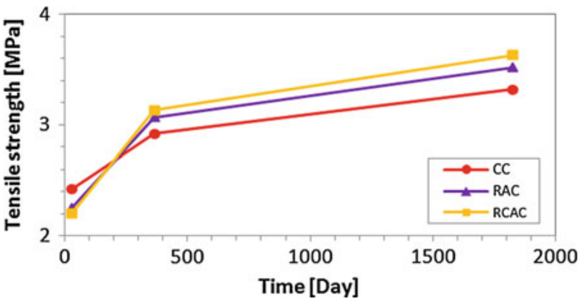


Fig. 5.23 STS of concrete due to increasing curing time (Kou et al. (2011a)

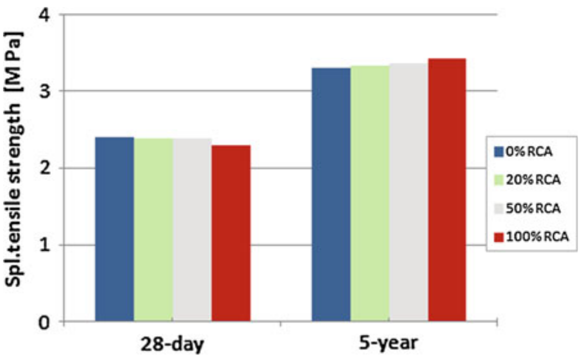


Fig. 5.24 STS of concrete after 28 days and 5 years of curing for various replacement of coarse NA by RCA (Kou and Poon 2008)

higher STS in RCAC than in conventional concrete after 5 years of curing, which also increased with the replacement ratio of coarse NA by RCA (Fig. 5.24). Conversely, Yong and Teo (2009) observed an improvement in the STS of concrete due to the replacement of 100 % of natural aggregate by coarse normal or saturated surface dry RCA at the early stage (up to 28 days) of curing; however, strength development of both types of RCAC slowed down in the 28–56 days curing period.

Gonzales-Fonteboa et al. (2011) observed an effect of w/c ratio on the STS performance of concrete containing RCA (Table 5.12 and 5.13). They observed higher STS in RCAC than in conventional concrete at the w/c ratios of 0.50; on the other hand, the STS of conventional concrete was higher than that of the RCAC at the w/c ratio of 0.65. However, Kou and Poon (2006) observed higher STS for concrete prepared by replacing various percentages of natural coarse aggregate by RCA at high w/c ratio than that prepared at low w/c ratio. They justified their results with the variability in surface texture of RCA, which gave the paradoxical results.

Yang et al. (2008) observed decreasing STS of RCAC with increasing water absorption capacity of incorporated RCA. Similarly to CS, Gomez-Soberon (2002) and Kou et al. (2011a) observed an inverse relationship between STS and porosity of RCAC. The relationship between open porosity and STS observed by Gomez-Soberon (2002) study is presented in Fig. 5.25.

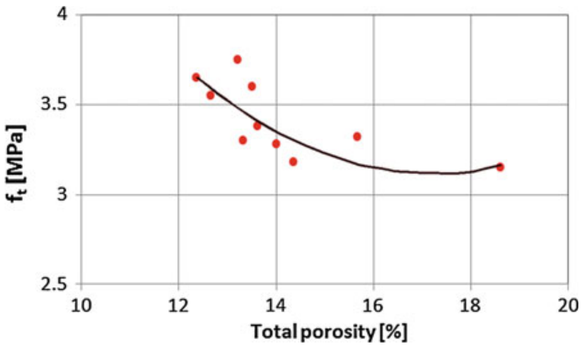


Fig. 5.25 STS versus total porosity curve of RCAC (Gomez-Soberon 2002)

The presence of impurities in RCA such as crushed clay brick and tiles has positive or negative effects on the STS of resulting RCAC depending on the substitution ratio. Yang et al. (2011) observed lower 7- and 28-day STS in RCAC prepared by replacing 20 and 50 % of RCA by crushed brick than in conventional RCAC (Fig. 5.26), owing to the higher porosity of crushed brick aggregate than of RCA. In crushed brick containing concrete, most of the tensile failures occurred within the crushed brick particles while for other mixes, failure seemed to occur between the aggregate and the mortar matrix interfaces. The STS of concrete paving blocks containing RCA decreased with increasing replacement of fine or coarse RCA by crushed clay brick in the Poon and Chan (2006) study too. Kou et al. (2011a) also observed lower STS for RAC than for RCAC especially at the later stages of hydration.

However, Poon and Chan (2007) observed higher STS in concrete paving blocks due to the 10 % replacement of RCA by crushed tiles or a 1:1 mixture of crushed tiles and bricks. Mixing crushed glass with tiles or tiles/bricks gave strength comparable to that of conventional RCA concrete. On the other hand, the addition of crushed wood drastically lowered the STS. Strengthening of the

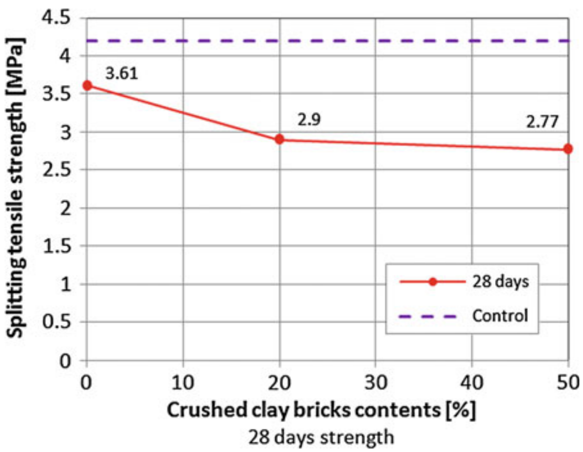


Fig. 5.26 Effect of crushed brick content in RCA on the STS performance (Yang et al. 2011)

cement paste/aggregate binding due to the penetration of cement paste into the porous tile and brick aggregate, filling of pores by fine tiles and brick aggregate as well as the presence of more tiles and bricks aggregate in concrete due to their lower density were the major causes for this improvement.

Nagataki et al. (2004) observed comparable STS for RCAC incorporating RCA with minimum adhered mortar content to that of conventional concrete made with the original aggregates. The RCA with minimum adhered mortar content even exhibited higher STS than conventional concrete. The smaller size, lower sand content as well as the elastic compatibility between RCA and cement paste were the causes of the good performance of RCAC. Padmini et al. (2009) observed lower STS in RCAC than in conventional concrete and the difference narrowed down as the CS decreased. In contrast to interfacial bond failure between cement mortar and aggregate observed in conventional concrete, RCAC exhibited both interfacial bond failure and aggregate failure in the STS tests (Padmini et al. 2009; Rao et al. 2011).

Tabsh and Abdelfatah (2009) reported that the STS for 50 and 30 MPa classes of conventional concrete as well as that of RCAC were similar when the RCA was generated from 50 MPa concrete. On the other hand, a drop by 25–30 % and 10–15 % in STS was observed for both concrete classes when RCA was generated from 30 MPa concrete. Tavakoli and Soroushian (1996) observed a negligible effect of aggregate size or dry mixing time on the STS of RCA concrete. The 28-day STS of RCAC with two types of RCA is either higher or statistically comparable to that of the control concrete for limited ranges of various experimental parameters such as size of coarse RCA, mixing time and w/c ratio.

The addition of several mineral admixtures such as silica fume, fly ash, rice husk ash does not have prominent beneficial effect on STS improvement as observed in CS (Gonzalez-Fonteboa and Martinez-Abella 2008; Thangchirapat et al. 2008). Gonzalez-Fonteboa and Martinez-Abella (2008) observed around 6.8 % higher STS in RCAC prepared at w/c of 0.55 than in conventional concrete due to the incorporation of silica fume as mineral admixture into cement but the improvement was not as significant as for CS (around 11.6 %). Ajdukiewicz and Kliszczzewicz (2002) observed improvement in STS of RCAC due to the addition of SF and superplasticizer, but the improvement was not as significant as in CS. Kou et al. 2007) also observed lower STS for RCAC using a blended cement prepared by replacing 25 % (by weight) OPC by FA than for RCAC using OPC. The increasing addition of FA into 35 % further lowered the strength. On the other hand, the same authors in another publication (2008) reported that the addition of fly ash as a replacement of 25 % of cement by weight can increase the STS. The major difference between these two studies was the larger amount of binder content in the mix containing FA in the later study than in the former one. The improvement was due to the pozzolanic activity of FA which densified the concrete matrix by improving porosity.

The replacements of 10 % OPC by SF or 15 % OPC by metakaolin (MK) gave higher STS to the resulting mixes prepared by replacing 50 and 100 % (by volume) of coarse natural aggregate by RCA than that of the control and of the RCAC

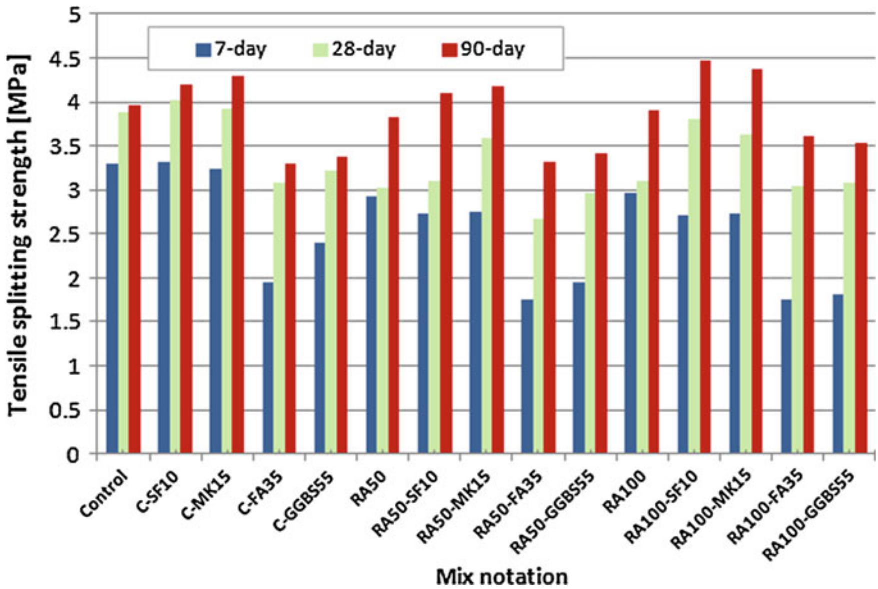


Fig. 5.27 STS of control concrete and RCAC containing various types of mineral admixtures (Kou et al. (2011b)

prepared with OPC at the curing ages of 7, 28 and 90 days (Kou et al. (2011b) (Fig. 5.27). On the other hand, the RCAC prepared by replacing 35 or 55 % of OPC by FA or ground granulated blast furnace slag (ggbfs) respectively had lower STS than the control concrete and the RCAC containing OPC at all the curing periods. The formation of more hydration products due to the SF and MK hydration and the consequent improvement of the microstructure of ITZ increased the binding of RCA and cement paste and hence improved the STS. The increase in STS between 7 and 90 days was higher for RCAC using blended cement than for the control concrete and the RCAC using OPC. The increase in tensile strength was higher for FA and ggbfs than for SF and MK too.

Ann et al. (2008) observed that the 28-day STS for RCAC was lower than for conventional concrete. The strength for RCAC using OPC-30 % pulverized fuel ash and OPC-65 % ground blast furnace slag as binder was similar but lower than that of the RCAC concrete. However, the ratios of STS to CS were comparable for all types of concrete (Fig. 5.28). In the Berndt's (2009) study, though the 28-day STS of RCAC using 50 and 70 % ggbfs as replacement of OPC and RCA as sole coarse aggregate was lower than that of concrete using slag cements and natural aggregate as sole coarse aggregate, the STS of mixes having former composition was higher than that of concrete using 100 % OPC and natural coarse aggregate as well as of concrete using 100 % OPC and 100 % coarse RCA.

The STS of concrete with coarse RCA obtained after treatment by polyvinyl alcohol followed by air-drying (PI-R(A) in Fig. 5.29) was higher than that of concrete with untreated RCA at the curing ages of 7–90 days. However, oven

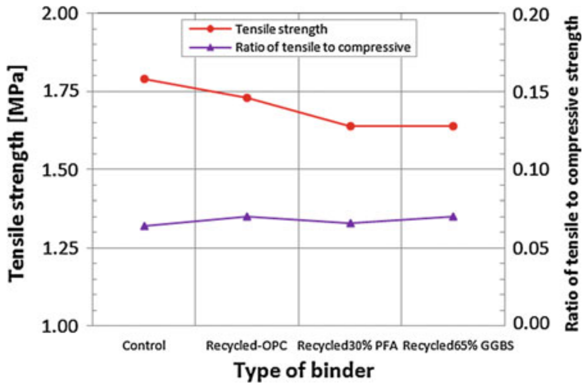


Fig. 5.28 STS performance and ratios STS/CS of conventional and RCA concrete (Ann et al. 2008)

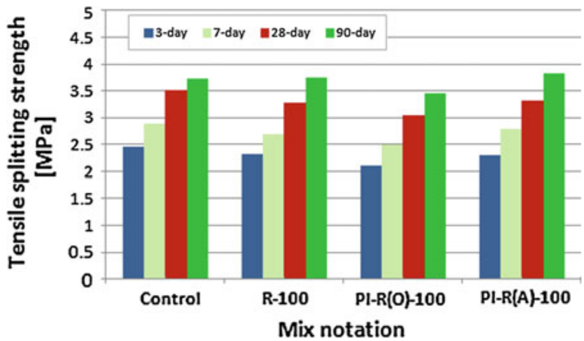


Fig. 5.29 STS of control concrete and RCAC containing normal or polyvinyl alcohol treated RCA (Kou and Poon 2010)

drying of polyvinyl alcohol treated RCA (PI-R(O) in Fig. 5.30) gave the resulting concrete lower strength than concrete with untreated RCA (Kou and Poon 2010). Tsujino et al. (2006) observed a beneficial effect of the surface treatment of coarse RCA by mineral oil in the STS performance of the resulting RCAC. On the other hand, silane treatments of RCA deteriorate the SRS of the resulting concrete.

The STS/CS ratio of RCAC was reported in some studies (Gonzalez-Fonteboa et al. 2011; Sagoe-Crentsil et al. 2001; Thangchirapat et al. 2008; Ravindrarajah and Tam 1985) to be similar to that of conventional concrete. Sagoe-Crentsil et al. (2001) observed ratios of STS to CS in the range of 0.89–1.21 depending on the type of cement used, cement content, curing age and RCA content and these values were similar to the range of 0.99–1.19 exhibited by conventional concrete at various curing ages. Thangchirapat et al. (2008) observed ratios in the range of 0.80–0.82 for RCAC in comparison to 0.95 for conventional concrete. According to Sagoe-Crentsil et al. (2001), the absence of detrimental effect of RCAC on the STS is partly indicative of good bond characteristics between the aggregate and the mortar matrix.

Paine et al. (2009), from their limited data, observed that the existing Eurocode 2 relationship between CS and STS could be applied to RCA concrete as to conventional concrete. Kou and Poon (2008) observed that the relationship between STS and CS presented in ACI 318-89 overestimates their STS data on

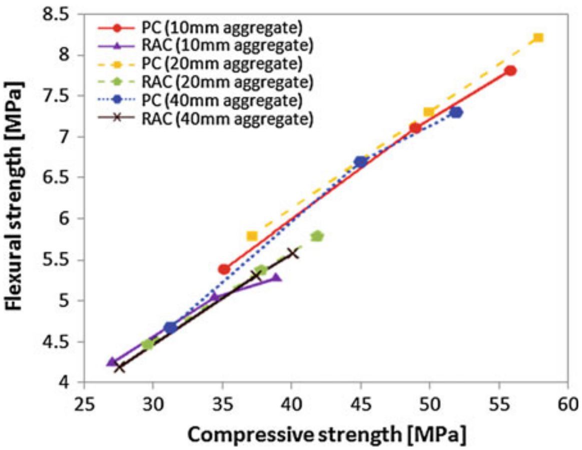


Fig. 5.30 Relationship between FS and CS of conventional concrete and RCAC (Padmini et al. 2009)

RCAC. They proposed the next relationship between STS $(f_{\rm sp})$ and CS $(f_{\rm cu})$ with a correlation coefficient of 0.87:

$$f_{\rm sp} = 0.0931\, f_{\rm cu}^{0.8842} \tag{5.2}$$

Xiao et al. (2006b) observed a significant overestimation of STS data using American ACI 318-02 and Chinese GB 50010-2002 codes. The authors proposed the following relationship between STS $(f_{\rm sp})$ and cubic CS $(f_{\rm cu})$:

$$f_{sp} = 0.24\, f_{cu}^{0.65} \tag{5.3}$$

5.3.3 Flexural Strength

Similarly to CS and the STS, the addition of CDW aggregate in concrete lowers the flexural strength (FS). However, in several studies, it is reported that this addition does not reduce FS as substantially as CS. The variation in FS between conventional concrete and concrete containing CDW aggregate was negligible in some studies and was lower than 30 % in others depending on the variations of different factors such as replacement amount, origin and quality of CDW aggregate, w/c ratio, design strength of concrete. Table 5.15 shows some of the results from various references.

Limbachiya et al. (2000, 2004) observed comparable 28-day FS for 50, 60 and 70 MPa concrete classes, prepared by replacing 0, 30, 50 and 100 % (by weight) of natural coarse aggregate by RCA. Safiuddin et al. (2011) did not observe any significant differences in 7- and 28-day FS of conventional and RCA concrete. The strength increased with the curing time like in conventional concrete. The improvement in interfacial bonding and mechanical interlocking due to the

Table 5.15 28-day flexural strength of concrete containing various amount of CDW aggregate

Reference	Type of aggregate	Tensile strength (MPa)/substitution level(%)/day	Comment
Limbachiya et al. (2004)	RCA/coarse	4.4/0/28; 4.3/30/28; 4.3/50/28; 4.5/100/28	–
Limbachiya et al. (2000)	RCA/coarse	5.2/0/28; 5.2/30/28; 4.9/50/28; 5.0/100/28[a]	[a, b, c] = 50, 60 and 70 MPa Concrete classes
		6.0/0/28; 6.1/30/28; 6.1/50/28; 6.0/100/28[b]	
		7.0/0/28; 6.9/30/28; 7.0/50/28; 7.2/100/28[c]	
Mas et al. (2012)	RCA/coarse	2.29/0/28; 2.73/20/28; 2.00/40/28[a]	[a, b, c] = concrete prepared at w/c of 0.65, 0.72 and 0.45
		3.83/0/28; 2.85/20/28; 2.70/40/28[b]	
		2.27/0/28; 2.09/25/28; 1.90/50/28; 1.81/75/28[c]	
Casuccio et al. (2008)	RCA/coarse 5-21 %	3.9/0/28; 3.7/100/28[a]; 3.2/100/28[b]; C18*	C18, C37 and C48 concrete classes; [a, b]: RCA from C55 and C30 concrete classes
		5.2/0/28; 5.3/100/28 [a]; 4.7/100/28[b]; C37	
		7.3/0/28; 6.0/100/28 [a]; 5.8/100/28[b]; C48	
Gupta et al. (2010)	RCA/coarse	6.0/0/28; 5.64/100/28; 6.2/0/28[a]; 6.18/100/28[a]; 4.78/100/28[b]	[a] and [b] = 10 and 20 % FA containing concrete
Gull (2011)	RCA/coarse + fine	6.67/0/3; 4.43/100/3; 6.38/100/3[a]	[a] = concrete contains water reducing admixture
		9.7/0/7; 6.0/100/7; 9.5/100/7[a]	
		13.0/0/28; 8.2/100/28; 13/100/28[a]	
Yang et al. (2011)	RCA/coarse	3.18/0/7; 4.19/0/28; 2.94/100/7; 3.61/100/28	–
Ahmed (2011)	RCA/fine	4.87/0/28; 4.77/25/28; 4.86/50/28; 4.03/75/28; 3.97/100/28	–
Heeralal et al. (2009)	RCA/coarse	4.19/0/28; 4.00/50/28; 3.46/100/28	–

*C18 indicates a concrete class with design 28-day strength of 28 MPa; the others are similar angularity and surface roughness of RCA aggregate as well as the effectiveness of interfacial bonding due to the orientation of larger coarse RCA along the specimen's length compensated the negative impact of the weakness of RCA and therefore maintained a FS similar to that of conventional concrete.

Chen et al. (2010) observed a slight increase in FS due to the replacement of up to 40 % of coarse NA by RCA and similar values to that of conventional concrete above this replacement level. In this study, the ratios of FS to CS were in the range of 0.11–0.13 when 10–100 % coarse NA was replaced by RCA. Ahmed (2011) observed similar 28 and 56 days FS of concrete due to the replacement of 25 and 50 % of natural fine aggregate by fine RCA. However, at 75 and 100 % replacement level, the FS was lower than that of conventional concrete. The 28-day FS of RCAC prepared by replacing 50 and 100 % of coarse NA by RCA in the Malesev et al. (2010) study were respectively 5.7 and 5.2 MPa in comparison to the 5.4 MPa of conventional concrete.

Yang et al. (2011) observed a 7.5–13.8 % reduction in FS due to replacement of 100 % coarse NA by RCA at various ages. Gull (2011) observed a reduction of around 37 % in 28-day FS of concrete due to the replacement of fine and coarse natural aggregates by RCA when both mixes were prepared at w/c ratio of 0.5. However, the 28 day FS of RCAC prepared at the same w/c ratio but by using a water reducing agent was similar to that of conventional concrete. Casuccio et al. (2008) observed a 5–21 % reduction in 28-day FS of concrete due to the replacement of 100 % coarse NA by RCA. Mas et al. (2012) observed 20, 13 and 30 % reductions in FS due to the replacement of up to 75 % (by volume) of coarse natural aggregate by low quality RCA in three types of concrete prepared at w/c of 0.65, 0.72 and 0.45 respectively.

Singh and Sharma (2007) observed a 4–15 % reduction in 1- to 28-day FS of 20 and 25 MPa concrete mixes due to replacement of coarse natural aggregate by RCA aggregate. James et al. (2011) observed a 28-day FS about 2.5 % lower due to the replacement of 25 % by mass of NA by RCA at a w/c ratio of 0.55. Like in conventional concrete, the FS increased with curing time. The differences in FS between conventional concrete and RCAC are lower at higher w/c ratios than at lower ones. The authors did not observe any effect of the w/c ratio on the CS of RCAC either.

Yong and Teo (2009) observed higher 3-day FS for RCAC than for conventional concrete up to a 100 % substitution level of coarse NA by RCA. However, the FS of conventional concrete was higher than that of RCAC when the curing age increased to 28 days. They also reported that the FS performance of RCAC was not as good as that observed for CS and STS due to the lower modulus of elasticity of RCA than NA's; therefore RCA tended to deform more than NA. In comparison to CS, Akbarnezhad et al. (2011) observed a lower reduction in the modulus of rupture as the replacement of coarse NA by RCA increased. At 100 % replacement, the reduction in modulus of rupture and CS was 15 and 30 % respectively. The higher water absorption capacity of RCA might enhance the bond strength between the new mortar and aggregate, which can partially

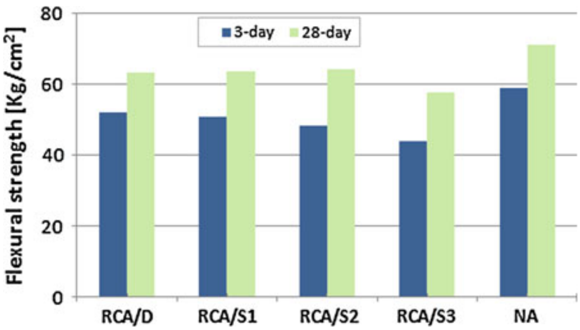


Fig. 5.31 FS of concrete containing dry NA and RCA with various moisture content (D: dry; S1, S2 and S3: 89.5, 88.1 and 100 % water saturated RCA) (Oliveira and Vazquez 1996)

compensate the negative effect related to the weakness of the old ITZ in RCA as the FS is largely dependent on the bond strength between aggregate and mortar matrix.

The concrete containing RCA from high-strength concrete exhibited higher FS than the one containing RCA from low strength concrete Limbachiya et al. (2000). Topçu and Sengel (2004) observed a systematic decrease in the FS of 16 and 20 MPa (cylindrical) concrete classes as the content of coarse RCA in concrete increased. The reduction was 13 and 27 % for the 16 and 20 MPa mixes respectively at a 100 % substitution level. Padmini et al. (2009) observed lower FS for RCAC than for conventional concrete and the differences in terms of CS and FS decreased with a reduction of the design CS (Fig. 5.30).

Takavoli and Soroushian (1996) observed higher FS for RCAC using smaller coarse RCA than for bigger coarse RCA and in some cases the FS was higher than for the control concrete. Oliveira and Vazquez (1996) observed a reduction from the control concrete of about 10 % in the 3- and 28-day FS of mixes made with coarse RCA with different moisture levels (Fig. 5.31). The FS of concrete containing dry RCA or around 90 % saturated RCA were comparable; however, the FS of concrete containing saturated surface dried RCA was significantly lower than that of the others.

Katz (2003) observed similar 28-day FS for conventional and concrete with coarse RCA and a mixture of fine RCA and natural sand, obtained from 1-day cured concrete; however, the FS of RCAC with RCA obtained from 3- and 28-day concrete was 11.5 % lower than that of conventional concrete. On the other hand, the reduction in FS was respectively 29.9, 20.9 and 31.3 % in concrete with white Portland cement as binder for 1-, 3- and 28-day cured RCA. Yang et al. (2011) observed 3 and 9 % reductions from the control concrete in the 7- and 28-day FS of concrete with RCA as sole coarse aggregate. The addition of crushed brick up to a 50 % substitution level of RCA slightly increased the FS due to the low Young modulus of brick and therefore improved the tensile stress along the matrix-aggregate interface (Fig. 5.32). The failure modes for control and RCA concrete occurred at the aggregate and mortar matrix's interface while both interface and aggregate failure occurred in RCAC containing crushed brick.

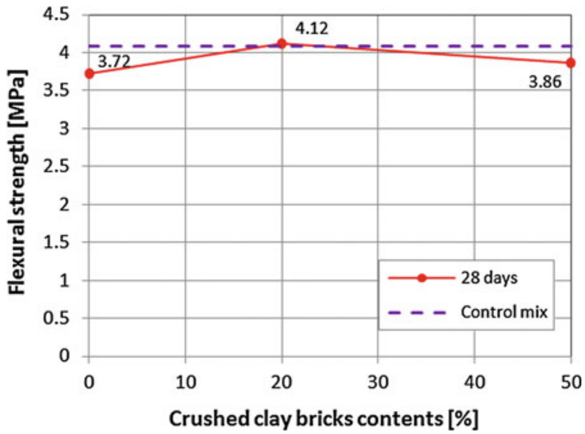


Fig. 5.32 Effect of crushed brick content in coarse RCA on the FS performance of concrete (Yang et al. 2011)

The FS of RCAC in the Chen et al. (2003) study was about 78 % that of conventional concrete when both mixes had a w/c ratio of 0.5. The substitution of 17, 33, 50 and 67 % of RCA by a mixture of bricks and tiles can slightly increase the FS as shown in Fig. 5.33. The differences in FS between conventional and RCA mixes gradually decreased with the w/c ratio. The FS of RCAC was similar to that of conventional concrete at the w/c ratio of 0.67 and significantly higher at w/c of 0.8. The FS of concrete containing RCA with sand-sized particles as well as other impurities like bricks and tiles was much lower than that of conventional concrete especially at lower w/c ratio.

Ahmed (2011) observed that the replacement of 30 % by mass of cement by FA in concrete containing 25 % fine RCA and the replacement of 30 and 40 % by mass of cement by FA in concrete containing 50 % fine RCA could improve the FS (Fig. 5.34). In the Jemas et al. study, the incorporation of FA to replace 10 and 15 % of OPC improved the FS of RCAC and the FS of RCAC prepared at a w/c ratio of 0.55 was even better than that of conventional concrete (Fig. 5.35).

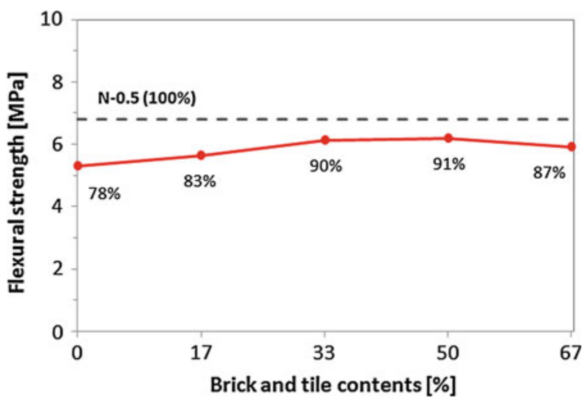


Fig. 5.33 Effect of brick and tile content on the FS of RCAC (Chen et al. 2003)

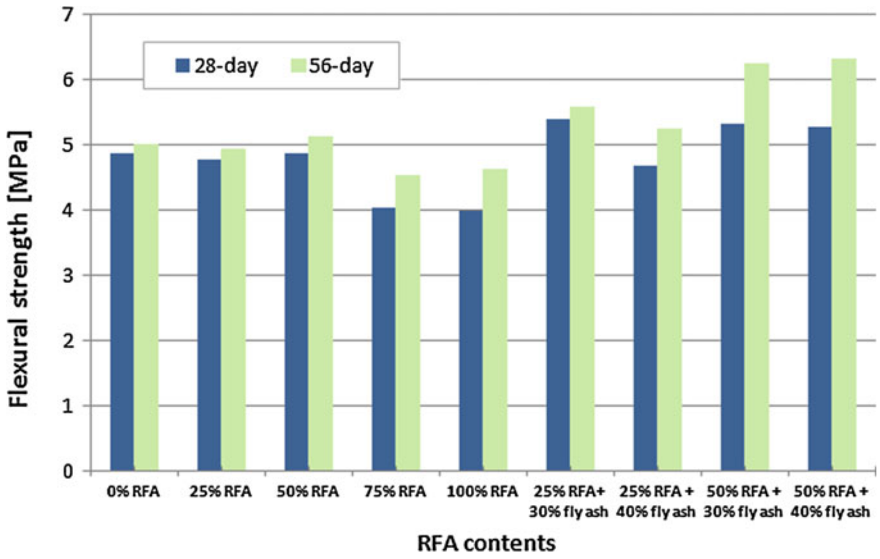
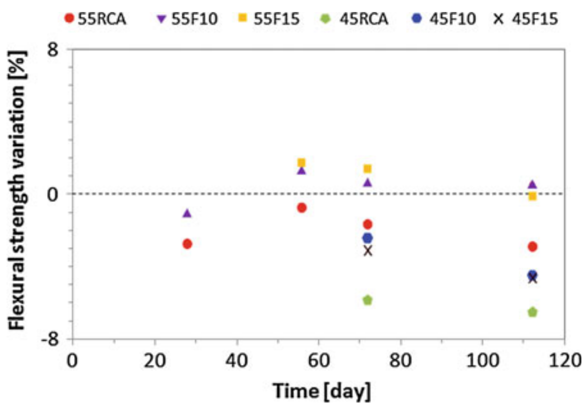


Fig. 5.34 Effect of FA on the FS performance of concrete with fine RCA (Ahmed 2011)

Fig. 5.35 Variation of the FS of conventional concrete and RCAC with and without FA, prepared at w/c of 0.55 and 0.45 (James et al. 2011)



Gupta et al. (2011) observed a reduction of about 6 % in 28-day FS in comparison to a 21.5 % reduction in the corresponding CS, due to the replacement of 100 % coarse NA by RCA. On the other hand, the FS of RCAC with FA as a 10 % replacement of OPC was around 3 % higher than that of conventional concrete and similar to that of conventional concrete with 10 % FA. The FS of RCAC with 20 % FA, however, was about 20 % lower than that of conventional concrete with OPC only. Rao and Khan (2009) observed reductions of about 4 % in FS due to the replacement of 50 % (by mass) of coarse NA by coarse RCA. However, the incorporation of 0.01–0.03 % glass fibre improved the FS of the resulting RCAC.

Tam et al. (2007) observed advantages of the use of a two-stage mixing approach instead of the normal mixing approach to increase several mechanical

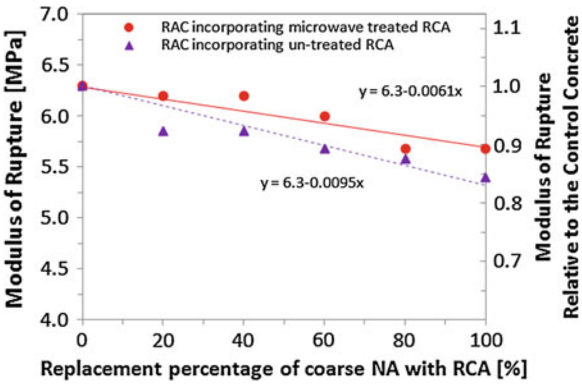


Fig. 5.36 Effect of microwave treatment of RCA on the FS performance of concrete (Akbarnezhad et al. 2011)

properties including the FS of RCAC. The substitution of coarse NA by RCA in the 25–40 % range yielded the optimal FS, along with the other mechanical properties of concrete using the two-stage mixing approach.

Akbarnezhad et al. (2011) observed that the FS of concrete with coarse RCA obtained after a microwave treatment was higher than that observed for concrete with untreated RCA as the microwave treatment can remove adhered mortar content as well as weak RCA particles from concrete (Fig. 5.36). Li et al. (2009) observed an improvement in the FS of RCAC due to the coating of RCA by blast furnace slag, fly ash and silica fume separately or an equal weight mixture of two of these three. These materials were used to replace 20 % of OPC (by weight) in concrete. The improvement was highest for the silica fume and fly ash mixture due to the higher packing density of this mixture.

Yang et al. (2008), after analysing 197 test results of available database along with their own experimental results, observed a decrease of the rupture modulus (indicative FS) of concrete as the water absorption capacity of CDW aggregate increased. The normalized rupture modulus, $f_r/\sqrt{f_c}$, where f_r and f_c are the rupture modulus and CS of concrete against water absorption capacity of RCA respectively, are presented in Fig. 5.37. They observed that the rupture moduli of the control concrete and of the concrete with grade I RCA according to the Korean standard satisfied the expression in the ACI 318-05 norm, regardless of their substitution level of coarse natural aggregate,. On the other hand, the rupture moduli of the concrete with 50 % grade II and of the concrete with grade III RCA were slightly lower than the ACI specified value. The reduction in the rupture modulus due to the incorporation of RCA was due to the weak binding between the components in the concrete matrix owing to the adhered cement paste on the RCA surfaces.

Takavoli and Soroushian (1996) observed lower FS for RCAC than that predicted from the CS of RCAC according to the American ACI Code 318 expression and the difference became larger at higher w/c ratios. Ahmed (2011) reported that the expression used in Australian code, AS3600-2009 could be used for concrete with fine RCA but the same expression for concrete with fine RCA and FA

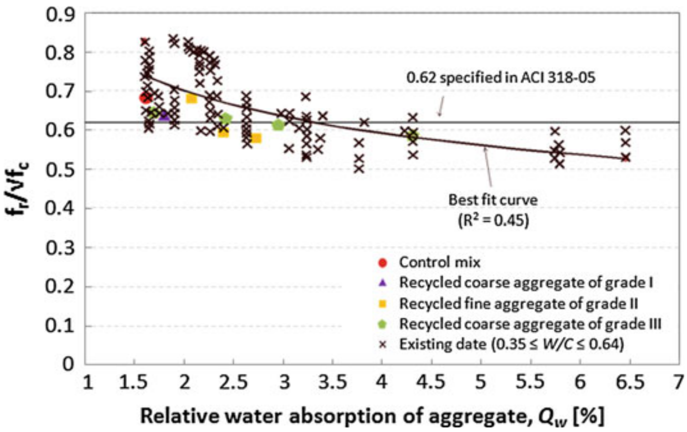


Fig. 5.37 Performance of normalized rupture modulus with the water absorption capacity of natural and CDW aggregate (Yang et al. 2011)

predicts a significantly lower FS value then the experimental value. In the James et al. (2011) study, the existing ACI 318 (2008) code underestimated their FS results. Katz (2003) observed higher values of 28-day FS of conventional concrete and RCAC than the value predicted according to ACI 363R.

5.3.4 Modulus of Elasticity

Similarly to the strength properties, the modulus of elasticity (MO) of concrete containing CDW aggregate is normally lower than that of conventional concrete and it decreases as the content of CDW aggregate in concrete increases. Some typical results are presented in Table 5.16. The causes for reduction in concrete's MO due to the incorporation of CDW aggregate indicated in various references are: (1) the loss of concrete stiffness, which depends on properties such as stiffness of mortar, concrete porosity and aggregate-cement paste bonding; these properties deteriorate due to the addition of RCA to concrete; (2) the lower MO of CDW aggregate than that of natural aggregate, since the concrete's MO is primarily dependent on the MO of the aggregate.

Depending on factors such as substitution ratio, quality and size of aggregate, w/c ratio, the reduction in MO of RCAC may reach 50 % when compared to conventional concrete. The reduction in concrete's MO due to the incorporation of CDW aggregate is generally higher than the corresponding CS reduction. Safiuddin et al. (2011) observed a reduction of about 17.7 % in the 28-day MO in comparison to a 12.2 % reduction in CS of concrete with 100 % coarse RCA when compared to conventional concrete. Chen et al. (2003) observed a 22 % reduction in MO in comparison to a 15 % reduction in CS of concrete due to the inclusion of

Table 5.16 MO of concrete containing various amount of CDW aggregate

Reference	Type of aggregate	Modulus of elasticity (GPa)/substitution level(%)/day	Comment
Evangelista and de Brito 2007	RCA/fine	35.5/0/28; 34.2/30/28; 28.9/100/28	Type of addition: v/v
Etxeberria 2007b	RCA/coarse	32.6/0/28; 31.3/25/28; 28.6/50/28; 27.8/100/28	Type of addition: v/v
Gonzalez-Fonteboa and Martinez-Abella 2002	RCA/coarse	29.7/0/28; 29.1/15/28; 27.8/30/28; 26.6/60/28; 26.7/100/28	Type of addition: v/v
Gonzales–Fonteboa et al. 2011	RA/coarse	29.6/0/28[a]; 28.2/20/28[a]; 26.4/50/28[a]; 24.3/100/28[a] 33.9/0/28[b]; 32.6/20/28[b]; 28.8/50/28[b]; 24.0/100/28[b]	[a] and [b] = water to cement ratios of 0.65 and 0.50 respectively; Type of addition: v/v
Gonzalez-Fonteboa and Martinez-Abella 2008		32.2/0/28[a]; 31.4/0/28[b]; 32.5/0/115[a]; 32.8/0/115[b]; 28.6/100/28[a]; 27.3/100/28[b]; 28.9/100/115[a]; 28.3/100/115[b]	[a] and [b] = concrete containing OPC and OPC-SF as binder respectively; Type of addition: v/v
Rao et al. 2011	RCA/coarse	31.2/0/28; 26.8/25/28; 26.7/50/28; 26.4/100/28	Type of addition: v/v
Domingo-Cabo et al. 2010	RCA/coarse	33.3/0/28; 32.4/20/28; 33.5/50/28; 33.3/100/28[, 1] 36.2/0/28; 32.4/20/28; 34.1/50/28; 31.0/100/28[b, 1] 32.2/0/28; 31.2/20/28; 31.2/50/28; 31.6/100/28[2]	[a] = cured at 100 % humidity condition and at 20 °C; [b] = initial 18 days in 100 % humidity and next 10 days in 65 % humidity and at 23 °C; [1] = prepared at constant w/c; 2: prepared at constant slump by considering absorbed amount of water by RCA; Type of addition: v/v
Thangchirapat et al. 2008	RCA/ coarse + fine	34.9/0/28; 31.0/100/28[a]; 29.8/100/28[b]; 26.7/100/28[c]	[a] = replaced coarse NA; [a] = replaced coarse NA + 50 % fine NA; [a] = replaced coarse NA + 100 % fine NA; Type of addition: w/w
Berndt 2009	RCA/coarse	47.2/0/28; 40.1/100/28; 45.6/0/28[a]; 36.2/100/28[a]	[a] = 30 % OPC-70 % slag as binder; Type of addition: v/v

RCA as 100 % (by volume) replacement of coarse NA when the concrete was prepared at w/c of 0.5. In contrast to the CS and STS performance, Etxeberria et al. (2007b) also observed a decrease of the MO caused by the addition of coarse RCA as a replacement of NA due to the lower MO and higher deformation of RCA than NA's. The percentage reduction in 28-day MO due to the replacement of 25, 50 and 100 % of NA by RCA was around 4, 12 and 15 % respectively. Rahal (2007) observed a reduction of only 3 % in the MO for concrete with cylindrical strength between 25 and 30 MPa due to replacement of coarse NA by RCA.

Gonzales-Fonteboa et al. (2011) observed considerable decrease in the MO due to the addition of coarse RCA in concrete, because of the lower MO of RCA than NA's. They observed 4.7, 10.9 and 18.0 % reduction in MO due to the replacement of 20, 50 and 100 % of coarse NA by RCA in concrete at w/c of 0.65. These values were 3.8, 14.9 and 29.2 % at w/c of 0.50. Berndt (2009) observed a reduction of about 15 % in the MO due to a 100 % replacement of coarse NA by RCA. According to them, the low MO of RCAC might have an impact on the structural response (e.g. a low stiffness material is less susceptible to cracking). Corinaldesi (2011) observed a reduction of around 17 % in 28-day MO due to the replacement of 30 % coarse NA by RCA. Xiao et al. (2006a) observed a 45 % reduction in the MO of concrete due to the incorporation of coarse RCA as a 100 % replacement of coarse NA when compared to conventional concrete. Frondistou-Yannas (1977) observed a reduction of up to 40 % in the MO in comparison to a 4–14 % reduction in CS of concrete due to the incorporation of coarse RCA as a full replacement of NA.

In the Rao et al. (2011) study, the MO decreased with the replacement ratio of coarse NA by RCA due to the weaker ITZ between RCA and cement mortar and the lower MO of RCA than NA's. The reduction percentage in 28-day MO of concrete due to the replacement of 25, 50 and 100 % NA by RCA were 14.3, 14.4 and 15.4 % respectively. Evangelista and de Brito (2007) observed a reduction of about 3 % in the 28-day MO of concrete containing fine RCA as a replacement of 30 % by volume of fine NA; however, the MO of concrete containing 100 % fine RCA was 18.6 % lower than that of conventional concrete. The loss of concrete stiffness, which depends on properties such as stiffness of mortar, concrete porosity and aggregate-cement paste bonding was not as significant for smaller incorporation ratios of fine RCA as for higher ones and therefore the MO of concrete was slightly affected for small ratios of fine RCA.

Kou and Poon (2008) observed a 17–23 % reduction in 28-day MO of concrete containing three types of coarse RCA as a 100 % replacement of NA when compared to conventional concrete; however, after 5 years of curing, the reduction dropped to around 10 % indicating higher gain over time in MO of RCAC than that observed in conventional concrete. The increase in MO of the mixes containing three types of RCA as a 100 % replacement of coarse NA between 28 days and 5 years was in the range of 33–40 % in comparison to a 20 % improvement in conventional concrete. Gomez-Soberon (2002) observed a gradual decrease of the MO as the replacement of coarse NA by RCA increased up to 60 % and then became similar at 100 % replacement level (Fig. 5.38). The development of the

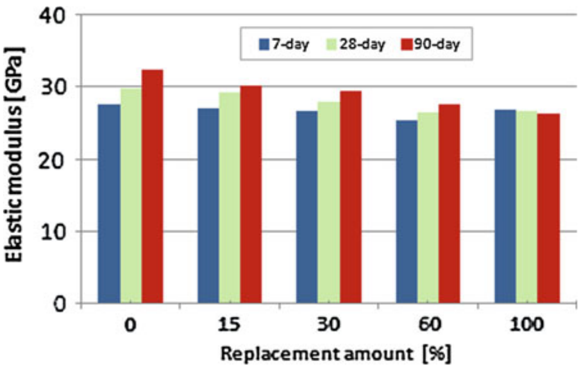


Fig. 5.38 MO of concrete at various ages due to the replacement of coarse NA by coarse RCA (Gomez-Soberon 2002)

MO gradually slowed down with increasing content of RCA in concrete. The MO of concrete containing RCA at 100 % replacement level was almost the same at the various curing ages. The authors could not establish a relationship between total porosity and MO even though the MO decreased as the open porosity increased up to around a 15 % porosity level. Safiuddin et al. (2011) observed a smaller increase in MO as curing time increased than that observed for FS and STS. In this study, the 28-day MO of RCAC was only 11.2 % higher than 7-day value, whereas the FS and STS increases were respectively 40.3 and 17.3 %. The MO increased with the concrete's CS too.

Domingo-Cabo et al. (2010) found a decrease of the 28-day MO due to the incorporation of a good quality coarse RCA as a 0, 20, 50 and 100 % (by volume) replacement of coarse NA in mixes with similar w/c ratio. Unlike most investigations, the RCA used in this one was not pre-saturated before concrete mixing; instead a super-plasticizer was used to prepare a workable mix. After the RCAC were prepared at constant slump by considering the amount of water absorbed by RCA, the 28-day MO was similar to that of conventional concrete. Padmini et al. (2009) observed a significant reduction in MO of concrete with the incorporation of RCA as coarse aggregate owing to the increase of porosity of concrete due to that incorporation. The higher reduction in percentage of the MO was observed for concrete made with smaller coarse RCA due to their higher porosity. However, no effect was detected of the strength of original concrete from which the RCA were generated on the MO of RCAC. Corinaldesi (2010) observed a reduction of around 23 and 13 % in 28-day MO of concrete prepared at w/c of 0.40 and 0.45 respectively, due to the replacement of 30 % of fine and coarse gravel (6–12, FR and 11–22 mm, CR) by similar sized RCA (Fig. 5.39). However, these values for fine and coarse gravel RCA became 22 and 32 % respectively at the w/c ratio of 0.60. Thangchirapat et al. (2008) observed a reduction of around 11 % in the 28-day MO due to a 100 % replacement by weight of coarse NA by RCA. The replacements of fine NA by fine RCA in the concrete with 100 % coarse RCA further lowered the MO. The reduction in MO of mixes with 50 and 100 % fine RCA was respectively around 14 and 24 %.

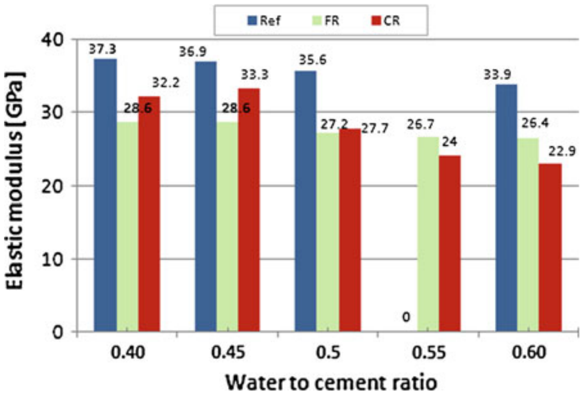


Fig. 5.39 MO of concrete with NA and RCA at different w/c ratios (Corinaldesi 2010)

Table 5.17 MO (E-modulus) of conventional and RCA-containing concrete (Hansen and Boegh 1985)

Item[a]	H	H/H	H/M	H/L	M	M/H	M/M	M/L	L	L/H	L/M	L/L
E- modulus (GPa)	43.4	37.0	36.3	34.8	38.5	33.0	32.0	30.0	30.8	27.5	22.3	22.6
Reduction (%)	–	14	16	20	–	14	17	22	–	11	28	27

[a] Details about H, H/M etc. are in Fig. 5.12

Kou and Poon (2008) observed a decreasing of the MO of conventional as well as of RCA mixes with the water to binder ratio. In comparison to conventional concrete, Hansen and Boegh (1985) observed an 11–28 % reduction in the MO of three different classes of concrete with RCA from three different classes of concrete as a full replacement of coarse NA when the various types of concretes were subjected to 47-day accelerated curing conditions. Depending on the strength of the original concrete, the reduction percentage slightly varied, as presented in Table 5.17.

The reduction in MO of concrete due to the incorporation of RCA was more pronounced in water curing than in steam curing especially at the smaller ratios of RCA (Poon et al. 2006) (Fig. 5.40). Fonseca et al. (2011) reported that the MO of concrete decreased as the incorporation of coarse RCA as a replacement of NA increased, due to the increase in porosity of concrete. The effect of curing conditions on the MO of conventional and RCA concrete was determined in this investigation. The authors used four curing conditions: water immersion, wet chamber, outer environment and laboratory. They observed the lowest MO of both types of concrete when the specimens were cured in laboratory conditions, the driest condition in this study (Fig. 5.41). The variations in MO of both types of concrete were not significant in other curing conditions. The cause of the low MO of concrete observed in laboratory curing condition was the formation of a porous microstructure of cement pastes due to low humidity.

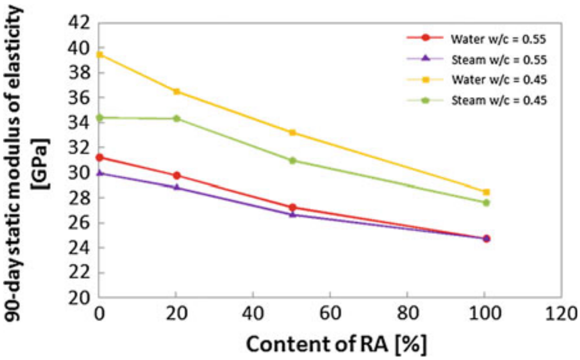


Fig. 5.40 Effect of curing conditions on the MO of concrete (Poon et al.2006)

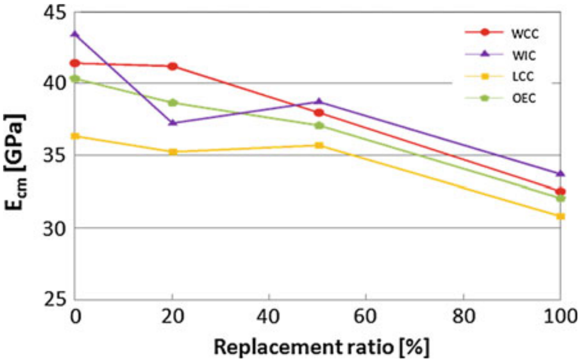


Fig. 5.41 Effect of curing conditions on the MO of concrete containing various ratios of coarse RCA (Fonseca et al. 2011)

Oliveira and Vasquez (1996) observed comparable 28-day MO for concrete containing coarse RCA with various moisture levels; the MO of RCAC was about 75 % that of conventional concrete. Chen et al. (2003) observed insignificant influence of the quality of RCA whether it was washed or unwashed with impurities such as sand particles, bricks and tiles on the MO of the resulting concrete. The differences in MO of conventional and RCA concrete were also similar with the w/c ratio (Fig. 5.42). Chen et al. (2003) also found an insignificant effect of brick and tile contents on the MO of RCAC (Fig. 5.43).

Tam et al. (2007) observed higher MO for concrete prepared using a two-stage mixing approach (in which mixing of the water was divided into two parts: the first one added to the mixed aggregate and the remaining part to the mixed aggregate and cement) than that observed for concrete prepared using a one-step mixing approach (in this approach the whole amount of water was added to the mixed aggregate and cement). In this study, the replacement of 31.3 % of NA by RCA gave the highest improvement in 28-day MO of RCAC prepared by the two-stages mixing when compared to conventional concrete. In another study, Tam and Tam (2008) observed an improvement of about 16 % in the 28-day MO of concrete containing coarse RCA as a 30 % replacement of NA due to the incorporation of silica fume as a 2 % replacement of OPC where the concrete was prepared by a

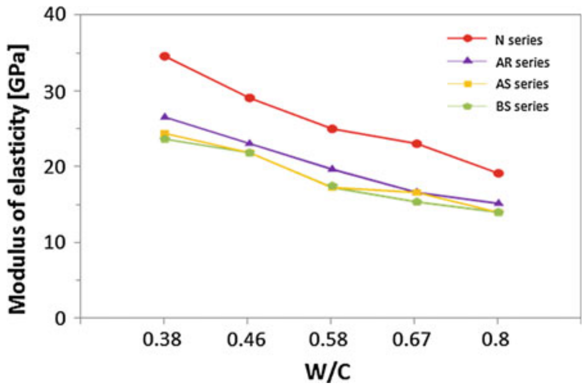


Fig. 5.42 Effect of w/c ratio on the MO of concrete containing NA (N), washed (AR) and unwashed (AS) RCA (Chen et al. 2003)

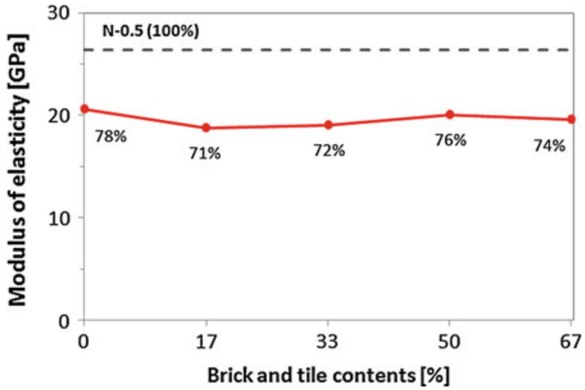


Fig. 5.43 Effect of brick and tile content in RCA on the MO of concrete (Chen et al. 2003)

two-stage mixing approach, when compared to the MO of equivalent RCAC containing 2 % SF prepared by a one-step mixing approach. In comparison to the MO of RCAC prepared by conventional mixing method, Razaqpur et al. (2010) observed 11 and 15 % improvements in the MO of RCAC prepared using the equivalent mortar volume (EMV) method (details are given in the CS section). The MO of RCAC produced by EMV method was comparable or even better than that of conventional concrete because the total natural aggregate volume was the same in both types of concrete. Akbarnezhad et al. (2011) observed a significant improvement in the MO of RCAC due to the use of microwave treated RCA (MRCA). In comparison to conventional concrete, the reduction in MO of RCAC with MRCA as the only coarse aggregate was around 10 % whereas it was around 25 % for RCAC with untreated RCA.

In several references, it was reported that the use of mineral additions could not improve the MO of RCAC although mineral additions normally improve the various strength properties. According to Thangchirapat et al. (2008), the concrete's MO mainly depends on the properties of the aggregate rather than the strength of the cement paste. In their study, they did not observe any beneficial effect of rice husk ash addition into OPC on the MO performance of concrete.

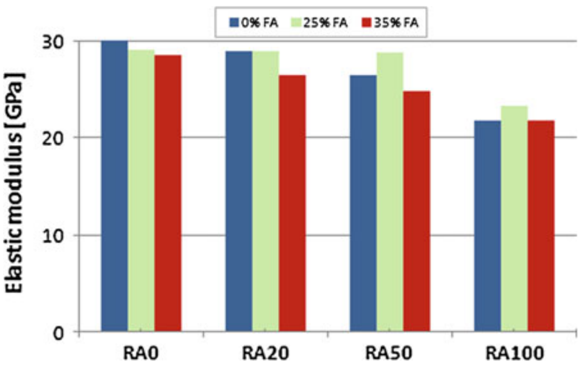


Fig. 5.44 Modulus of elasticity of concrete containing various amount of coarse RCA aggregate due to FA addition Kou and Poon (2006)

Similarly, Gonzalez-Fonteboa and Martinez-Abella (2008) did not observe any improvement of the MO of concrete containing RA as coarse aggregate due to the replacement of 8 % (by weight) of OPC by silica fume. They observed respectively reductions of about 15 and 13 % in the 28- and 115-day MO of concrete with SF while similar reductions of around 11 % were observed for OPC mixes after 28 and 115 days of curing. Berndt (2009) observed a lower MO of RCAC due to the incorporation of blast furnace slag as 50 and 70 % replacement of OPC. At 70 %, the MO of RCAC was around 20 % lower than that of the concrete containing 70 % slag and without RCA.

Kou and Poon (2006) observed a slight improvement of the 28-day MO of concrete containing RCA as a 20, 50 and 100 % replacement of coarse NA due to the incorporation of FA as a 25 % replacement of OPC (Fig. 5.44). However, for 35 % FA addition, the MO of RCAC was lower than the one of RCAC without FA.

Lopez-Gayarre et al. (2009) applied the analysis variance (ANOVA) method to analyse their experimental results on the use of RCA as a replacement of coarse NA in concrete. Experimental variables such as quality of aggregate, replacement ratio, size distribution, declassified content, strength of original concrete and concrete slump value were considered for analysis. Out of these parameters, they observed significant influence of the quality of aggregate on the MO of RCAC when the 100 % coarse NA was replaced by RCA. However, the aggregate's quality had negligible effect at 20 and 50 % replacement levels.

In several studies, the existing relationships between CS and MO (E) proposed in various standard specifications were applied to check the validity of these relations in concrete containing RCA. As for example, the MO of concrete containing various types of CDW aggregate from the Pain et al. (2009) study was around 20 % lower than the MO values estimated by using the Eurocode 2 expression. Rahal (2007) reported that the expression presented in ACI 318-02 for the relationship between CS and MO of concrete overestimated the experimental results for conventional as well as RCA concrete mixes. Kou et al. (2007) and Kou and Poon (2008) also reported that the existing ACI 363R-92 expression overestimated their experimental results. On the other hand, the experimental MO of conventional as well as RCA concrete from the Oliveira and Vasquez (1996) study were consistent with the CEB-FIP model code (The International Federation for Structural Concrete).

Several expressions were also proposed to describe the relationship between MO (E) and cubic CS (f_{cu}) in earlier studies, some of which are presented below:

Ravindraraja and Tam (1985):

$$E = 7770 \times f_{cu}^{0.33} \tag{5.4}$$

Kakizaki et al. (1988):

$$E = 1.9 \times 10^5 \times f_{cu} + \left(\frac{\rho}{2300}\right)^{1.5} \times \sqrt{\frac{f_{cu}}{2000}} (\rho = \text{density}) \tag{5.5}$$

Dhir et al. (1999):

$$E = 370 \times f_{cu} + 13100 \tag{5.6}$$

Some new equations were recently proposed to establish the relationship between MO (E) and cubic CS (f_{cu}) of RCAC. Corinaldesi (2011) observed that the existing relationship between 28-day cubic CD and MO (f_{cu} and E respectively) as described in Italian norms, NTC 2008 and presented in Eq. (5.7)) could be applied to the results obtained for conventional concrete but a different relationship (Eq. (5.8)) was necessary for RCAC:

$$E = 22.0 \sqrt[3]{\frac{0.83 \cdot f_{cu}}{10}} \tag{5.7}$$

$$E = 18.2 \sqrt[3]{\frac{0.83 \cdot f_{cu}}{10}} \tag{5.8}$$

In another study, Corinaldesi (2010) proposed the following expressions to establish relationships between 28-day cubic CS and MO of RCAC containing fine and coarse RCA, respectively:

$$E = 18.8 \sqrt[3]{\frac{0.83 \cdot f_{cu}}{10}} \tag{5.9}$$

$$E = 909 \times f_c + 8738 \tag{5.10}$$

From their experimental results, Evangelista and de Brito (2007) observed that the inclusion of concrete's density (ρ) according to the following equation was necessary to establish a relationship between cubic CS (f_c) and MO (E):

$$E = a \times (f_c + 8)^{\frac{1}{3}} \times \left(\frac{\rho}{b}\right)^2 \tag{5.11}$$

Where a, b and s are the regression coefficients whose values are 8917, 2348 and 0.85 respectively.

From the experimental results in various references and also their own, Xiao et al. (2006b) proposed the following expression to relate the CS and the MO of RCAC:

$$E = \frac{10^5}{2.8 + \frac{40.1}{f_{cu}}} \tag{5.12}$$

5.3.5 Flexural and Shear Performances

Several investigations were undertaken to understand the flexural and shear performances of concrete containing RCA. Here, some results will be highlighted from relevant references.

Razaqpur et al. (2010) observed higher ultimate FS in reinforced RCAC beams than in conventional concrete beams regardless of the source of RCA or the tension and steel contents of the beam. The RCAC were prepared using a new mixing method where the total mortar content (new and old) in RCAC and conventional concrete were the same. The flexural failure modes and cracking patterns of both types of concrete were similar. The mid-span deflections of both types of concrete also met the American ACI 318 M-05 specification limit. However, the RCAC beams showed lower cracking moments and slightly smaller crack spacing than the conventional concrete beam.

Sato et al. (2007) observed higher flexural deflection for reinforced concrete beams containing RCA than for conventional concrete under the same moment and w/c value. In these conditions, they observed similar ductility factors, ultimate moments and crack spacing in the control concrete beam as well and the concrete beams containing coarse or fine RCA but the crack width of concrete containing coarse or fine RCA were larger than that observed in the conventional concrete beam. The crack spacing and crack width of the RCAC beam containing coarse RCA were in the ranges of 0.92–1.37 and 0.57–1.3 times those of the conventional concrete beam respectively. The same parameters for RCAC containing fine RCA were 0.74–1.26 and 1.1–1.7 times those of the conventional concrete. They did not detect any cracking or deflection for 1 year under wet conditions but observed many cracks and two times more deflection than in the conventional concrete beam under dry condition when the concrete beams containing fine RCA and conventional concrete were kept under sustained bending moment equivalent to 100 N/mm^2 in tension rebar stress in reinforced concrete sections. Regardless of the type of aggregate, the ultimate moments of the concrete beams can be predicted from the Japanese code, JSCE 2002e.

Tsujino et al. (2007) investigated the flexural performance of concrete containing untreated and oil-coated low and medium qualities RCA as coarse

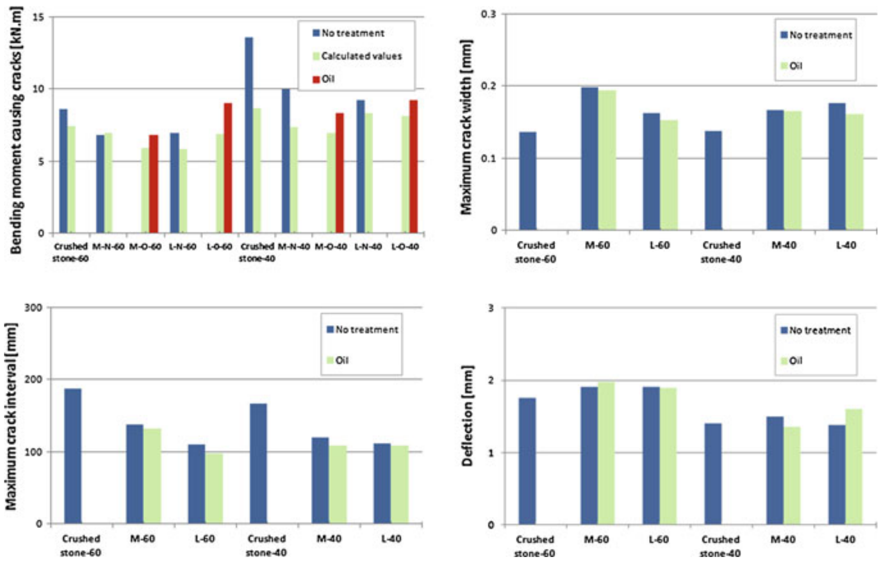
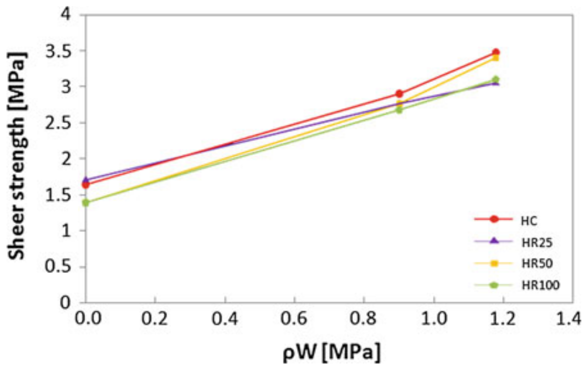


Fig. 5.45 Flexural behaviour of conventional concrete and concrete containing different types of RCA (M and L: medium and low quality RCA; N and O: untreated and oil treated RCA, 60 and 40: w/c of 0.6 and 0.4, respectively) (Tsujino et al. 2007)

aggregates along with conventional concrete at w/c ratios of 0.6 and 0.4 and their results are presented in Fig. 5.45. The cracking moments of concrete containing RCA were slightly lower than that observed for conventional concrete; the differences in cracking moment between conventional concrete and RCAC were higher at low w/c ratio. However, the cracking moment of RCAC can be predicted by conventional equation. The use of RCA as coarse aggregate in concrete also lowered the maximum crack spacing and increased deflection and maximum crack width. However, the maximum crack widths of all types of RCAC were significantly lower than 0.3 mm, the allowable crack width limit for reinforced concrete as specified by the Japanese Architectural Institute. The deflections of RCAC were marginally higher than that of conventional concrete at the w/c ratio of 0.4, indicating problems related to deflection can be overcome by lowering the w/c of RCAC. The authors did not find any beneficial effect on the flexural performance of RCAC due to the use of surface treated RCA.

Etxeberria et al. (2007b) studied the shear behaviour of concrete beams prepared by replacing 0, 25, 50 and 100 % (by volume) of coarse NA by RCA. The beams were prepared with and without transverse reinforcement. They observed a negligible influence of a 25 % replacement of NA by RCA on the shear strength of concrete beam, especially for the beam without transverse reinforcement; however, the shear strength decreased at higher replacement levels (Fig. 5.46). They also concluded that modifications in the concrete composition such as an increase in the cement amount and a decrease of the w/c ratio were necessary to control the shear strength loss due to the incorporation of RCA in concrete.

Fig. 5.46 Effect of transverse reinforcement in concrete beams made with NA only (HC) and by replacing 25, 50 and 100 % (HR25, HR50, HR100) of coarse NA by RCA



5.3.6 Stress–Strain Relationship

The analysis of the stress–strain curve (SSC) of concrete can yield data such as strength and toughness performance and therefore the evaluation of SSC is essential for structural design of concrete. Several studies were done to evaluate the SSC of concrete with CDW aggregate. The results from a few studies are presented next.

Topçu and Guncan (1995) determined several factors such as toughness, plastic energy capacity and elastic energy capacity from the stress–strain curves of conventional concrete and concrete containing various amount of RCA as a replacement of coarse NA. They observed a gradual decrease of toughness, plastic and elastic energy capacities of the latter as the incorporation of RCA in concrete increased.

Xiao et al. (2006a) observed a significant influence of RCA and replacement ratio of coarse NA by RCA on the stress–strain curve of the resulting concrete (Fig. 5.47). The incorporation of RCA increases the peak strain but significantly decreases the ductility of concrete i.e. ultimate strain. At 100 % replacement of NA by RCA, the increase in the peak strain was 20 %. The higher peak strain was due to the lower stiffness of RCA than that of NA. They also observed that the

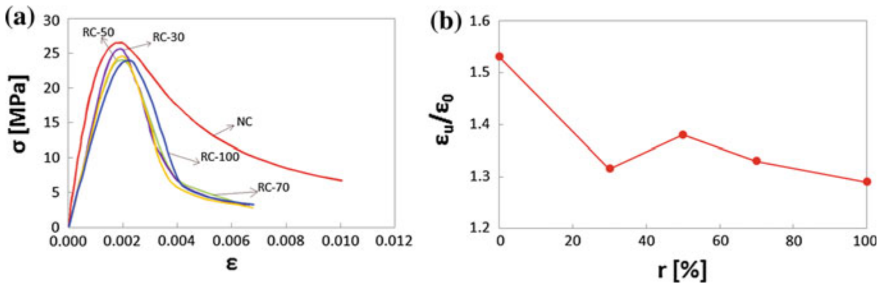


Fig. 5.47 Stress (σ)–strain (ε) curves and ultimate strain of concrete with replacement of coarse NA by coarse RCA (Xiao et al. 2006a). **a** Stress–strain curve. **b** Ultimate strain

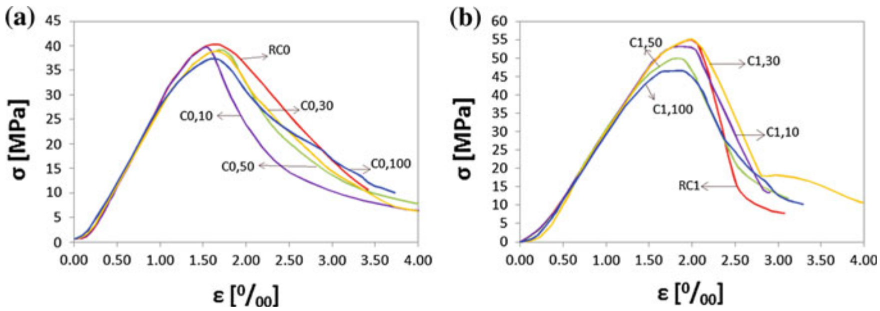


Fig. 5.48 Stress–strain curve of concrete with replacement of fine NA by fine RCA: **a** Without superplasticizers; **b** With superplasticizers (Pereira et al. 2012)

commonly used relationship between CS and MO (E) for NAC was not applicable for RCAC. They also successfully applied the analytical relationship proposed in the Chinese Code GB 50010 for uniaxial compression of NAC after a slight modification to predict the stress–strain curve of RCAC in similar condition.

Tsujino et al. (2006, 2007) studied the load–deflection curves of reinforced concrete beams of conventional concrete as well as of concrete containing untreated and oil treated coarse RCA. They found marginal differences between the NA and the RCA or between the treated and untreated RCA in terms of the plastic behaviour of concrete made with them namely load deflection curves, ultimate concrete strain at compression fibres, ultimate bending moment and toughness ratio or ductility factor. The ultimate concrete strains at compression fibres, bending moment and ductility factors were around 3500 μ, 30 kN.m and 5 respectively for all types of concrete.

Pereira et al. (2012) did not observe any major differences in the stress–strain curve of concrete due to the incorporation of fine RCA as a replacement of fine NA in concrete (Fig. 5.48a). However, the RCAC exhibited earlier stiffness losses than the NAC due to the lower CS of RCAC and the fragile adhered cement paste that facilitates the propagation of cracks. However, the use of superplasticizers increased the yield stress but decreased the yield path length in the stress–strain curves of NAC and RCAC (Fig. 5.48b). Ajdukiewicz and Kliszczzewicz (2002) reported that RCAC with recycled basalt aggregate exhibited more brittle behaviour than RCAC with recycled granite aggregate. The stress–strain curves of high-strength conventional concrete and RCAC with a chemical admixture were more linear than that of concrete without the admixture.

5.3.7 *Creep of Concrete*

Creep of concrete is defined as the deformation of structure under sustained load. Creep is dependent on various factors including the properties of aggregate and the

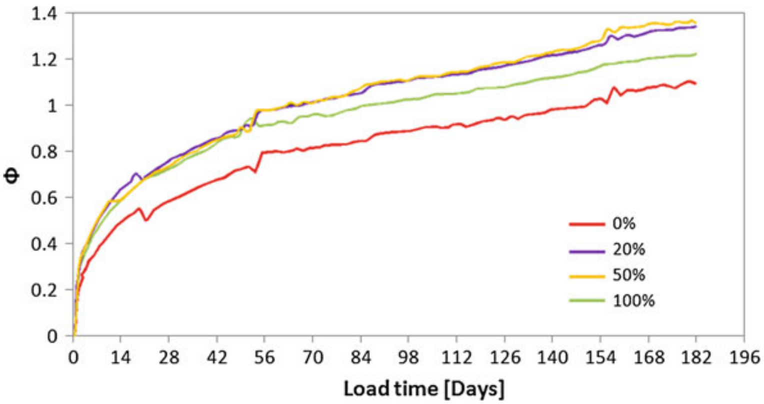


Fig. 5.49 Creep coefficient of concrete with replacement of coarse NA by coarse RCA (Domingo-Cabo et al. 2010)

amount of cement paste. Generally low creep is observed in concrete containing strong aggregate and aggregate with a high stiffness. As the properties of CDW aggregate are different from those of conventional aggregate, the incorporation of CDW aggregate significantly changes the creep of concrete. The incorporation increases the total paste content in concrete too, which also has some effect on the creep performance of concrete. In general the creep of concrete increases with the incorporation of CDW aggregate in concrete. Some results are highlighted next.

Domingo-Cabo et al. (2010) determined total creep deformation, creep coefficient and specific creep deformation of concrete due to the replacement of coarse NA by coarse RCA. They observed a gradual increase of the above parameters with the content of RCA in concrete. They observed a 35, 42 and 51 % higher total creep deformation due to a 20, 50 and 100 % replacement of NA by RCA, respectively, when the specimens were loaded for 180 days. Similarly, the specific creep deformation was also increased by 25, 29 and 32 % due to a 20, 50 and 100 % replacement of NA by RCA. The creep coefficients for various concrete mixes are presented in Fig. 5.49.

After comparing their experimental results with various relationships from ACTM C512-02 Code-2002, RILEM Model B3 Code-1995, CEB-FIP Code-1990 and a model developed by Gardner and Lockman (2001), Domingo-Cabo et al. (2010) concluded that these models were conservative to determine the deformation in the NAC and the RCAC's except the CEB-FIP Code for RCAC with 50 and 100 % coarse RCA. Their results are presented in Fig. 5.50.

The creep of RCAC in the Wesche and Schulz (1982) study, where RCA was used as integral replacement of coarse NA, was 50 % higher than that of conventional concrete. Kou et al. (2007) observed that creep strain increased with the incorporation of coarse RCA in concrete due to the gradual increase of mortar content. The addition of fly ash as a 25 % replacement (by weight) of cement lowers the creep strain of conventional as well as of RCA concrete. The creep

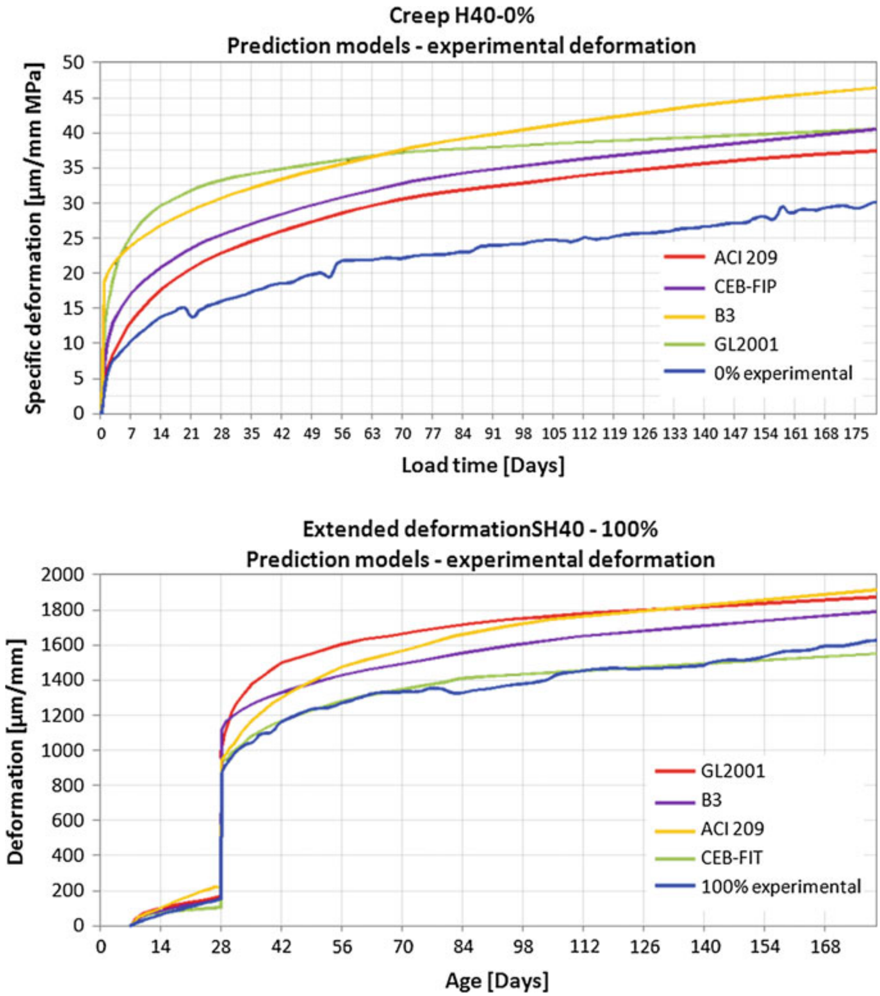


Fig. 5.50 Experimental and predicted creep deformation of NAC and RCAC with 100 % coarse RCA (Domingo-Cabo et al. 2010)

coefficient and specific creep of RCAC's with complete replacement of coarse NA by two types of RCA generated from concrete containing river gravel of limestone gravel in the Fathifazl et al. (2011) study were comparable to or lower than those of the equivalent NAC when the RCAC's were prepared by the EMV method. On the other hand, both parameters for RCAC's prepared by the usual method are higher than those of the NAC. The creep coefficient of RCAC also depended on the residual mortar content in RCA. Concrete with RCA with higher mortar content had higher creep. The authors also proposed a residual mortar factor to fit their experimental results with those predicted by ACI-209 code-1982 and CIB-FIP model code.

Table 5.18 Creep of concrete containing CDW aggregate

Reference	Type of aggregate	Creep coefficient/substitution level(%)/day	Comment
Limbachiya (2010)	RCA/coarse	1.25/0/28; 1.24/30/28; 1.41/50/28; 1.93/100/28;a	a = Concrete with 30 MPa design strength
Limbachiya et al. (2000)	RCA/coarse	1.14/0/90[a]; 1.15/30/90[a]; 1.31/50/90[a]; 1.88/100/90[a]; 0.81/0/90[b]; 0.83/30/90[b]; 0.99/50/90[b]; 1.08/100/90[b];	[a, b] = Concrete with 50 and 60 MPa respectively

Limbachiya (2010) observed almost equal 28-day creep strain for concrete with 30 MPa design strength, when 30 % of coarse NA was replaced by coarse RCA (Table 5.18). However, in comparison with conventional concrete, the creep strain of concrete increased 13 and 54 % when the replacement level was increased to 50 and 100 % of coarse NA, respectively. Limbachiya et al. (2000) observed higher creep in high-strength concrete with coarse RCA as aggregate than that in conventional concrete and creep increased with increasing RCA content (Table 5.18). They identified two reasons for this trend: the increase in cement content to reduce w/c value and achieve the same 90-day CS as conventional concrete and the presence of old cement paste in RCA. Like for conventional concrete, they observed lower creep for RCAC with higher CS (Table 5.18).

Paine et al. (2009) reported that the creep coefficient of concrete containing three types of RA coarse aggregate and a type of RCA coarse aggregate was slightly higher than that of conventional concrete due to differences in the stiffness of NA and CDW aggregates as well as in the CS of concrete containing conventional and CDW aggregates. The creep of RAC and RCAC were in the range of 1.9–2.6 at 100 days. They also observed that the experimentally determined 100-day creep values of concrete containing conventional and CDW aggregates were about 20 % lower than the value predicted by the Eurocode 2 relationship.

Regardless of the w/c ratios and quality of the aggregate, Tsujino et al. (2006) observed higher creep strain in concrete containing RCA as the only coarse aggregate than that in conventional concrete, possibly due to higher paste content in RCAC. The use of oil treated RCA had little effect on the creep strain of the resulting concrete; however, the creeps of concrete containing silane treated RCA were very high at various experimental conditions possibly due to lower bond strength between aggregate and cement paste.

However, in contrast with the above results, Ajdukiewicz and Kliszczzewicz (2002) observed up to 20 % lower creep of 1-year old high-performance/high-strength (hp/hs) concrete containing RCA as full replacement of NA (coarse and fine) or full replacement of coarse aggregate than that of conventional hp/hs concrete. This difference was more visible for concrete prepared using a chemical admixture.

5.3.8 Surface Hardness

The hardness of the surface of concrete has several practical repercussions and therefore several tests are performed to evaluate this property using parameters such as Schmidt hardness and rebound number, abrasion resistance, impact value. In this section, this property of concrete containing CDW aggregate is highlighted.

The Schmidt hardness test, which is a non-destructive method, can yield indirect data on surface hardness and penetration resistance as well as CS of concrete. A few results are available on the evaluation of this property in RCAC too.

Topçu and Sengel (2004) observed decreasing Schmidt rebound hammer with increasing coarse RCA content in two types of concrete with design CS of 16 and 20 MPa. The rebound hammer values of conventional concrete and RCAC were around 20 and 19 respectively for both classes of concrete. The 28-day rebound hammer values of conventional concrete and concrete with RCA as the only fine and coarse aggregate in another study of Topçu (1997) were respectively 21.3 and 11.6.

Rao et al. (2011) observed decreasing rebound numbers with increasing replacement of coarse NA by RCA. The rebound number for conventional concrete and RCAC containing 25, 50 and 100 % coarse RCA were 30.28, 16.80, 16.23 and 14.95, respectively. According to the authors, this may be due to the porous nature of RCA linked to the weak adhered cement paste. They also observed a linear relationship between rebound number and CS of RCAC, which is presented in Fig. 5.51. Sagoe-Crentsil et al. (2001) observed a volume loss about 12 % higher in the abrasion resistance of concrete due to complete replacement of coarse basalt aggregate by RCA aggregate (Fig. 5.52). The use of 35 % slag containing OPC or the increase of 5 % in cement content in the preparation of RCA marginally improved the abrasion behaviour of concrete.

Limbachiya (2010) also observed a gradual increase of abrasion depth of two concrete with design strength class of 35 and 45 MPa with the replacement level of coarse NA by RCA. Like the NAC, the abrasion depth of the RCAC's also decreased with as the CS increased (Table 5.19). Limbachiya et al. (2000) from

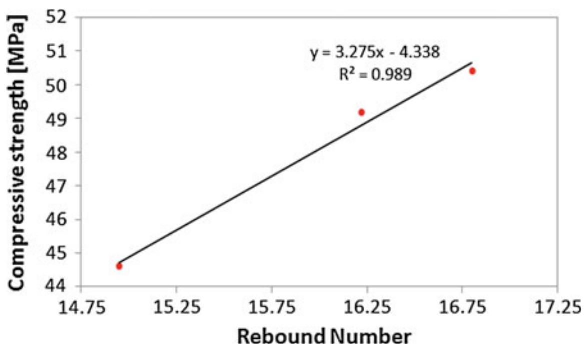


Fig. 5.51 Rebound number *versus* compressive strength for RCAC (Rao et al. 2011)

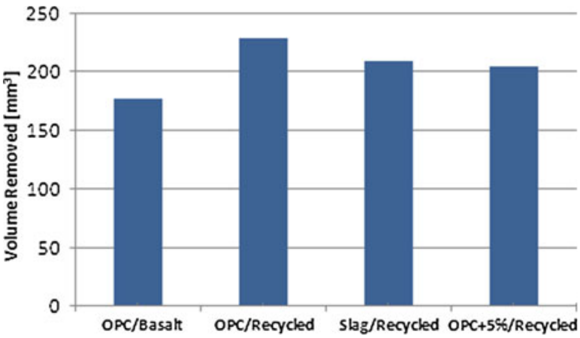


Fig. 5.52 Abrasion resistance of conventional concrete and RCAC's (Sagoe-Crentsil et al. 2001)

their results of concrete with three strength classes (50, 60 and 70 MPa), concluded that the incorporation of coarse RCA as partial or full replacements of NA did not change the abrasion resistance of concrete. The depth of abrasion of RCAC with 100 % coarse RCA at the design strength of 50 and 60 MPa were respectively 0.03 and 0.04 mm lower than that of the conventional concrete. On the other hand, Maas et al. (2012) did not observe any differences in the abrasion behaviour of two types of concrete prepared at w/c ratios of 0.72 and 0.45 by replacing 20 % of coarse NA by low grade RCA. However, the abrasion resistance of concrete deteriorated when 40 % coarse NA was replaced by RCA.

Fonseca et al. (2011) observed comparable abrasion resistance of conventional concrete and RCAC's with a 20, 50 and 100 % replacement by volume of coarse NA by coarse RCA when the concrete specimens were subjected to various curing conditions (Fig. 5.53). However, even though statistically insignificant, concrete containing RCA as the only coarse aggregate had marginally lower wear than the conventional concrete for all curing conditions. According to them, this is due to the better bond between RCA and the cement paste because of the porous nature of RCA.

Poon and Chan (2006) observed similar abrasion resistance but lower skid resistance in concrete paving blocks prepared by using RCA as the only aggregate than in concrete where a part of fine and coarse RCA was replaced by FA. The effect of FA on the behaviour of paving blocks containing crushed brick at 25, 50 and 75 % replacement of RCA was similar. However, the skid and abrasion resistances of paving blocks with or without FA as a partial replacement of RCA normally decrease and increase respectively with the incorporation of crushed brick. The observed improvement of skid resistance of concrete blocks by using FA was due to the formation of a more homogeneous concrete mix along with the smoother surface texture of the concrete blocks.

Table 5.19 Abrasion depth of concrete containing CDW aggregate (Limbachiya 2010)

Type of aggregate	Concrete strength (MPa)	Abrasion depth in mm/substitution level(%)/day
RCA/coarse	35	0.69/0/28; 0.73/30/28; 0.75/50/28; 0.78/100/28;
	45	0.49/0/28; 0.51/30/28; 0.48/50/28; 0.54/100/28;

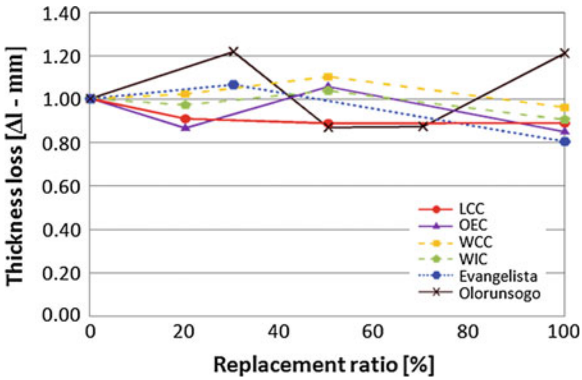


Fig. 5.53 Thickness loss due to abrasion of concrete with various replacement ratios of coarse NA by RCA for several curing conditions (Fonseca et al. 2011)

Poon and Chan (2007), in another study, observed comparable abrasion resistance but higher skid resistance of concrete due to incorporation of impurities such as crushed tiles or mixtures of crushed tiles with glass, brick and wood as 10 % replacement of RCA in concrete paving block. The higher skid resistance was due to the rough surface texture of the contaminating particles. Still all the RCAC paving blocks with and without contaminants met the standard specifications of Hong Kong.

Poon and Lam (2008) observed that the skid resistance of RCAC paving blocks made with an aggregate to cement ratio (A/C) of three was lower than that of blocks made with an A/C of four and six due to higher cement content and therefore smoother surface texture; still the skid resistance of all RCAC paving blocks met the standard specifications of Hong Kong. The quality of RCA did not have any effect on the skid resistance as aggregates were embedded in the cement matrix. Similarly, the abrasion resistance of the RCAC paving blocks made with an A/C of three and four were better and satisfactory than the blocks made with an A/C of six.

Topçu (1997) observed damage depths of 20–30 mm for conventional concrete and 100–130 mm for concrete containing RCA as the only coarse and fine aggregate, when both types of concrete were subjected to an impact test. The higher damage observed in the RCAC concrete was due to the presence of weak adhered mortar.

5.3.9 Other Mechanical Properties

Ajdukiewicz and Kliszczzewicz (2002) observed lower bond stress at failure for high-strength RCAC than for conventional high-strength concrete, which was more prominent for 220 MPa round bars than for 440 MPa ribbed bars. The bond stress was on average about 20 % lower when concrete was prepared with coarse and fine RCA and 8 % lower when RCA was used to replace coarse NA only.

Frondistou-Yannas (1977) observed lower aggregate to mortar bond strength for RCAC than for conventional concrete. The ranking of bond strength *versus* type of coarse aggregate can be arranged as: new granite (56 ± 15 lb) > recycled granite from demolished concrete (49 ± 18 lb) > RCA (39 ± 14 lb) > recycled mortar (3139 ± 8 lb).

Razaqpur et al. (2010) determined the bond performance of conventional concrete and concrete with RCA as the only coarse aggregate prepared by the normal and the EMV methods according to the ASTM A944.99–2003 procedure. They observed lower bond strength in RCAC prepared by the normal method. On the other hand, the bond strength of RCAC prepared by the EMV method was similar or even better than that of conventional concrete. However, they did not observe any effect of aggregate type on the bond performance of the RCAC.

Gonzalez-Fonteboa and Martinez-Abella (2005) observed poorer fatigue performance of RCAC than that of conventional concrete. Concrete fatigue was evaluated via an indirect test where the compressive strength of cylindrical specimens was determined using two loading rates: standard loading rate of 8.66 kN/s and one at 0.06 kN/s. They observed higher strength losses (9.03 %) and therefore poorer fatigue performance for RCAC than for conventional concrete (4.77 %).

In the Xiao and Falkner (2007) study, the shape of the load versus slip curve between RCAC and steel rebars was comparable to that observed for NAC and steel rebars. They also observed a 12 and 6 % reduction in bond strength between RCAC and steel rebars as compared to the strength observed for NAC and steel rebars when 50 and 100 % of NA was replaced by RCA (Fig. 5.54). On the other hand, for deformed rebars, the bond strength was similar for all types of concrete irrespective of the replacement level of coarse NA by RCA. For equivalent CS of NAC and RCAC, they observed higher bond strength between RCAC with 100 % RCA than between NAC and steel rebars.

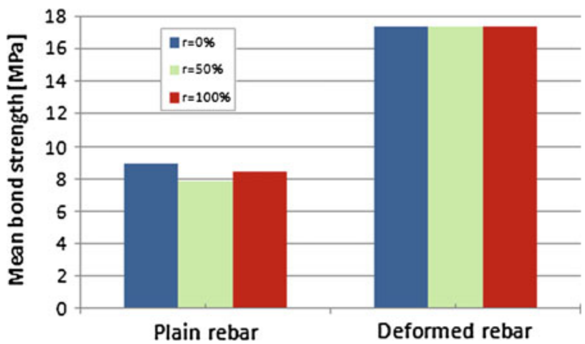


Fig. 5.54 Bond strength between concrete and steel rebars for various types of concrete (Xiao and Falkner 2007)

5.3.10 Failure Mode

Due to adhered mortar in RCA, the failure mode of RCAC is different than that of conventional concrete. The failure of RCAC in the Berndt (2009) study occurred through the old mortar particles and the gravel whereas in conventional concrete (NAC) it occurred through the stone-mortar interface and the gravel .

By examining the surfaces of RCAC after splitting tensile strength tests, Etxeberria et al. (2007b) reported that the failure of concrete (medium strength) mainly occurred through the RCA and it never happened through new interfacial zone. According to them, the weakest point of RCAC was the RCA and in particular the adhered mortar as the new aggregate-paste bond was stronger than the old mortar or the RCA-old cement paste bond due to the use of high quality cement as well as the quality of RCA, since it came from a concrete with low strength and therefore high w/c ratio. Yang et al. (2011) observed that the failure of RCAC made by replacing 50 % coarse RCA by crushed brick aggregate (CBA) during tensile and flexural strength tests occurred within the CBA due to the poor strength of porous CBA. On the other hand, the failure of RCAC with or without the 20 % CBA occurred in the aggregate-mortar interface like in the NAC.

5.3.11 Ultrasonic Pulse Velocity

The assessment of the ultrasonic pulse velocity (UPV) through concrete can give important information such as strength and elastic performance of concrete. This is an inexpensive and quick non-destructive test method to assess the quality of concrete. The influence of CDW aggregate on the UPV of concrete is reported in various references, some of which are highlighted next.

Zega and Di Miao (2009) observed comparable UPV of RCAC's containing three types of coarse RCA as 75 % replacement by volume of NA and their equivalent conventional concrete. Kwan et al. (2012) observed a decreasing UPV of concrete due to the replacement of coarse NA by RCA; however, like in conventional concrete, the UPV of RCAC also increased with curing time. The UPV of all types of concretes was in the 3.66–4.58 km/s range after 28 days of curing and is classed as good category. According to Malhotra (1976), a good category concrete does not contain any large voids or cracks, which would affect the structural integrity of concrete. Rao et al. (2011) observed a decreasing UPV as the replacement of coarse NA by RCA increased. Khatib (2005) observed a systematic decrease of UPV as the replacement of fine NA by fine RCA and fine CBA increased. At the same replacement level, the UPV of concrete containing CBA was higher than that observed for RCA concrete. The authors observed a sharp increase in UPV at the curing age of 1–7 days and then slowed down as the curing time increased. They also proposed an exponential relationship between CS and UPV, which fitted especially well for large replacement levels of fine NA by fine

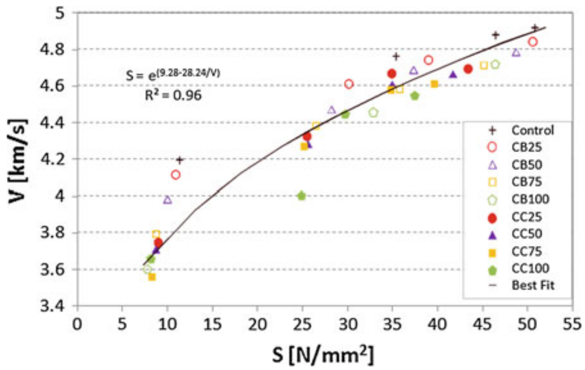


Fig. 5.55 Relationship between the UPV (V) and CS (S) of concrete (Khatib 2005)

RCA and CBA (Fig. 5.55). However, Kwan et al. (2012) commented that the polynomial or exponential relationships were not appropriate to relate UPV with CS of concrete containing RCA as there was a certain UPV value after which an increase in UPV would not necessarily mean an increase in CS. Tu et al. (2006) observed a lower UPV for high-performance (high-strength concrete (hpc/hs) with RCA as replacement of both fine and coarse NA than that observed for hpc/hs with RCA as the only coarse aggregate. The UPV increased with the w/c ratio.

Regardless of the type of mineral addition used as binder, Kou et al. (2011b) observed a decrease in the UPV of 28-day concrete as the replacement of coarse NA by RCA increased. However, the UPV of RCAC at all replacement levels of coarse NA by RCA and with SF and MK as mineral additions was significantly higher than that of conventional concrete with OPC as single binder or RCAC with or without FA and ggbfs. The UPV of RCAC with FA and ggbfs was slightly lower than that of conventional concrete. The gain in UPV between 28 and 90 days of curing for RCAC with mineral additions was higher than that for concrete with OPC as single binder owing to the formation of more hydration products and consequent improvement of the microstructure of RCAC due to a pozzolanic reaction. However, contrary to these results, Topçu (1997) observed a gradual increase of UPV in concrete due to the replacement of all NA by RCA, linked to wider air-voids because of RCA incorporation. The UPV in conventional concrete and concrete with 100 % RCA was around 70 and 93 µs, respectively.

5.4 Durability Performance

The durability of concrete is defined as the ability of concrete to withstand chemical attack and external environmental and physical actions. A concrete with a long service life must have a good durability performance. The durability of concrete depends on various factors such as the properties of concrete's constituents and their proportioning, the curing conditions and external environmental conditions. Several tests are performed to evaluate the durability of concrete. A vast work is already available on the evaluation of various durability behaviours of concrete containing CDW aggregate. In this section these properties are presented based on the collected references.

5.4.1 Drying Shrinkage

Concrete begins to shrink as soon as the hardening process starts by losing unconsumed water (i.e. that does not take part in the cement hydration reaction). The shrinkage of concrete can affect several mechanical and other durability properties of concrete due to the formation of micro cracks. In this section, the shrinkage (especially drying shrinkage) performance of concrete containing CDW aggregate is presented based on the literature. Normally, the incorporation of CDW aggregate in concrete increases the drying shrinkage. The increase in paste content in concrete due to the incorporation of CDW aggregate was identified as the main reason for drying shrinkage of concrete to increase (Kou et al. 2011b; Limbachiya et al. 2000). Table 5.20 shows a few typical examples of drying shrinkage of concrete containing RCA as coarse aggregate.

Limbachiya et al. (2000, Limbachiya (2010) observed higher drying shrinkage in RCAC with 30, 60 and 70 MPa design strength than in equivalent conventional concrete (Table 5.20). The shrinkage of concrete increased with the RCA content and design strength. The reasons for this trend were: the increase in cement content to reduce the w/c value and achieve the same 90-day strength as conventional concrete; and the presence of old cement paste in RCA. Like in conventional concrete, they observed lower shrinkage in RCAC with higher CS. In comparison to conventional concrete, Hansen and Boegh (1985) observed an increase of about 40–60 % in 440-day drying shrinkage of three different classes of structural concrete containing three types of coarse RCA as the only coarse aggregate by comparison with the parent concrete from which the RCA were generated (Table 5.20). The reasons for this trend were the same as in the previous research. Hasaba et al. (1982) observed a drying shrinkage about 70 % higher in concrete with RCA as a full replacement of fine and coarse NA. In comparison to conventional concrete, Poon et al. (2006) observed about 33 and 20 % higher 112-day drying shrinkage of two types of RCAC where 100 % by volume of coarse NA were replaced by RCA and at w/c of 0.55 and 0.45, respectively.

Sagoe-Crentsil et al. (2001) observed a 35 % increase of the 365-day drying shrinkage due to the complete replacement of coarse basalt aggregate by RCA in concrete; however, the decreasing trend of drying shrinkage of both types of concrete with time was similar. The 56-day drying shrinkage strain of both types of concrete was less than the 700 μ, the recommended limit in the Australian standard, AS 3600. Khatib (2005) observed a higher drying shrinkage of conventional concrete and RCAC's containing fine RCA at various levels in the first 10 days of curing (Fig. 5.56). The shrinkage increased with the content of RCA in concrete. Kou and Poon (2009a) observed increasing drying shrinkage of concrete

Table 5.20 Drying shrinkage of concrete containing CDW aggregate

Reference	Type of aggregate	Shrinkage strain/substitution level(%)/day	Comments
Limbachiya et al. (2000)	RCA/coarse	596/0/90[a]; 600/30/90[a]; 625/50/90[a]; 673/100/90[a];d 718/0/90[b]; 728/30/90[b]; 768/50/90[b]; 785/100/90[b];d 769/0/90[c]; 781/30/90[c]; 792/50/90[c]; 818/100/90[c];d	[a, b, c] = concrete with 30, 60 and 70 MPa design strength respectively; d = shrinkage ($\times 10^{-6}$)
Hansen and Boegh (1985)	RCA/coarse	4.0/0/440 [h]; 4.3/0/440 [m]; 5.1/0/440 [l];a 6.4/100/440 [h]; 6.0/100/440 [m]; 5.8/100/440 [l];b 6.1/100/440 [h]; 6.6/100/440 [m]; 6.3/100/440 [l];c 7.9/100/440 [h]; 7.0/100/440 [m]; 7.5/100/440 [l];d	[h, m, l] = aggregate from high, medium and low strength concrete; a, b, c, d = shrinkage ($\times 10^{-4}$) of original concrete, high, medium and low strength RCAC
Limbachiya et al. (2012)	RCA/coarse	280/0/91; 320/30/91; 425/50/91; 810/100/91;a 195/0/91; 250/30/91; 425/50/91; 695/100/91;b	a, b = Concrete with OPC and OPC-FA as binder; Shrinkage in μ strain.

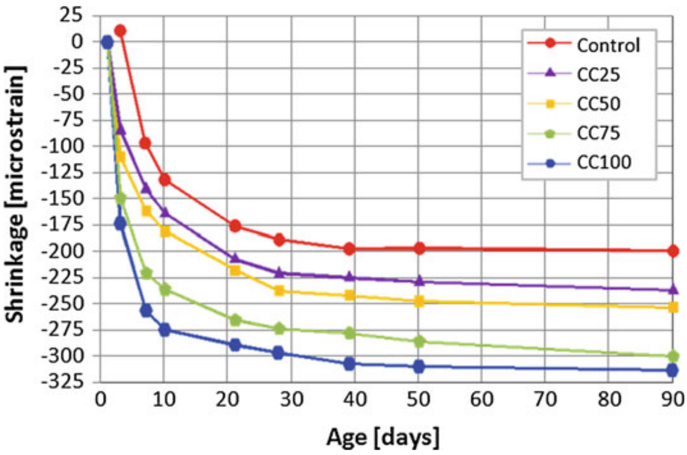


Fig. 5.56 Drying shrinkage of conventional and RCA concrete (CC) *versus* time (Khatib 2005)

at constant w/c ratio and near constant slump due to the replacement of fine NA by fine RCA. Ajdukiewicz and Kliszczzewicz (2002) observed a significant influence of RCA on the shrinkage performance of the resulting high-performance/high-strength concrete. The shrinkage of concrete with RCA generated from two types of concrete with granite and basalt as the coarse aggregates as complete replacement of NA (fine and coarse) was 35–45 % higher than that of conventional concrete. Kou et al. (2011b) reported that the incorporation of RCA as partial or full replacement of coarse natural aggregate in concrete increased the 112-day drying shrinkage of the resulting concrete due to the presence of old cement paste and the low stiffness of RCA.

Domingo-Cabo et al. (2010) measured the drying shrinkage strain of NAC and RCAC with coarse RCA as 20, 50 and 100 % replacement of coarse NA. After 180 days they observed 20 and 70 % higher shrinkage strain of the concrete mixes with 50 and 100 % incorporation of RCA, respectively. The increase in volume of the cement paste and of the porosity of concrete due to the incorporation of RCA was the reason for the higher drying shrinkage of the RCAC. These results are presented in Fig. 5.57.

Zega and Di Miao (2011) observed similar drying shrinkage strains in conventional concrete and RCAC containing fine RCA as 20 % replacement of fine NA when both types of concrete after 180 days of drying shrinkage testing. On the other hand, the shrinkage strain of RCAC containing 30 % fine RCA was slightly lower than that of the NAC due to a lower effective w/c ratio. Regardless of the type of aggregate in concrete, Yang et al. (2008) observed a higher rate of shrinkage strain within the first 10 days of testing and then it gradually slowed down. The shrinkage strain of concrete containing coarse or fine RCA as a 100 % replacement of coarse or fine NA respectively was also lower than that of conventional concrete in the first 10 days due to higher water absorption capacity; however, at a later stage, shrinkage was higher for RCAC due to lower stiffness of

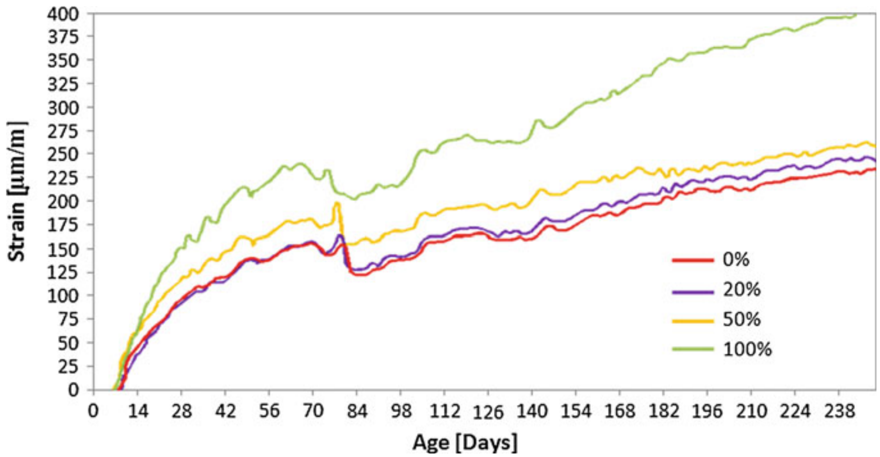
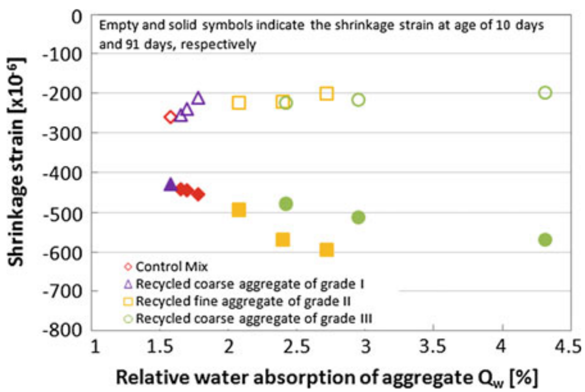


Fig. 5.57 Drying shrinkage of NAC and RCAC *versus* time (Domingo-Cabo et al. 2010)

the RCA than the NA. Moreover, the shrinkage strain of concrete containing coarse RCA with low water absorption capacity was lower than that of concrete containing coarse RCA with higher water absorption capacity or concrete containing fine RCA with higher water absorption capacity. The authors also observed an increasing trend of long-term shrinkage strain of the RCAC as the water absorption capacity of RCA increased (Fig. 5.58).

Corinaldesi (2010) reported that the 180-day shrinkage of concrete containing NA and RCAC prepared by replacing 30 % NA by fine and coarse RCA at the w/c ratios of 0.4–0.5 were almost the same and then shrinkage gradually increased for the w/c ratio of 0.60 (Fig. 5.59). The difference in shrinkage between conventional concrete and RCAC was also higher at low w/c ratio. On the other hand, for equal CS, the 180-day shrinkage of RCAC containing fine and coarse RCA were respectively 23 and 14 % lower than the conventional concrete (Fig. 5.60).

Fig. 5.58 10- and 90-day shrinkage strain of concrete with various w/c ratios (Yang et al. 2008)



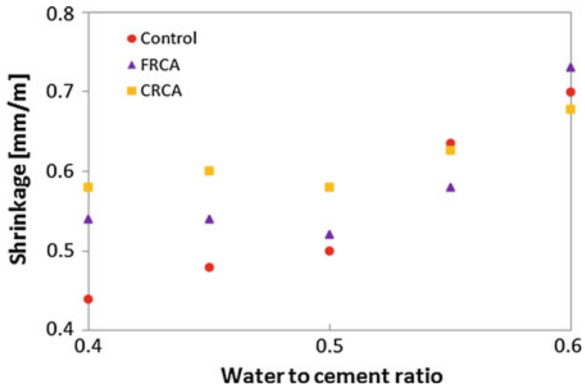


Fig. 5.59 Shrinkage of concrete *versus* w/c ratio (FRCA and CRC: fine and coarse RCA) (Corinaldesi 2010)

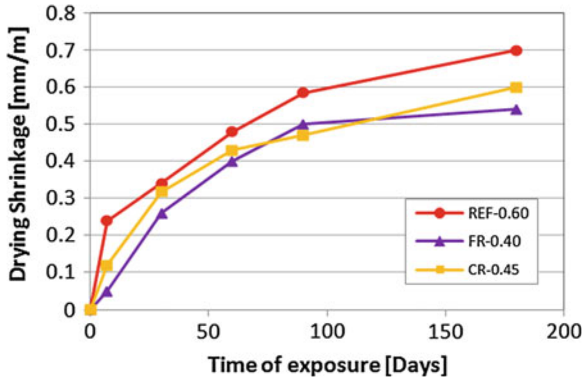


Fig. 5.60 Shrinkage of equal CS conventional concrete and RCAC (FR and CR: fine and coarse RCA, 0.60, 0.45, 0.40 indicate w/c) (Corinaldesi 2010)

Poon et al. (2009) observed increasing drying shrinkage of concrete blocks prepared using fine RA as the only coarse aggregate as the replacement level or the soil content in the RA increased. On the other hand, the drying shrinkages of concrete blocks decreased as the aggregate to cement ratio decreased and the quality of fine RA improved. Lowering of w/c ratio reduced the drying shrinkage of conventional concrete as well as RCAC (Kou et al. 2007, 2008). The drying shrinkage of RCAC increased with the coarse RCA content due to the higher mortar content in concrete (Kou et al. 2007).

In the Kou and Poon (2008) study, reducing the w/c ratio of RCAC from 0.55 to 0.40 was more effective to mitigate drying shrinkage than replacing 25 % of cement by FA. The authors observed an inverse relationship between improvement in CS and increase in drying shrinkage of RCAC (Fig. 5.61). Poon et al. (2006) reported that the drying shrinkage of steam cured concrete containing RCA as the only coarse aggregate and w/c ratios of 0.55 and 0.45 was respectively 14 and 15 % lower than that of the corresponding normal water cured RCAC. Eguchi et al. (2007) observed an approximately linear increase of drying shrinkage strain of RCAC with the water absorption capacity of concrete.

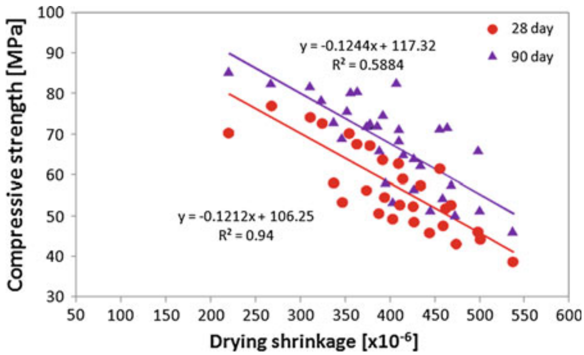


Fig. 5.61 Relationship between CS and drying shrinkage (Kou and Poon 2008)

Depending upon their hydration behaviour, mixing mineral additions with cement has positive or negative effect on the drying shrinkages of concrete containing RCA. Regardless of the replacement ratio of coarse NA by RCA, Kou et al. (2011b) reported that the drying shrinkages of concrete containing SF and MK as 10 and 15 % replacement of OPC was higher than that of the conventional concrete due to the formation of a higher amount of calcium silicate hydrate gel. On the other hand, the drying shrinkage of concrete containing FA and ggbfs was lower than that of the conventional concrete due to the lower hydration rate of FA and ggbfs as well as the restraining effect of unhydrated powder in cement paste. Their results are presented in Fig. 5.62.

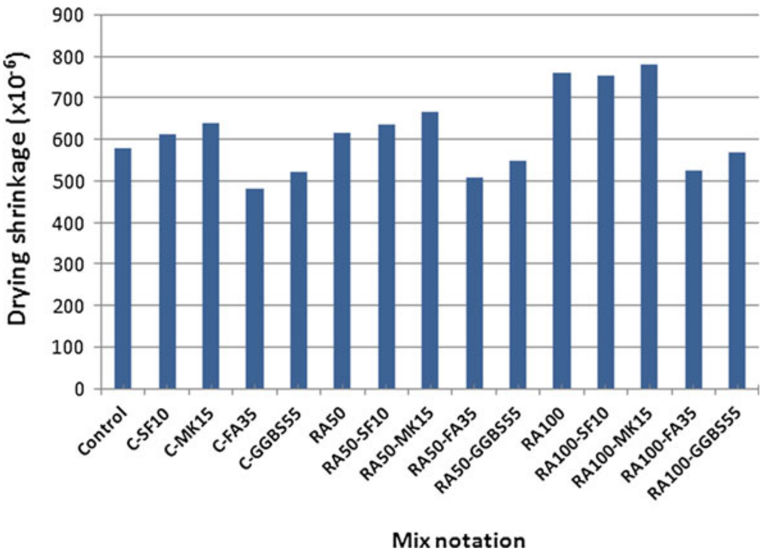


Fig. 5.62 Drying shrinkage of concrete containing 0–100 % of coarse RCA and various mineral additions (Kou et al. 2011b)

Limbachiya et al. (2012) also observed a significant decrease in the drying shrinkage strain of conventional concrete as well as of concrete containing coarse RCA as partial or full replacement of NA when 30 % by weight of OPC was replaced by FA due to the reduction in water demand in the concrete mix because of the lubricating effect of FA as well as the amount of available water the near pore network for any external drying (Table 5.19). However, regardless of the type of binder used, the shrinkage of concrete gradually increased with the RCA content due to the higher water absorption capacity and adhered mortar content of RCA. However, up to a 30 % replacement of coarse NA by RCA in two types of concrete of three different strength classes, the increase in shrinkage strain was not more significant (maximum 36.8 %) than that observed in higher replacement level (maximum 117.9 and 256.4 % at 50 and 100 % replacement level respectively). On the other hand, Sagoe-Crentsil et al. (2001) observed a significant increase of drying shrinkage especially after 91 days due to the replacement of 35 % of OPC by slag in RCAC. An increase in 5 % cement content in RCAC also increases the drying shrinkage but it was not as significant as that observed by slag addition.

The drying shrinkage performance of concrete containing RCA obtained after chemical treatment is reported in several references. Tsujino et al. (2006) stated that the surface coating of medium and low quality coarse RCA by a mineral oil could decrease the drying shrinkages of the resulting concrete, which was more prominent for concrete prepared at w/c of 0.4 than for that prepared at 0.6. In comparison to untreated medium quality RCA, the reduction was 10 % for oil treated medium quality RCA. The drying shrinkage of concrete containing silane coated medium quality RCA prepared at w/c of 0.6 and 0.4 was respectively comparable and lower than that of concrete containing untreated similar quality RCA. On the other hand, for low quality RCA, the drying shrinkage of concrete containing silane treated RCA at both w/c values was higher than that observed for concrete containing untreated RCA. In most of the cases except the concrete containing oil coated medium and low quality RCA prepared at w/c of 0.6, the drying shrinkage of concrete containing treated or untreated RCA was lower than that of conventional concrete.

Kou and Poon (2010) observed higher drying shrinkage of concrete prepared by replacing all coarse NA by untreated or polyvinyl alcohol treated RCA (Fig. 5.63) and cured for 112 days. However, the shrinkage of both concrete mixes with oven dried treated RCA (PI-R(O)-100) and air-dried treated RCA (PI-R(A)-100) was around 15 % lower than that of the concrete with untreated RCA (R-100) due to the lower water absorption capacity of treated RCA than that of the untreated one. Shayan and Xu (2003) observed that the drying shrinkage of concrete with coarse RCA as full replacement of coarse NA or fine RCA as 50 % replacement of fine NA was higher than that of conventional concrete. The use of sodium silicate and lime treated coarse or fine RCA in concrete further increased the drying shrinkage. However, all the concrete compositions met the Australian standard specifications. The drying shrinkage of concrete containing treated coarse and fine RCA as full replacement of coarse NA and as 50 % replacement of fine NA respectively was

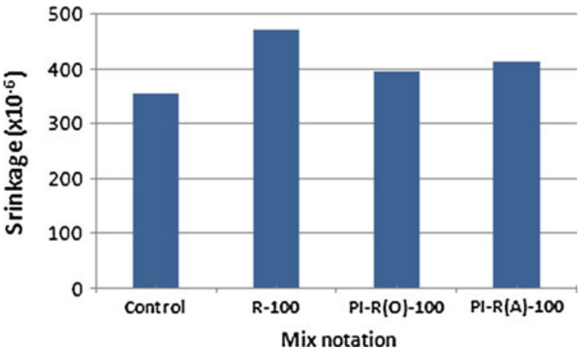


Fig. 5.63 Shrinkage of concrete containing untreated and polyvinyl alcohol treated coarse RCA and conventional concrete (Kou and Poon 2010)

higher than the standard specifications' limit. An OPC blended with 8 % SF was used as binder in this investigation. Fathifazl et al. (2011) observed lower shrinkage strain of NAC and RCAC when both types of concrete were prepared by the EMV method than when they were prepared by the normal method. They also observed lower shrinkage strain in concrete containing RCA originated from a concrete made with river bed gravel than from one made with limestone gravel.

5.4.2 Permeability Properties

The durability of concrete is greatly influenced by its permeability behaviour. A concrete with low permeability has better durability performance. Lower permeability means lower void content in concrete, and therefore water and some other corrosion agents cannot penetrate easily into concrete. Several properties such as water absorption, gas permeability and chloride penetration are evaluated to determine the permeability performance of concrete. Alexander et al. (1999) classified concrete according to its performance in various permeability tests. This classification is presented in Table 5.21.

The types of pores present in concrete also influence the permeability behaviour of concrete: discontinuous capillary pores are desirable for low permeable concrete. Thus the evaluation of porosity of concrete can provide data on the permeability behaviour of concrete. The factors, which improve concrete porosity,

Table 5.21 Concrete durability classification based on permeability parameters (Alexander et al. 1999)

Durability class	Oxygen permeability index (log scale)[a]	Sorptivity (mm/$\sqrt{h}$)	Chloride conductivity (mS/cm)
Excellent	>10.0	<6	<0.75
Good	9.5–10.0	6–10	0.75–1.50
Poor	9.0–9.5	10–15	1.50–2.50
Very poor	<9.0	>15	>2.50

[a] Negative logarithm of oxygen permeability coefficient

can also improve the permeability related durability performance of RCAC. Concrete porosity depends on various factors including aggregate's porosity. The use of CDW aggregate in concrete can change the porosity of concrete due to the higher porosity of CDW aggregate than that of the NA.

Gomez-Soberon (2002) observed an increase in total porosity of concrete with the replacement level of coarse NA by RCA. Total porosity was determined by mercury intrusion porosimetry. After 90 days of curing, total porosity of concrete containing RCA as the only coarse aggregate was around 3.8 % higher than that of conventional concrete. They also observed a significant decrease in total porosity as curing time increased, which was more prominent for RCAC. Kou and Poon (2006) observed that incorporation of coarse RCA in concrete gradually increased the total porosity and average pore's diameter and shifted the pore size distribution to larger pores. The replacement of 25 and 35 % OPC by FA respectively decreased and increased the open porosities of conventional and RCA concrete. On the other hand, decreasing water to binder ratio decreased the open porosities and average pore diameters of both types of concrete. Kou et al. (2011a) observed improvement of total porosity of RCAC as curing time increased.

Properties such as water absorption capacity by immersion and capillarity, chloride and other gas permeation of concrete with CDW aggregate are discussed in this section.

5.4.2.1 Water Absorption

The water absorption capacity of concrete is an important property, which provides data on the water accessible porosity of concrete; concrete with high water absorption capacity is less durable in aggressive environmental conditions. Since the water absorption capacity of CDW aggregate is higher than that of natural aggregate, concrete containing CDW aggregate has higher water absorption capacity than conventional concrete. The water absorption is evaluated by an immersion test, which measures the open porosity of concrete specimens, and by capillarity test, which measures the capillary water absorption due to a difference in pressure occurred between the liquid on the concrete's surface and inside the capillary pores of concrete. Several references are available on the evaluation of water absorption capacity of concrete containing various types of CDW aggregates.

Kwan et al. (2012) observed increasing water absorption capacity of concrete as the replacement level of coarse NA by RCA increased (Fig. 5.64). They state that the replacement of 30 % by weight of coarse NA by RCA led to a water absorption capacity below 3 %, i.e. a concrete considered to have low water absorption capacity. For an 80 % replacement, the water absorption capacity of RCAC was 2.2 times higher than that of conventional concrete. Rao et al. (2011) also observed a gradual increase of water absorption capacity of concrete with the incorporation of coarse RCA to replace coarse NA due to the higher water absorption capacity of RCA, which was about 3.5 times higher than that of NA.

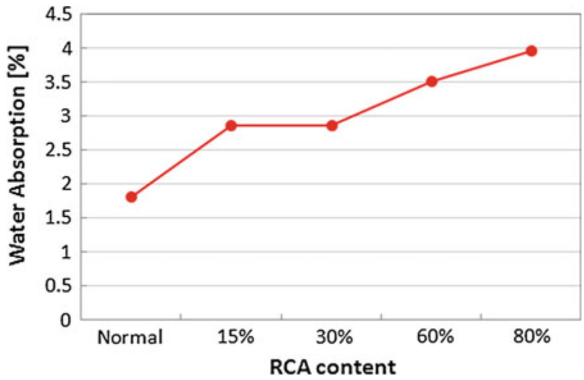


Fig. 5.64 Water absorption capacity of concrete versus coarse RCA content (Kwan et al. 2012)

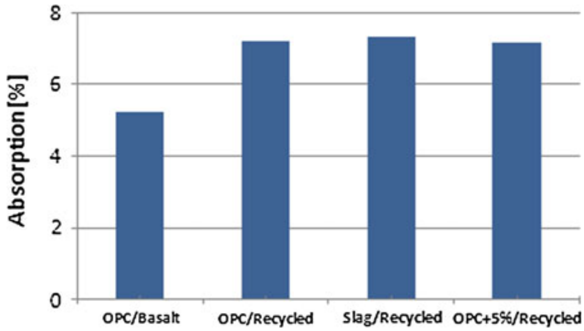


Fig. 5.65 Water absorption due to immersion of conventional concrete and RCAC's (Sagoe-Crentsil et al. 2001)

Sagoe-Crentsil et al. (2001) observed water absorption about 25 % higher due to the complete replacement of coarse basalt aggregate by RCA (Fig. 5.65). The concrete specimens used in this research were moist cured for 6 days after demoulding and then cured at 23 °C at 50 % room humidity before determining the water absorption capacity. Adding slag cement or increasing 5 % OPC content did not have any beneficial effect on the water absorption capacity of RCAC. Grdic (2010) observed water absorption capacity about 0.15–0.37 % higher in self-compacting concrete (SCC) due to a 50–100 % replacement of coarse NA by RCA. They did not observe any water penetration for 50 and 100 % coarse RCA based SCC; on the other hand, the control SCC (with NA only) had a penetration of 10 mm.

Soutsos et al. (2011) observed increasing water absorption capacity of concrete paving blocks as the replacement ratio of coarse and fine NA by similar sized RCA and recycled masonry aggregates (RMA) increased. This increase was higher for concrete with fine recycled aggregate than with coarse one. These results are presented in Fig. 5.66. The authors observed that the replacement of 55 or 25 % of coarse or fine NA by coarse and fine RCA respectively in concrete paving blocks allows maintaining the water absorption capacity below the BS EN1338 specified maximum limit of 6 % along with reasonably good CS and STS. For recycled masonry aggregates, these threshold replacement ratios were respectively 50 and

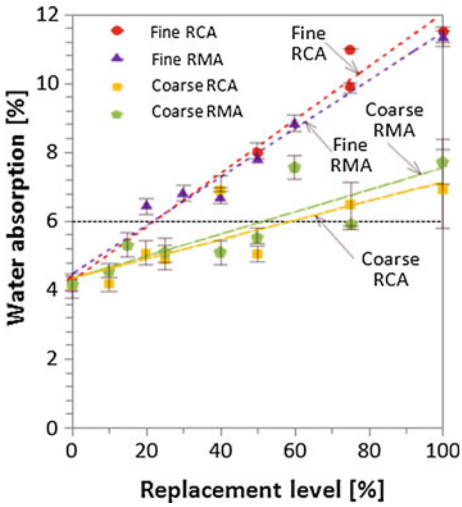


Fig. 5.66 Water absorption capacity of concrete containing various types of CDW aggregate (Soutsos et al. 2011)

20 % for coarse and fine aggregates. The water absorption capacity of conventional concrete and RCAC prepared by replacing 30 and 100 % by volume of fine NA by fine RCA in Evangelista and de Brito (2010) was respectively 11.3, 13.2 and 16.5 %. The water absorption capacity of concrete with 30 and 100 % fine RCA was about 16 and 46 % higher than that of the conventional concrete respectively. Yaprak et al. (2011) also observed a gradual increase of the water absorption capacity of concrete as the replacement of fine NA by RCA increased.

Poon and Lam (2008) observed increasing water absorption capacity of concrete paving blocks containing NA or coarse RCA as the aggregate to cement ratio increased. The water absorption capacity of the concrete also increased with the water absorption capacity of aggregate. A significant reduction in water absorption of the concrete blocks containing coarse RCA was observed when the coarse RCA was partially replaced by low water absorbing aggregates such as coarse NA or coarse recycle glass aggregate. Gomes and de Brito (2009) observed a linear increase of water absorption capacity of concrete with the replacement of coarse NA by RCA and crushed brick and mortar recycled aggregate (CBMRA). The water absorption capacity of concrete containing CBMRA was significantly higher than that observed for RCAC due to the higher porosity of the former type of aggregate.

Poon and Chan (2006) observed a higher cold and hot water absorption capacity of concrete paving blocks due to a 25, 50 and 75 % replacement of RCA by crushed clay brick aggregate and the water absorption increased with the RCA replacement level due to the high water absorption capacity of clay brick aggregate. However, the incorporation of FA to replace part of RCA in both types of concrete (with or without crushed clay brick aggregate) can improve the water absorption capacity of concrete due to the filling of voids within the concrete mixes. In another reference, Poon and Chan (2007) observed higher water absorption capacity of RCAC paving blocks due to the addition of contaminants such as crushed tiles, glass, brick and wood. The water absorption capacity of all types of paving blocks increased with the aggregate to cement ratio.

Mas et al. (2012) observed a gradual increase of water penetration depth in concrete prepared at w/c ratios of 0.45, 0.65 and 0.72 due to the replacement of coarse NA by low quality RCA, which contains 20–30 % of ceramic materials. Yang et al. (2011) observed a water permeability index about 20 % higher in concrete containing RCA as the only coarse aggregate than in conventional concrete. The replacement of RCA by crushed brick gradually increased the water permeability of the resulting concrete due to the higher porosity of crushed brick aggregates than that of RCA (Fig. 5.67). The water permeability of concrete containing RCA as the only coarse aggregate and of concrete containing crushed brick as a 50 % replacement of coarse RCA were 2.31×10^{-7} $m^3/min^{1/2}$ and 3.98×10^{-7} $m^3/min^{1/2}$, respectively. The water permeability coefficient of 84-day cured NAC and RCAC where RCA was used as the only coarse aggregate in the Berndt (2009) study was in the range of 1–1.4 ($\times 10^{-10}$) cm/s and 1.7–1.9 ($\times 10^{-10}$) cm/s, respectively. The marginal increase in water permeability coefficient of RCAC was due to the residual mortar content in RCA. The addition of blast furnace slag to replace 50 and 70 % cement did not have an influence on the permeability performance of concrete. According to the author, the observed permeability of NAC and RCAC's was within the acceptable range for durable concrete, i.e. it was $<3 \times 10^{-10}$ cm/s.

Buyle-Bodin and Zaharieva (2002) observed higher initial water absorption and sorptivity of RCAC's than of conventional concrete (Fig. 5.68). The test was performed in concrete samples obtained after two types of curing, normal water curing and air curing. Both properties of RCAC prepared by replacing only coarse

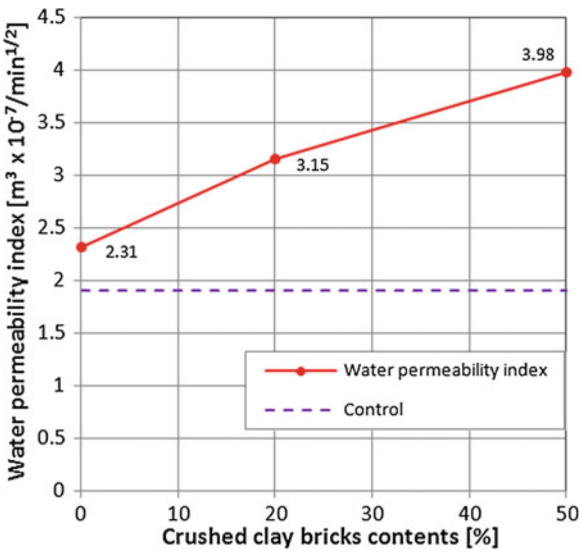


Fig. 5.67 Water permeability of RCAC *c* content of crushed brick as a coarse RCA replacement (Yang et al. 2011)

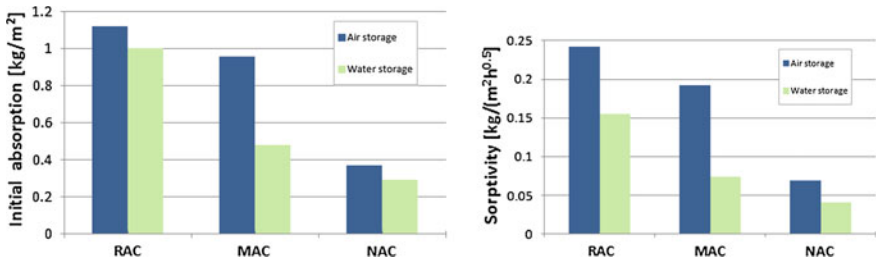


Fig. 5.68 Initial water absorption and sorptivity of conventional and RCA concrete at air- and water-curing conditions (Buyle-Bodin and Zaharieva 2002)

NA (MAC) were considerably better than those of the RCAC prepared by full replacement of fine and coarse NA (RAC), especially when the concrete samples were water cured. In terms of curing conditions, the RCAC's and the conventional concrete (NAC) obtained after water-curing both exhibited better performances due to the finer pore structure.

Zaharieva et al. (2003) observed about two to three times higher water permeability in concrete due to the replacement of coarse and fine NA by RCA and the water permeability of RCAC also increased with the amount of fine RCA. The water permeability of NAC was around 0.8×10^{-20} m^2. According to them, even though the incorporation of RCA in concrete increased the water permeability, the NAC and RCAC's tested could be considered as feebly permeable.

Limbachiya et al. (2000) and Limbachiya (2010) did not observe any effect of a 30 % replacement of coarse NA by RCA on the initial surface water absorption measured at 10 min in five classes of concrete with design compressive strength of 20, 30, 50, 60 and 70 MPa. Their results for the 20 and 30 MPa mixes are presented in Table 5.22. The surface water absorption increased with the RCA content but decreased as the design strength increased. The gradual increase in water absorption of concrete due to the content of RCA was attributed to the raise of cement paste content as more cement was added to reach the design strength of concrete. The same authors (Limbachiya et al. 2012) observed significant reduction in the 10 min initial surface water adsorption of three classes (with 20, 30 and 35 MPa design CS) of conventional concrete as well as mixes with various ratios of coarse RCA as partial or full replacement of NA due to the use of 30 % FA as a mineral addition. This improvement was attributed to the pozzolanic reaction of FA and a refinement of the pore structure and it was especially prominent for the full replacement of coarse NA by RCA.

Table 5.22 Initial surface water absorption at 10 min ($ml/m^2/s \times 10^{-2}$) of concrete containing RCA (Limbachiya 2010)

Type of aggregate	Strength class (MPa)	Water absorption/substitution level (%)
RCA/coarse	20	50/0; 50.5/30; 55/50; 66/100
	30	32/0; 32/30; 36.5/50; 51/100

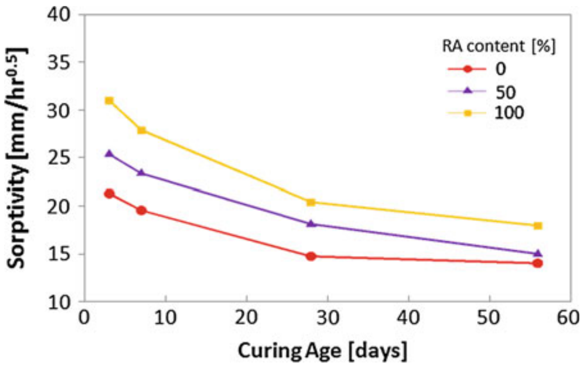


Fig. 5.69 Sorptivity of concrete containing RCA at 0, 50 and 100 % replacement of coarse NA (Olorunsogo and Padayachee 2002)

Olorunsogo and Padayachee (2002) observed a gradual increase in water sorptivity of concrete due to a 50 and 100 % replacement of coarse NA by RCA (Fig. 5.69). The sorptivity of concretes also decreased with increasing curing time. The difference in sorptivity between NAC and RCAC decreased with increasing curing time, indicating more improvement in RCAC than NAC with increasing curing time. However, according to Alexander et al. (1999) classification (Table 5.21), the concrete containing NA and RCA could be classified as poor and very poor respectively.

Evangelista and de Brito (2010) observed an increase of around 34 and 70 % in capillary water sorptivity due to the replacement of 30 and 100 % by volume of fine NA by fine RCA, respectively, attributed to the formation of more capillary pores due to its high porosity (Fig. 5.70a). Zega and Di Miao (2011) also observed a 13 % increase of capillary water absorption capacity of concrete due to the replacement of 20 and 30 % by volume of fine NA by fine RCA because of the higher porosity of RCA than that of NA (Fig. 5.70b). However, the capillary water

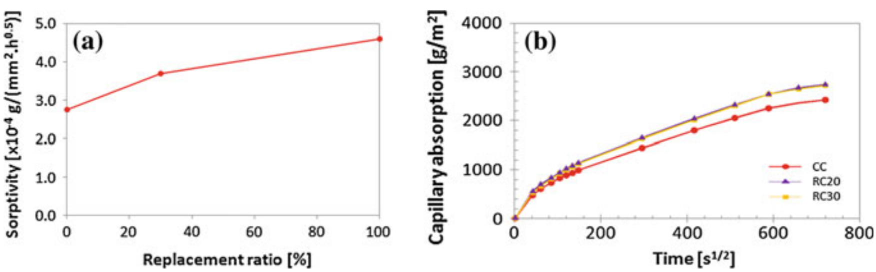


Fig. 5.70 **a** Capillary water sorptivity (Evangelista and de Brito, 2010) and **b** capillary water absorption (Zega and Di Miao 2011) of conventional concrete and RCAC containing RCA as a replacement of 30 and 100 % fine NA

sorptivity of RCAC was below the Argentinean specified maximum value, i.e. lower than 4 $g/m^2/s^{1/2}$.

Gomes and de Brito (2009) observed a progressive increase of the capillary water absorption coefficient with the content of RCA and CBMRA; the maximum increase of capillary water absorption coefficient for concrete with RCA as the only coarse aggregate and for concrete with CMBRA as a 50 % replacement of coarse NA was respectively 16.6 and 71.5 % as compared to conventional concrete. Gonçalves et al. (2004) also observed higher capillary water absorption coefficient of concrete containing RCA than that of conventional concrete; however, they did not observe any effect of increasing the coarse RCA content in concrete on the capillary water absorption coefficient.

5.4.2.2 Chloride Permeability

The determination of diffusivity of chloride ions through concrete can provide data on the permeability performance of concrete. Lower chloride permeability is desirable for durable concrete structures. Another important property that needs to be evaluated for reinforced concrete structure is chloride-induced corrosion. Several studies were undertaken to understand the chloride permeation and chloride-induced corrosion performance of concrete containing CDW aggregate. In several researches, it was reported that the incorporation of CDW aggregate in concrete increases the chloride permeability of concrete; however, some results also indicate negligible influence of CDW aggregate incorporation on the chloride permeability and chloride-induced corrosion performance of concrete.

Olorunsogo and Padayachee (2002) observed a significant increase in chloride conductivity with the replacement of coarse NA by RCA. The chloride conductivity of concrete containing RCA as the only coarse aggregate was 86.5 % higher than that of the conventional concrete. However, the chloride conductivity of RCAC also decreased with curing time (Fig. 5.71). The conventional concrete and the RCA concrete after 56 days of curing are classified as good and very poor, respectively in terms of chloride ion conductivity as presented in Table 5.21. Poon

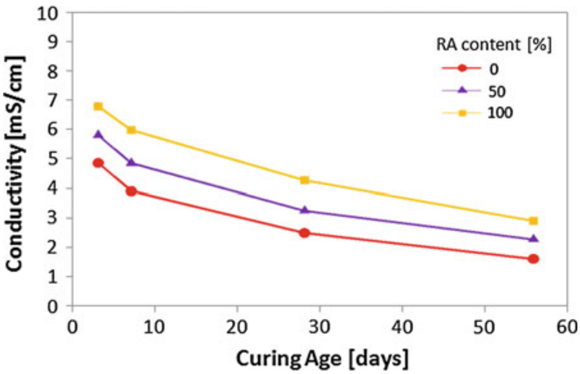


Fig. 5.71 Chloride ion conductivity of conventional and RCA concrete (Olorunsogo and Padayachee 2002)

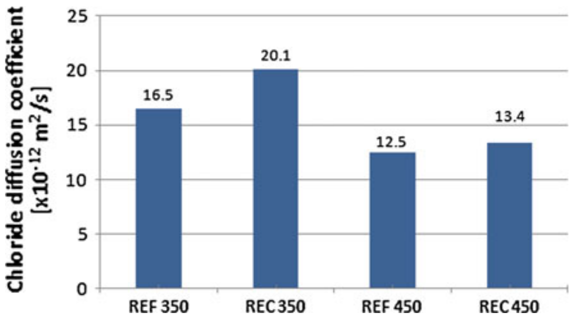


Fig. 5.72 Chloride diffusion coefficient of conventional concrete and RCAC with various cement contents (Gonçalves et al. 2004)

et al. (2006) observed higher resistance against chloride penetration of steam cured RCAC than that observed in normal water cured RCAC, which might be due to the formation of tortuous interconnecting capillary pores because of non-uniform calcium silicate gel formation. The chloride ion penetration resistivity decreased as the coarse RCA content in concrete increased.

Gonçalves et al. (2004) observed a lower chloride ion penetration coefficient with full replacement of coarse NA by RCA. The difference in chloride ion penetration coefficient between conventional concrete and RCAC was lower at higher cement contents indicating a higher influence of paste quality than that of aggregate porosity (Fig. 5.72).

Kou and Poon (2009a) observed lower chloride penetration resistance of concrete due to the incorporation of fine RCA as a replacement of fine NA at constant w/c ratio, which might be due to the poor microstructure formation because of adhered mortar content in RCA. On the other hand, at constant slump, the resistance to chloride penetration of RCAC was comparable to that of conventional concrete. The resistance to chloride penetration of RCAC decreased as the RCA content in concrete increased. Evangelista and de Brito (2010) also observed a linear increase of chloride migration coefficient with the replacement of fine NA by fine RCA because of increasing porosity of concrete (Fig. 5.73). The chloride migration coefficient of RCAC containing fine RCA at 30 and 100 % by volume replacements of fine NA was around 12 and 34 %, respectively.

Rao et al. (2011) observed an increase in chloride penetration depth with the replacement of coarse NA by coarse RCA. At 100 % replacement level, the increase in chloride penetration depth was 14 % as compared to that of conventional concrete. According to the authors, the presence of old porous mortar as well as interfacial transition zone in RCA formed a permeable concrete internal structure. Limbachiya et al. (2012) observed a marginal difference in the resistance to chloride ingress due to the replacement of 30 % of coarse NA by RCA; however, this difference became significant at 50 and 100 % replacement levels. Significant improvement of chloride resistance performance of RCAC was observed, when 30 % of OPC was replaced by FA. Gomes and de Brito (2009) observed higher chloride ion penetration depths and chloride permeability for concrete mixes containing RCA or CBMRA than for conventional concrete

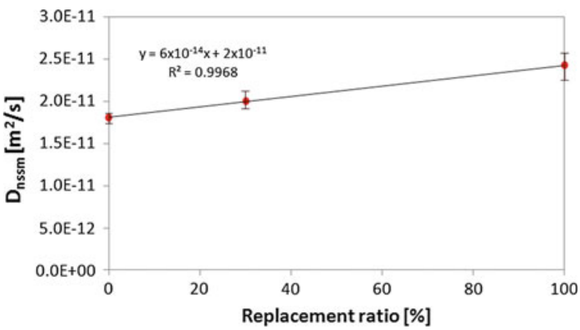


Fig. 5.73 Relationship between replacement ratio of fine NA by fine RCA and chloride migration coefficient (Evangelista and de Brito 2010)

(Table 5.23). The values for the concrete containing CBMRA were significantly higher than those observed for the RCA concrete due to the higher porosity of CBMRA than that of RCA. They also observed an increase in the chloride penetration depth with the water absorption capacity of concrete. The chloride permeability coefficient of concrete mixes containing 50 % RCA and 25 % CBMRA was respectively 5.6 and 18.8 % higher than that of the conventional concrete.

Limbachiya et al. (2000) did not observe any negative effect on the chloride diffusion performance of concrete due to the incorporation of coarse RCA as partial or full replacement of NA in three classes of high-strength concrete with design strength of 50, 60 and 70 MPa; the difference in chloride diffusion coefficients between the conventional and the RCA concrete mixes was below 1×10^{-11} m^2/s. Limbachiya et al. (2000) observed similar chloride-induced corrosion of conventional concrete and RCAC with design strength of 50 MPa except for RCAC containing RCA as the only coarse aggregate when the conventional concrete and RCAC cubes were exposed to a 2.5 M NaCl solution at 20 °C. However, the corrosion current of steel in RCAC with RCA as the only coarse aggregate and the corrosion initiation time were slightly higher and slightly shorter than for the conventional concrete and the RCAC with 50 % RCA, respectively.

Tu et al. (2006) observed higher chloride penetration (CP) for high-performance concrete (hpc) with RCA as replacement of both fine and coarse NA than that observed for hpc with RCA as the only coarse aggregate and NA as the only

Table 5.23 Chloride ion penetration depth of conventional concrete and RCAC's (Gomes and de Brito 2009)

Concrete mix	Porosity of aggregate (%)	Water absorption capacity of concrete (%)	Chloride penetration depth (mm)	Permeability coefficient (m^2/s) ($\times 10^{-12}$)
Conventional	2.29	~13.0	12.50	6.31
RCAC1	8.49	~17.0	13.21	6.66
RCAC2	16.34	~15.5	24.30	7.50
RCAC3	–	~17.0	14.39	7.26

RCAC1, RCAC2, RCAC3: concrete prepared by replacing 50, 25 and 37.5 % by volume of coarse NA by RCA, CBMRA and a 2:1 mixture of RCA and CBMRA, respectively

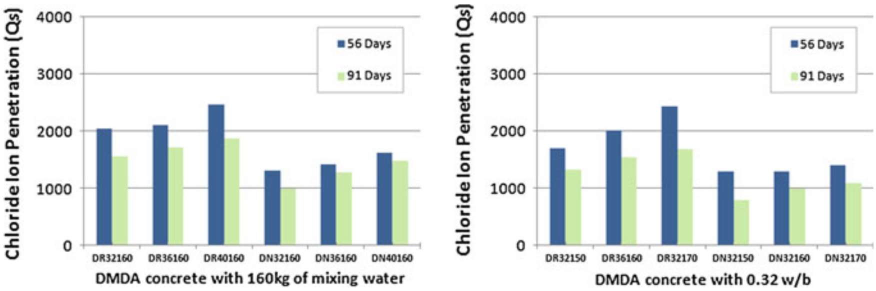


Fig. 5.74 Chloride ion resistivity of RCAC (*DR* Concrete with RCA as fine and coarse aggregate; *DN* Concrete with coarse RCA and fine NA) (Tu et al.2006)

fine aggregate due to the lowering of residual mortar content (Fig. 5.74). At fixed water content, the CP increased with the water to binder ratio. On the other hand, at fixed water to binder ratio, the lower CP was observed for RCAC containing lower water content. The CP of concrete with RCA as coarse NA replacement or as a replacement of fine and coarse NA after 91-day of curing was below 2000 Coulombs, the specified amount according to ASTM C 1202.

Kou et al. (2008) observed a decrease in resistance to chloride ion penetration with the coarse RCA content due to the porous nature of RCA. In the Kou and Poon (2010) study, the chloride permeability of concrete containing RCA as complete replacement of coarse NA was significantly higher than that of the conventional concrete, especially after 28 days of curing. However, the resistance to chloride permeability of concrete with oven dry and air-dried polyvinyl alcohol treated RCA was respectively 32 and 35 % higher than that of concrete with untreated RCA and comparable to that of conventional concrete. The chloride ion permeability of 90-day cured of both types of concrete (conventional and RCAC) was lower than that of 28-day cured concrete and was more significant for concrete with untreated RCA. Kou et al. (2007, 2008) also observed an increase in resistance to chloride ion penetration of normal water cured or steam cured RCAC with decreasing water to binder ratio and increasing curing time.

Like in conventional concrete, the resistance to chloride permeability of concrete with RCA can be improved by using mineral additions (Kou and Poon 2006; Kou et al. 2011b; Poon et al. 2007; Ann et al. 2008; Berndt 2009). This is due to the improvement of the watertightness of concrete (due to a microstructure improvement) and of the chloride binding capacity of cement paste because of the formation of high amounts of calcium silicate hydrate and calcium alumina silicates (Kou et al. 2011b). Kou et al. (2011b) observed a significant improvement of chloride permeability resistance in conventional concrete as well as in RCAC due to the replacement of 10, 15, 35 or 55 % of OPC by SF, MK, FA or ground granulated blast furnace slag (ggbfs), respectively and the ranking of improvement was: ggbfs > FA > MK > SF. The measurement was done for concrete specimens obtained after 28 and 90 days of conventional curing. Berndt (2009) also observed a significant decrease in the chloride diffusion coefficient of NAC and

RCAC after exposure for 1-year to artificial sea water due to the replacement of 50 and 70 % OPC by ggbfs. The chloride diffusion coefficient of NAC and RCAC containing OPC and OPC-slag as binder was in the range of 2×10^{-12} and 8×10^{-13} m^2/s, respectively. Ann et al. (2008) observed a significant increase in chloride ion permeability of concrete due to the incorporation of RCA as complete replacement of coarse NA when both types of concrete (conventional and RCAC) were subjected to rapid chloride ion penetration test (Fig. 5.75). However, at a replacement of 30 or 60 % of OPC by pulverised fuel ash (PFA) and ggbfs, respectively, the chloride ion permeability of RCAC decreased considerably and was even lower than that of the conventional concrete. They also observed improved resistance against chloride-induced corrosion of RCAC containing PFA and ggbfs due to the refinement of the pore structure as well as the increased chloride binding capacity of the cement paste.

Shayan and Xu (2003) reported that concrete with RCA had lesser chloride resistivity and higher negative half-cell potential than conventional concrete although both parameters for RCAC were within acceptable limits. The corrosion current densities of conventional concrete and RCAC were also similar and very low indicating that both types of concrete were resistant to chloride induced corrosion. The chloride ion penetration depth of concrete containing RCA aggregate was also higher than that of the conventional concrete. The depth of penetration of RCAC containing coarse RCA as a full replacement of coarse NA was slightly higher than that of RCAC containing fine RCA as a 50 % replacement of fine NA. On the other hand, the depth of chloride penetration for RCAC containing sodium silicate plus lime treated RCA was higher than that of the RCAC containing untreated RCA. The RCAC had inferior performance than the conventional concrete in the rapid chloride permeability test too; however, the corrosion protection of all types of concrete can be categories as high. The authors did not find any advantages of sodium chloride treatment of RCA on the corrosion

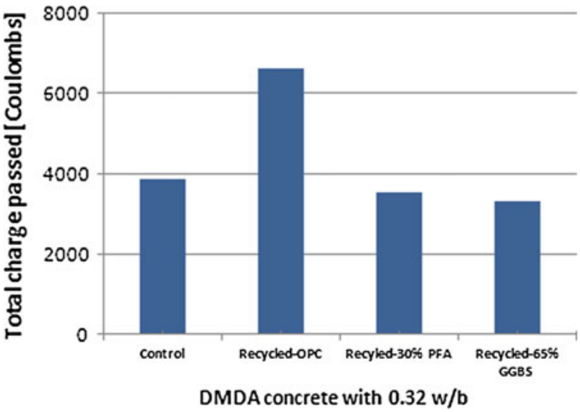


Fig. 5.75 Chloride ion permeability of conventional concrete containing OPC and RCAC containing OPC and PFA and ggbfs (Ann et al. 2008)

resistance of the resulting concrete; however, they found an important role of pre-coating with silica fume.

Kong et al. (2010) observed comparable resistances to chloride ion penetrations of concrete with RCA as the only coarse aggregate and conventional concrete when coarse RCA was pre-coated with FA or slag before mixing. They applied three mixing methods to prepare concrete: normal mixing (NM); double mixing (DM), which is described in Sect. 5.2.1. (for details, see Tam and Tam 2008); triple mixing (TM) in which fine and coarse aggregates were initially mixed for 15 s with part of the mixing water, then the mineral addition was added and mixed for another 15 s to coat the surface of aggregate, after this cement was added and mixed for another 30 s; finally, the remaining water along with the superplasticizer were added. Their results are presented in Fig. 5.76. Razaqpur et al. (2010) observed marginally higher apparent chloride diffusion coefficient of RCAC prepared by the EMV method than by the conventional mixing method; however, the chloride diffusion coefficient of both types of RCAC was lower than or comparable to that observed for conventional concrete. Abbas et al. 2009 reported that the apparent chloride diffusion coefficients of RCAC's prepared by the EMV method were found to be of the same order of magnitude of 10^{-12} m^2/s as the conventional structural grade concrete. They observed higher resistance to chloride diffusion of RCAC prepared by the EMV method due to the addition of FA or ggbfs with OPC and this resistance was significantly better for slag-based RCAC than FA-based RCAC.

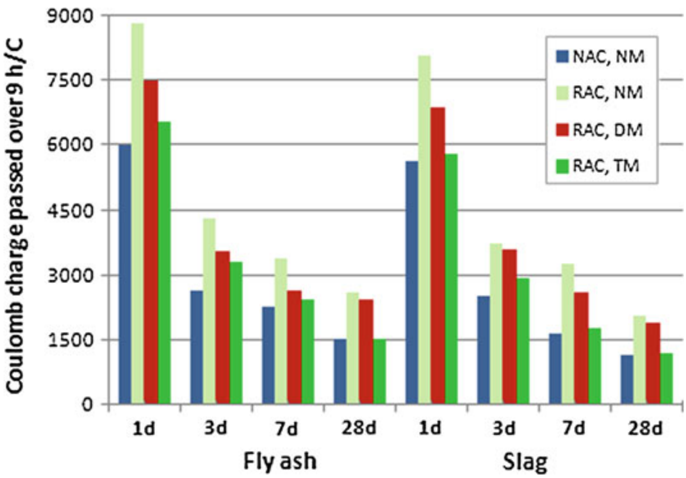


Fig. 5.76 Effect of the mixing method on the resistance to chloride penetration of conventional concrete and RCAC (Kong et al. 2010)

5.4.2.3 Gas Permeability

The gas permeability performance of concrete with CDW aggregate is also reported in the literature. Results are available for several types of gases: air, oxygen, nitrogen.

Kwan et al. (2012) determined the intrinsic permeability of concrete containing various percentages of coarse RCA by using nitrogen gas penetration. They observed a gradual increase in intrinsic permeability of concrete with the content of coarse RCA and the curing time. The difference in permeability between conventional concrete and RCAC decreased with the curing period. They also observed a parabolic inverse relationship between CS and intrinsic permeability, from which they concluded that concrete would achieve a constant permeability as the maturity of concrete increased. Limbachiya et al. (2000) did not observe any effect of 30 % replacement of coarse NA by RCA on the intrinsic air-permeability performance of three classes of high-strength concrete with design strength of 50, 60 and 70 MPa. Air permeability increased with RCA content but decreased with design strength. The increase of air permeability of concrete due to the incorporation of RCA was attributed to the increase of cement paste content as more cement was added to reach the design strength of concrete.

Buyle-Bodin and Zaharieva (2002) observed lower air permeability of concrete with complete replacement of coarse NA by RCA than that of concrete with RCA as complete replacement of fine and coarse aggregates (Table 5.24). Similarly, both types of concrete and conventional concrete as well exhibited considerably lower air permeability after water curing than after air curing. Zaharieva et al. (2003) observed that pre-soaking of RCA can improve the air permeability of concrete as dry aggregate can absorb hydrating water and therefore can hinder the hydration reaction. The air permeability of NAC and RCAC increased significantly due to an increase in pre-treatment temperature i.e. oven drying of concrete specimens at different temperatures after curing (Fig. 5.77).

Gonçalves et al. (2004) found a significant increase in oxygen permeability of concrete due to the replacement of coarse NA by RCA. Permeability as well as the difference of permeability between conventional concrete and RCAC decreased with the cement content in concrete due to a cut in porosity. These results are presented in Fig. 5.78.

Olorunsogo and Padayachee (2002) evaluated the oxygen permeability of concrete various replacement ratios of coarse NA by RCA. They presented their results in terms of oxygen permeability index (OPI), which is defined as the

Table 5.24 Air permeability of concrete containing RCA and conventional concrete at various experimental conditions (Buyle-Bodin and Zaharieva 2002)

Concrete type	Conventional		RCAC with RCA as the only coarse aggregate		RCAC with RCA only (coarse and fine)	
Curing condition	Water	Air	Water	Air	Water	Air
Air permeability ($\times 10^{-18}$) m^2	6.00	20.0	2.80	3.10	1.04	1.45

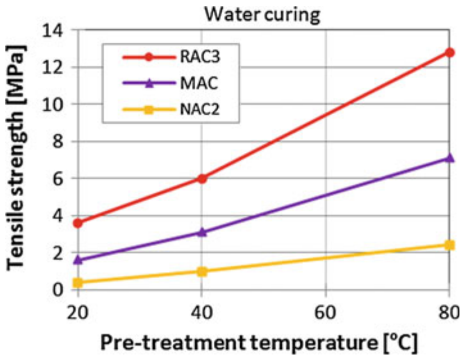


Fig. 5.77 Air permeability of NAC and RCAC at various pre-treatment temperatures (Zaharieva et al. 2003)

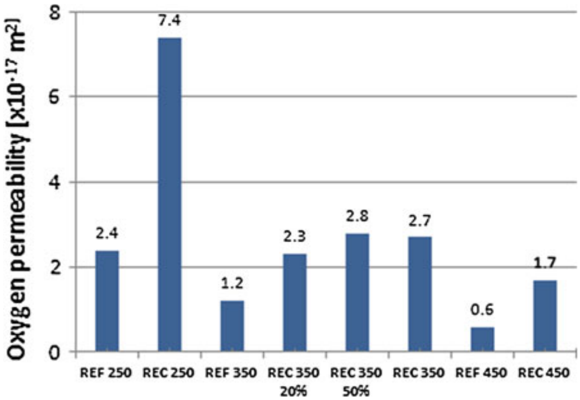


Fig. 5.78 Oxygen permeability of conventional and RCAC's with various cement contents (Gonçalves et al. 2004)

negative logarithm of oxygen permeability coefficient. They observed a gradual decrease of OPI as the replacement of NA by RCA increased; however, the increase in OPI of RCAC's with respect to the curing time was similar to that of conventional concrete. After 56 days of curing, the OPI of conventional concrete was 10 % higher than that of the RCAC with 100 % RCA. These results are presented in Fig. 5.79. The concrete mixes containing 0, 50 and 100 % coarse RCA after 56 days of curing can be considered as excellent, good and poor, respectively, according to the Alexander et al.'s (1999) classification (Table 5.21).

5.4.3 Depth of Carbonation

In a reinforced concrete structure, the steel reinforcement is chemically protected from corrosion by a passive oxide layer due to the presence of the surrounding alkaline environment. However, with time and in the presence of other chemical and physical factors, the alkali content in concrete gradually decreases due to the carbonation of concrete by atmospheric carbon dioxide and therefore the corrosion

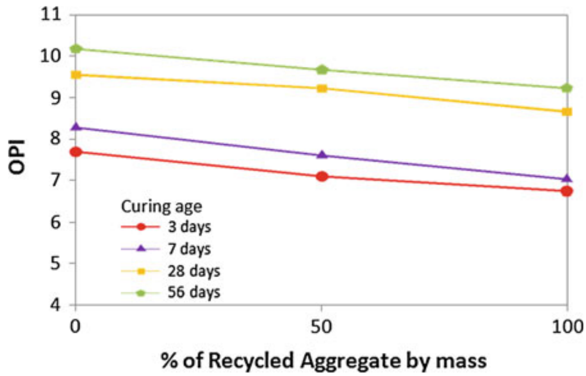


Fig. 5.79 Oxygen permeability index of concrete versus replacement ratio of coarse NA by RCA (Olorunsogo and Padayachee 2002)

Table 5.25 Carbonation depth (in mm) of conventional concrete and RCAC's

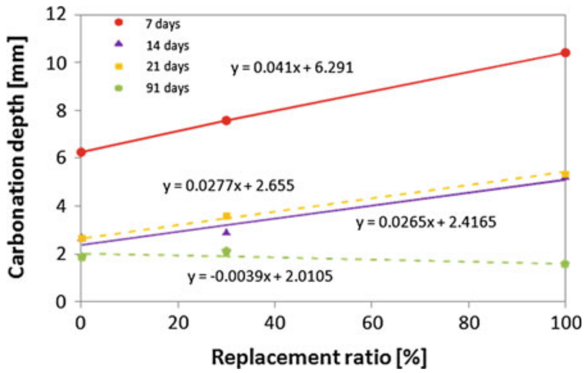
References	Type of aggregate	Concrete type	Carbonation depth/substitution level (%, volume)
Limbachiya 2010	RCA/coarse	C30	21/0; 21/30; 20/50; 18.5/100
		C35	18/0; 18/30; 17.5/50; 16.5/100
Gomes and de Brito 2009	RCA/coarse	Conventional	5.13/0
		RCAC1	5.63/50
		RCAC2	5.56/25
		RCAC3	6.57/37.5

C30, C35 Concrete with design strength of 30 and 35 MPa; RCAC1, RCAC2, RCAC3: concrete prepared by replacing coarse NA by RCA, CBMRA and a 2:1 mixture of RCA and CBMRA, respectively

resistance of reinforced structures goes down. The presence of micro cracks and pores in concrete generally enhances the rate of carbonation. Several reports are available on the evaluation of carbonation resistance of concrete. Normally RCAC has higher rate of carbonation than conventional concrete. Hansen (1992), after analysing various studies, concluded that RCAC had 4 times faster carbonation rate than that of conventional concrete. Two typical examples of the effect of RCA on the carbonation depth of concrete are presented in Table 5.25.

Limbachiya (2010) observed similar carbonation depth of two classes of air-entrained conventional concrete and concrete with a 30 % replacement of coarse NA by RCA. However, the carbonation depth of concrete decreased when the replacement level of coarse NA increased to 50 and 100 %. The author gave two reasons for resistance against carbonation to improve due to the incorporation of coarse RCA: increase in calcium hydroxide content with more attached cement paste content and increase in alkalinity due to increased cement content in RCAC to reach equal strength of concrete as well as to reduce the w/c ratio. These results are presented in Table 5.24. In this study, the concrete samples were exposed for

Fig. 5.80 Carbonation depth of concrete *versus* replacement ratio of fine NA by fine RCA (Evangelista and de Brito 2010)



20 weeks to a carbon dioxide atmosphere at 20 °C and 55 % room humidity. Gomes and de Brito (2009) observed higher carbon dioxide penetration depth in concrete with coarse RCA or coarse CBMRA than in conventional concrete (Table 5.24). Evangelista and de Brito (2010) observed a linear increase of carbonation depth with the replacement ratio of fine NA by fine RCA similarly to the capillary water absorption and chloride permeability performances (Fig. 5.80). Zega and Di Miao (2011) observed similar carbonation depth of NAC and RCAC's prepared by replacing 20 and 30 % by volume of fine NA by fine RCA, when concrete was exposed for 620 days to urban-industrial environmental conditions. Shayan and Xu (2003) observed comparable depth of carbonation of conventional concrete and RCAC with coarse RCA as full replacement of coarse NA or fine RCA as 50 % replacement of fine NA, even though a marginally higher carbonation depth was observed in concrete containing fine RCA. However, the use of sodium silicate and lime treated coarse or fine RCA significantly increased the depth of carbonation of the resulting concrete.

Sagoe-Crentsil et al. (2001) observed higher carbonation depth in concrete with RCA as complete replacement of coarse NA than in conventional concrete. The use of slag cement or a 5 % increase in cement content can decrease the carbonation depth of the RCAC, which was more pronounced for RCAC with higher cement content (Fig. 5.81).

Buyle-Bodin and Zaharieva (2002) observed significantly higher carbon penetration depth of concrete due to the complete replacement of coarse and fine NA by RCA (Fig. 5.82a). The carbonation depth of water cured RCAC was around half that of air-cured RCAC. They also observed that the kinetics of carbonation for conventional concrete and RCAC can both be designed according to basic law of diffusion (Fig. 5.82b):

$$x = C \cdot \sqrt{t}$$

where, x, C and t are depth, rate and time of carbonation.

Razaqpur et al. (2010) observed comparable or even lower carbonation depth in RCAC prepared by mixing two methods (conventional and EMV) than in

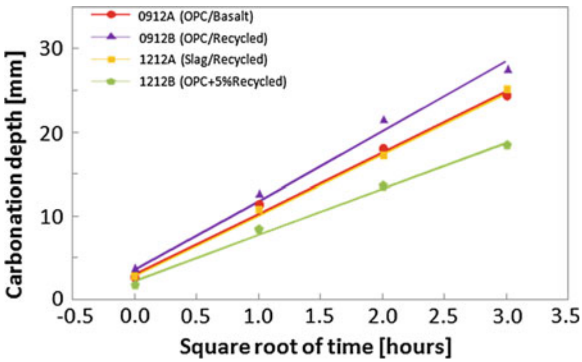


Fig. 5.81 Depth of carbonation versus square root of time of conventional concrete as well as RCAC's during accelerated carbonation test (Sagoe-Crentsil et al. 2001)

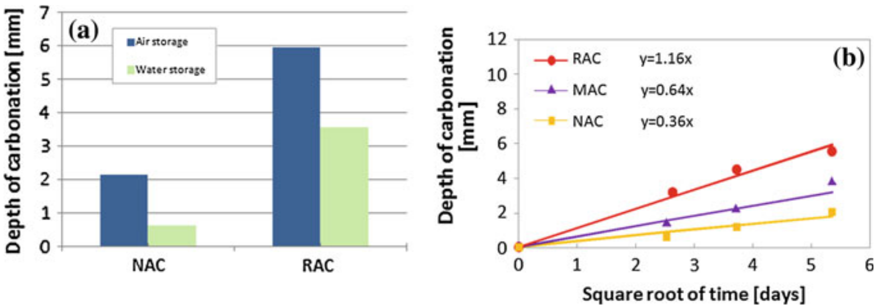


Fig. 5.82 **a** Depth of carbonation and **b** relationship between depth of carbonation and time (Buyle-Bodin and Zaharieva 2002)

conventional concrete, due to the difference in composition of cement of residual and fresh mortars. On the other hand, the carbonation coefficient of RCAC prepared by the conventional method was lower than that prepared by the EMV method due to lower fresh cement content in the later mix. Abbas et al. (2009) reported that the addition of FA and slag increases the carbonation depth due to the consumption of calcium hydroxide because of the pozzolanic reaction. The depth of carbonation of RCAC with or without FA and slag prepared by the conventional method as well as the EMV method and with 140 days of exposure fell in the range of structural grade concrete, i.e. about 0–7 for conventional concrete and 7–15 mm for concrete containing FA and slag.

Limbachiya et al. (2012) did not observe significant differences in the carbonation of conventional concrete and RCAC with various RCA contents when the design strength was 20 MPa. However, the depth of carbonation increased with the replacement ratio of coarse NA by RCA for the 30 and 35 MPa concrete classes and this was more prominent for concrete prepared with 80 % OPC and 20 % FA due to the pozzolanic reaction of FA, which lowered the Portlandite content and thus the pH of the pore solution. The increase in carbonation in RCAC was due to

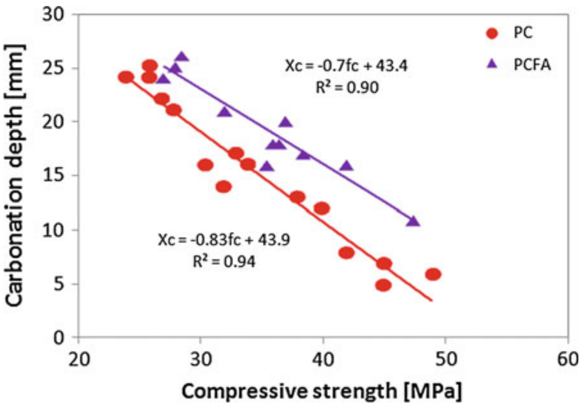


Fig. 5.83 Relationship between CS and carbonation depth (Limbachiya et al. 2012)

the higher water absorption capacity of RCA, which releases water throughout the hydration period and increase the humidity level of concrete. They also observed a linear inverse relationship between CS and carbonation depth of concrete (Fig. 5.83). Like Buyle-Bodin and Zaharieva (2002), they observed that the carbonation depth of RCAC can be predicted by a basic diffusion law.

5.4.4 Freeze–Thaw Durability

The freeze–thaw phenomenon is an important issue for cold region concrete. It occurs due to development of stress in a closed space such as pores in cement paste due to the expansion of water when it freezes. Some cracks may be formed if the stress is higher than the cement paste's strength; the damage can further increase if freezing and thaw cycles continue. The resistance of concrete containing CDW aggregate against freeze–thaw cycle is reported in several references.

Nagataki and Iida (2001) observed decreasing freeze–thaw resistance of concrete due to the incorporation of coarse RCA as a replacement of NA. However, the freeze–thaw resistance was satisfactory as the freezing–thawing factor for RCAC after 300 cycles was >70. Gokce et al. (2004) observed poor freeze–thaw durability of air-entrained concrete with coarse RCA produced from non-air-entrained concrete as the relative modulus of elasticity RCAC was below 60 % after 30 cycles of freeze–thaw. According to them, the presence of a small amount of non-air-entrained RCA can drastically deteriorate the freeze–thaw resistance of air-entrained concrete. The poor freeze–thaw resistance was due to the conversion of overall pore system of concrete containing coarse RCA with adhered mortar and air voids into a partial non-air-entrained void system. On the other hand, regardless of the adhered mortar content in RCA, the freeze–thaw resistance of concrete with coarse RCA originated from air-entrained concrete was even better than that of the conventional concrete after 500 cycles of freeze–thaw cycles even though the RCA has higher permeable voids and water absorption capacity.

Gokce et al. (2004) observed a marginal improvement in the freeze–thaw resistance of concrete due to the use of coarse RCA with small adhered mortar content. The decrease in w/c ratio from 0.55 to 0.33 can significantly improve the freeze–thaw resistance of the resulting concrete with coarse RCA and small adhered mortar content even though still unsatisfactory for long-term exposure. The addition of metakaolin to the above mix (i.e. with RCA with small adhered mortar content and at w/c of 0.3) can lead to a resistance over the standard durability limit of 300 cycles of freeze–thaw; however, a similar RCAC containing silica fume had less resistance than the conventional concrete.

Nagataki and Iida (2001) observed that the freeze–thaw resistance of medium and low strength RCAC as well as of concrete with RCA obtained by primary crushing only (poor quality) was lower than that of high-strength RCAC and RCAC with RCA by a two-stage crushing process. The freeze–thaw resistance of RCAC improved with curing time. After 1 year of curing, the freeze–thaw resistance of RCAC's was similar to that of conventional concrete. Limbachiya et al. (2000) and Limbachiya (2010) observed similar freeze–thaw resistance in air-entrained conventional and high-strength concrete with design strength in the range of 30–50 MPa and in an equivalent type of concrete with RCA at 20–100 % replacement of coarse NA. Figure 5.84 shows the durability factor of 50 MPa NAC and RCAC evaluated with the British standard test where concrete samples are exposed to 300 freeze–thaw cycles. The NAC and RCAC with design strength of 50 MPa also met the British specification (BS 5328 Part 1–1991) for heavy duty external paving blocks.

Oliveira and Vasquez (1996) observed significant influence of moisture content in RCA on the freeze–thaw durability of the resulting concrete. The conventional concrete and the concrete containing RCA with 89.5 % moisture content as the only coarse aggregate resisted more than 100 cycles when both were exposed to freeze–thaw cycles. The RCAC with coarse RCA with 100, 0 and 88 % moisture contents failed after 20, 40 and 80 cycles respectively. Topçu and Sengel (2004) observed marginal deterioration of Schmidt hardness, CS and flexural strength when the conventional concrete and RCAC with 16 and 20 MPa design strength

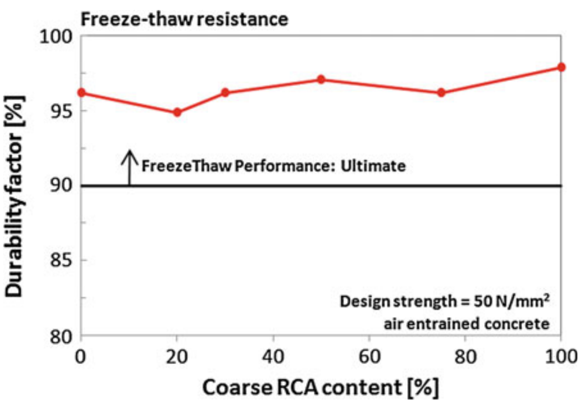


Fig. 5.84 Durability factor evaluated from freeze–thaw resistance tests of concrete *versus* replacement ratio of NA by coarse RCA (Limbachiya et al. 2000)

were exposed to cycles of freeze–thaw (−20 °C for 8 h and then 20 °C for 16 h) for 8 days. Ajdukiewicz and Kliszczzewicz (2002) observed similar or even better freeze–thaw resistivity in high-performance concrete with RCA than in conventional hpc.

Razaqpur et al. (2010) observed similar freeze–thaw resistance of conventional concrete and concrete with RCA as full replacement of coarse NA and prepared by the conventional and EMV methods. The RCAC prepared by the EMV method on the other hand exhibited better freeze–thaw resistance than the RCAC prepared by the conventional method due to the lower mortar content in the former RCAC (Abbas et al. 2009).

5.4.5 Alkali-Aggregate Reactivity and Resistance to Harsh Chemical Substances

A few references are available on the evaluation of the resistance of RCAC to several harmful chemical reactions or chemical environment such as alkali-aggregate reaction and sulphate resistance. Shayan and Xu (2003) observed marginally higher expansion of concrete prisms with replacement of coarse or fine NA by untreated or sodium silicate plus lime treated coarse or fine RCA than of conventional concrete when the specimens of all the types of concrete were subjected to the alkali-aggregate reactivity test for 1 year; however, the expansion of all types of concrete was below 0.024 % and well within the limit, 0.04 %, considered to be indicative of deleterious alkali-aggregate reaction.

Shayan and Xu (2003) observed satisfactory sulphate resistance of concrete with untreated or sodium silicate plus lime treated coarse or fine RCA concrete along with conventional concrete when concrete specimens were stored in a 5 % sodium sulphate solution for 1 year. Limbachiya (2010) observed a comparable expansion of two classes of conventional concrete and concrete with a 30 % replacement of coarse NA by RCA and design strength of 10 and 20 MPa, when both were immersed in a 3 % sodium sulphate solution for 6 months. However, the sulphate-induced expansion of RCAC increased as the replacement level of NA by RCA increased to 50 and 100 %. Limbachiya et al. (2012) observed lower sulphate resistance potential of RCAC than of conventional concrete when both were exposed to a 3 % sodium sulphate solution for 60 days. They observed gradually higher expansion of concrete as the replacement ratio of coarse NA by RCA increased (Fig. 5.85a). However, the addition of FA as a 30 % replacement of OPC slightly improved the sulphate resistance of RCAC due to the reduction in mono-sulphoaluminate and Portlandite contents in the cement paste and the prevention of reaction between free lime and sodium sulphate because of the pozzolanic property of FA (Fig. 5.85b).

Berndt (2009) observed lower dynamic elastic modulus of concrete with RCA as the only coarse aggregate after 12 months exposure into a 5 % sodium sulphate

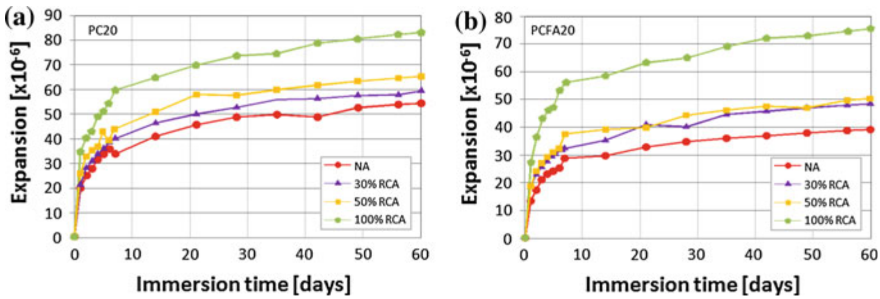


Fig. 5.85 Expansion of concrete with 20 MPa design strength: **a** OPC as binder **b** OPC-30 % FA as binder (Limbachiya et al. 2012)

solution due to the replacement of 50 and 70 % of OPC by ggbfs (Fig. 5.86). Lee et al. (2008) observed lower expansion of cement mortar due to the replacement of 50 % of fine NA by two types of fine RCA with different water absorption capacity when the hardened specimens were kept under sodium sulphate and magnesium sulphate solutions up to 15 months. On the other hand, at 100 % replacement level, depending upon the quality of RCA, the expansion was comparable or significantly higher than that of the conventional mortar. The mortar with higher water absorption capacity has higher expansion than the other one (Table 5.26).

Lee (2009) reported that the magnesium sulphate resistance of cement mortar with replacement of NA by fine RCA depended on the replacement ratio; the loss of CS and the expansion of RCA mortar (RCAM) with 25 and 50 % replacement of NA by RCA were lower and those of the RCAM with 75 and 100 % RCA were higher than the corresponding values of the conventional cement mortar when the specimens were cured in a 4.24 % magnesium sulphate solution for 1 year

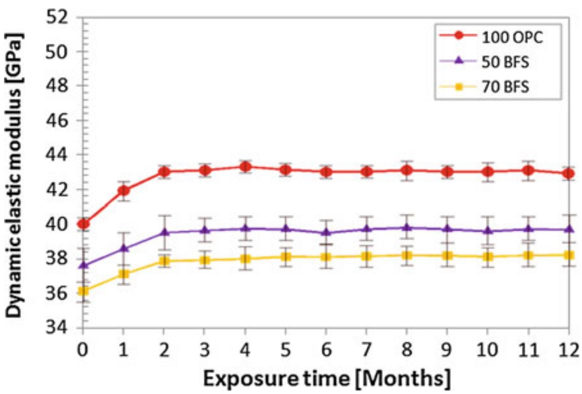


Fig. 5.86 Dynamic elastic modulus of RCAC containing OPC and OPC-slag binder up to 12 months exposure in a 5 % sulphate solution (Berndt 2009)

Table 5.26 Expansion of cement mortar immersed for 9 and 15 months in sodium and magnesium sulphate solutions (Lee et al. 2008)

Type of solution	Replacement amount	Expansion (%) of					
		Conventional mortar		RCA-A		RCA-B	
		9 months	15 months	9 months	15 months	9 months	15 months
Sodium sulphate	0	0.192	1.032				
	50			0.118	0.696	0.134	0.517
	100			0.954	Collapsed	0.287	0.974
Magnesium sulphate	0	0.105	0.523				
	50			0.070	0.386	0.085	0.303
	100			0.202	1.122	0.205	0.757

RCA-A and RCA-B Mortar with RCA with water absorption capacity of 10.35 and 6.59 % respectively

(Fig. 5.87). They also reported that the RCAM with less porous fine RCA has higher resistance to magnesium sulphate attack.

5.4.6 Other Durability Properties

Regardless of type of aggregate, in the Kwan et al. (2012) study, conventional concrete and RCAC slightly shrunk in the initial 24 h of wet curing and then expanded with further curing as well as with increasing replacement level of coarse NA by RCA (Fig. 5.88). The higher expansion in RCAC than in conventional concrete was due to the development of high hydrostatic pressure in the specimen because of the higher water absorption capacity of RCA than of NA.

Tu et al. (2006) observed significantly higher resistivity of high-performance concrete with RCA as complete replacement of fine and coarse NA or RCA as complete replacement of coarse NA than the minimum value for durable concrete, 20 kΩ-cm on or after 28 days of curing. The resistivity of concrete containing

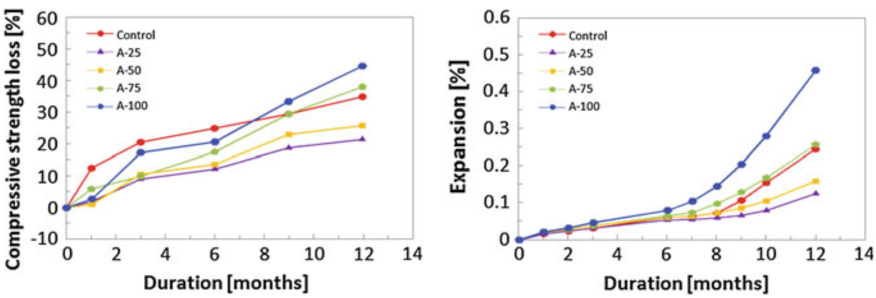


Fig. 5.87 CS loss and expansion due to magnesium sulphate attack of cement mortar with various contents of fine RCA (Lee 2009)

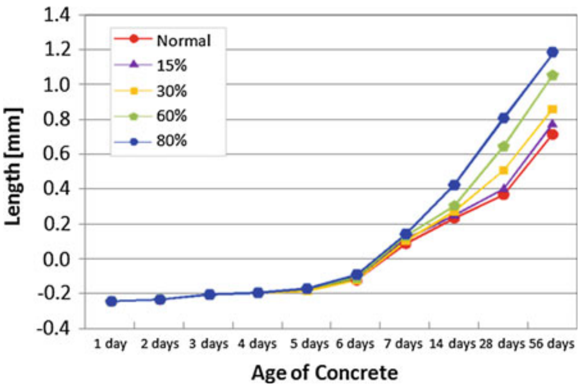


Fig. 5.88 Shrinkage followed by expansion of concrete due to water curing of concrete (Kwan et al. 2012)

RCA as a complete replacement of coarse NA was higher than that of the concrete containing RCA as a complete replacement of fine and coarse NA at different w/c ratios. Sani et al. (2005) observed a lower calcium leaching rate in RCAC prepared by completely replacing coarse NA and a part of fine NA by RCA when water was percolated through both types of concrete, despite the higher porosity of RCAC than of NAC (Fig. 5.89). The addition of FA further improved the leachability of ions for both types of concrete due to the pozzolanic activity.

Vieira et al. (2011) observed no significant differences in thermal response and mechanical properties namely CS, STS and MO of NAC and RCAC with replacement of 20, 50 and 100 % by volume of coarse NA by RCA when they were exposed for 1 h to temperatures of 400, 600 and 800 °C. On the other hand, Zega and Di Miao (2009) observed marginally good post-fire performances of CS, MO (E) and UPV for three types of RCAC's with a replacement of 75 % by volume of coarse NA by RCA from concrete containing three types of coarse aggregate (granitic crushed stone, quartzite crushed stone and siliceous gravel), when compared to equivalent conventional concrete with the same types of coarse NA, when the concrete specimens were exposed to a temperature of 500 °C for 1 h. The losses in percentage of various properties of the NAC and RCAC's due to

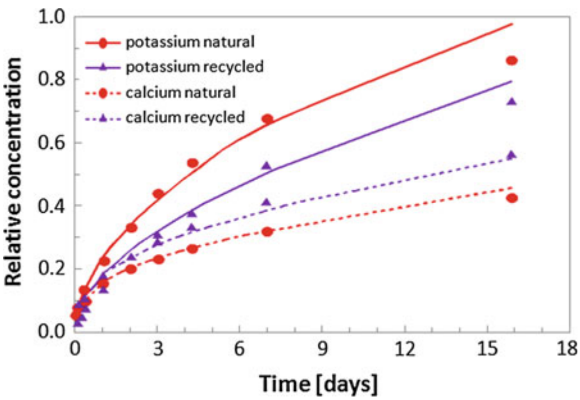


Fig. 5.89 Calcium and potassium leaching for NAC and RCAC (Sani et al. 2005)

Table 5.27 Loss in some mechanical properties (%) of NAC and RCAC due to exposure to 500 °C (Zega and Di Miao 2009)

Property	w/c	Loss (%)					
		GG		SG		QG	
		NAC	RCAC	NAC	RCAC	NAC	RCAC
CS	0.4	22	10	23	20	6	5
	0.7	15	13	24	21	24	16
MO(E)_static	0.4	46	28	46	28	26	23
	0.7	52	48	55	49	47	47
UPV	0.4	34	27	41	35	27	21
	0.7	41	38	47	45	39	38
MO(E)_dynamic	0.4	61	56	72	73	50	48
	0.7	68	69	79	77	72	67

GG, SG and QG Granitic gravel, siliceous gravel and quartzite gravel, respectively

the exposure to high temperatures are presented in Table 5.27. The authors also observed a better performance of RCAC's than NAC when concrete were prepared at w/c of 0.4 than at 0.7. Moreover, the RCAC containing recycled quartzite coarse aggregate exhibited better performance than the RCAC's with the other two types of recycled coarse aggregate at w/c of 0.4 but behaved similarly at w/c of 0.7.

Poon et al. (2009) studied the high temperature performance of concrete with two types of RA as the only coarse aggregate (CRA1 and CRA2) by varying several parameters during the preparation of the concrete mixes. The residual CS of various series of concrete after heating at 800 °C for 1 h is presented in Fig. 5.90. The authors observed a gradual increase of residual CS of concrete due to the variation of these parameters: gradual simultaneous increase of fine RA (FRA1) at aggregate to cement ratio of 10:1; gradual increase of aggregate to cement ratio at 100 % CRA2 and FRA2 contents and gradual increase of FRA1 content in concrete having CRA1 as the only coarse aggregate at aggregate to cement ratio of 12:1 (A, B, and C series in Fig. 5.90 respectively). The soil content

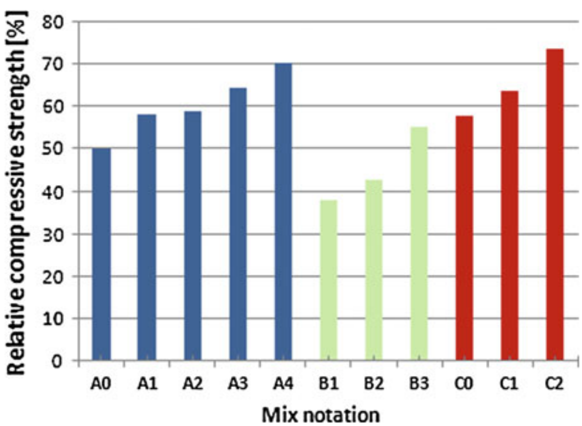


Fig. 5.90 Residual CS of RCAC after heating at 800 °C (Poon et al. 2009)

in these series also increased with the FRA1 content. According to the authors, the low residual CS of the B series in comparison to the other two series was due to relatively low soil content and high residual mortar content in FRA2 than in FRA1. The gradual improvement of the performance of concrete due to the increase of soil content was due to the formation of crystalline calcium aluminate silicates at high temperature.

5.5 Conclusions

The recent developments on the properties of concrete with CDW as aggregate were discussed thoroughly in this chapter. The results are presented in three different sections to present fresh and hardened properties of concrete. Aggregates generated from recycled concrete or other ceramic-based waste materials are reported to be used as partial or full replacements of fine and coarse aggregates in concrete. The use of these aggregates as coarse aggregate is more versatile than that as fine aggregates. Both normal and high-strength concrete can be produced with CDW aggregates. In several investigations, it was reported that 30 % replacement of natural aggregates by aggregates generated from waste concrete did not substantially deteriorate the mechanical and durability performances of the resulting concrete. However, the presence of impurities such as clay brick, tiles and other ceramics can strongly jeopardize the performance of concrete and therefore in this case replacement ratios should be lower than the one mentioned above. Some important conclusions are indicated below:

1. The use of any type of CDW aggregates substantially lowers the workability of concrete. This is mainly due to the higher water absorption capacity of these types of aggregates than that of NA because of adhered porous mortar. Therefore several mixing procedures have been developed to improve the workability performance of CDW aggregates-based concrete; out of these, pre-saturating aggregates for 10 min before mixing or using 85–90 % humid CDW aggregate are the most widely used techniques. The density of concrete also mildly decreases with the replacement of NA by CDW aggregates;
2. All the strength properties and modulus of elasticity of concrete are also deteriorated by the incorporation of CDW aggregates as partial or full replacement of NA. However, in comparison to the reduction in compressive strength, the reduction in flexural and splitting tensile strength is not so prominent. The reduction in these properties of concrete is due mainly to the presence of adhered mortar in CDW aggregate. The reduction in modulus of elasticity is also very high due to the lower modulus of elasticity of CDW aggregates than that of NA. However, pre-saturation of the aggregates before mixing can increase the strength properties and the modulus of elasticity of concrete;

3. The load-deflection curve of concrete containing CDW aggregate with NA is marginally different from the load-deflection curve of conventional concrete. Concrete with CDW aggregates normally has lower toughness and ductility performances but higher creep than those of conventional concrete. The surface hardness properties such as Schmidt hardness, skid resistance, impact and abrasion resistances of concrete with CDW aggregates are normally inferior to those of conventional concrete, primarily due to the presence of adhered mortar. However, several researchers observed similar abrasion behaviour of CDW aggregates-based and natural aggregates-based concrete;
4. The incorporation of CDW aggregates in concrete increases its drying shrinkage due to the higher paste content. This incorporation increases total porosity and therefore increases various permeability properties such as water absorption, chloride migration, depth of carbonation, various gas permeability of the resulting concrete. Concrete containing CDW aggregates has poorer freeze–thaw resistance than that of conventional concrete. The resistance of CDW aggregates-based concrete to some harsh chemical solutions is also poorer than that of NA-based concrete;
5. However, by improving the concrete mixing preparation techniques, almost all properties of concrete with CDW aggregates as a replacement of NA can be substantially improved and in some cases property results are comparable to those of conventional properties. The replacement of cement up to given amount by several mineral additions such as fly ash, blast furnace slag, silica fume, metakaolin can improve almost all properties of concrete with CDW aggregate at various stages of curing depending on their reactivity. The properties of concrete with CDW aggregate as partial replacement of NA up to given replacement ratios (30 %, in most of the cases) are similar to the equivalent properties of conventional concrete.

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