Chapter 4
Use of Industrial Waste as Aggregate: Properties of Concrete

4.1 Introduction

As indicated in an earlier section, aggregates account for the largest part of the concrete volume and therefore play a substantial role in almost all concrete properties such as workability, strength, dimensional stability, and durability. Recently, several waste materials have been studied to be used as aggregate in concrete. The use of waste as aggregates can consume vast amounts of waste materials as this is the major component of cement mortar and concrete.

In this section, properties of concrete with various types of waste aggregates generated from various industries will be presented. The major focus will be given on the behaviour of concrete with various waste industrial aggregates; however, if information is not available on a particular property of concrete, the same or similar property of cement mortar with that particular waste will be considered.

4.2 Coal Bottom Ash

Many references are available on the properties and use of fly ash (FA) as a mineral addition in conventional Portland cement concrete. However, not much has been reported on the use of FA and coal bottom ash (CBA) as aggregate in concrete. CBA falls into the bottom of the furnace in modern large thermal power plants and constitutes about 20 % of total ash content of the coal fed into the boilers. The properties of CBA depend on the coal type, pulverising system, combustion conditions, temperature, type of furnace, minerals in coals and milling system and these are already presented in detail in Chap. 2. Here, the effect on CBA aggregate on the several concrete properties will be discussed.

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4.2.1 Fresh Concrete Properties

4.2.1.1 Workability/Slump Behaviour

The slump behaviour of concrete due to the incorporation of CBA is probably dependent on its shape, porosity and surface texture. Therefore, two parallel views exist concerning the slump behaviour of concrete with CBA.

Agarwal et al. (2007) measured the workability of concrete with CBA as replacement of fine aggregate using the compacting factor test, described in Indian standard, IS 1199-1959. They found that the workability of concrete decreased as the replacement level of the fine aggregates with CBA increased (Fig. 4.1a). The increase in the specific surface due to increased fineness of fine aggregate as well as a greater amount of water needed for the mix of ingredients to get closer packing result in decrease of the workability of the mix.

Kim and Lee (2011) reported contrasting slump values for concrete with fine and coarse CBA used as a partial or full replacement of natural fine and coarse aggregate, respectively. They concluded that the flow characteristics of fresh concrete were slightly reduced by the use of coarse CBA, whereas the effect of fine CBA can be neglected. They observed a 20.8 % reduction of slump of fresh concrete that contains coarse CBA only as coarse aggregate. More complicated shape and rougher surfaces of CBA than normal aggregate and a lowering of aggregate–cement paste lubrication effect due to absorption of some free cement paste and water by porous CBA are the major causes of this reduction. On the other hand, the porosity and water absorption capacity of fine CBA is lower than

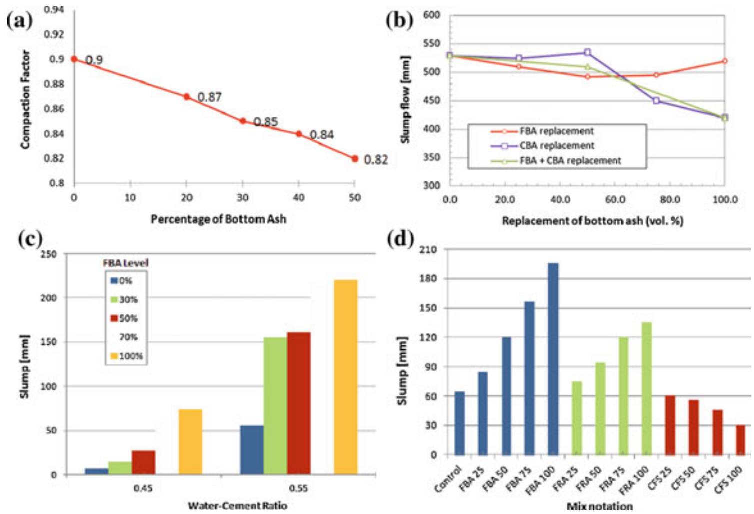


Fig. 4.1 Workability behaviour of concrete with coal bottom ash aggregate. **a** Aggarwal et al. 2007; **b** Kim and Lee 2011; **c** Bai et al. 2005; **d** Kou and Poon 2009

that of coarse CBA and therefore concrete with fine CBA absorbs negligible amounts of cement paste and water during mixing, which does not affect the slump value of the resulting concrete mix. Their results are presented in Fig. 4.1b.

Kasemchaisiri and Tangtermsirikul (2008) also observed reduction in slump value due to incorporation of CBA as a partial substitution of fine aggregate. According to these authors, this was due to an increase in frictional forces between aggregate particles as CBA is highly irregular in shape and it has rough surface texture.

Bai et al. (2005) reported an increase in slump due to the partial substitution of sand with fine CBA in the concrete mix (Fig. 4.1c). According to the authors, the presence of a "ball-bearing effect", due to the replacement of irregular shaped normal sand by spherical shaped fine CBA aggregate increases the slump value of the concrete mix.

Kou and Poon (2009) also reported increasing slump of fresh concrete mix due to the incorporation of fine CBA as partial or total replacement of normal sand (Fig. 4.1d). An increase in free water content in the concrete mix with fine CBA by comparison with that of the conventional concrete mix, due to the high water absorption capacity of CBA, increases slump.

4.2.1.2 Bleeding Behaviour

Andrade et al. (2009) observed bleeding of water during the preparation of a fresh concrete mix with CBA as aggregate. The authors reported that the concrete mix started to segregate (i.e. the aggregates and cement particles tended to occupy the bottom of the container) during the concrete preparation and moulding process due to the difference in weight of various constituents in the concrete mix. The water loss due to the addition of CBA as partial replacement of sand fraction in the concrete mix is presented in Fig. 4.2a. The bleeding of water increases with increasing content of CBA. Ghafoori and Bucholc (1996) observed a similar behaviour for concrete with fine CBA as partial substitution of sand (Fig. 4.2b). According to these authors, the higher bleeding of fresh concrete mix due to

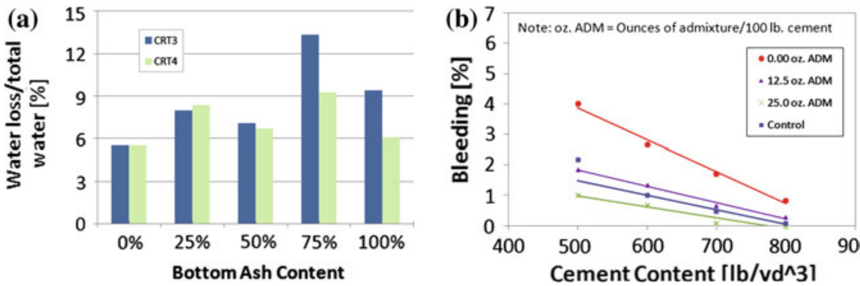


Fig. 4.2 Water loss due to bleeding of concrete mixes with various amounts of fine CBA aggregates. **a** Andrade et al. (2009); **b** Ghafoori and Bucholc (1996)

addition of CBA by comparison with conventional concrete is due to the increased demand of water during the mixing of concrete with CBA.

Andrade et al. (2007) reported that the water, absorbed during mixing, is desorbed at a later stage and increases bleeding. Decreases in water/cement (w/c) ratio and addition of air-entraining admixture can significantly decrease bleeding of concrete with CBA (Andrade et al. 2009; Ghafoori and Bucholc 1996).

4.2.1.3 Density

As the density of CBA is considerably lower than that of normal fine and coarse aggregates, the inclusion of CBA aggregate in concrete decreases its unit weight or density. Another factor that is pointed out in some of the studies is the higher w/c ratio of concrete with CBA than in conventional concrete, which introduces more air bubbles in the concrete mix. Figure 4.3 shows the density of concrete with two different size ranges (Lee et al. 2010). The size ranges of the CBA aggregate present in concrete mixes F1 and F2 are, respectively, 0–2 and 2–8 mm. A significant decrease in density was observed due to the incorporation of CBA aggregate in concrete. Yüksel et al. (2007) reported about 30 % reduction in fresh density of concrete briquette (block) with CBA used to replace 50 % (in volume) of 0–4 mm sand.

4.2.2 Hardened Concrete Properties

4.2.2.1 Density of Concrete

Just like for fresh-state density, the incorporation of CBA aggregate also decreases the dry density of hardened concrete due to the low bulk density of CBA aggregate. Experimental results of two different types of concrete are presented in Fig. 4.4.

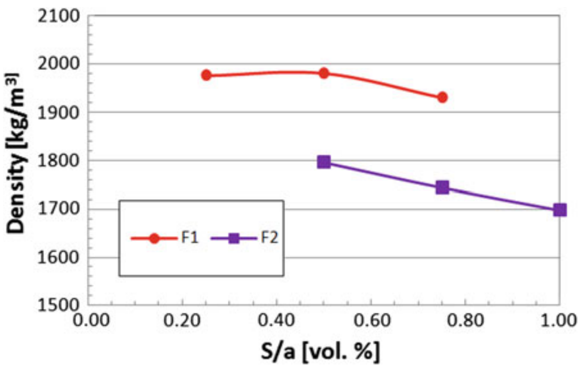


Fig. 4.3 Density of concrete with CBA aggregate (Lee et al. 2010)

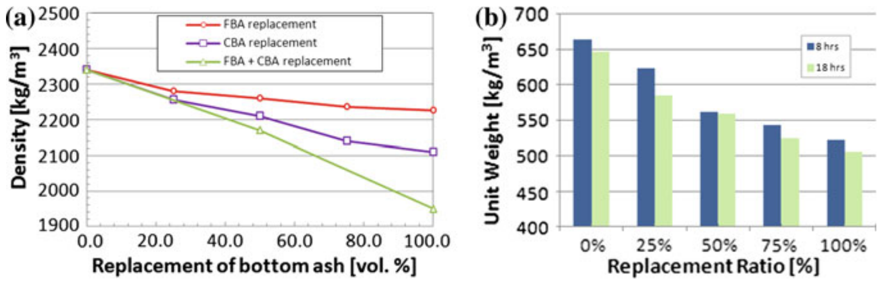


Fig. 4.4 Density of two different types of hardened concrete. **a** Normal (Kim and Lee 2011); **b** Autoclaved aerated (Kurama et al. 2009)

4.2.2.2 Compressive Strength

Variations in compressive strength of concrete due to the incorporation of CBA aggregate were observed depending on the method of preparation of concrete. Bai et al. (2005) observed lower compressive strength of concrete with CBA aggregate than that of conventional concrete at constant w/c ratio, while the two types of concrete exhibited almost similar compressive strength at constant slump (Fig. 4.5).

The same authors concluded that 30 % of natural sand could be replaced with CBA aggregate to produce concrete in the 40–60 N/mm^2 compressive strength range without detrimentally affecting the permeation and drying shrinkage properties of structural concrete. Yüksel et al. (2011) also observed a decreasing trend

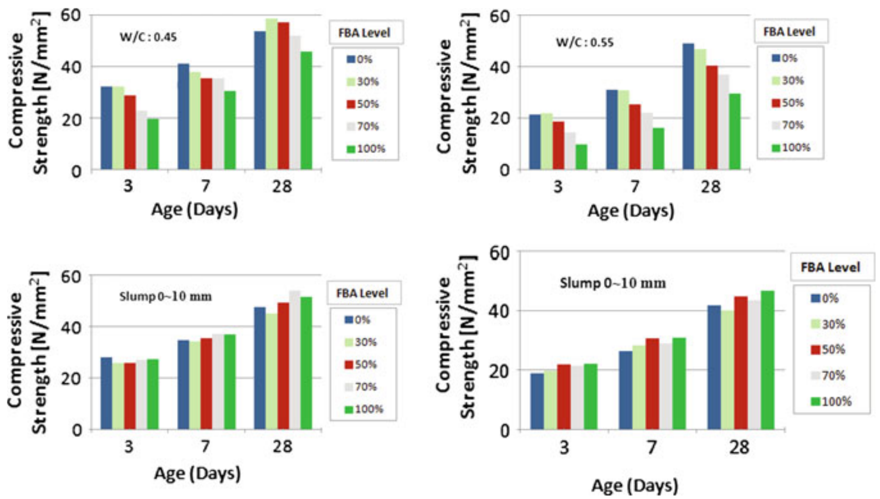


Fig. 4.5 Compressive strength of concrete with CBA at constant w/c and slump values (Bai et al. 2005)

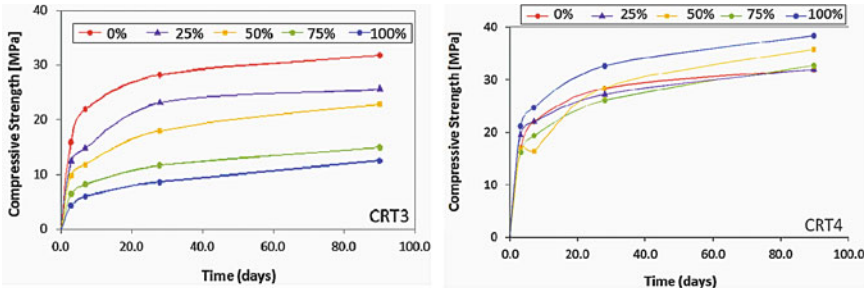


Fig. 4.6 Compressive strengths of concrete with CBA (Andrade et al. 2007)

in compressive strength of concrete due to increasing addition of CBA for constant w/c.

Kim and Lee (2011) did not observe any significant changes in the compressive strength of concrete with constant w/c value due to the incorporation of coarse and fine CBA as partial and full replacement of coarse and fine aggregate, respectively. According to the authors, this was due to the presence of higher amounts of cement paste than those observed in other studies. Kuruma et al. (2009) observed higher compressive strength in autoclaved aerated concrete with fine CBA replacing 50 % of natural sand than in similar conventional concrete, and compressive strength further decreased with increasing replacement level. This was mainly due to the pozzolanic activity of CBA, which increased at autoclaved aerated conditions and therefore forms additional amounts of products like calcium silicate hydrate gel, and strengthen the structure.

Andrade et al. (2007) reported that the consideration of water content in CBA during the preparation of a concrete mix has profound effect on the compressive strength of hardened concrete. Their results are presented in Fig. 4.6. The authors prepared two types of concrete: in one type the moisture content in CBA was not considered to determine the water amount in the mix (CRT3) and in the other type the moisture content in CBA was considered for that effect (CRT4).

As the CBA is slightly pozzolanic by nature, the strength development pattern with respect to elapsed time for concrete with CBA is different from that of conventional concrete. In most studies, it was reported that this type of concrete gains strength at a slower rate in the initial period of curing and grows faster at the latter stage of curing (Andrade et al. 2007; Agarwal et al. 2007; Ghafoori and Bacholc 1996). According to these authors, bottom ash takes parts in hydration reaction at the latter stages of curing and forms other products. In Fig. 4.6, the strength development behaviour of two types of concrete with CBA aggregate is presented. Park et al. (2009) reported that the failure of concrete with coarse CBA aggregate was predominantly by aggregate fracture instead of binder fracture and interface fracture, due to the lesser hardness of CBA versus that of normal aggregate.

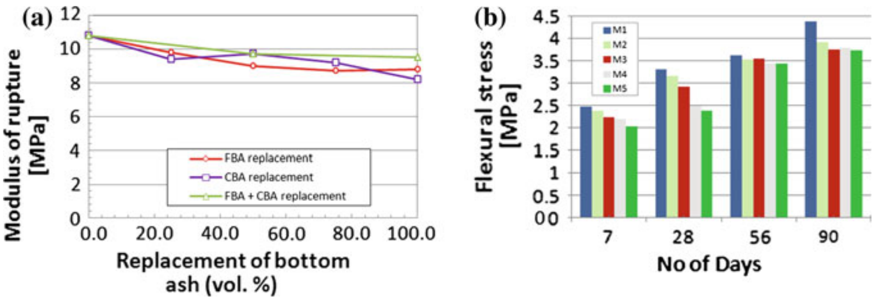


Fig. 4.7 Flexural strength of concrete with CBA aggregate (M1, M2, M3, M4, M5 in Fig. 4.7b represents concrete with CBA replacing 0, 20, 30, 40 and 50% by weight of natural sand). **a** Kim and Lee 2011; **b** Aggarwal et al. 2007

4.2.2.3 Flexural Strength

Some authors observed a significant reduction in the flexural strength of concrete due to the incorporation of CBA, as well as to increasing CBA content, even though the effect observed in compressive strength behaviour was insignificant (Kim and Lee 2011, Agarwal et al. 2007). Some experimental results are presented in Fig. 4.7. On the other hand, Triches et al. (2007) observed an increase in flexural strength of roller compacted concrete (RCC) due to the addition of CBA as partial substitution of natural sand because of the pozzolanic activity of CBA as well as improvements of aggregate arrangement in the concrete matrix. Kuruma et al. (2009) observed higher flexural strengths for concrete prepared by replacing 50 % (by weight) of natural sand with fine CBA.

Ghafoori and Bacholc (1996) reported that conventional concrete exhibited higher flexural strength at low content of cement than concrete with CBA fine aggregate. However, they were able to reduce this difference by increasing cement content in concrete mix as well as by adding chemical admixture in the concrete mix

Table 4.1 Flexural strength of concrete (psi) with natural sand and CBA as fine aggregates (Ghafoori and Bucholc 1996)

Curing age (day)	Cement content in concrete (lb/yd^3)				Cement content in concrete (lb/yd^3)			
	500	600	700	800	500	600	700	800
	Natural sand (C)				CBA aggregate (BA)			
7	505	646	716	818	376	501	641	752
28	595	722	797	881	481	622	748	916
90	688	830	945	985	573	707	815	925
	CBA aggregate + 12.5 oz ADM/100 lb cement (ADM1)				CBA aggregate + 25.0 oz ADM/100 lb cement (ADM2)			
7	469	588	743	809	452	675	809	951
28	674	788	840	926	636	830	967	1054
90	711	826	904	963	721	882	1021	1137

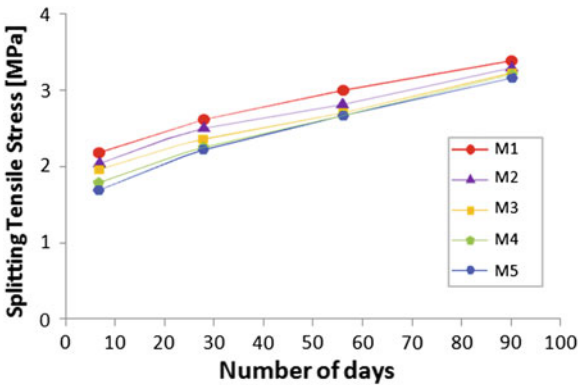


Fig. 4.8 Splitting tensile strength of concrete with CBA aggregate (Aggarwal et al. 2007)

with CBA. For a given content of admixture, the flexural strength of CBA concrete became higher than that of control concrete. Their results are presented in Table 4.1.

4.2.2.4 Splitting Tensile Strength

The splitting tensile strength of concrete decreases as replacement percentage of fine aggregate by CBA rises and increases with the curing age (Agarwal et al. 2007). The highest and lowest gains in splitting tensile strength were observed at 20 and 50 % replacement of fine aggregates with bottom ash, respectively (designated M2 and M5). Plain concrete reaches 64, 77 and 88 % of 90-day strength at 7, 28 and 56 days of curing, respectively, whereas these values for concrete with CBA at 20, 30, 40 and 50 % replacement levels were in the ranges of 62–86 %, 60–83 %, 56–83 % and 53–84 %, respectively. Their results are presented in Fig. 4.8.

Ghafoori and Bacholc (1996) reported that the inclusion of CBA in concrete had more influence on splitting tensile strength than on compressive strength. At low cement content (500 lb/yd^3 of concrete), the splitting tensile strength of concrete with CBA aggregate was lower than that of conventional concrete at the early ages of curing and was similar after 56 days of curing. However, the initial dormant period of CBA concrete can be overcome by adding admixtures to this type of concrete. On the other hand, the splitting tensile strength of both types of concrete was similar for the concrete mix with 600 lb/yd^3 of cement. These results are presented in Fig. 4.9.

4.2.2.5 Static Elastic Modulus

The static elastic modulus of concrete with CBA is significantly lower than that of conventional concrete. Ghafoori and Bacholc (1996) reported that the higher elastic modulus value of conventional concrete than that of CBA concrete was due to the lower paste porosity of conventional concrete than that of CBA concrete, as the w/c value of conventional concrete was lower than that of CBA concrete, as well as to the higher bulk density of natural sand aggregate than that of CBA

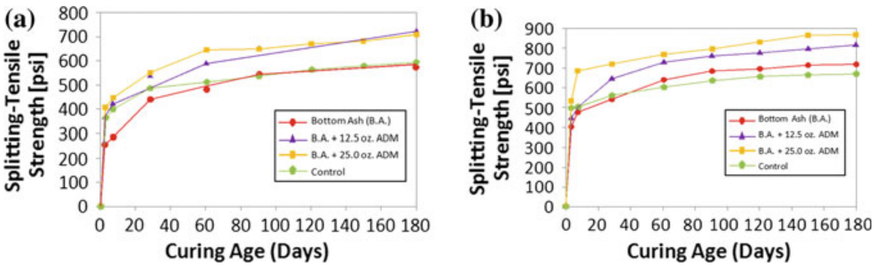


Fig. 4.9 Splitting tensile strength of normal and CBA concrete (Ghafoori and Bucholc 1996). **a** Cement content: 500 lb/yd^3; **b** Cement content: 600 lb/yd^3

Table 4.2 Modulus of elasticity of different types of concrete (Ghafoori and Bucholc 1996)

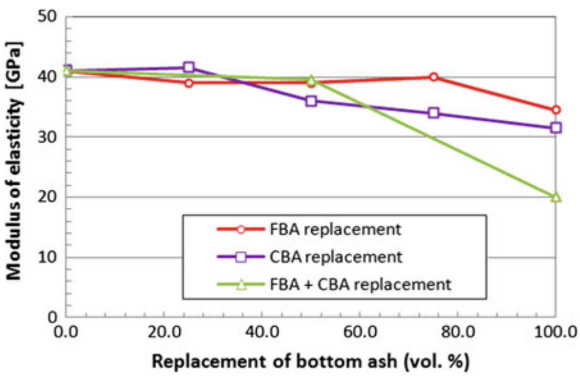
Cement content (lb/yd^3)	Modulus of elasticity, (psi) of concrete ($\times$ 10^6)			
	C	CBA	ADM1	ADM2
500	5.02	3.32	3.64	3.83
600	5.58	3.80	4.04	4.36
700	5.80	3.86	4.32	5.05
800	5.74	4.25	4.47	5.20

Details about concrete mix proportions are presented in Table 4.1

aggregate. Their results are presented in Table 4.2. The authors achieved a significant improvement of the modulus of elasticity for CBA concrete by using a higher amount of cement along with the addition of an admixture.

Kim and Lee (2011) also observed a similar modulus of elasticity of high-strength concrete (HSC) prepared by replacing 50 % by volume of aggregate with CBA, beyond which it dropped quickly (Fig. 4.10). The authors observed a higher reduction in the modulus of elasticity using coarse CBA aggregate than for fine CBA. The reduction in modulus of elasticity of concrete due to 100 % replacement of fine natural aggregate (NA) by fine CBA was about 15 %, whereas these values

Fig. 4.10 Modulus of elasticity of concrete with various amounts of CBA aggregates (Kim and Lee 2011)



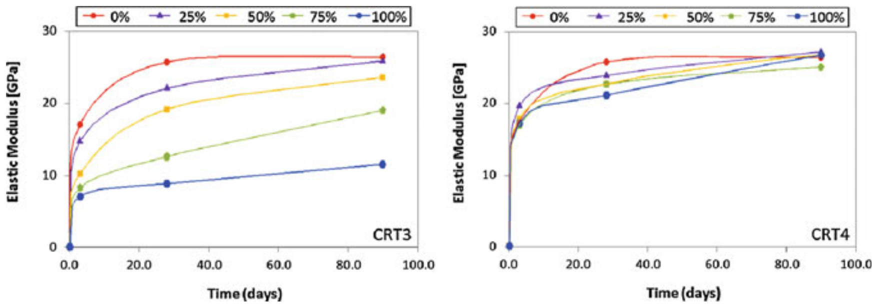


Fig. 4.11 Modulus of elasticity of concrete with CBA (Andrade et al. 2007)

for 100 % replacement by coarse CBA and 100 % replacement by a mixture of fine and coarse CBA were, respectively, 22.5 and 51 %.

Andrade et al. (2007) also observed significant reduction in elastic modulus value for concrete with fine CBA as partial or full replacement of fine aggregate fraction (Fig. 4.11). This reduction was prominent at the early stage of curing. However, at the latter stages of curing the pozzolanic reaction of CBA made the microstructure of concrete denser and improved the mechanical properties including the elasticity modulus. However, changes in water content during concrete the mix preparation by considering the water content in CBA can improve the elastic modulus behaviour of hardened concrete. The authors refer this fraction as CRT4 in Fig. 4.11.

The same authors plotted stress–strain curves for concrete with CBA aggregates. However, they did not find too much difference for CRT4 type concrete, but other types gave scattered results at all ages.

4.2.3 Durability Behaviour

4.2.3.1 Drying Shrinkage

The drying shrinkage of concrete is generally affected by the addition of CBA aggregates, as this material is porous by nature and therefore absorbs a large amount of water. Bai et al. (2005) reported that the concrete with CBA as a replacement of sand fraction at constant w/c value exhibited lower drying shrinkage than conventional concrete. This is due to the release of moisture absorbed by CBA during dry condition that keeps the mortar in a moist condition. On the other hand, for constant slump value, shrinkage increased with increasing content of CBA. However, in this condition, the authors found a comparable drying shrinkage of concrete with CBA replacing 30 % by weight of natural sand. Their results are presented in Fig. 4.12.

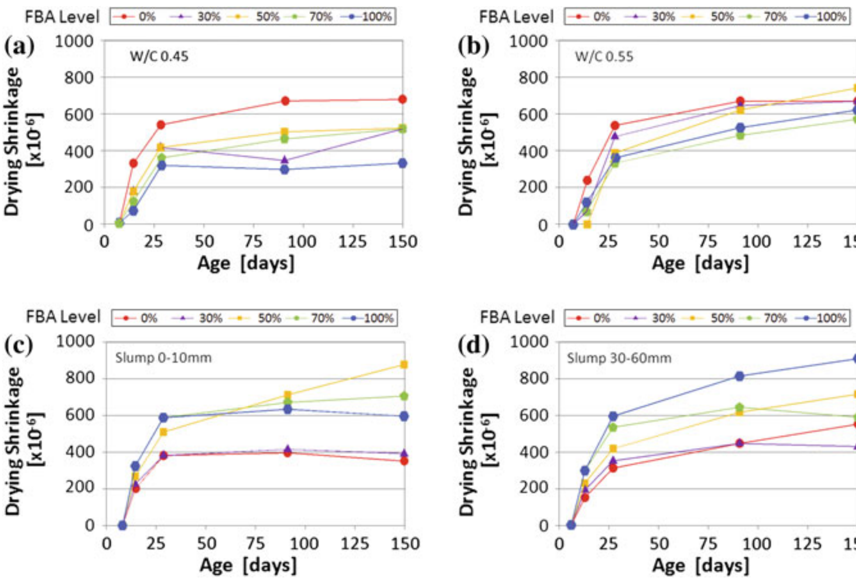


Fig. 4.12 Drying shrinkage of concrete with fine CBA aggregates (Bai et al. 2005). **a** W/C 0.45. **b** W/C 0.55. **c** Slump range 0–10 mm. **d** Slump range 30–60 mm

Kim and Lee (2011) observed similar results for concrete with CBA prepared at constant w/c value and slump value. The lower shrinkage value of concrete due to an increasing content of CBA aggregate at constant slump was due to the decrease in free water content in concrete with CBA. Ghafoori and Bacholc (1996) also reported lower drying shrinkage for CBA concrete than the conventional concrete despite the higher w/c value of the former concrete.

The plastic shrinkage (early volume change) of concrete is also affected by the inclusion of CBA aggregates. Ghafoori and Bacholc (1996) found about 35 % reduction in plastic shrinkage of concrete with fine CBA aggregates than that observed for normal concrete due to higher bleeding of the former type of concrete. Incorporating a low content of chemical admixture had little effect on the plastic shrinkage of CBA concrete but it could be increased considerably by adding a higher dosage of admixture. Bleeding water was significantly reduced for higher admixture content, and therefore increased the shrinkage value. Andrade et al. (2009) also found a reduction in the plastic shrinkage of concrete due to the incorporation of CBA aggregates. However, the shrinkage of concrete prepared by considering the moisture and free water content in CBA aggregates was higher than that of the reference concrete. The reduction in shrinkage was due to the higher content of bleeding water as well as to the absorption of water by CBA.

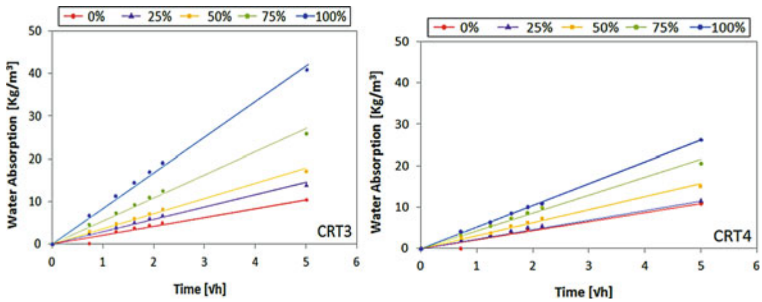


Fig. 4.13 Capillary water absorption of concrete with CBA aggregates (Andrade et al. 2009)

4.2.3.2 Capillary Water Absorption

Andrade et al. (2009) reported higher capillary water absorption values for concrete with various amounts of CBA as fine aggregates than for the reference concrete. According to the authors, the addition of porous CBA aggregate in concrete not only provides some free water due to bleeding but it also provides a pore system that is different from that of the reference concrete. However, capillary water can be reduced significantly if the water content in bottom ash is considered during concrete mixing. Their results are presented in Fig. 4.13. They also measured the sorptivity coefficient for different concrete mixes from capillary absorption data, which are presented in Table 4.3. Yüksel et al. (2007) also observed increasing capillary water absorption coefficients with increasing CBA content except for the 10 % replacement level of fine aggregate by volume. At this level, the pozzolanic activity of CBA decreases the porosity of concrete but at higher substitution levels the porosity of CBA increases the overall porosity of concrete and therefore increases the permeability.

4.2.3.3 Chloride Permeability

Ghafoori and Bacholc (1996) observed about 120 % higher current flow in CBA concrete than in conventional concrete in the rapid chloride permeability test. The authors also reported that the addition of an admixture reduced the chloride permeability of CBA concrete; in this concrete, the current flow was about 61 % higher than in the reference concrete. Kou and Poon (2009) reported contrasting

Table 4.3 Water sorptivity coefficient (kg m^{-2} $h^{0.5}$) of different type of concrete (Andrade et al. 2009)

Type of concrete	Amount of CBA in concrete (mass%)				
	0	25	50	75	100
CRT3	2.2	3.1	3.8	5.7	8.9
CRT4		2.3	3.2	4.4	5.3

behaviour of chloride permeability of concrete with CBA aggregate prepared at constant w/c value and at constant slump. At constant w/c value, the chloride permeability of CBA concrete was higher than that of conventional concrete and it increased with the CBA content, because of the looser microstructure of CBA concrete than that of the control concrete due to its higher free water content. On the other hand, at constant slump, the free water content in CBA concrete was not as high as in the reference concrete and therefore chloride permeability was reduced. Kasemchaisiri and Tangtermsirikul (2008) observed an increase in chloride permeability of 7-day cured self-compacting concrete with increasing CBA aggregate content. However, the authors observed a similar permeability for CBA concrete and the reference concrete at latter stages of curing, possibly due to the pozzolanic reaction of CBA.

4.2.3.4 Carbonation Depth

Kasemachaisiri and Tangtermsirikul (2008) measured the carbonation depth of 28- and 56-day cured concrete with CBA as a partial replacement of fine aggregates by using accelerated carbonation test. They found a slightly higher carbonation depth for concrete with 10 % CBA than that for the reference concrete. However, these values were much higher for concrete with 20 and 30 % CBA than for the reference concrete. The increase in porosity of concrete due to the addition of porous CBA in concrete led to deeper carbonation. However, the difference between the carbonation depth of the conventional concrete and the concrete with CBA aggregate decreased as curing time increased due to the pozzolanic activity of CBA.

4.2.3.5 Resistance to Chemical Attack

Kasemachaisiri and Tangtermsirikul (2008) reported higher resistance to sulphate attack of self-compacting concrete with CBA aggregates than of the reference concrete. The sulphate resistance of CBA concrete increased with increasing content of CBA. The observed improved performance of CBA concrete by comparison with the reference concrete was due to the predominance of sulphate enhanced pozzolanic activity of CBA aggregate over the porosity induced by porous CBA. Ghafoori and Bacholc (1996) did not observe any substantial differences between the expansions of CBA concrete and conventional concrete after 6-month exposure of concrete specimens in sulphate solution. The test was performed according to the ASTM C 1012 standard method.

4.2.3.6 Abrasion Resistance

Ghafoori and Bacholc (1996) observed higher depth of wear for concrete with CBA fine aggregates than for conventional concrete. The average depth of wear

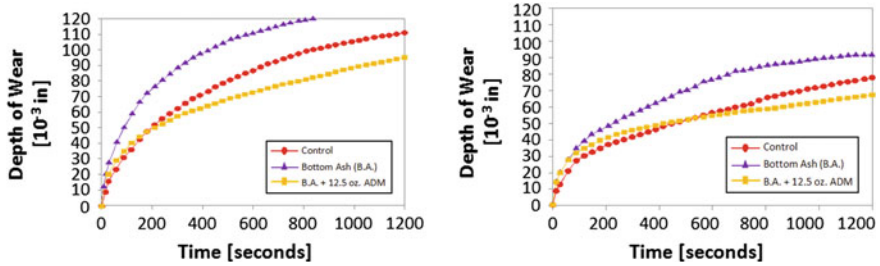


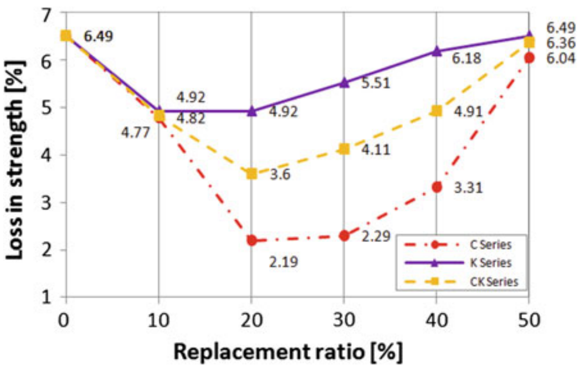
Fig. 4.14 Depth of wear of concrete with CBA and conventional concrete for two cement contents (Ghafoori and Bucholc 1996)

for CBA concrete was about 40 % higher that of the conventional concrete. However, a low content admixture can improve the abrasion resistance of CBA concrete and make it exhibit better performance than conventional concrete. These results are presented in Fig. 4.14. Yüksel et al. (2007) observed similar abrasion values in concrete with various percentage of CBA as fine aggregates replacement and conventional concrete.

4.2.3.7 Resistance to Freeze–Thaw and Dry–Wet Cycles

Yüksel et al. (2007) observed better performance of concrete with CBA aggregate than of conventional concrete subjected to freeze–thaw cycles. The strength loss for the concrete with CBA at 10 and 20 % replacement levels was almost the same but at higher substitution level the strength loss again increased and at 100 % replacement level it became similar to that of conventional concrete. They also observed further improvement of CBA concrete by mixing it with blast furnace slag aggregate (BFS). Their results are presented in Fig. 4.15. Ghafoori and Bucholc (1996) also observed better performance of concrete with CBA as fine aggregates than conventional concrete when the concrete specimens were intermittently subjected to freeze–thaw cycles.

Fig. 4.15 Freeze–thaw resistance of concrete with CBA (K-series), BFS concrete (C-series) and concrete with a mixture of CBA and BFS (CK-series) (Yüksel et al. 2007)



Yüksel et al. (2007) observed higher compressive strength losses for concrete with CBA aggregates than that observed for conventional concrete when both types of concrete were subjected to intermittent wet–dry cycles due to the increase in porosity of CBA concrete. This increase further increases the strength loss. Mixing BFS aggregate with CBA aggregates improved the performance of CBA concrete under these environmental conditions.

4.2.3.8 High Temperature Behaviour

The addition of CBA as a partial substitution of fine aggregates in concrete up to a certain level improved its high temperature performance (Yüksel et al. 2007, 2011). The percentage of residual compressive strength of CBA concrete at 20 % replacement level was the highest, then it gradually decreased at higher substitution rates and it was similar to that of conventional concrete at 40 % replacement level. Compared to the surface of the post-fired reference concrete, which had several randomly distributed cracks, the surfaces of post-fired CBA concrete contained very few cracks.

4.2.4 Coal Fly Ash

The use of FA as a pozzolan is well documented and many standard code of practice already recommend its use as pozzolanic material in concrete. However, limited studies are available on the use of FA as fine aggregates in concrete preparation. In this section, the concrete properties will be briefly highlighted.

As FA consists of very fine spherical particles, a concrete mix with FA as aggregates is more workable, cohesive, mobile, compactable and pumpable than conventional concrete (Ravina 1997; Pofale and Deo 2010). However, the water requirement of reference concrete to reach similar consistency to concrete with FA as partial replacement of sand depends on the size of the sand fraction (Ravina 1997). Ravina (1997) observed similar water requirements for concrete mixes with FA replacing fine sand and higher water requirements for concrete mixes where FA replaced relatively coarse sand. Siddique (2003a, b) observed a decreasing slump trend due to the incorporation of increasing replacement contents of fine aggregates by FA aggregates.

Siddique (2003a) also observed decreasing air-content values with increasing content of FA as partial replacement of fine aggregates. However, concrete fresh density increased with increasing FA contents. These results are presented in Table 4.4.

Bleeding of concrete due to the addition of FA aggregates was similar to that of conventional concrete. The addition of water reducers and retarders significantly increased bleeding and the addition of superplasticizers reduced bleeding (Ravina 1997).

Table 4.4 Fresh concrete properties of concrete with fly ash as partial replacement of fine aggregates (Siddique 2003a)

Properties	Amounts of sand replaced by fly ash (%)					
	0	10	20	30	40	50
Water/cement	0.47	0.48	0.49	0.49	0.49	0.50
Slump (mm)	100	90	65	40	30	20
Air content (%)	5.2	4.8	4.4	4.0	3.8	3.2
Density (kg/m^3)	2308	2310	2314	2314	2316	2319

The addition of class F FA as a partial replacement of fine aggregate improved the compressive, flexural and splitting tensile strengths as well as the modulus of elasticity of the resulting concrete and the more so the greater the curing time (Siddique 2003a). Similar compressive strength behaviour of concrete due to the addition of low calcium FA was also observed by Maslehuddin et al. (1989). This increase is due to the densification of microstructure due to the pozzolanic reaction of FA. Figure 4.16 shows the compressive, splitting tensile and flexural strengths of concrete with FA as partial substitution of fine aggregates. Papadakis (1999) observed higher compressive strength development of mortar after 14 days of curing when low calcium FA was used as partial replacement of fine aggregates. On the other hand, the strength development of concrete was observed only after 91 days when the same FA was used to replace cement. The same author observed higher compressive strength, bound water and total porosity of mortar when a high calcium FA was used to partially replace the fine aggregates, whereas the strength was almost similar during the experimental curing period when the same FA used to replace cement (Papadakis 2000).

The addition of FA as fine aggregates replacement also decreases the depth of wear during the abrasion resistance test (Fig. 4.16). The improvement of abrasion resistance of concrete is due to the increase in compressive strength due to the addition of FA (Siddique 2003b). Seo et al. (2010) observed similar drying shrinkage cracking of concrete when coal FA replaced part of cement or fine aggregates and slightly higher performance than that of conventional concrete. Maslehuddin et al. (1989) observed significantly higher chloride corrosion resistance of concrete with FA as fine aggregates than that of conventional concrete (Fig. 4.17). Hwang et al. (1998) reported that the addition of coal FA as aggregate in mortar improved the carbonation behaviour if the w/c ratio was properly maintained.

4.2.5 Other Coal Ash

Dhir et al. (2000) reported the use of pulverised fuel ash (PFA) as partial replacement of the sand fraction in concrete. The authors found lower slump for concrete with moist-cured PFA aggregates and the slump further decreased as the

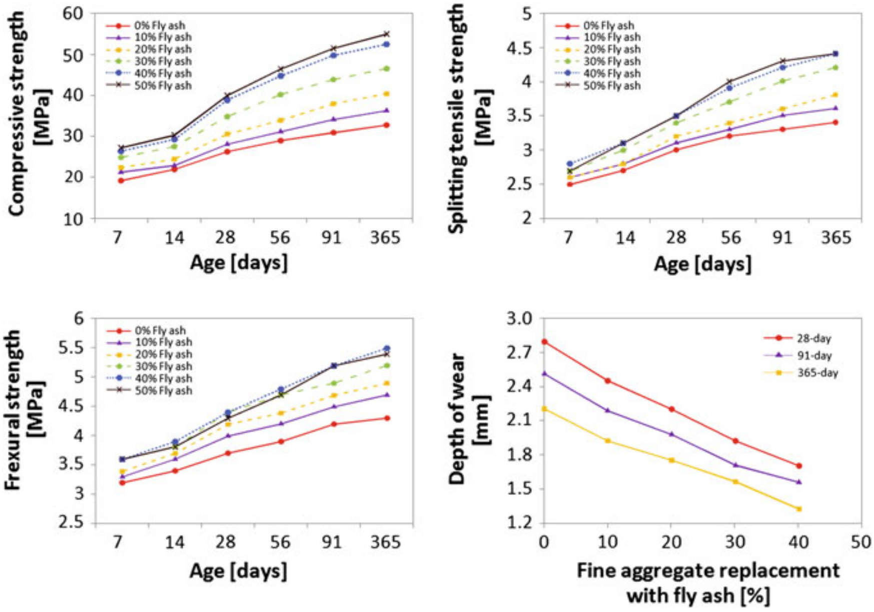


Fig. 4.16 Properties of concrete with fly ash aggregates (Siddique 2003a, b)

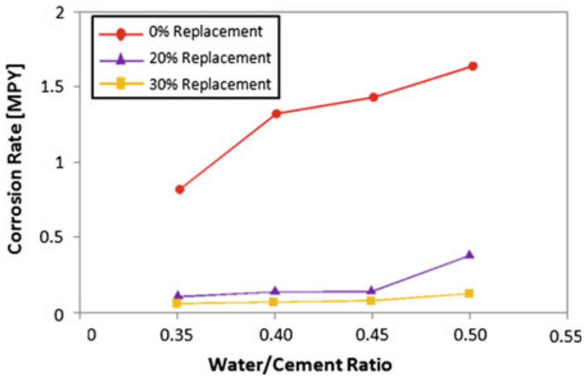


Fig. 4.17 Chloride corrosion rate of concrete with FA as partial replacement of fine aggregates (Maslehuddin et al. 1989)

content of PFA rose due to the increase of the fine content in the concrete mix. For dry PFA, the slump was not significantly affected at PFA/sand ratio of 0.05; but slump was reduced due to the addition of a higher amount of dry PFA. However, the authors observed similar cohesion and finishability of the concrete with PFA aggregates and the conventional concrete mix. To control the slump of the concrete mix with PFA, the authors suggested using a superplasticizer. They observed better bleeding performance of concrete mix with both dry and moist PFA aggregates than that of the conventional concrete mix.

The compressive strength of concrete with PFA aggregates was higher than that of the conventional concrete and differences increased as the PFA content increased as well as the curing age. The authors observed a particle size effect of PFA on the compressive strength of PFA-based concrete; the compressive strength of concrete with fine PFA was higher than that observed for concrete with coarse PFA. According to the authors, the higher strength was due to the pozzolanic effect of PFA.

Nataraja et al. (2007) reported the use of a burnt coal cinder as full substitution of NA in concrete. The authors found a substantial reduction in compressive strength of concrete due to the addition of coal cinder. The strength of concrete was 27.6 MPa for 7 days for burnt coal cinder whereas with crushed NA it was 35.5 MPa. At 28 days of curing, the strength of burnt coal cinder concrete increased to 38.5 MPa with a corresponding value for crushed aggregate concrete of 55.0 MPa. Compared to natural coarse aggregate, burnt coal cinder had a low crushing value and therefore the failure of concrete with burnt coal cinder occurred mostly due to aggregate crushing which decreased the compressive strength.

4.3 Steel Slag

Several types of steel slag are generated in the steel making process. The generation of these slags and their properties are already presented in Chap. 2. The presence of free calcium and magnesium oxides in some types of slag restrict their use as an aggregate in concrete. In a recent study, it was concluded that the use of electric arc furnace (EAF) slag as coarse and fine aggregates in concrete preparation can be considered while ladle furnace slag cannot be used for this purpose due to controversial results obtained after durability tests (Polanco et al. 2011). Several studies also reported the presence of high amounts of free lime and magnesium oxide in basic oxygen furnace steel slag. In this section, the properties of concrete with EAF-slag or steel slag, which do not exert too deleterious effects on the durability performance of concrete, will be presented. Significant information is available on the behaviour of conventional as well as HSC with steel slag as aggregates, and it will be discussed systematically in this section.

4.3.1 Fresh Concrete Properties

Changes in workability behaviour (slump) of fresh concrete mixes with slag aggregates were observed due to the large variation in aggregate properties such as water absorption capacity, size and shape and surface texture of various steel slags that were reported in various studies.

A concrete mix with a small amount of EAF-slag had a similar slump value to the one of a conventional concrete; however, increasing the addition of EAF-slag significantly reduces slump (Etxeberria et al. 2010). Manso et al. (2004) reported that concrete mixes with EAF-slag as the only fine and coarse aggregates lacked cohesion, and therefore collapsed during mixing. The complete substitution of coarse aggregate EAF aggregates of similar size and the substitution of the 0–4 mm fraction of NA by a 1:1 mixture of EAF-slag and limestone filler with particle size <1 μm in the concrete mix can eliminate this problem. Qasrawi et al. (2009) also observed marginal reduction in slump for concrete mixes with steel slag replacing up to 50 % by weight of fine NA and concrete can be classified as having moderate slump. However, concrete with 100 % slag was sticky with slump almost nil. The increase in the fine content and angular particle content of the concrete mix due to the addition of slag as well as the slightly higher water absorption capacity of slag by comparison with that of natural sand were the causes of the observed slump loss. On the contrary, Al-Negheimish et al. (1997) did not observe any significant difference between the slump of concrete with steel slag as coarse aggregates and conventional concrete at equal w/c value.

The bulk density of the majority of steel slags is significantly higher than that of NA, and therefore the dry density of concrete with steel slag is generally higher than that of conventional concrete. According to Papayianni and Anastasiou (2010), heavyweight concrete with a density of 2750 kg/m^3 could be produced by using EAF-slag. Masleduddin et al. (2003) reported that the density of a fresh concrete mix with EAF-slag with a 3.51 specific gravity replacing 45–65 % by weight of crushed limestone aggregates were in the range of 2436–2769 kg/m^3, whereas the density of concrete with crushed limestone aggregates with a 2.54 specific gravity was 2330 kg/m^3. Al-Negheimish et al. (1997) observed a similar increase in density due to the replacement of natural coarse aggregate by steel slag. However, Qasrawi et al. (2009) observed a very slight increase (<5 %) in the density of concrete with steel slag, used to replace up to 50 % by weight of fine NA, and the resulting concrete was reported to be normal weight according to ASTM specifications.

4.3.2 Hardened Concrete Properties

4.3.2.1 Compressive Strength

In several references, it was reported that the compressive strength of concrete with EAF-slag as coarse and fine aggregates replacement was similar to or even higher than that of conventional aggregate. However, contrasting results are also available on compressive strength behaviour due to use of EAF-slag as aggregates in concrete.

Al-Negheimish et al. (1997) observed similar compressive strength behaviours in concrete with coarse conventional and with coarse EAF-slag aggregates as curing time increased and for three different curing conditions, namely moist curing at 21 °C, curing at 28 °C with 45 % humidity and curing at 55 °C and 10 %

humidity. However, curing conditions had a significant effect on the compressive strength of steel slag concrete. Standard moist curing of this type of concrete exhibited the highest compressive strength, followed by moderate and high temperature curing. Moreover, at the latter stages of curing, deterioration of compressive strength was observed in moist-cured samples possibly due to the formation of expansive products.

Pellegrino and Gaddo (2009) observed higher compressive strength in concrete with EAF-slag than in conventional concrete after 7, 28 and 74 days of curing. However, in this study, significantly higher amounts of fluidifying agent and slightly higher amounts of aerating agent were used during preparation of concrete with EAF-slag aggregate than those used in conventional concrete preparation. The compressive strength of conventional concrete stabilized after 28 days of curing while the compressive strength of concrete with EAF-slag increased with curing time up to 74 days.

The 28-day compressive strengths of conventional concrete and concrete with unprocessed steel slag in the Maslehuddin et al. (2003) study were 39.7 and 41.6 MPa, respectively. These concrete mixes were prepared using a similar composition with coarse aggregate to total aggregate ratio of 0.60. The 28-day compressive strength of concrete with slag aggregates with coarse aggregate to total aggregate ratios of 0.45, 0.50, 0.55 and 0.65 were, respectively, 31.4, 37.7, 37.6 and 42.7 MPa. The authors also concluded that a coarse aggregate to total aggregate proportion of 50 % may be adopted to minimise the weight effect of heavy steel slag aggregates.

Almusallam et al. (2004) and Beshr et al. (2003) compared the compressive strength of concrete with steel slag coarse aggregates to that of concrete with three types of limestone aggregates. Concrete was prepared with a w/c ratio of 0.35 and slump of 50–75 mm using a superplasticizer so that the compressive strength performance can be related with the mechanical properties of the aggregates. After 28 days of curing, the compressive strength of concrete specimens prepared with calcareous, dolomitic, and quartzitic limestone and steel slag aggregates were 43, 45, 47 and 54 MPa, respectively. According to the authors, for HSC the bulk of the compressive load is borne by the aggregate rather than the cement paste alone and therefore failure occurs through the aggregate. Thus, the compressive strength of HSC depends on the mechanical prosperities of the coarse aggregates. Since the steel slag aggregate had better mechanical properties than the other aggregates, the incorporation of steel slag in concrete improved its compressive strength.

Papayianni and Anastasiou (2010) determined a 28-day compressive strength of 64.2 and 70.3 MPa for HSC with crushed limestone aggregate (reference concrete) and concrete with coarse EAF aggregates, respectively. The compressive strength of concrete with EAF-slag as fine and coarse aggregates was 77.9 MPa and it was about 21.3 % higher than that of the reference concrete. The authors also observed a higher rate of strength gain for concrete with slag aggregates during the initial periods (0–7 days) of curing (89.2–92.2 % of 28-day strength) than that observed for the reference concrete (81.8 % of 28-day strength).

Etxeberria et al. (2010) observed lower compressive strength for concrete with EAF-slag as the only coarse aggregates than that of a conventional concrete due to a higher effective w/c value (0.69) than that of the conventional concrete (0.65). However, in comparison with conventional concrete, the authors observed slightly higher and almost equal compressive strength of concrete mixes prepared by replacing, respectively, 25 and 50 % by volume of coarse nature aggregates by slag. On the other hand, the compressive strength of concrete prepared at lower w/c value increased with higher content of EAF-slag used to replace 0, 25, 50 and 100 % by volume of coarse NA (w/c equal to 0.57, 0.58, 0.59, and 0.60, respectively).

Manso et al. (2006) observed low compressive strength of concrete with EAF-slag as fine and coarse aggregates, due to its very poor workability behaviour. But the compressive strength of concrete with EAF-slag was comparable to that of conventional concrete at latter stages of curing (6 months and 1 year) when the fine and coarse NA in concrete were replaced according to the following methods: (1) complete replacement of coarse NA by similar size fractions of EAF-slag; (2) complete replacement of coarse NA by similar size fractions of EAF-slag along with the replacement of an equal amount of fine limestone aggregates by EAF-slag fine aggregates. In the case of the second method, the grain size of limestone aggregates was below 1 mm and therefore they act as a filler material.

Qasrawi et al. (2009) observed higher compressive strength for three different types of concrete (with design cube strength of 25, 35 and 45 MPa) prepared by replacing 15 and 30 % by weight of fine aggregates by steel slag than that for conventional concrete. However, at the replacement ratios of 50 and 100 % by weight, the compressive strength for all concrete types with slag aggregate were lower than for conventional concrete. The increase in compressive strength of concrete with EAF-slag up to a certain replacement level was due to the higher angularity of steel slag aggregates compared to NA, which therefore increased the binding between cement paste and aggregates. However, for higher slag incorporation levels, the percentage of the 0.15 mm aggregates fraction in concrete increased due to the higher content of this fraction in slag (about 40 % of total content). Thus, less cement was available to coat the slag particles and therefore the paste–aggregate bonding decreased, which ultimately reduced the compressive strength.

4.3.2.2 Splitting Tensile Strength

Several authors reported that the incorporation of steel slag as aggregates in concrete increases the splitting tensile strength just like it does the compressive strength as discussed in the previous section. However, results are also available where improvements of compressive strength but deterioration of splitting tensile strength was observed.

Al-Negheimish et al. (1997) observed higher 28-day splitting strength for concrete with steel slag coarse aggregate than for conventional concrete at three different curing conditions. This difference was more significant for curing conditions with moderate and high temperatures along with dry environment than that for normal moist curing conditions. Almusallam et al. (2004) and Beshr et al. (2003) also observed higher splitting tensile strength for concrete with steel slag aggregates than that for three other types of concretes using calcareous, dolomitic and quartzitic coarse aggregates when the authors investigated the behaviour of coarse aggregate type on the mechanical performance of concrete. Papayianni and Anastasiou (2010) observed 28-day splitting strength of 5.20, 5.52 and 5.89 MPa for HSC with crushed limestone aggregates (reference concrete), concrete with coarse EAF-slag aggregates and concrete with EAF-slag as fine and coarse aggregates, respectively. Pellegrino and Gado (2009) observed higher splitting tensile strength for concrete with steel slag with 2–22.4 mm size range as aggregates than for concrete with NA.

Etxeberria et al. (2010) found lower splitting tensile strength for concrete with various amounts of steel slag as coarse and fine aggregates than for conventional concrete with effective w/c ratios of 0.55 and 0.50. However, in the same study, higher compressive strength for concrete with steel slag than for conventional concrete with w/c ratio of 0.50 was reported. A slight improvement of compressive strength while a slight deterioration of splitting tensile strength due to the addition of steel slag aggregates in concrete was also observed in the Maslehuddin et al. (2003) study. The author obtained compressive strength of 41.6 MPa and splitting tensile strength of 6.26 MPa for concrete with steel slag aggregates and compressive strength of 39.7 MPa and splitting tensile strength of 6.33 MPa for conventional concrete.

4.3.2.3 Flexural Strength

Al-Negheimish et al. (1997) reported a slightly higher 28-day flexural strength for concrete with steel slag as coarse aggregates than that for conventional concrete for various curing conditions. The authors also observed significant effect on the flexural behaviour of concrete with steel slag aggregates due to changes in curing conditions. Papayianni and Anastasiou (2010) found 28-day flexural strength of 8.30, 9.13 and 9.96 MPa, respectively, for HSC with crushed limestone aggregates (reference concrete), concrete with coarse EAF-slag aggregates and concrete with EAF-slag as partial replacement of fine NA and complete replacement of natural coarse aggregates. Maslehuddin et al. (2003) observed lower flexural strength for concrete with coarse steel slag aggregates that added up to 60 % of total aggregates than that for conventional concrete with coarse crushed limestone aggregates. However, a coarse steel slag content of 65 % of total aggregates in concrete gave a higher flexural value (4.21 MPa) than in concrete with coarse limestone aggregates (3.96 MPa).

Qasrawi et al. (2009) observed an increase in flexural strength in concrete with increasing replacement of natural sand by fine steel slag aggregates up to a 50 % ratio by weight. However, replacement of 100 % sand by slag was not beneficial when compared to the other replacement levels at the ages of 28 and 90 days. For all replacement ratios, concrete with slag aggregates exhibited higher flexural tensile strength than conventional concrete. This can be attributed to the better mechanical properties of steel slag in addition to its higher angularity by comparison with NA, which increased the bond between aggregates and paste.

4.3.2.4 Young's Modulus of Elasticity

Al-Negheimish et al. (1997) observed a significantly higher 28-day Young's modulus of elasticity for concrete with slag as coarse aggregates than for conventional concrete with natural gravel as coarse aggregates. In this study, the 28-day Young's modulus of elasticity for concrete with slag and conventional concrete was 34.3 and 27.9 GPa, respectively. This higher modulus of elasticity was caused by the increase in concrete weight associated with the higher bulk density and modulus of elasticity of slag aggregate in comparison with NA. Beshr et al. (2003) and Almusallam et al. (2004) observed higher Young's modulus of elasticity for concrete with steel slag as coarse aggregates when a comparison was made between this modified concrete and concrete with three types of limestone aggregates. The authors determined Young's modulus of elasticity values of 29.6, 21.6, 24.4 and 28.8 GPa for concrete with steel slag, calcareous limestone, dolomitic limestone and quartzitic limestone as a coarse aggregates, respectively. This was due to the better mechanical properties of steel slag aggregates than those of the other aggregates. The addition of silica fume to cement further increases the Young's modulus and for 15 % replacement level of Portland cement this value was 40.4 GPa for concrete with steel slag aggregates (Almusallam et al. 2004). According to the authors, the type of aggregates had a more significant effect on the modulus of elasticity than on the compressive strength of concrete. Etxeberria et al. (2010) observed similar modulus of elasticity for concrete with EAF-slag and conventional concrete, particularly at lower water to cement ratios. Two types of concrete were prepared by varying the amount of cement and w/c ratio (w/c values of 0.5 and 0.55). The EAF-slag was used to replace 25, 50 and 100 % by volume of coarse NA. Their results are presented in Table 4.5.

Table 4.5 Modulus of elasticity of concrete with EAF-slag aggregates (Etxeberria et al. 2010)

Concrete type	Amount of EAF-slag to replace coarse aggregate (in volume) (%)	Young's modulus of elasticity (GPa)	
		w/c = 0.50	w/c = 0.55
Ref	0	36.4	30.1
EAF25	25	35.6	30.3
EAF50	50	36.2	26.3
EAF100	100	36.2	23.5

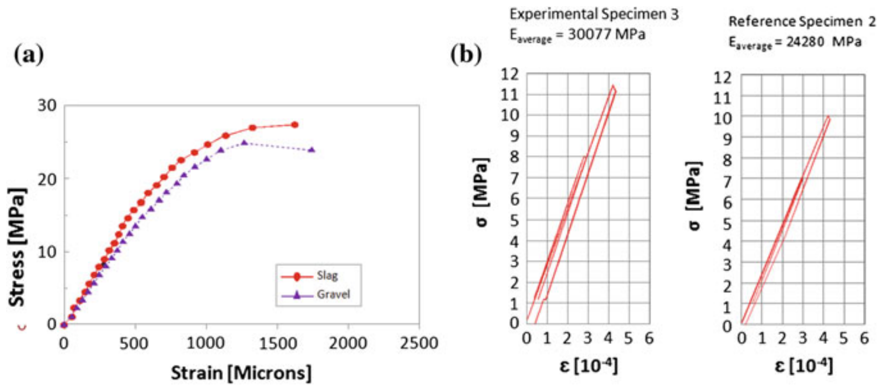


Fig. 4.18 Stress–strain curve of conventional concrete and concrete with steel slag as coarse aggregates. **a** Al-Negheimish et al. 1997; **b** Pellegrino and Gaddo 2009

Al-Negheimish et al. (1997) also observed an increase in the stiffness of concrete, which was clear in the plotted stress–strain curve (Fig. 4.18a). Pallegrino and Gaddo (2009) also plotted a stress–strain curve to determine the Young's modulus of elasticity (Fig. 4.18b). The value calculated from this graph for concrete with EAF-slag and conventional concrete was 30.7 and 24.1 GPa, respectively. The higher stiffness of EAF-slag concrete compared to conventional concrete was due to the higher density and roughness of steel slag aggregates compared to NA.

4.3.2.5 Abrasion Behaviour

Few data are available on the abrasion behaviour of concrete with steel slag aggregates. Papayianni and Anastasiou (2010) reported that concrete made with EAF-slag as coarse aggregates improved the abrasion resistance compared to reference concrete by 73.9 %. This improvement can be further improved to 77.4 % if the concrete is prepared by partially replacing fine aggregates and using EAF-slag as coarse aggregates.

4.3.3 Durability Properties

As steel slag contains some deleterious components like free lime and magnesia (periclase), the expansion of concrete with steel slag at various experimental conditions was reported to evaluate the effect of these components on the resulting concrete. In this section, those properties along with others will be discussed.

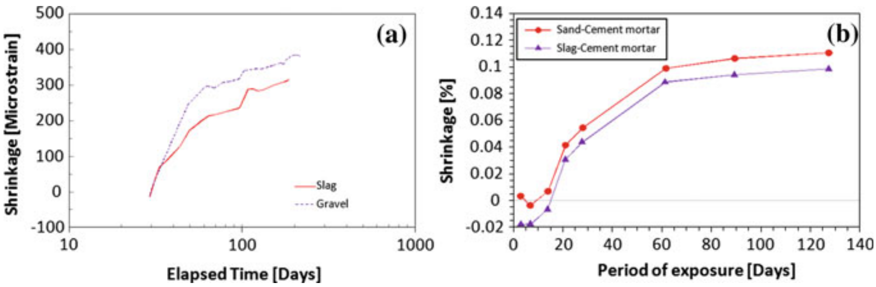


Fig. 4.19 Drying shrinkage of gravel and slag concrete. **a** Al-Negheimish et al. 1997; **b** Maslehuddin et al. 2003

4.3.3.1 Drying Shrinkage

Al-Negheimish et al. (1997) found lower drying shrinkage for concrete with steel slag aggregates than for natural gravel concrete due to their angular particle shape and honeycomb surface texture in comparison to irregular shaped and smooth surface textured gravel aggregates (Fig. 4.19a). The higher modulus of elasticity of slag aggregates by comparison with gravel aggregates was also responsible for the low shrinkage of concrete with slag aggregates. Masleduddin et al. (2003) also observed lower shrinkage for cement mortar with slag aggregates than for normal cement mortar (Fig. 4.19b). After 120 days of curing at 25 °C and 50 % room humidity, the shrinkage of slag and normal cement mortars was 0.097 and 0.11 %, respectively. The incorporation of steel mill scale into mortar as fine aggregates also lowers its drying shrinkage (Al-Otaibi 2008).

4.3.3.2 Expansion of Concrete with Steel Slag Aggregates

Normally, steel slag aggregates contain potentially expansive oxides like free lime and magnesium oxide (periclase); therefore, several tests are performed at normal moist as well as accelerated curing conditions to evaluate the expansion behaviour of concrete with steel slag aggregates. That information will be discussed here.

Lee and Lee (2009) reported the formation of pop-outs of concrete with EAF-slag as fine aggregates. Combined thermogravimetric and EDX analyses of the materials in the pop-out portion revealed the formation of expansive $Ca(OH)_2$ and $Mg(OH)_2$ due to the hydration of free CaO and MgO, present in EAF-slag aggregates.

Length Change Due to Moist Curing

Etxeberria et al. (2010) observed similar length change behaviour for concrete with various percentages of slag and conventional aggregates after submersing

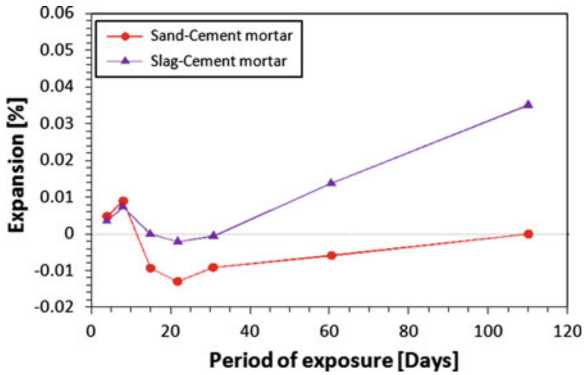


Fig. 4.20 Expansion of cement mortar exposed to moist environment (Maslehuddin et al. 2003)

them in water from 12 to 56 weeks. The specimens suffer a slight length change after 12 weeks of curing and then it almost stabilizes up to 56 weeks of curing. On the other hand, Maslehuddin et al. (2003) observed slightly higher length change for cement mortar with steel slag aggregates than for mortar with NA when both types of mortars were exposed to moist environment for 4 months. The length change observed for concrete with steel slag aggregates was 0.034 %, which was lower than the ASTM C33 prescribed limit of 0.05 %. Their results are presented in Fig. 4.20. The observed expansion of concrete due to the incorporation of steel slag aggregates was due to calcium carbonate, present in steel slag, which expanded on absorption of water.

Effect on Accelerated Ageing

Pellegrino and Gaddo (2009) used three kinds of accelerated ageing conditions to evaluate the effect of deleterious free lime and periclase, present in steel slag aggregates, on the expansion behaviour of the resulting concrete. They initially used the accelerated ageing method described in the ASTM D 4792 standard and found a reduction in strength of about 5 % for concrete with EAF-slag aggregates and an increase of 9 % for conventional concrete. The surface of the concrete with EAF-slag aggregates also exhibits higher efflorescence than conventional concrete due to the formation of white powder of calcium and magnesium hydroxides. However, ageing of concrete specimens obtained after the test by 3-month moist curing at room temperature improves their strength. Their results are presented in Table 4.6. Manso et al. (2006), on the other hand, did not observe any significant differences in compressive strength of conventional concrete as well as of concrete with steel slag aggregates after using the method described in ASTM D 4792 standard followed by 90 days of moist curing (Table 4.6).

Manso et al. (2006) also reported the results of a vigorous accelerating test where conventional concrete and concrete with EAF-slag replacing various percentages of fine and coarse NA were initially subjected to autoclave test followed by 90 days of weathering. Their results indicated that concrete with limestone

Table 4.6 Compressive strength behaviour of concrete after accelerated ageing

Concrete type	Compressive strength (MPa)		
	Before ageing	After 32 days curing at 70 °C	After curing at 70 °C for 32 days plus moist curing for 90 day
Pellegrino and Gaddo (2009)			
Control	30.4	33.1	32.9
EAF-slag	44.4	41.9	43.4
Manso et al. (2006)			
Control	38.5		39.6
EAF-slag-1	33.7		35.9
EAF-slag-2	35.3		39.4
EAF-slag-3	30.2		33.5
EAF-slag-4	30.7		34.1

Table 4.7 Compressive strength before and after autoclave ageing followed by weathering (Manso et al. 2006)

Concrete type	Compressive strength (MPa)		
	Before ageing	After ageing	Appearance
Control	38.5	18.4	Superficial cracking
EAF-slag-1	33.7	20.9	Slight superficial cracking
EAF-slag-2	35.3	23.8	Slight superficial cracking

aggregates exhibited poorer compressive strength than concrete with EAF-slag aggregates due to the difference in shape of these two aggregates. Their results are presented in Table 4.7.

Manso et al. (2004) evaluated the soundness of cement mortar with EAF-slag as partial substitution of fine aggregates in concrete according to ASTM C1012, in which cement mortar were subjected to ten cycles of repeated immersion in a saturated Na_2SO_4 solution followed by drying in an oven. They observed larger deterioration in mortar with EAF-slag than in conventional mortar.

4.3.3.3 Freeze–Thaw Resistance

Manso et al. (2006) reported that concrete with steel slag aggregates exhibited poorer performance than conventional concrete after 25 cycles of freezing and thawing. Their results are presented in Table 4.8. Out of four concrete mixes with steel slag aggregates, the authors observed better performance for concrete with EAF-slag-2, which exhibited slightly higher compressive strength as well as lower porosity. The addition of an air-entraining admixture improved the freeze–thaw resistance of concrete with steel slag aggregates. Pellegrino and Gaddo (2009) observed about 7 % reduction in compressive strength for concrete with EAF-slag,

Table 4.8 Compressive strength behaviour of concrete after freeze–thaw cycles

Concrete type[a]	Compressive strength (MPa)		Strength change (%) (strength gain: +) (strength loss: −)	Appearance
	Before ageing	After freezing and thawing		
Manso et al. (2006)				
Control (13)	38.5	32.7	−15	Good
EAF-slag-1 (16.2)	33.7	20.6	−39	Significant damage
EAF-slag-2 (16.0)	35.3	27.2	−23	Slight damage
EAF-slag-3 (17.6)	30.2	16.9	−44	One sample cracked
EAF-slag-4 (19.6)	30.7	16.0	−48	Significant damage
Pellegrino and Gaddo (2009)				
Control	30.4	33.9	+11.5	
EAF-slag	44.4	41.2	−7.3	

[a] Data in parenthesis indicates the porosity of concrete in percentage

when the concrete specimens were subjected to repeated freeze–thaw cycles for 25 days. Their results are also presented in Table 4.8. The lesser reduction in compressive strength in comparison to Manso et al.s' study (2006) was due to the incorporation of an air-entrainment agent, which caused the formation of closed pores in the specimens and therefore concrete specimens gained resistance against freezing and the thermal/expansive stress decreased.

4.3.3.4 Resistance Against Wet–Dry Cycles

Pellegrino and Gaddo (2009) observed a reduction of about 26.5 % in compressive strength of concrete with EAF-slag aggregates in comparison to a reduction of 7.7 % for conventional concrete when both types of concrete specimens, after 28 days normal curing, were subjected to 30 cycles of repeated 16-h moist curing followed by 8-h oven drying at 110 °C. The presence of free calcium and magnesium oxides in EAF-slag favours the more serious degradation of the resulting concrete than that observed in the conventional concrete. Manso et al. (2006) also observed significantly higher strength reduction in concrete with EAF-slag aggregates (except for one slag concrete composition) than in the control concrete when the four types of concrete mixes along with a control mix were subjected to a similar type of wet–dry cycles to that performed by Pellegrino and Gaddo (2009). These results are presented in Table 4.9.

Maslehuddin et al. (2003) observed a reduction of 3–7 % in compressive strength of concrete with limestone and EAF-slag as coarse aggregates, when the 28-day hardened concrete specimens were exposed to 120 cycles of mild thermal cycles. The concrete specimens were exposed 8 h at 70 °C followed by 16 h at

Table 4.9 Compressive strength behaviour of concrete after wetting and drying cycle

Concrete type	Compressive strength (MPa)		Strength loss (%)	Appearance
	Before ageing	After freezing- thawing		
Manso et al. (2006)				
Control	38.5	27.3	29	Good
EAF-slag-1	33.7	19.9	41	Slight damage
EAF-slag-2	35.3	24.7	30	Good
EAF-slag-3	30.2	16.6	45	Slight damage
EAF-slag-4	30.7	15.6	49	One sample cracked
Pellegrino and Gaddo (2009)				
Control	30.4	28.7	5.60	
EAF-slag	44.4	32.7	26.52	

25 °C to complete one thermal cycle. However, a reduction in pulse velocity and an increase in water absorption after the completion of thermal cycles indicated better performance of concrete with steel slag aggregates than of the limestone aggregates concrete due to the denser microstructure of the former concrete compared with the latter.

4.3.3.5 Other Durability Behaviour

Manso et al. (2006) investigated the alkali-aggregate reaction of EAF-slag to be used as aggregates in concrete by using the ASTM C1260 method. According to the authors, slag contains a significant amount of glassy phase, which can react with alkalis present in cement. The average value of expansion was 0.14 % after 16 days and 0.15 % after 28 days, both well below the specified value of 0.2 %. However, the presence of free CaO and MgO in EAF-slag overestimate the expansion value.

Maslehuddin et al. (2003) detected a better performance in concrete with steel slag aggregates than in limestone concrete when both were subjected to chloride induced corrosion. The time to initiation of reinforcement corrosion and time to cracking of concrete specimens were, respectively, 190 and 517 h for conventional concrete and 198–367 h and 509–774 h for steel slag aggregates concrete. This was mainly due to a denser microstructure in steel slag aggregates concrete than in NA concrete. Lower water absorption capacity and higher ultrasonic pulse velocity in concrete with steel slag aggregates than in NA concrete were also observed.

4.4 Blast Furnace Slag

The use of BFS as aggregates in concrete is not as common as its use as a component in cement. However, some recent reports indicate that this material (particularly air-cooled BFS) can be used as an aggregate in concrete preparation.

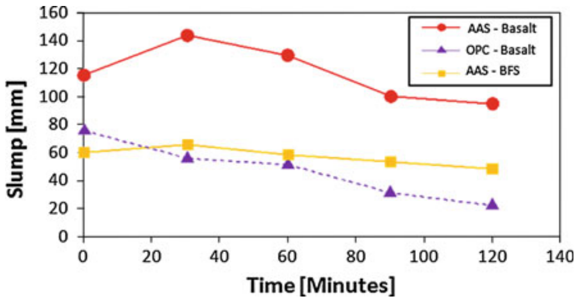


Fig. 4.21 Comparison of slump of AAS concrete with BFS and basalt as coarse aggregates and conventional concrete with basalt coarse aggregates (Collins and Sanjayan 1999)

4.4.1 Fresh Concrete Properties

There are vast differences in results presented in various references on the slump behaviour of concrete mix due to the addition of BFS aggregates. Etxeberria et al. (2010) observed a slight increase in slump when 25 % by volume of coarse NA were replaced by BFS aggregates. The mix was workable just like conventional concrete. On the other hand, replacing 50 % by volume of NA by BFS aggregates considerably reduced the slump of the resulting concrete. However, slump increased again when natural coarse aggregates were completely replaced by BFS aggregates. Collins and Sanjayan (1999) observed a slump of 65 mm for alkali-activated slag (AAS) concrete with BFS coarse aggregates and a slump of 115 mm for a similar concrete with basalt coarse aggregates. In this study, the BFS aggregates were presaturated with water before being used as aggregates due to their higher water absorption capacity (4.4 %) in comparison to that of basalt aggregates (1.2 %) (Fig. 4.21). The observed low slump of BFS aggregates was due to the differences in surface texture, shape and porosity from basalt aggregates. However, in some studies, no significant difference was found in terms of slump or workability between conventional concrete and concrete with BFS as coarse aggregates (Demirboga and Gul 2006; Haque et al. 1995). The incorporation of BFS as fine aggregates replacement also decreases the slump of the resulting concrete and increasing their content further decreases it (Yüksel et al. 2011).

The density of concrete with BFS aggregates depends on the bulk density of BFS aggregates. Demirboga and Gul (2006) observed an increase in fresh density of about 7.9–8.5 % in HSC with different w/c values due to BFS-aggregates incorporation. The bulk density of BFS aggregates was equal to 2.78 g/cm^3 and higher than that of natural coarse aggregates. Etxeberria et al. (2010) reported lower dry density for concrete with BFS aggregates than for conventional concrete. The dry density was further decreased as the content of BFS aggregates increased due to the lower bulk density of BFS aggregates (2.36 g/cm^3) than that NA (2.56 g/cm^3). The air content of alkali-activated concrete with BFS coarse aggregates and basalt coarse aggregates were 1.6 and 1.2 %, respectively (Collins and Sanjayan 1999).

4.4.2 Hardened Concrete Properties

Etxeberria et al. (2010) observed contrasting compressive strength behaviour, when concrete mixes were prepared by replacing 0, 25, 50 and 100 % by volume of natural coarse aggregates by BFS aggregates at low and high w/c ratios. At high w/c, the compressive strength for concrete with BFS aggregates used to replace 25 and 100 % by volume of coarse aggregates was lower than that of the conventional concrete. But the compressive strength of concrete with BFS aggregates used to replace 50 % by volume of coarse aggregates was higher than that of the conventional concrete. On the other hand, at low w/c, the compressive strength of concrete with BFS aggregates was higher than that of the conventional concrete and strength increased with the content of slag aggregates in concrete. In both cases, the w/c value of conventional concrete was lower than those of the BFS-aggregates concrete mixes.

Demirboga and Gul (2006) found higher compressive strength for HSC with BFS as coarse aggregates than for conventional concrete for various w/c values. However, the difference in strength between BFS-aggregates concrete and conventional concrete decreased as the w/c value increased (Fig. 4.22). Haque et al. (1995) reported that compressive strength of concrete with BFS aggregates could be as high as 107 MPa. Yüksel and Bilir (2007) observed similar compressive strength for concrete pavement blocks where 60–80 % by volume of fine aggregates were replaced by equal-size BFS aggregates and for a conventional pavement block. Yüksel et al. (2011) observed a decreasing trend of compressive strength with increasing substitution level of fine NA by BFS aggregates.

Collins and Sanjayan (1999) found higher 1-day compressive strength for AAS concrete with BFS as coarse aggregate than for coarse basalt aggregates (NA) AAS concrete when both were cured by immersion. However, the compressive strength of NA AAS at the later stages of immersion curing and sealed curing at all

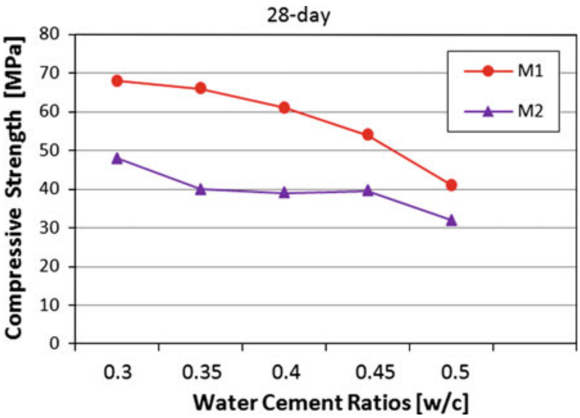


Fig. 4.22 Compressive strength of concrete with natural and BFS coarse aggregates (Demirboga and Gul 2006)

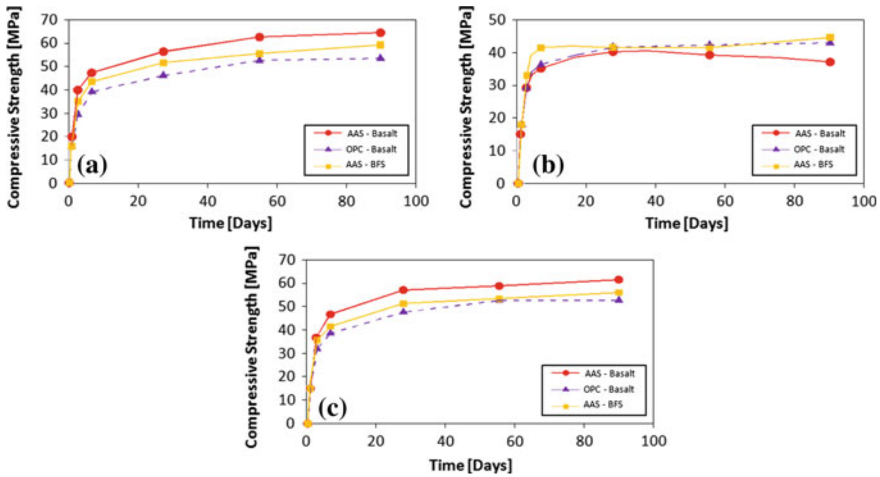


Fig. 4.23 Compressive strength of various types of concrete at three different curing conditions: **a** at 23 °C in immersion; **b** at 23 °C in 50 % humidity; **c** at 23 °C with sealed specimens (Collins and Sanjayan 1999)

curing period was higher than that of AAS with BFS aggregates. On the other hand, the compressive strength of AAS with BFS aggregates was higher than that of NA AAS at the whole curing period when both were cured at 23 °C with 50 % room humidity due to internal curing effect, where the moisture present in BFS aggregates came out at low humidity conditions (Fig. 4.23).

Etxeberria et al. (2010) observed lower modulus of elasticity and splitting tensile strength for concrete with BFS aggregates than for conventional concrete at two different ranges of w/c values. These values further decreased as the content of BFS aggregate in concrete increased. Lower modulus of elasticity was reported for BFS-aggregates concrete in comparison to conventional concrete in Haque et al.s' (1995) study too. On the other hand, in comparison to conventional concrete, about 8–10 % higher splitting tensile strength and higher elastic modulus were recorded for concrete with BFS coarse aggregates prepared at various w/c ratios.

Ashby (1996) observed similar elastic modulus of elasticity but marginally higher Poisson's ratio for concrete with air-cooled slag aggregates than for natural gravel aggregates concrete. The specific creep for grade 20 concrete with air-cooled aggregates was lesser but for grade 40 concrete it was similar to that of the natural gravel concrete.

Yüksel et al. (2007) observed higher abrasion resistance of concrete with BFS as partial replacement of fine aggregates. In this study, the fine aggregates were replaced by BFS aggregates up to a replacement level of 50 % (by weight). Maximum abrasion resistance was observed when 10 % of fine aggregates were replaced by BFS aggregates.

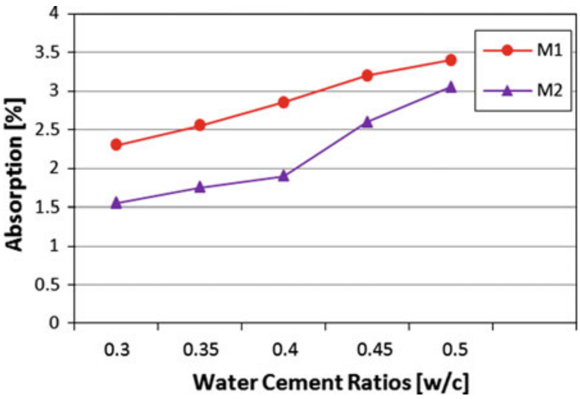


Fig. 4.24 Water absorption capacity of conventional concrete (M1) and concrete with BFS aggregates (M2) (Demirboga and Gul 2006)

4.4.3 Durability Performance

Etxeberria et al. (2010) observed almost identical length change behaviour for concrete with BFS coarse aggregates and conventional concrete when both were immersed in water for 12 and 56 weeks: all types of concrete specimens suffered a slight length change after 12 weeks of immersion and then it stabilized up to 56 weeks of immersion. However, in comparison to conventional concrete, BFS-aggregates concrete suffered lesser length change in the early 12-week period.

Etxeberria et al. (2010) observed lower capillary water absorption for AAS concrete with BFS aggregates than for conventional concrete. The lowest and highest absorptions were observed for mixes with 50 and 100 % by volume replacement of coarse aggregates by BFS aggregates, respectively. Demirboga and Gul (2006) also observed lower water absorption capacity for HSC with BFS as coarse aggregates than for conventional concrete at different w/c values (Fig. 4.24). On the other hand, Yüksel et al. (2007) reported higher capillary water absorption for concrete with BFS aggregates replacing 20–50 % by weight of fine aggregates than for conventional concrete; however, at 10 % replacement level the capillary water absorption of BFS concrete was lower than that of the conventional concrete due to pozzolanic reactions of some constituents of BFS aggregates with free lime. At high substitution level, concrete became porous due to porous aggregates addition and therefore increased capillary water absorption. Haque et al. (1995) observed lesser water absorption and water penetration for a high-performance concrete with air-cooled BFS aggregates than for conventional concrete.

Etxeberria et al. (2010) observed significantly higher residual strength for concrete with BFS aggregates than for conventional concrete when both were exposed to 800 °C for 4 h. The residual compressive strength for mixes where 25, 50 and 100 % by volume of NA were replaced by BFS aggregates was, respectively, 48, 53 and 51 % in comparison to 33 % for conventional concrete. The concrete with BFS as partial replacement of fine aggregates exhibited similar or slightly better compressive strength and dynamic elastic modulus behaviour than the conventional concrete when both were exposed to 800 °C (Yüksel et al. 2011).

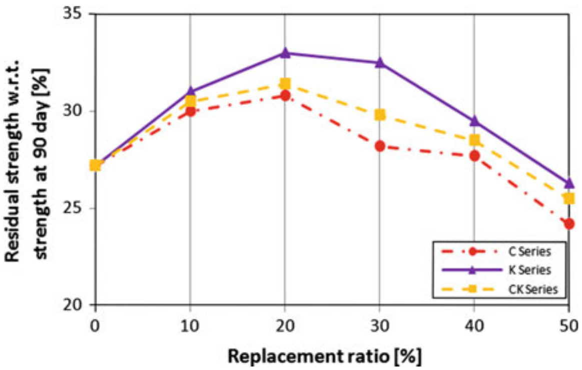


Fig. 4.25 Residual compressive strength of concrete after high temperature exposure of BFS-aggregates concrete (C-series) along with CBA aggregates concrete (K-series) (Yüksel et al. 2007)

Increasing addition of aggregates slightly improved these properties, which was more noticeable in the dynamic elasticity modulus results. In another work, Yüksel et al. (2007) observed higher residual strength for concrete where up to 40 % by weight of fine aggregates were replaced by BFS aggregates than for conventional concrete. In this study, the maximum strength was observed at 20 % replacement level. The residual strength was lower than that of the conventional concrete for 50 % replacement of fine aggregates by BFS aggregates, possibly due to changes in concrete microstructure and the generation of more porosity because of the substitution of fine aggregates by porous BFS aggregates. However, the residual strength of concrete with CBA as partial replacement of fine aggregates was better than that of the BFS-aggregates concrete (Fig. 4.25).

Yüksel et al. (2007) found higher freeze–thaw resistance for concrete with BFS as partial replacement of fine aggregates than for conventional concrete as well as CBA aggregates with concrete. The specimens were subjected to 50 cycles of repeated freezing and thawing. This resistance increased with the replacement ratio of fine aggregates by BFS aggregates and at 20 % replacement level it reached a maximum; further increasing the replacement level up to 50 % by weight decreased the resistance even though the performance was still better than that of conventional concrete. Their results are presented in Fig. 4.26.

Yüksel et al. (2007) observed an insignificant effect of wet–dry cycles on the strength loss of concrete with BFS fine aggregates as 0–50 % by weight replacement of NA, when the specimens were subjected to 25 cycles. Collins and Sanjayan (1999) observed lower drying shrinkage for AAS concrete with BFS as coarse aggregates than for concrete with basalt aggregates. The experiment was undertaken at 23 °C with 50 % room humidity. However, similar autogenous shrinkage in concrete with basalt and BFS aggregates indicated that the improvement of shrinkage because of BFS-aggregates addition was attributed to the internal curing caused by the moisture present in these aggregates. Ashby (1996) also observed lower drying shrinkages for concrete with air-cooled BFS aggregates than for conventional concrete with river gravel up to a period of 56 days. Haque et al. (1995) reported lower drying shrinkage for high-performance concrete with air-cooled BFS aggregates than for conventional concrete.

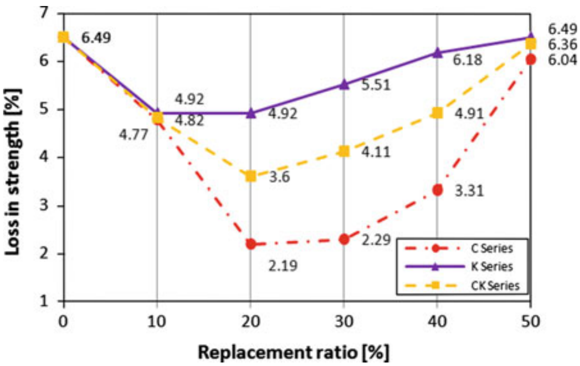


Fig. 4.26 Loss in strength after freeze–thaw testing: C-Series—BFS-aggregates concrete; K-series—CBA aggregates concrete (Yüksel et al. 2007)

4.5 Non-Ferrous Slag

4.5.1 Copper Slag as Aggregate in Concrete

Several reports are available on the use of copper slag as fine and coarse aggregates in concrete. Both normal and HSC are prepared with copper slag aggregates. In this section, normal and high-performance concrete properties will be discussed in the same section.

4.5.1.1 Fresh concrete properties

The incorporation of copper slag as a partial or full substitution of fine aggregate in concrete increases the slump value (Al-Jabri et al. 2011; Wu et al. 2010; Pezhani and Jeyaraj 2010). Al-Jabri et al. (2011) observed increased slump of concrete as the incorporation of copper slag as replacement of fine aggregates rose. The slump of conventional concrete and concrete with copper slag at 100 % fine aggregate level were, respectively, 65.5 and 200 mm. The improvement in slump was due to the presence of a greater amount of free water in slag concrete than in conventional concrete as the water absorption capacity of copper slag was lower than that of natural fine aggregates. However, segregation and bleeding was observed in fresh concrete mixes with high amount of copper slag, i.e. in this case concrete mixes prepared with 80 and 100 % of fine NA replaced by copper slag. Al-Jabri et al. (2009a, b) also observed a significant reduction in w/c ratio in HSC due to the replacement of fine aggregates by equal-size copper aggregates. The w/c value of conventional concrete and concrete with copper slag at 100 % fine aggregates replacement level were 0.35 and 0.27, respectively. The addition of copper slag as coarse aggregates also increased the slump of concrete (Khanjadi and Behnood 2009).

Increased bleeding was also reported in the Ishimaru study (2005) when the content of copper slag used to replace natural fine aggregates increased. Bleeding increased with copper slag incorporation due to the high bulk density, glass-like surface properties and irregular grain shape of copper slag (Shoya et al. 1997). Bleeding depends on several factors such as w/c ratio, air content and slag content in concrete. According to Shoya et al. (1997) 40 % copper slag can be used as partial replacement of aggregates to control the amount of bleeding to less than 5 l/m^2. The addition of an admixture with cellulose ether and powder-like material, such as limestone powder, is highly effective to improve the bleeding performance of concrete with copper slag as aggregates (Shoya et al. 1997). Hwang and Laiw (1989) obtained a concrete mix with satisfactory workability and minimal bleeding at the optimum fineness modulus of a mixture of copper slag and natural fine aggregates, which was roughly equal to 2.6.

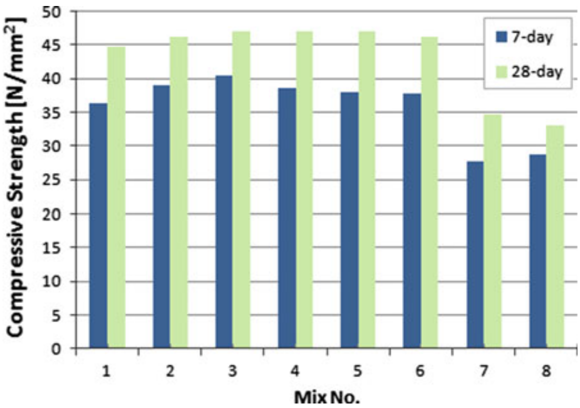
The addition of copper slag as fine or coarse aggregates in concrete increases its fresh density due to the higher bulk density of copper slag than that of NA (Al-Jabri et al. 2011; Khanjadi and Behnood 2009). Al-Jabri et al. (2011) observed an increase in the density of fresh concrete of about 5 % when fine aggregates were totally replaced by copper slag. Khanjadi and Behnood (2009) reported a density of 2310 and 2668 kg/m^3 for HSC mixes with limestone and copper slag as coarse aggregates, respectively. Khanjadi and Behnood (2009) also observed air-content values of 2.5 and 2.4 % for natural and copper slag concrete, respectively. The difference in air content between control concrete and copper slag concrete increases with addition of silica fume with cement.

4.5.1.2 Hardened Concrete Properties

Al-Jabri et al. (2011) observed an increasing trend of 28-day compressive strength of concrete as the content of copper slag rose up to a 40 % replacement level (mix No. 4 in Fig. 4.29) of fine aggregates. Their results are presented in Fig. 4.29. Further increment of the content of copper slag decreased the compressive strength of resulting concrete and at a 60 % replacement level (mix No. 6) it became slightly higher than the compressive strength of conventional concrete. The mix with 40 % copper slag content yielded the highest 28-day compressive strength of 47.1 N/mm^2 compared with 45 N/mm^2 for the control mix, whereas the lowest compressive strength of 34.8 N/mm^2 was obtained for the mix with 80 % copper slag (mix No. 7). This reduction in compressive strength for concrete mixes with high copper slag contents was due to the increase in free water content that resulted from the low water absorption characteristics of copper slag in comparison with sand, which caused a considerable increase in the workability of concrete and thus reduced the compressive strength as shown in Fig. 4.27.

In the Birindha and Nagam (2011) study, the 28-day compressive strength of concrete prepared by replacing 40 % of fine aggregates by copper slag aggregates was 46.7 MPa in comparison to the equivalent compressive strength of 35.1 MPa for the control concrete. However, at 60 % substitution level of fine aggregates by copper slag the compressive strength decreased to 39.7 MPa. Similar observations were reported in some other studies, where copper slag was used as fine aggregates

Fig. 4.27 Compressive strength of concrete with copper slag fine aggregates (Al-Jabri et al. 2011)



(Ayano and Sakata 2000; Caliskan and Behnood 2004; Hwang and Laiw 1989; Li 1999; Shoya et al. 1997; Zong 2003). Wu et al. (2010) observed slightly higher dynamic compressive strength of concrete with copper slag replacing 20 % of fine aggregates, which at 40 % replacement level became similar to that of the control concrete. The dynamic compressive strength continuously decreased when the replacement amount exceeded 40 %. According to the authors, the observed improvement at lower substitution level was due to the presence of angular sharp edged particles in copper slag as well as the improvement of cohesion between the cement paste and the aggregates. However, at higher copper slag contents the amount of free water increased due to the low water absorption capacity of copper slag, which increased bleeding, internal voids and capillary pores in concrete.

The 7- and 28-day compressive strength of HSC with copper slag up to 50 % replacement ratio (of natural sand) as fine aggregates were similar (or slightly better) than those of HSC with natural sand. The compressive strength was reduced significantly beyond that level of substitution, due to the separation of particles of the constituents and the formation of pores in concrete by excess free water present in concrete with copper slag aggregates (Fig. 4.28) (Al-Jabri et al. 2009a, b).

The compressive and splitting tensile strengths of HSC with copper slag as coarse aggregates prepared at constant w/c ratio are higher than those of HSC with limestone aggregates, due to the higher strength of copper slag aggregates compared to limestone aggregates and also the porous and rough surface texture of copper slag. This surface texture may produce a superior bond and transition zone in comparison with that of the limestone aggregates (Khanzadi and Behnood 2009). The incorporation of silica fume with cement can produce a stronger transition zone between the copper slag aggregates and the cement paste due to its pozzolanic reaction, and therefore increase the compressive strength (Khanzadi and Behnood 2009).

The tensile splitting strength and flexural strengths of HSC with copper slag behave similarly to compressive strength. However, by comparison with compressive strength a higher rate of development of splitting tensile strength was observed for HSC with copper slag coarse aggregates than for limestone

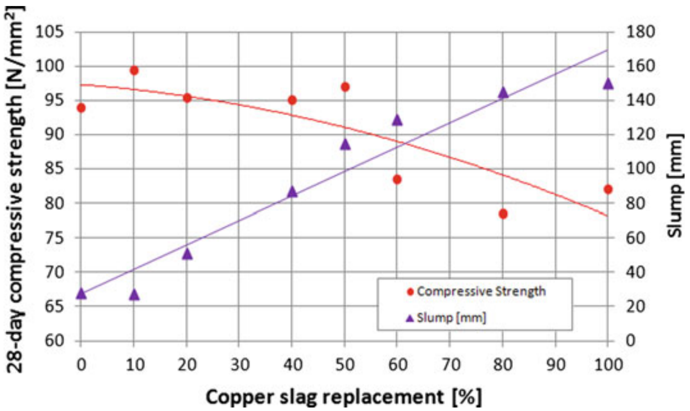


Fig. 4.28 Effect of fine copper slag aggregates addition on the compressive strength and slump of high-strength concrete (Al-Jabri et al. 2009b)

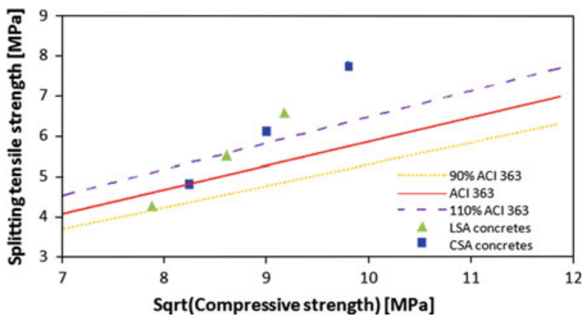
aggregates concrete (Fig. 4.29) (Khanjadi and Behnood 2009). Birindha and Nagam (2011) observed an increase in tensile strength of about 36.5 % when 40 % of natural fine aggregates were replaced by copper slag. However, the increase in percentage became 17 % at 60 % replacement level.

The abrasion resistance for cement mortar with copper slag aggregates is better than for conventional concrete (Tang et al. 2000). Khanjadi and Behnood (2009) also reported a rebound hammer value about 2.6 % higher for HSC with copper slag as coarse aggregates than for natural limestone aggregates concrete. The improvement of properties due to the addition of copper slag as aggregates is due to the higher hardness of copper slag aggregates in comparison with NA.

4.5.1.3 Durability Properties

Birindha et al. (2010) observed increasing ultrasonic pulse velocity of conventional concrete as the replacement of fine aggregates by copper slag aggregates rose. In this study, the copper slag aggregates replaced fine aggregates up to a 60 % level. The optimum value was observed for concrete where 40 % of fine

Fig. 4.29 Splitting tensile strength and compressive strength relationship according to ACI 363 (Khanzadi and Behnood 2009)



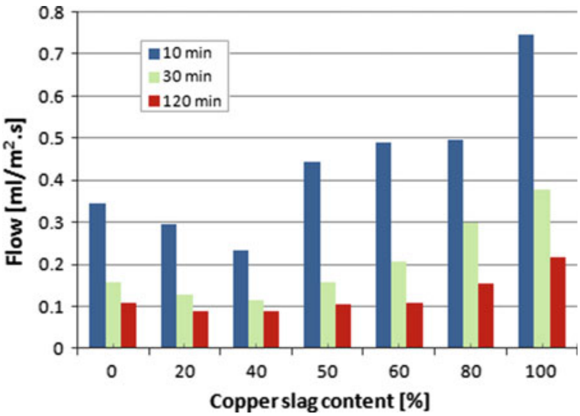


Fig. 4.30 Surface water absorption of HSC due to addition of copper slag as sand replacement (Al-Jabri et al. 2011)

aggregates were replaced, which indicates that the densest microstructure was observed at this replacement level. The drying shrinkage of concrete with copper slag as fine aggregates is similar or even less than that of specimens without copper slag (Ayano and Sakata 2000). Al-Jabri et al. (2011) observed a decreasing trend of water permeable voids in HSC with increasing replacement of sand by copper slag fine aggregates up to a 50 % replacement level; the voids increased again as the content of copper slag continued to rise. The same authors observed a decreasing trend of surface water absorption by HSC with increasing replacement of sand by copper slag aggregates up to a 40 % replacement level; however, after this substitution level, the surface absorption increased abruptly due to the presence of pores created by excessive free water (Fig. 4.30).

The freeze–thaw resistance of concrete with copper slag aggregates is lower than that of conventional concrete due to the internal defects originated by the upflow of bleeding water. However, the addition of an admixture and limestone powder improves this property (Shoya et al. 1997). Birindha et al. (2010) observed higher chloride corrosion rate of uncoated rebar in concrete with copper slag as partial replacement of fine aggregates than in control concrete. The corrosion rate increased with the slag content. But when the rebar was coated with zinc phosphate paint, no corrosion was observed in the corrosion period. The authors also observed higher penetration rate of chloride ions at 40 and 60 % replacement rates of fine aggregates by copper slag aggregates even though the amounts for all types of concrete were very low according to the ASTM C1202 specification. The sulphuric acid resistance capacity of concrete with copper slag aggregates was also observed to be low in comparison to control concrete and decreased as the content of copper slag in concrete increased (Fig. 4.31) (Birindha et al. 2010). The resistance to sulphate attack and the rate of carbonation of concrete with copper slag aggregates are similar to (or even better than) the ones of concrete with conventional aggregates (Ayano and Sakata 2000, Hwang and Laiw 1989).

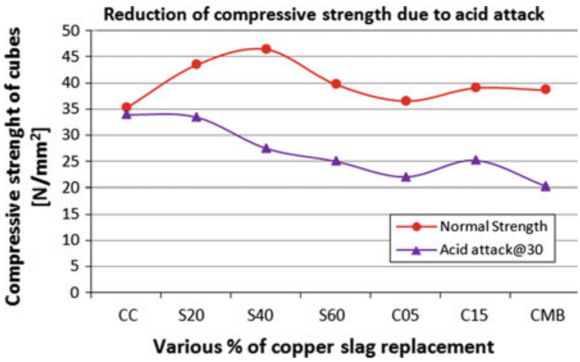


Fig. 4.31 Compressive strength resistance behaviour of concrete with copper slag due to 5% sulphuric acid attack (Brindha et al. 2010)

4.5.2 Other Non-Ferrous Slag

The use of several other non-ferrous industrial slags as aggregate in concrete is also reported. Here, some results will be highlighted.

Atzeni et al. (1996) observed no significant differences between the 30-day compressive strength of conventional concrete and concrete with two types of lead and zinc slag, used as partial replacement of sand. However, leaching of significant amounts of lead from concrete with slag is a big problem when using this material as aggregates in concrete, and therefore it needs to be addressed at disposal of demolished slag added concrete. Penpolcharoen (2005) reported the use of a slag, produced during processing of lead batteries, as partial and full replacement of coarse limestone aggregates and partial replacement of Portland cement. The partial and full replacement of limestone aggregates by lead slag increased compressive strength and rising slag addition further increased the strength due to the superior mechanical properties and better packing of slag over limestone aggregates, and the magnetic nature of slag (Fig. 4.32). On the other hand, the water absorption capacity of concrete with slag aggregates was higher than that of NA

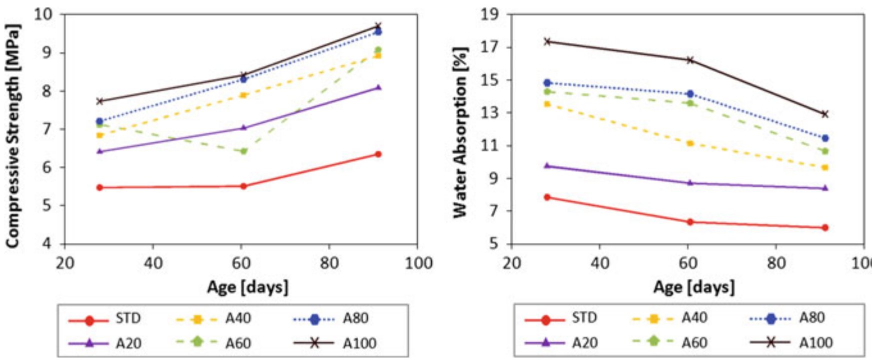


Fig. 4.32 Compressive strength and water absorption of concrete with lead slag (Penpolcharoen 2005)

concrete and rising slag addition further increased the water absorption due to the magnetic nature of slag aggregates (Fig. 4.32).

Metwally et al. (2005) reported the use of a slag produced during smelting of lead batteries as partial replacement of fine aggregates and full replacement of coarse aggregates. The slump of concrete with lead slag was lower than that of conventional concrete and increasing slag addition as replacement of fine aggregates further reduced slump. The highest compressive and tensile strengths of concrete with lead slag as fine aggregates were obtained at 20 % replacement level, when the slag was used as coarse aggregates. The same occurred at 40 % replacement level of sand by fine slag aggregates. An increase in cement content could increase the strength of slag aggregates concrete. An increasing addition of slag as aggregates in cement mortar lowers the abrasion resistance. Concrete with slag as 20 % replacement of fine aggregates showed almost equal absorption of a ray of α and β particles but better absorption of a γ-ray than conventional concrete with 100 % sand. The slag concrete absorbed 88 % of a ray of α and β particles and 90 % of a ray of γ particles in comparison to 86 % for α and β particles ray and 10 % for γ-ray absorption by conventional concrete.

Sorlini et al. (2004) reported the use of two slags (raw as well as 6 months weathered) generated during processing of EAF dust produced in steel production, which contains very high amounts of Zn. Their results suggest that slag addition in concrete does not change the concrete's mechanical performance (compressive, tensile and flexural strength) or in some cases improves these properties. However, slag addition lowers the modulus of elasticity of concrete. Saikia et al. (2008, 2012) also observed higher compressive strength and bending strength and lower water absorption of cement mortar with a slag obtained from lead and zinc smelting as partial replacement of aggregates than those of normal cement mortar.

The ferrosilicate slag, generated during the production of zinc in Imperial Smelting Furnace (ISF-slag) is successfully used as aggregate in concrete road construction (Morrison et al. 2003; Morrison and Richardson 2004). Monosi et al. (2001) observed a negligible reduction in strength, when a ground or unground slag obtained from zinc smelting was used to replace 20 % of sand and 15 % of Portland cement by weight. The reduction in compressive strength due to slag addition as aggregates was negligible especially at the later curing periods (7-day and onwards). However, the compressive strength significantly decreased in the whole curing period, when 15 % of Portland cement was replaced by an equal amount of ground slag in the concrete mix with 20 % slag aggregates. There was practically zero strength at the early periods (1–3 days) of curing, which indicates that the reduction in cement hydration due to deleterious component of slag was the major cause of strength reduction.

The use of other slags, e.g. ferronickel slag, ferrochromium slag, ferromolybdenum slag, aluminium with salt slag as fine or coarse aggregates, is also reported (Boheme and Van Den Hende 2011; Pereira et al. 2000; Shoya et al. 1997; Zelic 2005).

The concentrations of toxic elements present in the leachate generated from lead slag with cement mortar and concrete are generally higher than their concentrations in the leachate generated from conventional concrete; however, the concentrations generally meet the standard specifications (Atzeni et al. 1996; Penpolcharoen 2005; Saikia et al. 2008, 2012; Monosi et al. 2001). However, the use of these slags as aggregates in concrete can be considered only after thorough analysis of their environmental as well as economic suitability. Information on the long-term mechanical and durability performances of concrete with slag is also necessary for the effective application of these materials as aggregates in concrete, which can solve problem related to their disposal.

4.6 Plastic Waste

Significant work has been done on the use of various plastic wastes as aggregates or fibres or fillers in concrete. In this section, the properties of concrete with plastic waste as aggregates from existing literature data will be presented. The concrete properties will be discussed in three main sections: fresh concrete properties, mechanical and durability of hardened concrete properties. Some special properties of concrete will be highlighted in another section. Details about the properties of plastic as aggregates, the generation of plastic aggregates and other related issues are presented in Chap. 2.

4.6.1 Fresh Concrete Properties

The incorporation of plastic aggregates in concrete strongly affects the various fresh concrete properties due to their organic nature as well as their shape, size, porosity and lightweight nature. In this section, some fresh concrete properties available in various references will be highlighted.

4.6.1.1 Slump

Slump is used to measure the workability or consistency of fresh concrete mix. Being an important property, the slump of concrete and cement mortar mixes with plastic aggregates was studied extensively.

There are two parallel views on the workability behaviour of concrete with plastic aggregates. In the majority of the studies, a lower slump value of fresh concrete due to the incorporation of several types of plastic aggregates than that of conventional concrete was observed and increasing the incorporation level of plastic aggregates further lowers the slump (Albano et al. 2009; Batayneh et al. 2007; Frigione 2010; Ismail and Al-Hashmi 2008a; Kou et al. 2009). The reason for this is the sharp edge and angular particle size of plastic aggregates.

On the other hand, in a few studies, an increase in slump value due to the incorporation of plastic aggregates is reported (Al-Manaseer and Dalal 1997; Choi et al. 2005, 2009). According to Al-Manaseer and Dalal (1997) the increased slump of concrete mixes due to the incorporation of plastic aggregates is due to the presence of more free water in the mixes with plastic waste than in that with NA, since unlike NA, plastic aggregates cannot absorb water during mixing. Choi et al. (2005, 2009) reported an increase in slump of concrete with increasing content of two types of treated PET-bottle aggregates, due to the spherical shape of the PET-aggregates as well as the slippery surface texture, which decreases the inner friction between the mortar and the PET-aggregates and therefore increases the flowability.

Saikia and de Brito (2010) reported that the slump of concrete with cylindrical PET-aggregates with very smooth surface texture is slightly higher that of concrete with NA. The authors also found decreasing slump values in concrete due to the addition of fine and coarse sized flaky plastic aggregates, attributed to the fact that these PET-aggregates have sharper edges compared to NA. Moreover in comparison to NA, these flaky aggregates are angular and non-uniform by nature. The slump further decreased as the size of flaky aggregates increased.

The addition of some types of plastic aggregates such as rigid polyurethane (PUR) foam waste or heat treated expanded polystyrene foam (MEPS) decrease the slump of the resulting concrete mix due to the presence of large amounts of surface pores in these aggregates (Fraj et al. 2010; Mounanga et al. 2008; Kan and Demiboga 2009).

4.6.1.2 Density

Irrespective of the type and size of substitutions, the incorporation of plastic as aggregates generally decreases the fresh density of the resulting concrete due to the lightweight nature of these aggregates (Al-Manaseer and Dalal 1997; Ismail and Al-Hashmi 2008a; Hannawi et al. 2010; Marzouk et al. 2007; Kou et al. 2009; Choi et al. 2005, 2009; Saikia and de Brito 2010).

Ismail and Al-Hashmi (2008a) reported that the fresh density of concrete with 10, 15, and 20 % plastic aggregates as replacement of fine aggregates tends to decrease by 5, 7, and 8.7 %, respectively, by comparison with the reference concrete. Al-Manaseer and Dalal (1997) also found 2.5, 6 and 13 % lower densities of concrete with 10, 30, and 50 % plastic aggregates, respectively. Saikia and de Brito (2010) observed a reduction of the density of fresh concrete with increasing volume of PET-aggregates incorporated. The authors found a trend of this density reduction for the three different types of PET-aggregates they used: pellet-size aggregates > fine fraction of flaky aggregates > coarse fraction of flaky aggregates.

According to Fraj et al. (2010) the fresh density of different concrete mixes with dry and water-saturated PUR-foam aggregates classifies them as lightweight concrete and these values were 27–33 % lower than the control concrete's density. The density values decreased as foam incorporation increased.

Hannawi et al. (2010) reported that there was a decrease in fresh and dry densities as the plastic aggregates content increased. Dry density decreased from 2173 kg/m^3 for mixes with 0 % plastic aggregates to 1755 and 1643 kg/m^3, respectively, for mixes with 50 % PET and polycarbonate (PC) plastic aggregates, mainly due to the lower bulk density of plastic. These values were below 2000 kg/m^3, the minimum dry density required for structural lightweight concrete according to RILEM LC2 classification.

4.6.1.3 Air Content

No report is available on the evaluation of air content of cement mortar or concrete mixes with untreated plastic waste as aggregates. Choi et al. (2009) reported the air content of concrete with sand stone coated PET as partial replacement of fine aggregates (Table 4.10). An air-entrainment agent was used during preparation of concrete. The air content of concrete mixes with PET-aggregates was slightly lower than that of the control concrete for the same w/c value and a reducing trend was observed with increasing PET-content in concrete.

4.6.2 Mechanical Properties

The addition of plastic drastically changes the various hardened concrete properties, which will be highlighted in this section. The properties presented are: compressive, splitting tensile and flexural strengths, Young's modulus of elasticity, toughness behaviour: stress–strain curve, failure characteristics and abrasion resistance.

4.6.2.1 Compressive Strength

The compressive strength of concrete and cement mortar is a fundamental property, thoroughly studied in almost all studies related to plastic aggregates. The

Table 4.10 Air content of fresh concrete (Choi et al. 2009)

Amount of sand replaced by PET aggregate (%)	Air content		
	w/c = 0.53	w/c = 0.49	w/c = 0.45
0	4.5	5.0	5.0
25	4.2	4.5	4.8
50	4.1	4.3	4.0
75	4.1	4.2	–

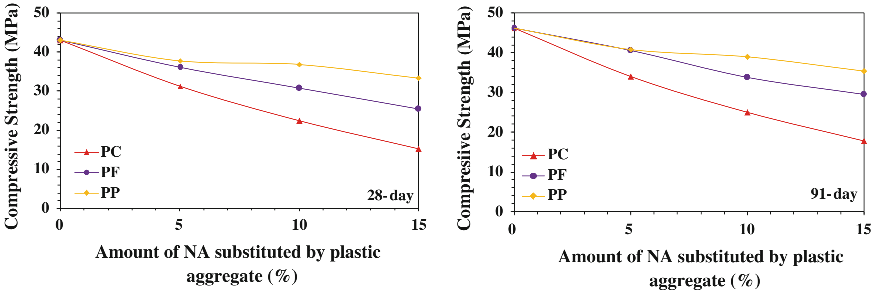


Fig. 4.33 Compressive strength of concrete with PET aggregates (Saikia and de Brito 2010)

incorporation of plastic as aggregates decreased the compressive strength of resulting concrete and mortar (Albano et al. 2009; Akcaozoglu et al. 2010; Batayneh et al. 2007; Choi et al. 2005, 2009; Fraj et al. 2010; Frigione 2010; Ismail and Al-Hashmi 2008a; Hannawi et al. 2010; Kan and Demirboga 2009; Kou et al. 2009; Marzouk et al. 2007; Panyakapo and Panyakapo 2008; Remadnia et al. 2009; Saikia and de Brito 2010). The compressive strength behaviour of concrete and mortar with three types of PET aggregate as partial substitution of fine and coarse NA are presented in Fig. 4.33.

The very low bond strength between the surface of plastic waste and cement paste as well as the hydrophobic nature of plastic waste, which can inhibit cement hydration reaction by restricting water movement, are the causes for low compressive strength of concrete with plastic aggregates. Another factor is the mismatch of particle size and shape between natural and plastic waste aggregates.

However, several authors reported that concrete with partial replacement of NA up to a certain level meet the standard strength values for various types of concrete such as concrete with moderate strength (Albano et al. 2009), minimum compressive strength requirement for structural concrete (Ismail and Al-Hashmi 2008a). Fraj et al. (2010) observed that concrete with dry PUR-foam aggregates almost satisfied the criteria for structural lightweight aggregates concrete as defined in ACI 318 and ASTM C 330. Panyakapo and Panyakapo (2008) reported that concrete with melamine waste aggregates as partial replacement of natural fine aggregates and FA as partial replacement of normal Portland cement (NPC) met most of the requirements for non-load-bearing lightweight concrete according to the ASTM C129-05 Type II standard. The percentage reduction of compressive strength of mortar and concrete due to partial replacement of natural fine aggregates by plastic aggregates at various substitution levels is presented in Table 4.11.

Akcaozoglu et al. (2010) investigated the use of shredded waste polyethylene terephthalate (PET) bottle granules as lightweight aggregates in mortar preparation using two types of binders: NPC and a 50:50 mixture of BFS and NPC. The authors found that the compressive strength of mortar with PET aggregate is higher for the NPC–BFS binder than for NPC only.

Table 4.11 Reduction of compressive strength of cement mortar and concrete (28-day) due to the substitution of natural aggregates by plastic aggregates

Reference	Types of substitution	Reduction in compressive strength for substitution level (%) of									
		3	5	10	15	20	30	45	50	75	100
Batayneh et al. (2007)	Fine/PET		23			72					
Frigione (2010)	Fine/PET		<2								
Hannawi et al. (2010)	Fine/PET	9.8		30.5		47.1			69		
	Fine/PC	6.8		27.2		46.1			63.9		
Kou et al. (2009)	Fine/PVC		9.1		18.6		21.8	47.3			
Saikia and de Brito (2010)	Fine flakes/PET		13.8	28.5	41.8						
	Coarse flakes/PET		28.3	47.9	64.4						
	Fine pellet/PET		12.2	14.6	22.4						

Fraj et al. (2010) observed a 57–78 % lower 28-day compressive strength of concrete with 8–20 mm rigid PUR foam as aggregate compared to a control concrete, due to the lightweight nature of the modified concrete as well as the low mechanical properties and the high porosity of PUR-foam aggregates. Prewetting the PUR-foam aggregates further lowers the compressive strength due to an increase in the mortar's porosity. Using a superplasticizer along with increasing cement content, on the other hand, increases compressive strength. The use of superplasticizer made it possible to decrease cement content by 15 % and to increase PUR-foam content by 33 % compared, with an acceptable reduction (15 %) of compressive strength.

Mounanga et al. (2008) reported that water curing concrete with PUR-foam aggregates and NA slightly improved the compressive strength compared to dry curing. For conventional lightweight concrete, the increase in strength was about 69 % and this improvement for concrete with 13.1, 21.2 and 32.7 % by volume of PUR-foam aggregates was 39, 34 and 5 %, respectively.

Kan and Demirboga (2009) reported that lightweight concrete with heat-treated expanded polystyrene (MEPS) waste aggregates exhibited a compressive strength 40 % higher than that of concrete with vermiculite or perlite aggregates at equal concrete density. However, the compressive strength of concrete with MEPS aggregates decreased with increasing addition of aggregates. The development of compressive strength of concrete with 100 % MEPS aggregates at 90 days with respect to that at 7 days was about 83 % whereas it was 69 % for concrete with 25 % MEPS aggregates, which might be due to the high heat of hydration of the former type of concrete because of low specific thermal capacity of the MEPS aggregates. The compressive strength of concrete with coarse MEPS aggregates was lower than that of concrete with fine MEPS aggregates as the coarse MEPS aggregates had higher porosity, and therefore were more brittle and weaker than the fine MEPS aggregates.

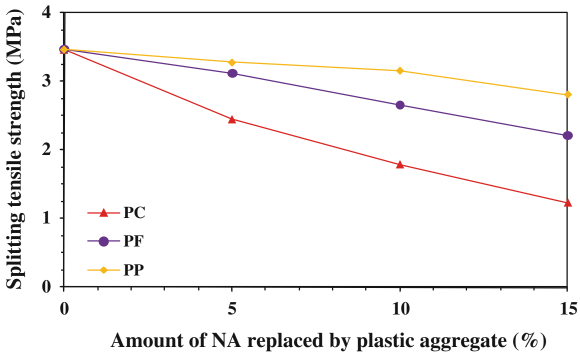


Fig. 4.34 Splitting tensile strength of concrete with plastic waste aggregates (Saikia and de Brito 2010)

4.6.2.2 Splitting Tensile Strength

Similarly to compressive strength, the incorporation of any type of plastic aggregates lowers the splitting tensile strength of concrete. The causes for the reduction observed in splitting tensile strength reported in various references were similar to those used to explain the decrease in compressive strength due to the addition of plastic aggregates. Some results on the tensile strength behaviour of concrete and mortar with various percentages of different types of plastic aggregates are presented in Fig. 4.34.

Kou et al. (2009) reported that splitting tensile strength was reduced with an increase in PVC content in a manner similar to that observed for compressive strength. According to them, the tensile splitting strength of concrete is influenced by the properties of the interfacial transition zone (ITZ); therefore, the smooth surface of the PVC particles and the free water accumulated at the surface of PVC granules could cause a weaker bonding between the PVC particles and the cement paste. According to Albano et al. (2009), the decrease in splitting tensile strength was due to the higher porosity of concrete caused by the increasing addition of PET-aggregates as well as the higher w/c value. Kan and Demirboga (2009) also reported that splitting tensile strength of concrete with heat treated expanded polystyrene (MEPS) aggregates decreases with their content in concrete, due to the generation of more porosity. Batayneh et al. (2007) reported a decreasing trend of splitting tensile strength but not as prominent as for compressive strength. Saikia and de Brito (2010) also reported lower 28-day tensile strength of concrete with three differently shaped PET-aggregates. The authors reported that the concrete cylinders with flakier PET-aggregate did not split into two fractions after the determination of tensile strength, which was generally observed for cylinders with natural and pellet-shaped plastic aggregates as the flaky shaped plastic aggregates could act as a bridge between the two split pieces (Fig. 4.35).

Kou et al. (2009) found an excellent correlation between 28-day splitting tensile strength and 28-day compressive strength of concrete with PVC aggregates as replacement of fine aggregates, which follows a linear relationship. Choi et al. (2009) also found an expression, $f_{st} = 0.23 \times f_c^{(1/3)}$, for the relationship between

Fig. 4.35 Concrete specimens after the determination of tensile splitting strength, from left to right: concrete with natural, pellet-shaped PET, fine and coarse PET flakes aggregates (Saikia and de Brito 2010)

28-day compressive strength and splitting tensile strength of concrete with PET aggregates and an expression, $f_{st} = 1.40 \times (f_c/10)^{(1/3)}$, for a similar relationship for conventional concrete.

4.6.2.3 Modulus of Elasticity

According to ASTM C 469, the modulus of elasticity is defined as a stress–strain ratio value for hardened concrete. The type of aggregates influences the modulus, since the deformation produced in the concrete is partially related to the elastic deformation of the aggregates.

From their study on the use of three different size fractions of PET waste aggregates in concrete production, where concrete was prepared at two different w/c values and at two different natural fine aggregates replacement levels, Albano et al. (2009) observed higher modulus at lower substitution rate of NA by PET-aggregates than at higher substitution rate and at low w/c value. They did not observe any effect of particle size. The modulus of elasticity of concrete with PET-aggregates met the requirement as described in "American Manual of Reinforced Concrete" (1952) except the concrete composition with 20 % large PET-aggregate at w/c of 0.60.

Frigione (2010) plotted the stress–strain curves (σ–ε curve) during the determination of the compressive strength of a reference concrete and a concrete with PET with w/c = 0.45 and cement content of 400 kg/m^3 to determine the modulus of elasticity. The calculated modulus was 48.1 and 41.8 GPa for the reference concrete and the concrete with PET, respectively.

Hannawi et al. (2010) found that increasing the plastic content in concrete decreased the resulting modulus of elasticity, probably due to the low stiffness of PET and PC plastics as well as the poor bond between the matrix and plastic aggregates (Fig. 4.36a). Saikia and de Brito (2010) also found lower modulus of elasticity for concrete with three differently shaped PET waste aggregates than for

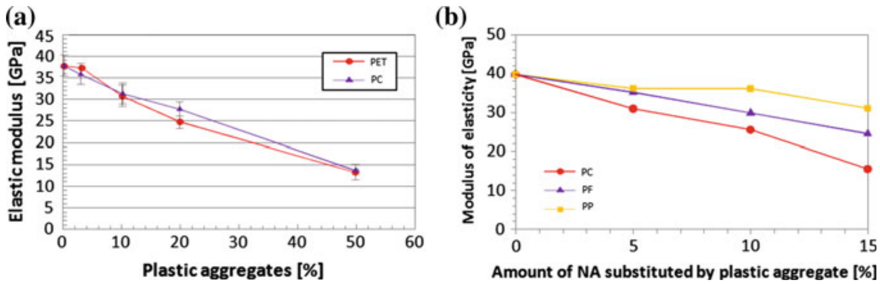


Fig. 4.36 Modulus of elasticity of concrete and cement mortar with plastic aggregates. **a** Hannawi et al. 2010; **b** Saikia and de Brito 2010

concrete with NA (Fig. 4.36b). According to them, the lower modulus of elasticity of concrete because of the incorporation of PET-aggregates is due to the lower stiffness of PET-aggregates than that of NA as well as to the higher porosity due to the high w/c value.

Compared to compressive strength, Fraj et al. (2010) observed a less significant effect on the modulus of elasticity due to the addition of fine expanded PUR-foam aggregates in lightweight concrete. The same authors found an increasing linear correlation between air-dry density and dynamic modulus of elasticity. As the PUR foam had a low stiffness due to its high porosity, increasing the content of PUR foam in concrete reduced its modulus of elasticity. Prewetting the PUR-foam aggregates, improving the cementitious matrix properties by using superplasticizer and decreasing the w/c value did not have an influence on the modulus of elasticity.

Increasing the replacement ratio of fine NA by PVC granules in concrete also reduced the resulting modulus of elasticity (Kou et al. 2009). The replacement of 5, 15, 30 and 45 % of fine NA by PVC granules reduced the modulus of elasticity by 6.1, 13.8, 18.9 and 60.2 %, respectively, when compared to that of the control concrete. According to the authors, the major causes of this reduction were (1) lower stiffness of PVC granules than of the cement paste; (2) lower compressive strength of the concrete with PVC than of the conventional concrete. They also reported that the prediction of the modulus of elasticity of concrete with PVC granules by using an equation suggested by ACI 318-83 overestimated the property value.

Choi et al. (2005) reported that the increasing addition of granulated BFS coated PET aggregates in concrete decreased the resulting modulus of elasticity. In another study, Choi et al. (2009) compared the relationship between the 28-day compressive strength and 28-day modulus of elasticity of concrete with different proportions of sand coated PET-aggregates as replacement of fine NA with CEB-FIP model code (CEB Bulletin Information No. 213/214: Committee Euro-international du Béton, Thomas Telford; 1993) and ACI code (ACI 318 M-05: Building code requirements for structural concrete and commentary. ACI Manual of concrete practice, ACI; 2005). The relationship between compressive strength and modulus of elasticity of concrete with plastic aggregates was in close agreement with the one suggested in ACI 318-05, in which the concrete's density was taken into consideration.

4.6.2.4 Flexural Strength

Flexural strength is defined as a material's ability to resist deformation under load and is measured in terms of stress. The flexural strength represents the highest stress experienced within the material in the moment of rupture. The transverse bending test is the most frequently employed, in which a rod specimen having either a circular or rectangular cross-section is bent until fracture using a three- or four-point flexural test technique.

Batayneh et al. (2007) reported a decreasing trend of flexural strength with increasing plastic waste aggregates content in concrete. However, this reduction was not as significant as for compressive strength. Ismail and Al-Hashmi (2008a) reported that the flexural strength of plastic waste concrete mixes at each curing age was prone to decrease with the increase of the plastic waste content in these mixes. Saikia and de Brito (2010) also found low flexural strength values for concrete with PET-aggregates than that for concrete with NA (Fig. 4.37).

Hannawi et al. (2010) did not find significant changes in the flexural strength of mortar specimens with up to 10 % PET-aggregates and up to 20 % PC-aggregates with similar composition. However, decreases of 9.5 and 17.9 % for mixes with 20 and 50 % PET-aggregates, respectively, were observed. For mixes with 50 % PC-aggregates a decrease of 32.8 % was measured. According to the authors, the elastic nature and the non-brittle characteristic under loading of the plastic aggregates might have an effect on the observed flexural strength. The bending strength of cement composites prepared by Laukaitis et al. (2005), using three different types of waste polystyrene granules followed a proportional relationship with its density.

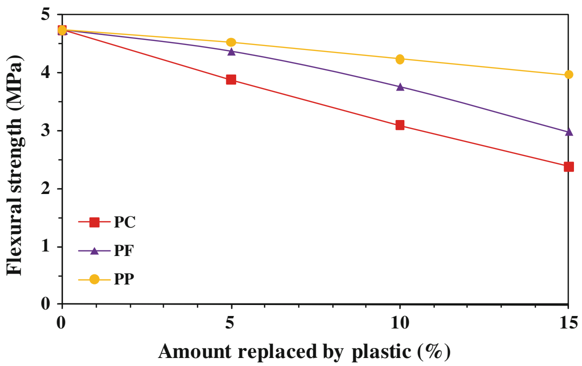


Fig. 4.37 Flexural strength behaviour of concrete with PET-aggregates (Saikia and de Brito 2010)

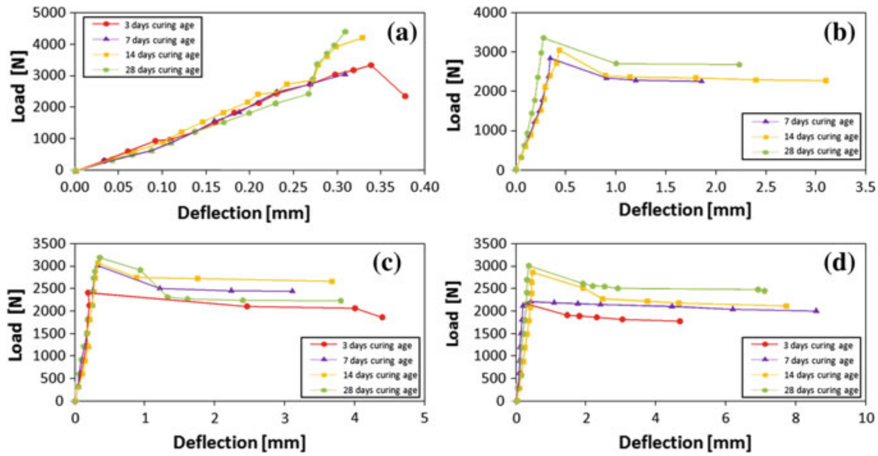


Fig. 4.38 Load–deflection curves of concrete with 0, 10, 15 and 15 % of fine aggregates by plastic aggregates (Ismail and Al-Hashmi 2008a). **a** 0 %, **b** 10 %, **c** 15 %, **d** 15 %

4.6.2.5 Toughness/Poisson's Ratio

Ismail and Al-Hashimi (2008a) plotted the load–deflection curves of a reference concrete and concrete mixes with 10, 15, and 20 % plastic waste as fine aggregate replacement at the curing ages of 3, 7, 14 and 28 days. The results are illustrated in Fig. 4.38. They show the propagation of microcracks is arrested by the introduction of plastic waste particles in concrete. The authors also determined the toughness indices for the concrete compositions with plastic waste aggregate at the curing ages of 3, 7, 14 and 28 days (Table 4.12).

The toughness indices of concrete mixes with plastic waste aggregates for all replacement levels after 14- and 28-day curing reached a plastic behaviour according to ASTM C1018, desirable for many applications that require high toughness.

Frigione (2010) plotted the stress–strain curves (σ–ε curve) during the determination of compressive strength of a reference concrete and a concrete with PET. Compared to the reference concrete, a higher strain value corresponding to the maximum stress was registered for the concrete with PET waste aggregates. The

Table 4.12 Toughness indices for concrete with plastics aggregates (Ismail and Al-Hashmi 2008a)

Percentages of plastic in concrete mixes (%)	Toughness indices at curing age											
	3-day			7-day			14-day			28-day		
	I_5	I_{10}	I_{10}:I_5	I_5	I_{10}	I_{10}:I_5	I_5	I_{10}	I_{10}:I_5	I_5	I_{10}	I_{10}:I_5
10	–	–	–	8.3	11.6	1.4	4.3	8.6	2.0	2.5	7.5	3.0
15	3.0	11.0	3.7	4.5	9.5	2.1	4.2	8.4	2.0	8.0	16.1	2.0
20	6.8	13.7	2.0	7.3	14.8	2.0	5.2	11.5	2.1	5.7	11.6	2.0

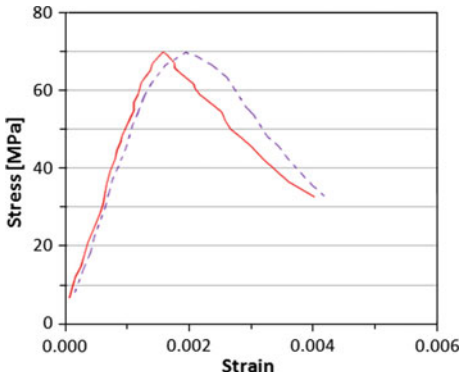


Fig. 4.39 Stress–strain curves for a reference concrete (*plain line*) and a concrete with PET waste aggregates (*dotted line*) (Frigione 2010)

peak shapes of the two curves also suggested that the concrete with PET waste aggregates is less brittle than the reference concrete and this type of concrete could withhold a larger deformation still keeping its integrity (Fig. 4.39). Kou et al. (2009) observed increasing Poisson's ratio values with increasing contents of PVC waste aggregates in concrete. Since the higher Poisson's ratios meant higher ductility, the addition of PVC improved the ductility of the resulting lightweight aggregates concrete, due to the elastic nature of PVC.

4.6.2.6 Failure Characteristics

After failure during the determination of compressive strength, specimens with plastic aggregates do not exhibit the typical brittle type of failure, obtained for conventional cement mortar and concrete. As the plastic aggregates content increased, the failure became more ductile. The specimens with plastic aggregates can carry load for a few minutes after failure without full disintegration, as was observed by various researchers (Hannawi et al. 2010; Marzouk et al. 2007, Saikia and de Brito 2010). The recycled PET-aggregates can delay crack initiation and prolong the crack propagation interval thereby increasing structural strength.

Albano et al. (2009) found various types of failure including normal cone type for concrete specimens with PET-aggregates, where 20 % of fine aggregates were replaced. As the smooth surface of the PVC particles and the free water accumulated at the surface of PVC granules may have caused weaker bonding between PVC particles and cement paste, most of the PVC granules in the concrete matrix did not fail but were debonded from the cement paste after reaching the ultimate strength of concrete (Kou et al. 2009). Fraj et al. (2010) reported that the rupture mechanism of concrete with PUR-foam aggregates was different from that of the normal weight control concrete: in the first case, the rupture occurred on the mortar matrix/PUR-foam aggregates interfaces as well as in the middle of the PUR-foam aggregates. In normal weight concrete, the rupture mainly took place in the ITZ because of the poor properties of this zone compared to the other concrete components. By observing the splitting behaviour of concrete blocks after tensile strength and flexural strength tests, Saikia and de Brito (2010) concluded that the flaky PET-aggregates can act as bridge between the two separated pieces of concrete block after failure, which was not observed for concrete blocks with natural as well as pellet-shaped PET-aggregates.

4.6.2.7 Abrasion Resistance

Compared to other properties, very few data are available on the abrasion resistance of concrete (or mortar with any type of plastic waste aggregates). Soroushian et al. (2003) reported the abrasion resistance of concrete with plastic waste fibres. The authors found a reduction of the abrasion resistance of concrete due to the addition of plastic waste fibres in concrete. However, the incorporation of commercial plastic aggregates in concrete improved its abrasion resistance (Nasvik 1991).

Recently, Saikia and de Brito (2010) reported that the incorporation of PET-aggregates can improve the abrasion resistance of concrete (Fig. 4.40a). The authors found that the abrasion resistance of concrete with pellet-shaped PET-aggregates increased with their content. On the other hand, for concrete with two types of flaky aggregates the best results were obtained for a 10 % substitution level. From the relationship between compressive strength and depth of wear for concrete with different types of plastic aggregates, the authors found a certain compressive strength level for concrete with PET-aggregates over which the abrasion resistance deteriorates (Fig. 4.40b).

4.6.3 Durability Performance

Several durability factors like permeability properties, shrinkage, carbonation resistance and resistance against freeze–thaw cycles are evaluated for concrete or mortar with plastic as aggregates. However, compared to the available information

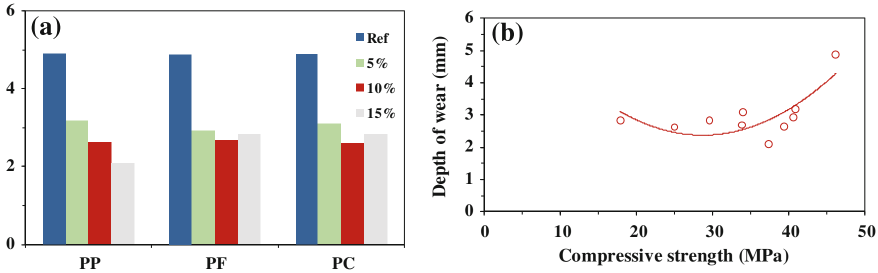


Fig. 4.40 **a** Depth of wear and **b** cubic compressive strength versus depth of wear of concrete with PET-aggregates after abrasion resistance test (Saikia and de Brito 2010)

on the mechanical performance of concrete with plastic aggregates, relatively less data are available on the durability behaviour.

4.6.3.1 Permeability Behaviour

Generally permeability of aggressive chemical species through the pores of concrete is the major factor, which controls several durability properties. Tests like water absorption, gas permeability and chloride permeability measurement can give information on the vulnerability of concrete for ingress of deleterious chemical species.

Water Absorption and Water Accessible Porosity

Albano et al. (2009) observed a higher water absorption value for concrete with PET-aggregates than that for concrete with NA. The water absorption value was further increased with increasing content of PET-aggregates in concrete, increasing size of PET-aggregates and increasing w/c value. According to the authors, the differences in size grading as well as in shape of plastic aggregates from the natural fine aggregates were responsible for this behaviour.

Choi et al. (2009) measured the sorptivity coefficient of 28-day cured cement mortars prepared by replacing 0, 25, 50 and 75 % of fine NA by sand powdered coated PET-aggregates. Their results indicated that the sorptivity of cement mortar with PET-aggregates at 25 % replacement level was lower than that of the control mortar and at 50 and 75 % replacement level it was higher than that of the control mortar. According to the authors, at 50 and 75 % replacement level the change in grading size of the fine aggregates mixture increased the inside porosity of mortar and thus increased the sorptivity.

Akcaozoglu et al. (2010) found higher water absorption and porosity values for a cement mortar with 100 % PET-aggregates than for a mortar with equal percentage in volume of PET-aggregates and sand. The authors found a similar trend for cement mortar with a mixture of equal weight of BFS and NPC though the BFS addition with NPC increased the water absorption and porosity of the resulting cement mortar. However, according to the authors, all the values for all types of mortar meet the range generally observed for lightweight concrete.

Fraj et al. (2010) recorded a higher value of the water accessible porosity of cement mortar with PUR-foam aggregates than that of the mortar with no plastic aggregates. The authors also reported that prewetting the PUR-foam aggregates further increased the porosity. However, the addition of superplasticizer in cement mortar with prewetted PUR-foam aggregates can decrease the porosity value.

Marzouk et al. (2007) reported that the volumetric substitution of plastic aggregates by less than 100 % decreased the rate of water adsorption with respect to the reference mortar that contained no waste. The authors found lower sorptivity for cement mortars with PET-aggregates than for mortars with no plastic waste.

The sorptivity further decreased with increasing volumetric substitution up to 50 %. Thus, their results suggest better durability performance of cement mortar with PET-aggregates than that of mortar with NA when in contact with aggressive solutions. Hannawi et al. (2010) measured the water absorption and apparent porosity values of the different mortar specimens with various amounts of PET and PC waste aggregates. Their results revealed that replacing 3 % by volume of sand by an equal volume of PET or PC do not exert influence either on water absorption or on the apparent porosity of the composites in comparison with the control mortar. However, apparent porosity and water absorption increased with increasing plastic content.

Gas Permeability

Fraj et al. (2010) reported higher gas permeability (2.2 times) of concrete with dry and prewetted PUR-foam aggregates than that of conventional concrete. Prewetting the PUR-foam aggregates can further increase the value considerably. Decreasing the w/c value and increasing superplasticizer content can reduce this value for concrete with prewetted PUR-foam aggregates.

Hannawi et al. (2010) found an increase of helium gas permeability coefficient with increasing plastic aggregates content in mortar, which indicated an increase of the percolated porosity of mortar due to the incorporation of plastic aggregates, because of weak bonding between the cement paste and plastic aggregates. They also reported greater helium gas permeability coefficient of mortar with PET aggregates than that of mortar with PC aggregates at the replacement level of 10, 20 and 50 % by volume of sand by plastic aggregates.

Chloride Migration

Kou et al. (2009) investigated the resistance to chloride-ion penetration of 28 and 91 days hardened concrete prepared by partially replacing natural fine aggregates by PVC waste granules. The chloride-ion penetration resistance of concrete was represented by the total charge passed in Coulomb during a test period of 6 h. Their results (presented in Fig. 4.41) indicated that this property improved with an increase in PVC content as well as with curing time. They found reduction of about 36 % in the total charge passed through the 28-day cured concrete, with 45 % replacement of NA by PVC granules in comparison to same-age concrete with no PVC granules. According to them, the increase in the resistance to chloride-ion penetration of concrete is attributed to the impervious PVC granules blocking the passage of the chloride ions.

Fraj et al. (2010) evaluated the chloride diffusion coefficient of concrete with rigid PUR foam as partial replacement of coarse NA. Their results are presented in Table 4.13. The authors observed lower chloride diffusion coefficient for concrete with dry PUR-foam aggregates than that of concrete with NA only. However, the

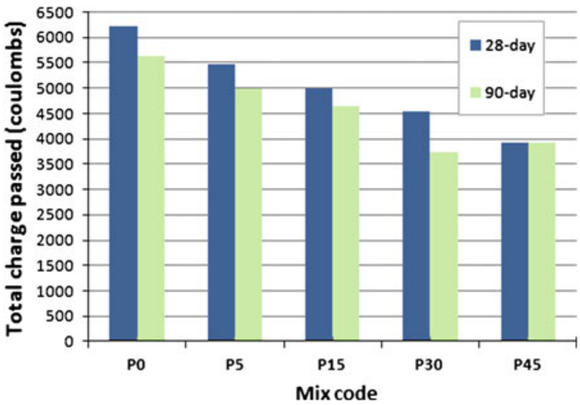


Fig. 4.41 Chloride penetration resistance of concrete with PVC waste granules (Kou et al. 2009)

Table 4.13 Chloride-ion penetration coefficient of concrete with PUR-foam aggregates (Fraj et al. 2010)

Volume of PUR-foam aggregate	w/c ratio	Cement content (kg/m^3)	Volume content of PUR foam (%)	Amount of superplasticizer (kg/m^3)	Effective chloride diffusivity coefficient (10^{-12} m^2/s)
Control	0.55	397	0	0	1.87
Dry PUR aggregate	0.55	397	34	0	1.62
Prewetted PUR aggregate	0.55	397	34	0	5.30
	0.44	415	35	1.405	2.70
	0.44	353	45	1.196	5.98

presaturation of PUR-foam aggregates in water resulted in a significant increase of the chloride diffusion coefficient, due to the increase in porosity of concrete with increasing incorporation of PUR-foam aggregates. They also reported that the reduction in w/c value and increase in cement content could significantly improve the chloride resistance performance of concrete with prewetted PUR-foam aggregates.

4.6.3.2 Carbonation

Akcaozoglu et al. (2010) measured the carbonation resistance of various types of cement mortars by measuring carbonation depth. A phenolphthalein solution was applied on the broken surfaces of the half pieces obtained after flexural tensile strength test. The compositions of various mixes along with the carbonation depth at various time periods are presented in Table 4.14. Irrespective of binder types, the carbonation depth of mortar with PET-aggregates only after 28 days of curing is lower than that of the mortar with an aggregate mixture of PET and sand. The authors also found a higher porosity for the mortar with sand and PET mixture than

Table 4.14 Carbonation depth of cement mortar specimens (Akcaozoglu et al. 2010)

Amount in mortar (%)					Depth of carbonation (mm) in			
Cement	Slag	PET-aggregate	Normal aggregate	Water	7 days	28 days	90 days	180 days
51.28	0	25.64	0	23.08	0.3	1.2	4.3	5.0
25.64	25.64	25.64	0	23.08	0.3	1.7	5.5	7.6
33.90	0	16.95	33.90	15.25	0.0	1.4	4.8	5.9
16.95	16.95	16.95	33.90	15.25	0.6	2.5	6.8	8.5

for the mortar with PET aggregates only. According to the authors, PET and sand aggregates used together did not combine sufficiently and the resulting mortar became porous. On the other hand, the depth of carbonation for concrete with slag is significantly higher than for the mortar prepared by using cement as the only binder.

4.6.3.3 Shrinkage

Frigione (2010) found an increase in drying shrinkage due to the incorporation of 5 % PET-aggregates in concrete at different experimental conditions due to the lower modulus of elasticity of concrete with plastic aggregates than that of conventional concrete. However, the shrinkage of concrete with PET-aggregates was acceptable for various uses as structural concrete.

From their experiments on the use of PVC waste granules as a partial volumetric replacement of natural sand in concrete, Kou et al. (2009) found decreasing drying shrinkage with increasing content of plastic aggregates (Fig. 4.42). According to the authors, PVC granules are impermeable and do not absorb water when compared to sand and do not shrink either, and hence are able to reduce the overall shrinkage of concrete.

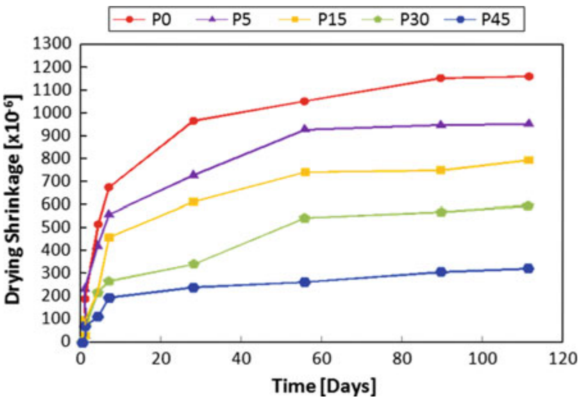


Fig. 4.42 Drying shrinkage of concrete with fine PVC aggregates (Kou et al. 2009)

Fraj et al. (2010) found higher drying shrinkage values for lightweight concrete with dry and prewetted polyurethane foam (PUR foam) as partial replacement of fine aggregates. Concrete with dry PUR-foam aggregate has 8.1 % higher 28-day drying shrinkage than control concrete. On the other hand, concrete mixes with prewetted PUR-foam aggregates at 34 and 45 % by volume replacement levels exhibited, respectively, 72.5 and 149.5 % higher 28-day drying shrinkage than control concrete. Lowering the w/c value or increasing superplasticizer, sand and cement content can reduce drying shrinkage of concrete with prewetted PUR-foam aggregates. In these conditions, the 28-day drying shrinkage of concrete with prewetted PUR-foam aggregates at 35 % by volume replacement level is 49.7 % higher than that of control concrete. According to the authors, the lower modulus of elasticity of PUR-foam aggregates and the higher amount of prewetting water in the case of concrete with prewetted aggregates are the causes of the higher drying shrinkage of concrete with PUR-foam aggregates.

Mounanga et al. (2008) reported higher drying shrinkage of lightweight concrete in which various fractions of fine aggregates were replaced by PUR-foam aggregates than that of control concrete. According to the authors, this behaviour was mainly due to effect of PUR-foam aggregates on the stiffness of concrete. However, other factors such as w/c value, sand content and thermal dilation during hydration also had a significant effect.

Akcaozoglu et al. (2010) observed significantly higher drying shrinkage of mortars with PET aggregates only than that of a mortar with equal percentage by weight of sand and PET-aggregates at the experimental drying periods. Mixing BFS with cement can reduce mortar shrinkage for both types of aggregates (PET only and sand-PET mixture).

4.6.3.4 Freeze–Thaw Resistance

Kan and Demirboga (2009) reported the freeze–thaw resistance of concrete with modified expanded polystyrene foam (MEPS) as partial or full substitution of natural fine and coarse aggregates by using standard ASTM 666 procedure B. The following conclusions were taken from the results: 1. increasing the MEPS aggregate ratio in the mixes the concrete is expected to exhibit a higher frost resistance and guarantee a higher durability; 2. coarse lightweight MEPS aggregates are more susceptible to freeze–thaw cycles than fine lightweight MEPS aggregates.

4.6.4 Other Properties

There are other concrete properties that are reported to be altered due to the incorporation of plastic aggregates. In this section, the fire behaviour and thermophysical properties of concrete with plastic aggregates are highlighted from the literature results.

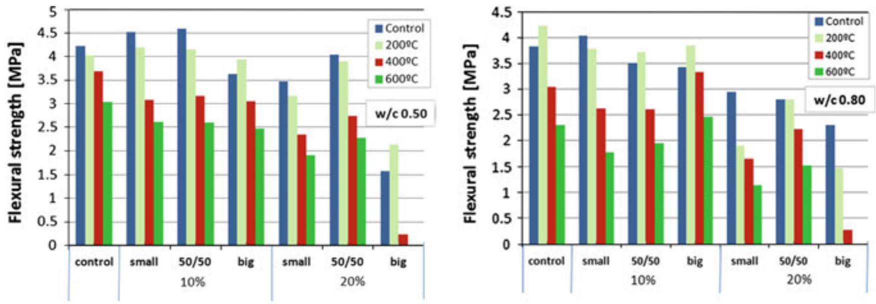


Fig. 4.43 Flexural strength behaviour of concrete with fine PET-aggregates before and after heat treatment (Albano et al. 2009)

4.6.4.1 Fire Behaviour

Albano et al. (2009) determined the fire behaviour of concrete with various percentages of shredded PET-aggregates as partial replacement of natural fine aggregates. The authors placed the cured slabs in a muffle furnace, the temperature inside the furnace was increased up to a given temperature, the slabs were kept at that temperature for 2 h and then the heating was stopped immediately. The temperatures chosen for this study were 200, 400 and 600 °C. After cooling the specimen to room temperature, the flexural strength was determined. In parallel, unheated specimens were tested. Their results are presented in Fig. 4.43.

As the temperature increased, the flexural strength decreased regardless of the substitution ratio and the PET particle size. However, the decrease in flexural strength was more significant when PET content was 20 % than 10 % due to the presence of more porosity (voids), which act as stress concentration spots. Moreover, PET-aggregates were more susceptible to temperature than natural fine aggregates. The volume change and the degradation of the PET particles produce less cohesion between concrete components and a greater number of voids. The decrease in flexural strength also increased with the w/c value. According to the authors, at high w/c value the thermal stability of PET-aggregates decreased due to the hydrolytic degradation of PET particles. The formation of carboxyl and hydroxyl end groups occurred due to the reaction of one water molecule with one PET molecule, which accelerated its decomposition. Besides, the water vapour was difficult to discharge at high temperatures, so the vapour pressure favours crack formation in concrete.

4.6.4.2 Thermophysical Properties

Mounanga et al. (2008) observed significantly low thermal conductivity for concrete with PUR-foam aggregates used to partially replace fine NA due to the porous nature of PUR-foam aggregates. These pores contain air, whose thermal conductivity is much lower than that of the other concrete constituents. The decrease in thermal conductivity was prominent for concrete with dry PUR-foam aggregates compared to concrete with saturated PUR-foam aggregates. Yesilata et al. (2009) observed an improvement of thermal insulation performance of plain concrete due to the incorporation of plastic aggregates, which was also dependent on the shape of the plastic aggregates.

The heat capacity of concrete with dry PUR-foam aggregates is lower than that of the reference concrete since the heat capacity of PUR-foam aggregates is also lower than that of the NA (Mounanga et al. 2008). On the other hand, the heat capacity of concrete with saturated PUR-foam aggregates is higher than that of the reference concrete due to the higher heat capacity of water present in the pores of prewetted PUR-foam aggregates.

4.7 Rubber Waste

Disposal of rubber tyre waste has become a serious problem due to the generation of huge amounts of tyres, which are non-biodegradable by nature. Tyre rubber in asphaltic concrete mixes, in incinerator to produce steam, to produce different plastic and rubber products, as a fuel for cement kiln, as feedstock for making carbon black, and as artificial reefs in marine environment are some attractive utilisation options (Siddique and Naik 2004). Extensive references including excellent reviews are available on the use of rubber tyre as coarse or fine aggregates or as a filler material for the preparation of various types of concrete (Kumaran et al. 2008; Siddique and Naik 2004). In this section, the properties of concrete with rubber tyre waste particles as aggregates will be discussed. The properties of these aggregates are presented in detail in Chap. 2.

4.7.1 Fresh Concrete Properties

The incorporation of rubber aggregates in concrete affects the various fresh concrete properties due to their organic nature as well as their shape, size and lightweight nature. In this section, changes in fresh concrete properties due to the addition of rubber aggregates available in the various references will be highlighted.

4.7.1.1 Slump

Just like for plastic aggregates, there are two parallel views on the workability behaviour of mortar and concrete mixes with rubber tyre aggregates. Sukontasukkul and Chaikaew (2006) observed lower slump for concrete with rubber aggregates and they added more water to obtain similar consistence to that of a conventional concrete. The water requirement increases with the rubber content and as the average particle size of the rubber aggregates decrease. Guneyisi et al. (2004) reported that the slump of concrete at two w/c values with and without silica fume gradually decreased with increasing rubber aggregates content. At a rubber content of 50 % by total aggregate volume the slump decreased near to zero and the mix was not workable so that an extra effort was required for the compaction of the concrete. The decrease in the slump was more remarkable for low w/c concrete mixes. Nayef et al. (2010) also found a near zero slump of a concrete mix with coarse rubber content of 20 % by total coarse aggregate volume and a very low slump value for a concrete mix with fine rubber aggregates (Fig. 4.44a). The slump of rubberized concrete mixes can be improved by adding 5 % microsilica. Taha et al. (2008) also observed heavy reduction in slump of concrete due to increasing substitution of NA by rubber aggregates (Fig. 4.44b).

Li et al. (2004) did not found any significant change in slump due to the replacement of 15 % coarse aggregates by rubber tyre chips or fibre. Khaloo et al. (2008) found contrasting slump behaviours of concrete mixes due to the incorporation of fine and coarse rubber tyre aggregates as a partial replacement of NA (Fig. 4.44c). The slump increased with the replacement ratio of sand by fine rubber aggregates up to 15 %, beyond which slump decreased. On the other hand, slump of concrete mixes with coarse rubber aggregates decreases to a minimum with tyre aggregates contents of 15 % and then it fluctuates slightly over the minimum value for higher rubber aggregate contents.

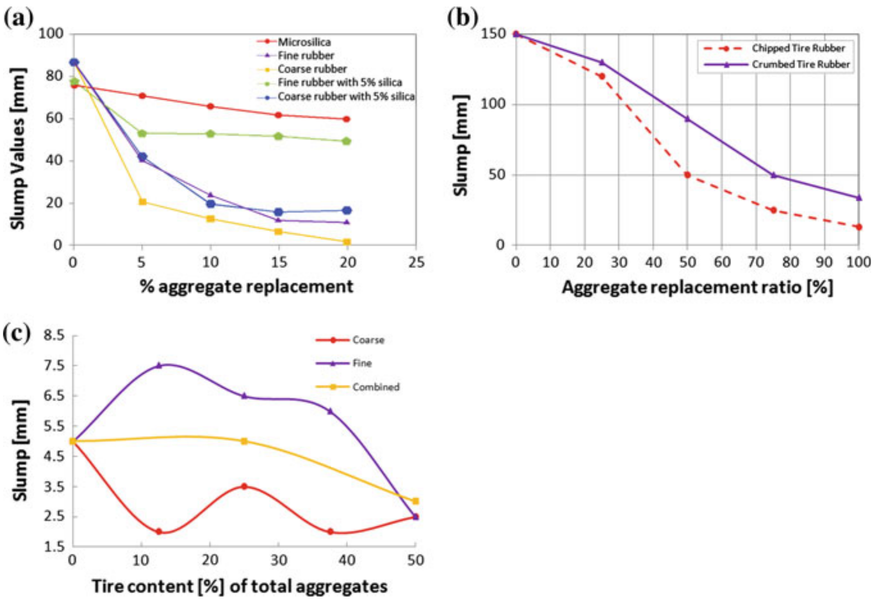


Fig. 4.44 Slump of concrete with rubber aggregates

Turki et al. (2009) reported a decreasing w/c value (for same slump) and therefore increasing slump (for same w/c value) of mortar mixes with increasing rubber aggregates replacement of sand up to 30 %. However, increasing the replacement ratio to 40 % does not change the w/c value observed for 30 %. On the other hand, the w/c value increased slightly at 50 % replacement of sand. Aiello and Leuzzi (2010) also found a slightly improved workability of fresh concrete with partial substitution of coarse or fine aggregates by rubber shreds. Raghvan et al. (1998) achieved comparable or better workability for rubber tyre aggregates cement mortar mixes than for control mortar.

In several investigations, it was reported that the rubberized concrete specimens have acceptable workability in terms of ease of handling, placement, and finishing (Khalloo et al. 2008; Li et al. 2004, Raghvan et al. 1998; Aiello and Leuzzi 2010). According to Fattuhi and Clark (1996), the process of mixing concrete with rubber by hand (i.e. manually) was easy and less strenuous than mixing concrete with natural stone aggregates. They did not encounter any problems with placing and compacting concrete with rubber aggregates. However, the workability of mixes with rubber crumbs was slightly better than that of mixes with low-grade rubber, possibly due to the small surface of rubber crumbs as well as the presence of a lesser amount of textile fibres.

4.7.1.2 Density

In general, using rubber aggregates in concrete and mortar mixes decreases their density due to the replacement of much heavier NA by lighter rubber tyre aggregates. Increasing the rubber content further reduces the density of concrete. The lightweight nature of concrete with rubber aggregates can be used for several purposes like in structures to reduce earthquake damage, architectural applications such as false facades and interior construction.

Some typical experimental results are presented in Fig. 4.45. Although there is a global consensus that the addition of rubber aggregates reduces the density of resulting concrete, large variations in the scale of the reduction in density are observed in various studies. In some investigations, a heavy reduction in density due to incorporation of rubber aggregates was reported. For example, Guneyisi et al. (2004) found that the unit weight of concrete ranged from 2427 to 1805 kg/m^3 depending on the silica fume and rubber contents. At 50 % rubber content, the unit weight was as low as about 75 % of that of the conventional concrete, irrespective of the silica fume content. Nayef et al. (2010) reported similar reduction for a concrete mix with 20 % by volume replacement of NA. In the Khaloo et al. (2008) study, the unit weight of concrete mixes with coarse, fine and coarse–fine aggregates mix at 50 % were reduced by 45, 34 and 33 %, respectively, compared to reference concrete. Fattuhi and Clark (1996) observed unit weights of concrete mix in the range of 2380–1880 kg/m^3 due to the addition of rubber aggregates in the range 0–13 % of total concrete mix.

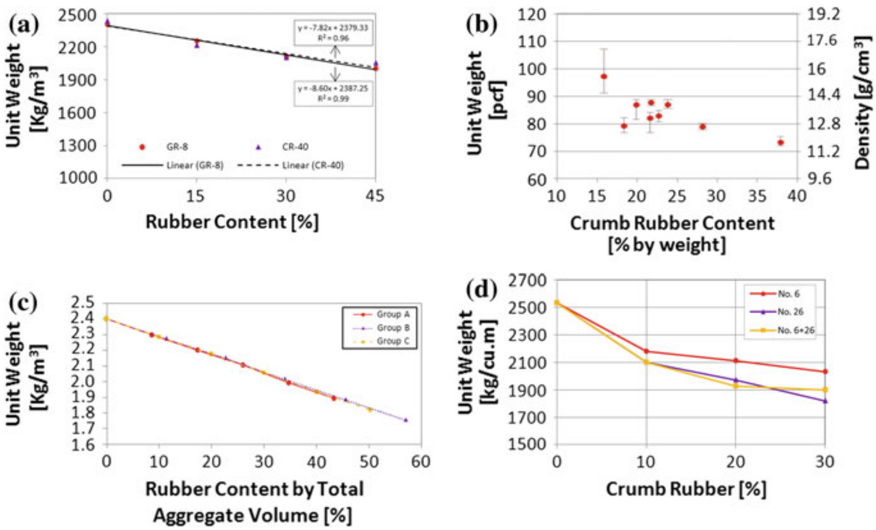


Fig. 4.45 Density and unit weight of concrete with rubber aggregates. **a** Zheng et al. 2008a; **b** Pierce and Blackwell 2003; **c** Khatib and Bayomy 1999; **d** Sukontasukkul and Chaikaew 2006

Aiello and Leuzzi (2010) observed a reduction in density of 5.8 and 6.0 % for concrete with 50 % by volume replacement of NA by coarse and fine rubber aggregates. They also observed an 8.8 and 8.3 % unit weight decrease for concrete with 75 % by volume replacement of NA by two-size fractions of rubber aggregates. Topcu (1995) also reported a 12.6 % reduction in unit weight for mixes with 45 % fine and coarse rubber chips by volume of total aggregate by comparison with the reference concrete. Ling (2011) found a density range of 2200–2000 kg/m^3 for concrete mixes with rubber aggregates content of 0–50 % by fine aggregates volume.

The size and quality of rubber aggregates also has some influence on the unit weight of rubberized concrete. In the Khaloo et al. (2008) study, a higher reduction in unit weight was observed for concrete with coarse rubber aggregates than that with fine rubber aggregates and a mixture of fine and coarse rubber aggregates. Fattuhi and Clark (1996) also reported that concrete with low-grade rubber aggregates had lower density than that with rubber crumb for similar rubber content and the difference in density increased with the rubber content. For example, at a rubber to cement ratio of 0.4 (by mass), the density of concrete with low-grade rubber was about 2 % lower than that of a similar concrete with rubber crumb. This difference may be due to the higher content of textile fibres in the low-grade rubber, and hence lower mass. On the other hand, Aiello and Leuzzi (2010) observed little difference in the density of concrete mixes prepared by partially replacing NA by fine and coarse rubber aggregates.

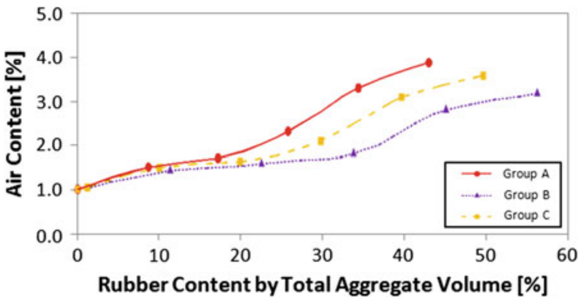


Fig. 4.46 Change in air content due to the addition of rubber aggregates in concrete: *Group A* fine aggregates replaced by crumb rubber, *Group B* coarse aggregates replaced by rubber chips, *Group C* fine and coarse aggregates replaced by crumb and chip rubber, respectively (Khatib and Bayomy 1999)

4.7.1.3 Air Content

The air content of concrete mixes with rubber particles is generally higher than that of conventional concrete with NA and it increases with the amount of rubber particles (Khatib and Bayomy 1999). Li et al. (2004) observed an increasing trend of air content of a concrete mix where 15 % by volume of natural coarse aggregates were replaced by rubber chips or fibres. Benazzouk et al. (2006) also reported a sharp increase in the air content of cement paste due to the incorporation of rubber particles. On the other hand, Figueiredo and Mavroulidou (2007) observed a reduction in air content of concrete due to the incorporation of crumb and fine rubber aggregates used to replace 10 % of coarse and fine aggregates, respectively. A typical air content behaviour of concrete due to the addition of rubber aggregate is presented in Fig. 4.46.

4.7.2 Hardened Concrete Properties

4.7.2.1 Dry Density

As the dry density of rubber aggregates is considerably lower than that of NA, similarly to fresh density, the dry density of hardened concrete decreases with increasing rubber aggregates content. Typical data are presented in Fig. 4.47.

Topcu (1995) observed a systematic decrease in the density of concrete with increasing contents of tyre chips. The average density of control concrete was 2300 kg/m^3. On the other hand, the values for concrete with 15, 30 and 40 % by volume replacement of fine and NA were 2220, 2140 and 2010 kg/m^3, respectively. Benazzouk et al. (2006) found 22–35 % reductions in 28-day dry density of concrete specimens with two types of rubber aggregates. The decrease in density was higher for concrete with small-size rubber aggregates. The effect was also more significant with expanded rubber type.

An increase in rubber aggregates content in concrete increases the air content which in turn reduces the dry density of the specimens (Yilmaz and Degirmenci

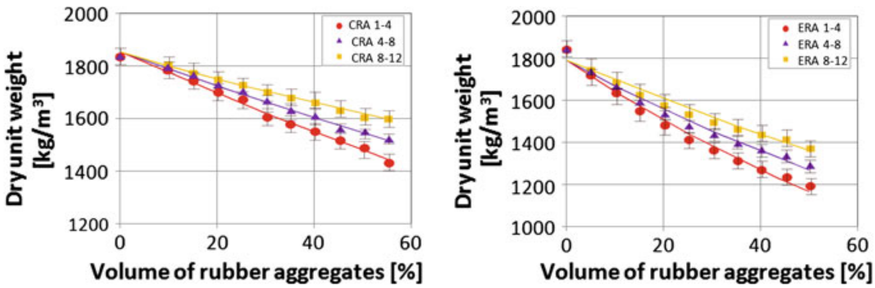


Fig. 4.47 Dry unit weight of rubberized concrete (Benazzouk et al. 2003)

2009). Sukontasukkul and Chaikaew (2006) speculated that the flocculation of the rubber particles during mixing of concrete with higher rubber contents may have some effect on the lower density of concrete specimens. Flocculation can create large voids inside the block, leading to a higher porosity which ultimately lowers the density. Due to the high water absorption of tyre particles, the ratio between the fresh density and the hardened density in rubber tyre concrete is greater than in conventional concrete. Therefore, rubber tyre concrete is expected to be more porous than conventional concrete.

4.7.2.2 Compressive Strength

It is universally accepted that the addition of rubber aggregates reduces the compressive strength of the resulting concrete and the increase of rubber content further deteriorates the compressive strength. In some of the studies, about 80–90 % reduction in compressive strength was reported depending on the size and type of rubber aggregates. Khatib and Bayomy (1999) observed about 90 % lower compressive strength in concrete with 100 % gravel replaced by chipped rubber than in conventional concrete. However, in a few studies like that of Ganjian et al. (2009) a slight improvement of 28-day compressive strength of concrete with rubber chips used to replace coarse NA in 5 % by volume was also reported, possibly due to the improvement of aggregates grading curve due to the incorporation of rubber particles.

Possible reasons for this strength reduction are (Ganjian et al. 2009): (i) reduction of the quantity of solid load-carrying material with increasing rubber content; (ii) the soft and smooth surfaces of rubber particles may significantly degrade the adhesion between the boundaries of rubber particles and cement paste, and thus increase the volume of the weakest phase and ITZ; (iii) non-uniform distribution of rubber particles at the concrete top surface tends to produce non-homogeneous samples and leads to a reduction in concrete strength at those parts, resulting in failure at lower stresses.

The compressive strength behaviour of concrete due to the incorporation of rubber aggregates reported in the literature is presented in Table 4.15. These

Table 4.15 Compressive strength of concrete with rubber aggregates

Reference	Size of aggregates/type of replacement	Type of concrete	Amount of replacement (%)	Compressive strength (MPa)
Aiello and Leuzzi (2010)	12.5–20 mm/volume	Normal	0	45.80
			25	23.90
			50	20.87
			75	17.42
	10–12.5 mm/volume	Normal	0	27.11
			15	23.97
			30	20.41
			50	19.45
			75	17.06
Bignossi and Sandrolini (2006)	Sand/volume	SCC	0	33.0
			22.2	24.7
			33.3	20.2
Emiroglu et al. (2007)	0–4 mm/volume	Normal	0	45.69
			5	41.71
			10	33.69
			15	24.75
			20	22.14
	4–8 mm/volume	Normal	0	45.69
			5	42.49
			10	37.30
			15	26.96
			20	23.91
Futtuhi and Clark (1996)[a]	Low grade rubber/mass	Normal	0	37.45
			~9.9	12.66
	Rubber crumb	Normal	~11.2	11.69

[a] Amount of replacement with respect to total solid content

results indicate that the size, proportions and surface textures of rubber particles noticeably affect the compressive strength of rubberized concrete mixes (Eldin and Senouci 1993; Topcu 1995). Gesoglu and Guneyisi (2007) observed a relatively higher strength development between 3 and 7 days of curing and the rate gradually decreased with curing age. However, the strength development pattern was almost similar in conventional concrete.

Benazzouk et al. (2003) found a sharp reduction in 28-day compressive strength of concrete due to the addition of different size fractions of two types of rubber aggregates. Some results of their investigation are presented in Table 4.16. They also found a high dependency of strength on several parameters such as substitution ratio, size and properties of rubber aggregates. The compressive strength of concrete specimens prepared by using compacted rubber aggregates was considerably higher than that using expanded rubber aggregates. Similarly, compressive strength decreased drastically when the content of rubber aggregates increased.

Table 4.16 28-day compressive strength of concrete with two types of rubber aggregates (Benazzouk et al. 2003)

Volume of rubber (%)	Size of rubber aggregates	Compressive strength (MPa)	
		CRA	ERA
0	–	82.5	82.5
5	1–4	68.0	59.0
	4–8	63.0	54.0
	8–12	60.5	51.0
10	1–4	55.0	42.0
	4–8	48.0	36.0
	8–12	43.0	32.0
25	1–4	26.0	15.0
	4–8	20.0	13.0
	8–12	15.0	11.0
50	1–4	6.5	3.4
	4–8	5.0	2.6
	8–12	3.5	2.0

CRA compacted rubber aggregates; ERA: expanded rubber aggregates

The rubber particles are less stiff than the surrounding cement paste which lowers the compressive strength of concrete. The cracks are initiated around the rubber particles, which accelerates the failure in the matrix. Larger incorporation of rubber particles in the concrete mix creates difficulty in the packing of lightweight rubber particles, and therefore voids are introduced in the matrix. The trend is slightly influenced by aggregate size; e.g. for a given amount of rubber, finer aggregates lead to lower losses in compressive strength than coarse aggregates.

Similar effect of particle size on the strength behaviours of rubberized concrete was also reported in other studies (Topcu 1995; Son et al. 2011; Khatib and Bayomy 1999; Ali and Goulias 1998; Ali et al. 1993). Khatib and Bayomy (1999) showed that rubberized concrete made with coarse chipped rubber replacing coarse aggregates has less strength than concrete made with fine crumb rubber. Ali and Goulias (1998) and Ali et al. (1993) also observed higher reduction in compressive strength due to the addition of coarse sized rubber aggregates than of fine rubber particles. This is due to the high compressibility of rubber particles, which generates localised stresses and bonding problems between them and the cement matrix. According to Topcu (1995), the interfacial bond in a coarse tyre rubber chips cement paste is weaker than in a fine tyre rubber chips cement paste, which ultimately affects the compressive strength. However, in some studies such as that of Emiroglu et al. (2007) the exact opposite effect of particle size is reported (Table 4.15).

Li et al. (2004) reported that the compressive strength of concrete with rubber chips replacing 15 % by volume of coarse NA was lower than that of concrete with an equal volume percentage of elongated or fibre type coarse rubber aggregates (Table 4.15). According to the authors, this is possibly due to the difference between their load transfer capabilities. Once debonded from the concrete matrix, chips do not have enough length to transfer the applied load through interfacial frictional force, while fibres have longer length to transfer the applied load, resulting in higher strength.

In a few studies, the effect of chemical or physical treatments of rubber aggregates on the compressive strength behaviour of the resulting concrete was also reported. This type of technique is generally adopted to improve the weak ITZ between rubber aggregates and cement paste. Li et al. (2004) reported that the surface treatment of fine rubber aggregates by NaOH solution increased the mechanical performance including compressive strength. However, this technique could not improve the properties for coarse rubber aggregates. The same author also tried to improve the mechanical performance by making holes in the rubber aggregates but it did not improve the studied properties. Naik and Singh (1991) also reported that the surface treatments of rubber particles could enhance the hydrophilicity of the rubber surface and therefore could improve mechanical performance including compressive strength.

4.7.2.3 Tensile Strength

Just like for compressive strength, the addition of rubber aggregates decreases the splitting tensile strength of the resulting concrete. The development of microcracks due to weak interfacial binding of rubber aggregates and cement paste as well as a surface segregation between rubber aggregates and cement paste due to the exerted stress are the major causes that lower the tensile strength of concrete due to the incorporation of rubber aggregates (Ganjian et al. 2009). However, for a given substitution ratio the reduction in splitting tensile strength of concrete with rubber aggregates is less prominent than that observed in compressive strength (Eldin and Senoucci 1993; Mavroulidou and Figueiredo 2010). The reduction in splitting tensile strength of concrete with fine rubber aggregates is also smaller than that with coarse rubber aggregates.

Ganjian et al. (2009) reported that the percentage reduction of tensile strength in concrete using chipped rubber as a partial replacement of NA was about twice than that in concrete using ground rubber particles for the same replacement level. The reduction in tensile strength with 7.5 % replacement was 44 % for concrete with chipped rubber and 24 % for concrete with ground rubber as compared to the control mix. In the Topcu (1995) study, the splitting tensile strength of C 20 type conventional concrete was 3.21 MPa, while it was 2.17, 1.53 and 1.13 MPa for concrete with fine rubber chips and, 1.50, 1.06 and 0.82 MPa for concrete with coarse rubber chips at the replacement ratios of 15, 30 and 45 %, respectively.

Instead of the brittle failure usually exhibited by conventional concrete specimens under compression, specimens with rubber aggregates generally show ductile failure due to the plastic behaviour of the rubber aggregates. Topcu (1995) found that the failed specimens withstood measurable post-failure loads during tensile strength test and underwent significant displacement, which was partially recoverable. Therefore, concrete specimens with rubber aggregates showed high capacity of absorbing plastic energy during the splitting tensile strength test. A similar type of tensile behaviour was reported by Eldin and Senoucci (1993). Kang et al. (2009) reported that the incorporation of rubber particles in RCC increased the tensile strength, as well as the ultimate tension elongation if the compressive strength was kept at the level of about 40 MPa. This was due to the higher deformation capability and lower modulus of elasticity of rubber particles than those of NA.

Kang et al. (2009) reported that the splitting tensile strength of concrete specimens is about one-tenth to one-fifteenth of cubic compressive strength.

4.7.2.4 Flexural Strength

The incorporation of rubber aggregate decreases the flexural strength of the resulting concrete and rising the rubber content further deteriorates the flexural strength due to the weak bond between cement paste and rubber particles. However, Benazzouk et al. (2003) observed higher flexural strength values for concrete prepared by replacing 20 % by volume of coarse and fine aggregates by two types of rubber aggregates with three size ranges than for conventional concrete. However, after substitution of 35 % by volume of NA by any type of rubber aggregates and any size range, the flexural strength decreased drastically due to the rupture of the rubber/cement matrix connection. Concrete with expanded rubber aggregate showed better flexural strength behaviour than concrete with compacted rubber aggregates, which was exactly the opposite trend of compressive strength of concrete with these aggregates.

In several studies, it was reported that the reduction in flexural strength was not as significant as that observed in the reduction of compressive strength of concrete due to the incorporation of rubber aggregates (Mavroulidou and Figueiredo 2010; Toutanji 1996). Toutanji (1996) found a significantly smaller reduction in flexural strength in comparison to compressive strength as the tyre chip content increased. Khatib and Bayomy (1999) observed a steeper initial rate of flexural strength reduction than that of compressive strength.

From the load–deflection curves during flexural strength measurement of concrete beam specimens with various amounts of rubber, several authors reported that the failure of specimens with rubber tyre chips exhibited a ductile mode of failure as compared to control specimens (Toutanji 1996; Sukontasukkul and Chaikaew 2006). The specimens with rubber could also withstand measurable post-failure loads due to the ability of the rubber aggregates to undergo large elastic deformation before the failure of the specimen took place.

Aiello and Leuzzi (2010) observed a larger reduction in flexural strength for concrete when coarse aggregates rather than fine aggregates were replaced by rubber particles. The rubberized concrete mixes prepared with 50 and 75 % by volume of coarse NA replacement both exhibited a decrease in flexural strength, referred to the control mix, of about 28 %. Whereas, mixes with substitution by volume of fine aggregates of 50 and 75 % showed a decay of about 5.8 and 7.3 %,

Table 4.17 Dynamic and static elastic moduli of concrete with ground and crushed rubber as coarse aggregates replacement (Zheng et al. 2008a)

Properties	Conventional	Concrete prepared by replacing coarse aggregates with					
		Ground rubber in volume (%)			Crushed rubber in volume (%)		
		15	30	45	15	30	45
E_d (GPa)	43.7	41.2	35.2	31.2	35.4	36.5	32.8
E_s (GPa)	31.8	27.1	24.1	22.3	23.1	24.3	22.1

respectively, referred to the control mix. A decrease in flexural strength with the increase in particle size of rubber aggregates was reported in other studies too (Benazzouk et al. 2003; Mavroulidou and Figueiredo 2010).

4.7.2.5 Modulus of Elasticity

Just like strength properties, the incorporation of crumb or chip rubber as aggregates in concrete considerably reduces both the static and dynamic moduli of elasticity. Aggregates characteristics affect the modulus of elasticity: concrete with aggregates with higher stiffness normally has high modulus of elasticity. Since the rubber aggregates have very low stiffness as compared to NA, the addition of rubber aggregates lowers the modulus of elasticity of the resulting concrete.

The type of rubber (i.e. chips or ground rubber) may have some effect on the modulus of elasticity. Zheng et al. (2008a) reported higher values of both static and dynamic moduli for concrete with 15 % by volume of coarse aggregate replaced by ground rubber than for concrete with crushed rubber at similar replacement level (Table 4.17). However, at higher replacement level the elasticity behaviour of concrete with ground and crushed rubber aggregates becomes similar. Skripkiunas et al. (2007) observed a reduction of about 11 % in the modulus of elasticity of concrete due to the addition of rubber aggregates that replaced fine aggregate by about 3 % by weight. Mavroulidou and Figueiredo (2010) observed a higher static modulus of elasticity for concrete with coarse rubber aggregates (19–10 mm) than for concrete incorporating finer rubber aggregates (10–4.75 mm). Both types of aggregates were used to replace 10 % by weight of natural coarse aggregates.

Azmi et al. (2008) found reductions in the modulus of elasticity with increasing rubber aggregates content in concrete as well as with increasing w/c value. The authors found a reduction of about 30 % in modulus of elasticity when the replacement ratio of fine aggregates by crumb rubber increased from 0 to 30 % by volume. According to the authors, the inclusion of crumb rubber implies defects in the internal structure of the composite material, producing a reduction of strength and stiffness. Benazzouk et al. (2003) reported that the decrease in dynamic modulus of elasticity was greater with expanded type rubber aggregates compared with compacted rubber aggregates for the same size and same amount of rubber content. Ganjian et al. (2009) reported lower modulus of elasticity for concrete with rubber aggregates than for conventional concrete. Kang et al. (2009) also observed a reduction in modulus of elasticity with increasing rubber content; however, the modulus of elasticity increased with the curing time. The reduction amount with respect to the modulus of elasticity of conventional concrete was slightly low at all substitution levels.

Guenisiyi et al. (2004) reported that the static modulus of elasticity of concrete decreased with increasing rubber content in a similar fashion to that observed in compressive and splitting tensile strengths. By increasing the rubber content to 50 % of the total aggregate volume, the modulus of elasticity dropped to about 6.5 and 8.0 GPa for w/c ratios of 0.60 and 0.40, respectively. These were respectively about 20 and 17 % of the modulus of elasticity of a similar type of conventional concrete. The use of silica fume slightly improved the modulus of elasticity of concrete even though the improvement was smaller than that observed for compressive and splitting tensile strengths. The results are presented in Fig. 4.48. Peisller et al. (2011) observed lower modulus of elasticity for concrete with various percentages of rubber aggregate used to replace fine NA in concrete. The reduction in the modulus of elasticity was smaller than that in compressive strength. The decrease in modulus of elasticity was 49 % on an average for the concrete with rubber by comparison with the reference concrete. The addition of 15 % silica fume with cement increased the modulus of elasticity but it was still lower than that observed for conventional concrete.

4.7.2.6 Stress–Strain Curve: Toughness Behaviour

It is consensual that the addition of rubber aggregates can substantially improve the post-cracking behaviour of concrete by absorbing a significant amount of energy. Thus special types of concrete can be prepared by incorporating rubber aggregate that can be used for applications where impact or blast resistance is

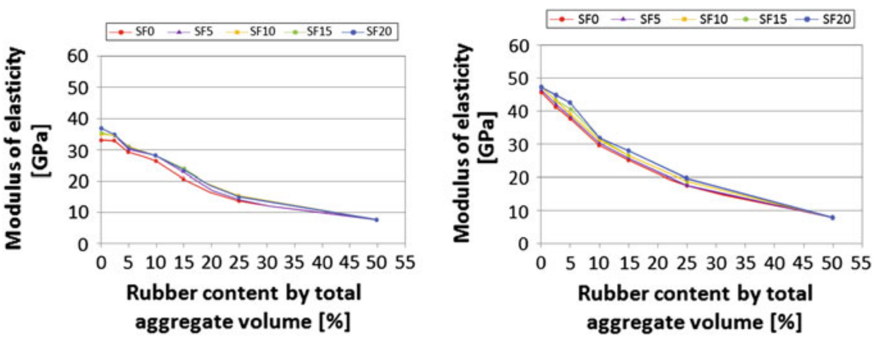


Fig. 4.48 Static modulus of elasticity of concrete with various percentages of rubber aggregates incorporation (Guneyisi et al. 2004)

Table 4.18 Toughness indices and some other parameters of rubberized concrete (Aiello and Leuzzi 2010)

Amount of rubber in concrete (%)	Toughness indices[a]			Residual strength factor[a]		Toughness (kN/mm^3)
	I_5	I_{10}	I_{20}	$R_{5,10}$	$R_{10,20}$	
50	4.06	8.72	14.4	93.2	56.8	113
75	4.96	9.92	17.8	99.2	78.8	196

[a] For details, see ASTM C1018-97

needed, such as bunkers and jersey barriers, or where vibration damping is required such as foundation pads in railway stations. Due to the positive influence of rubber aggregate, substantial work has been done to evaluate the stress–strain curve and the toughness behaviour of concrete with rubber aggregates. This behaviour is generally evaluated during the determination of various strength properties.

Aiello and Leuzzi (2010) observed substantial improvement of post-cracking behaviour of concrete due to the addition of coarse rubber aggregates. In Table 4.18, the toughness indices and energy absorption capacities (toughness) measured during the determination of the flexural strength of concrete with rubber aggregates used to replace 50 and 75 % by volume of natural coarse aggregates are presented. The toughness indices determined from the curves were in the specified limit of the standard range defined in ASTM C1018-97 and these increased with rubber content. However, in the same investigation insignificant enhancement of toughness behaviour due to the incorporation of fine rubber aggregates used to replace 25 and 50 % by volume of natural fine aggregates in concrete was also reported.

Batayneh et al. (2008) also found two distinct behaviours in the stress–strain curves of concrete depending on rubber content (Fig. 4.49). 0.075–4.75 mm rubber aggregates were used to replace fine NA in concrete. The stress–strain behaviour of specimens with rubber content up to 40 % follows a trend similar to that of the control specimen. In this case, concrete behaved like a brittle material i.e. there was a linear increase of stress until it reached its peak value before specimen's fracture. However, the curves became nonlinear for concrete mixes with 60 and 80 % rubber, which indicated that concrete behaves like a ductile material. Kang et al. (2009) observed a similar type of ductility behaviour for concrete with shredded rubber aggregates. Concrete with rubber aggregates did not disintegrate and some cracks closed after unloading.

Benazzouk et al. (2003) also observed increasing ductility in the stress–strain curve of concrete due to increasing addition of rubber aggregate as well as due to increasing particle size of rubber aggregates. The brittleness index (BI) was also measured to estimate the ductility of different concrete specimens. These values for different mixes as a function of rubber aggregates volume are presented in Fig. 4.50. The peak was obtained at a rubber addition level of 10 % for all aggregate sizes and characterised the transition from brittle to ductile material after

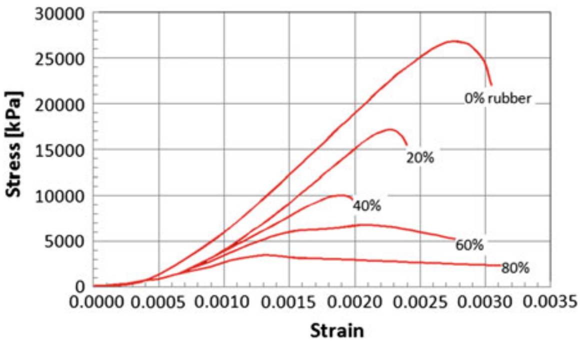


Fig. 4.49 Stress-strain behaviour of concrete with various percentage of rubber aggregates (Batayneh et al. 2008)

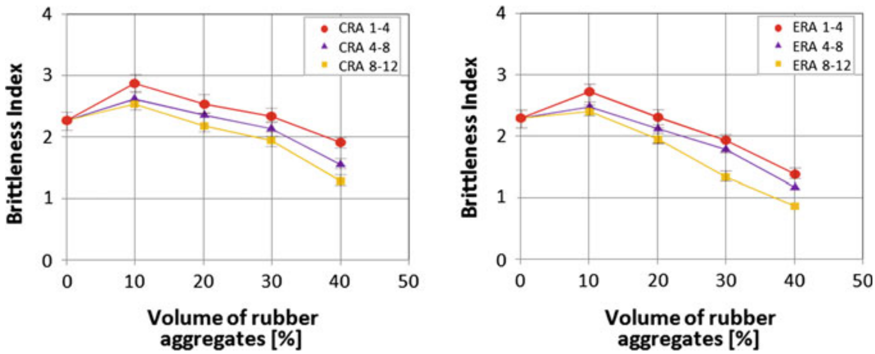


Fig. 4.50 Brittleness index of various types of concrete (Benazzouk et al. 2003)

this rubber content. The decrease in BI values with rubber content over 10 % reflected an increase in plastic deformation energy. This increase became even greater as the rubber size increased. For the same rubber content, the BI was lower for expanding type rubber aggregates than for compacted rubber aggregates. The alveolar character of rubber, therefore, helped to increase the deformability of cement–rubber composites.

Khaloo et al. (2008) observed increasing nonlinearity of stress–strain curves due to the incorporation of rubber aggregates in concrete. To compare the non-linearity between the control concrete and the rubber tyre concrete, a nonlinearity index was defined as the ratio between the slope of the line connecting the origin to 40 % of the ultimate stress and the slope of the line connecting the origin to the ultimate stress. A higher nonlinearity index implies a more nonlinear stress–strain curve. The nonlinearity index increases as the rubber content increases for all mixes. The substitution of rubber for mineral aggregates appears to allow more uniform crack development and provide gentler crack propagation, compared to conventional concrete. The authors also determined the toughness indices of the concrete mixes. Rubber tyre concrete exhibited greater toughness as compared to conventional concrete. Toughness indices maximise as rubber concentration approaches 25 % of the total aggregates volume. Beyond rubber concentrations of 25 %, toughness indices decrease due to the systematic reduction in strength.

4.7.2.7 Impact Resistance

Futtuhi and Clark (1996) evaluated the impact resistance of concrete with low-grade rubber, which contains textile fibres and dust as impurities. Two slabs were made and tested simultaneously. One slab was made with ordinary concrete (without rubber), while the other contained about 11 % of low-grade rubber relative to the total solids content by weight. The rubber to cement ratio was maintained at about 0.44. After impact by a hammer, examination of the slabs showed that both suffered cracking in all directions. However, the slab with rubber had a larger spread of cracks over the tension face. After the second hit, the maximum crack width in the ordinary concrete slab was 0.16 mm, while that for the slab with rubber was 0.50 mm. After the third hit, the maximum crack widths (at the same locations) increased to 0.3 and 2.0 mm for the plain concrete and rubberized concrete slabs, respectively. These results show that both slabs sustained the impact of the drop hammer; despite the compressive strength of the rubberized concrete slab being about 30 % of the strength of the ordinary concrete slab.

Ling et al. (2009) reported that using rubber aggregates as partial substitution of fine aggregates in concrete pavement blocks improved the impact resistance. Their results are presented in Table 4.19. They reported that the energy absorption and toughness of rubberized blocks were much larger than those of the control block. Extra forces were needed to fully open the blocks with high amounts of rubber aggregates because they maintained the integrity of the broken pieces even after a number of falling weight hits. The rubberized blocks exhibited higher displacement and did not show any clean split into two halves at failure mode as the rubber content increased.

Table 4.19 Numbers of hits that cause damage in concrete pavement blocks

Type of concrete	Type of damage							
	Small crack		Transverse crack		All directions crack		Completely broken	
	Block 1	Block 2	Block 1	Block 2	Block 1	Block 2	Block 1	Block 2
Type I	1	2	2	3	–	–	3	4
Type II	3	3	6	5	–	–	10	8
Type III	7	7	–	–	13	12	13	12
Type IV	9	10	–	–	15	16	15	16

Concrete slab prepared by replacing fine aggregates fraction by rubber aggregates that is used to replace 0, 10, 20 and 30 % by volume of total aggregates (type I, type II, type III and type IV, respectively)

Drop height 100 cm (Ling et al. 2009)

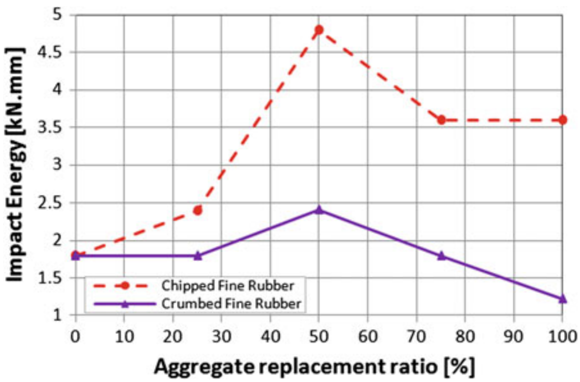


Fig. 4.51 Effect of rubber aggregates incorporation on the impact energy of concrete

Taha et al. (2008) evaluated the impact energy (the energy required to failure) of concrete prepared by replacing various percentages of coarse and fine aggregates, measured through an impact test. Their results suggested that increasing the replacement level of coarse aggregates by chipped tyre rubber particles up to 100 % significantly improved the impact energy of the concrete and that a maximum was reached for a replacement level of 50 % (Fig. 4.51). The impact resistance also peaked at 50 % replacement level for mixes with crumbed tyre rubber particles. However, it was lower than for the control mix at 75 and 100 % replacement levels. At low to medium replacement levels, the low stiffness of the tyre particles allowed the rubber-cement composite to have a relatively high flexibility, and thus absorb higher amount of energy than the conventional concrete.

4.7.2.8 Skid Resistance

Sukontasukol and Chaikaew (2006) evaluated the skid resistance of concrete using standard ASTM E303-93 and a pendulum type apparatus. 10 and 20 % by weight of coarse and fine aggregates of concrete were replaced by two sizes of crumb rubber (passing ASTM No. 6 and 20 sieves) and by a mixture of these two sizes. The results of the seven mixes are presented in Fig. 4.52a. Results show that crumb rubber concrete blocks (except those made with sieve No. 20 crumb rubber) exhibited better skid resistance than the control block. The highly elastic properties of rubber allowed the block surface to deform more and create more friction as the pendulum passed across it. Mixes with large rubber particles performed better than those with small particles.

On the other hand, a systematic reduction in skid resistance was observed for concrete slab where the fine aggregates were replaced by rubber aggregates at 0, 10, 20 and 30 % by volume of total aggregates (types I, II, III and IV respectively in Fig. 4.52b) (Ling et al. 2009). However, all the values met the minimum requirement in accordance with ASTM standard specification. No damage was

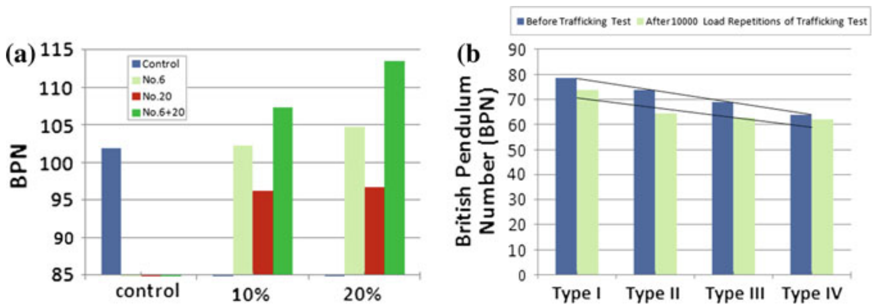


Fig. 4.52 Skid resistance of **a** concrete with rubber aggregates (Sukontasukkul and Chaikaew 2006); **b** concrete pavement block (Ling et al. 2009)

caused to any of the pavement blocks. The slightly higher skid resistance for low percentages of crumb rubber in pavement block was partly due to the rough surface texture of the paving blocks.

4.7.2.9 Abrasion Resistance

Sukontasukol and Chaikaew (2006) reported the abrasion resistance of concrete with various contents of rubber aggregates. The results in terms of percentile weight loss are shown in Fig. 4.53a. They found lower abrasion resistance for mixes with rubber crumb aggregates than for the control mix, as indicated by increasing weight loss with increasing crumb rubber content. Out of three types of rubber aggregates, abrasion resistance was lowest for the mixes of two-size fractions rubber and highest for the coarse rubber aggregates. Segre and Joekes (2000) reported that the NaOH treatment of rubber aggregates considerably improved the abrasion behaviour of rubberized concrete (Fig. 4.53b).

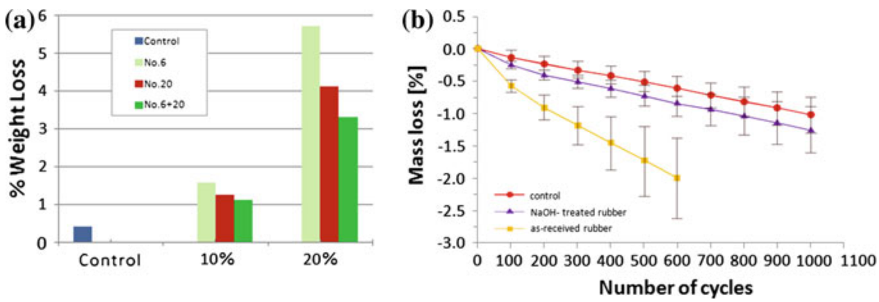


Fig. 4.53 Abrasion resistance of normal and rubberized concrete mixes

Table 4.20 Plastic shrinkage cracking behaviour of mortar (Raghvan et al. 1998)

Type of addition	Amount of cement replaced (%)	Number of cracks	Crack length (mm)			Average crack with (mm)			Time of first crack (min)
			1 h	2 h	3 h	1 h	2 h	3 h	
None	0	1	158	212	246	0.3	0.6	0.9	2
RS4.75	5	2	174	212	212	0.2	0.4	0.6	30
	10	2	156	203	203	0.2	0.4	0.4	60
	15	4	103	142	178	0.2	0.3	0.4	60
RS2.36	15	4	163	181	203	0.2	0.3	0.3	35
GR	15	3	107	204	219	0.2	0.2	0.4	45

4.7.3 Durability Parameters

Several durability parameters of concrete with rubber aggregates were reported in the literature. These include shrinkage, water absorption and water sorptivity, water and chloride permeability, and freeze–thaw resistance.

4.7.3.1 Drying Shrinkage

The incorporation of rubber aggregates decreases the drying shrinkage of concrete and increasing the amount of rubber aggregates further decreases it.

Raghvan et al. (1998) evaluated the plastic shrinkage of mortar with mass fractions of 0, 5, 10 and 15 % rubber shred with a size range of 4.75–2.36 mm. At 15 % rubber content, a fine fraction of rubber shred with size range of 2.36–1.18 mm and a fraction of granular rubber with about 2 mm diameter were also used to evaluate the properties. The width of the cracks for all the mixes was measured at 1, 2 and 3 h in the drying chamber. The results are summarised in Table 4.20. All the specimens cracked within the first 3 h of exposure and the cracks always occurred over the central stress raiser. After 3 h, the control mortar specimen developed a crack with an average width of about 0.9 mm, while the average crack width for the specimens with 5–15 % rubber shreds was 0.4–0.6 mm. The number of crack also increased due to the addition of rubber in cement mortar. The onset of cracking was delayed by the addition of rubber shreds. The content of rubber shreds in the mortar affected the onset time of cracking, the crack length, and the crack width.

Kang et al. (2009) reported that RCC with different contents of rubber tyre aggregates exhibited a shrinkage pattern similar to that of conventional concrete and the drying shrinkage developed at a higher rate in the first month than later on (Fig. 4.54). They found almost similar shrinkage for RCC with 50 kg/m^3 rubber aggregates and NA. However, higher shrinkage was recorded for rubber contents of 100 and 120 kg/m^3. Uygunoglu and Topcu (2010) reported lower drying

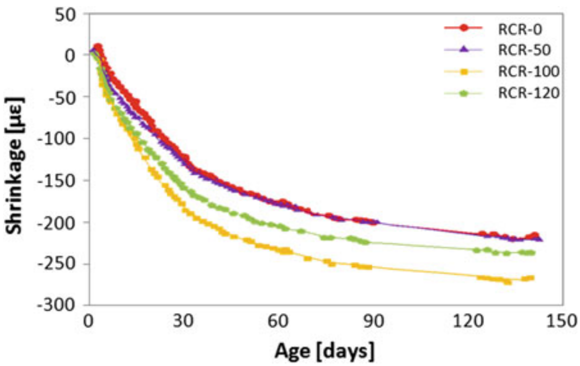


Fig. 4.54 Drying shrinkage of concrete with various amounts of rubber aggregates (Kang et al. 2009)

shrinkage of self-consolidating mortar prepared by replacing 10 and 20 % of sand by rubber aggregates. However, at higher substitution levels it increases sharply.

Ho et al. (2009), from a test according to ASTM standard C 1581-04, reported that the incorporation of rubber aggregate in concrete reduced the sensitivity of concrete to cracking due to shrinkage-related length change. This was due to the enhanced strain capacity of rubberized concrete. The compressive strain developed in the steel ring caused by the restrained shrinkage of the concrete specimen measured from the time of casting show that in comparison with the control concrete, the development of compressive strain in the steel ring slowed down for rubberized concretes, which confirms the stress relaxation resulting from the presence of rubber particles. The incorporation of rubber into concrete delayed the time of crack initiation and increasing rubber amounts further delayed it. These results are presented in Table 4.21.

4.7.3.2 Water Absorption

The amount of water absorbed is related to the porosity of the test specimens and gives an insight of the internal microstructure. Several reports are available on the water absorption behaviour of concrete due to incorporation of rubber aggregates.

The water absorption behaviour of concrete with rubber aggregates depends on their particle size. In general, the presence of large size rubber aggregates

Table 4.21 Effect of rubber aggregate on the cracking potential of concrete (Ho et al. 2009)

Mix	Time to cracking (day)	Average stress rate (MPa/day)	Potential for cracking[a]
C0R	9.25	0.39	High
C20R	15.50	0.16	Moderate-low
C40R	33.25	0.05	Low

[a] According to ASTM C1581-04

C0R conventional concrete

C20R and *C40R* concrete prepared by replacing 20 and 40 % of sand by an equal volume of rubber aggregates

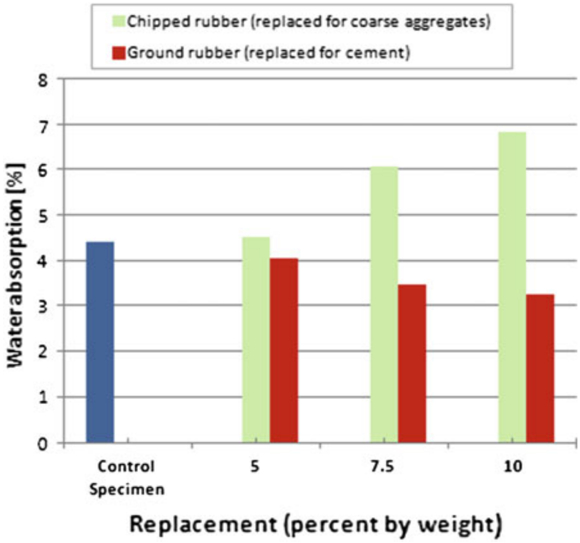


Fig. 4.55 Water absorption of concrete with rubber aggregates (Ganjian et al. 2009)

increases the water absorption of the resulting concrete due to the weak cement paste-rubber aggregates interactions, which is presented in Fig. 4.55 (Ganjian et al. 2009). Bignozzi and Sandrolini (2006) observed a slight increase in the water absorption of self-compacting concrete due to the incorporation of fine rubber aggregates, possibly due to deviations of rubber particles size from sand grain size distribution and an increase in air amount trapped during mixing procedures. Uygunoglu and Topcu (2010) also observed higher water absorption of self-consolidating cement mortar due to partial substitution of fine aggregates by rubber aggregates and these values further increased with the water to powder ratio. On the other hand, fine rubber particles can reduce the water absorbed. According to some authors, rubber particles do not absorb water, which ultimately lowers the amount of water absorbed (Yilmaz and Degirmenci 2009; Segre and Joekes 2000). Some authors argued that water reduction was due to a reduction in porosity of concrete as fine rubber particles filled the voids (Ganjian et al. 2009). Gesoglu and Guneyisi (2011) reported that the addition of FA can also reduce the amount of water absorbed in rubberized concrete due to the filling effect of FA at early ages and its pozzolanic reaction at later ages.

Segere et al. (2003) reported that using rubber aggregates at partial substitution of sand reduced the capillary water absorption (Fig. 4.56a). Rubber aggregate treated with NaOH can further improve the capillary water absorption behaviour (i.e. lower the absorption) of the resulting concrete due to an improvement in binding between rubber aggregates and cement paste. The authors reported a sorptivity coefficient of 0.29 mm/min$^{1/2}$ for conventional mortar and 0.06 mm/min$^{1/2}$ for the mortar with 10 % rubber particles. Benazzouk et al. (2007) also reported a decrease in capillary water absorption and in water absorption rate of cement composites with an increase in rubber content, which may be due to the

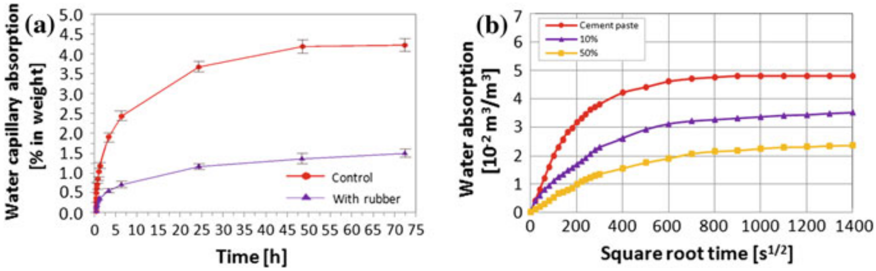


Fig. 4.56 Capillary water absorption of rubber-cement composites

capability of rubber to repel water (non-sorptive nature) and to an increase of air-entrainment, as manifested by closed empty pores that are not accessible to water (Fig. 4.56b). The decrease in water absorption is also attributed to a reduction in the porosity near particle/matrix interfacial zone, due to the high bonding between rubber aggregates and cement paste.

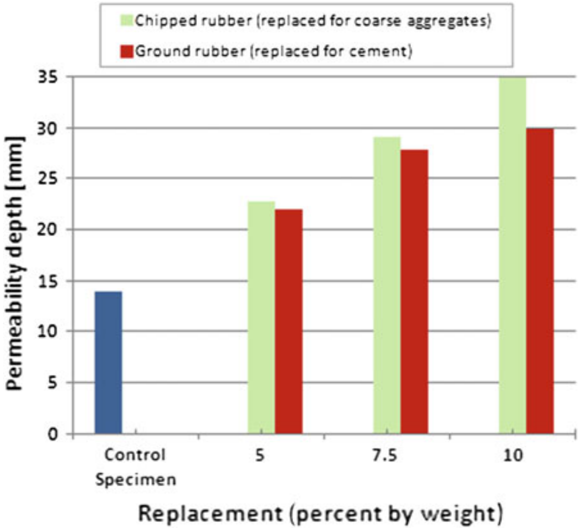
Bennazzouk et al. (2007) determined the sorptivity of cement composites with rubber particles. Value decreased from 0.193×10^{-3} m/s$^{1/2}$ for cement paste to 0.037×10^{-3} m/s$^{1/2}$ for specimens with 50 % of shredded rubber particles. The water sorptivity of concrete with crumb rubber aggregates was higher than that of composites with fine rubber particles; however, these values are lower than for conventional concrete. On the other hand, Gesoglu and Guneyisi (2011) found higher sorptivity coefficient of self-compacting concrete due to the addition of crumb rubber aggregates. The addition of FA can decrease the sorptivity of rubberized aggregate, which is particularly significant for concrete cured for 90 days.

In general, the depth of water penetration into concrete increased due to the incorporation of rubber aggregates. The increasing size of rubber particles further increases this parameter. The reasons for water absorption to increase are also accountable for depth of penetration. Ganjian et al. (2009) reported that concrete with replacements of 5 and 7.5 % of tyre rubber is classified as low permeability according to DIN 1048 standard but the mix with 10 % tyre rubber incorporation is classified as medium (Fig. 4.57).

4.7.3.3 Chloride Permeability

Very few data are available on chloride-ion permeability behaviour of concrete with rubber aggregates. Gesoglu and Guneyisi (2007) found a systematic increase in the depth of chloride penetration with increasing rubber content for concrete with and without silica fume, especially at high w/c ratio (Fig. 4.58). As the rubber content increased from 0 to 25 % by total aggregate volume, the chloride permeability of the rubberized concrete with and without silica fume was about 6–40 % and about 27–59 % greater than that of the controlled concrete at w/c ratio of 0.6 and 0.4, respectively, depending on the moist curing period. They also reported that increasing the moist curing period as well as adding silica fume

Fig. 4.57 Depth of water penetration (Ganjian et al. 2009)



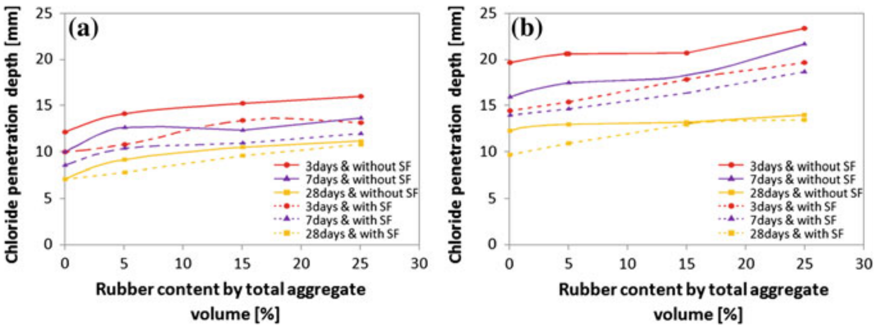


Fig. 4.58 Chloride permeability of concrete with rubber aggregates: **a** w/c: 0.4; **b** w/c: 0.6 (Gesoglu and Guneyisi 2007)

decreased the effect of rubber aggregate on the chloride-ion permeability of concrete.

The same authors (Gesoglu and Guneyisi 2011) reported the effect of FA on the chloride-ion permeability behaviour of self-compacting concrete with rubber aggregates. There was a progressive increase in the chloride-ion penetration as the rubber content rose. Extending the curing period from 28 to 90 days slightly improved the chloride-ion penetration behaviour. Incorporating FA slightly improved the chloride-ion permeability behaviour of the rubberized concretes at 28 days. However, when the curing period was prolonged to 90 days, incorporating the FA into the self-compacting rubberized concrete mixes significantly enhanced the resistance of the mixes against chloride-ion ingress. This finding was attributed to the long-term reaction of FA, which refined the pore structure of concrete and reduced the ingress of chloride ions.

4.7.3.4 Resistance to Chemical Attack

Topcu and Demir (2007) reported that the effect on the decrease of compressive strength of mortars with various amounts of rubber aggregates replacing natural sand was stronger in sea-water curing than in normal curing. The authors therefore recommended using sulphate resistant cement or high-strength cement in rubberized mortars to be used in sea-water environments.

4.7.3.5 Freeze–Thaw Resistance

Topcu and Demir (2007) also reported the effect of freeze–thaw cycles on the performance of rubberized concrete. In this study, concrete specimens had a cement content of 300 kg/m^3, a w/c ratio of 0.5, and 0, 10, 20 and 30 % replacement of fine aggregates by equal volume of rubber aggregates with size 1–4 mm. The results revealed that the concrete's compressive strength decreased with the increment of rubber incorporation after the freeze–thaw test. However, this reduction was slightly lower than the one observed for a similar concrete due to increasing addition of rubber aggregates before the freeze–thaw test. These reductions for all cylindrical specimens with 10, 20 and 30 % rubber incorporation compared to cylindrical control specimens not exposed to freeze–thaw cycles were, respectively, 16, 19 and 21 %. The reductions in cylindrical specimens with 10, 20 and 30 % rubber incorporation compared to control specimens, both exposed to freeze–thaw cycles, were respectively 15, 16 and 16. A similar behaviour was observed for cubic specimens. The authors also evaluated the freeze–thaw durability according to weight loss where they found that concrete prepared with 10 % replacement of fine aggregates by rubber aggregates exhibited better performance than conventional concrete.

4.7.3.6 High Temperature Behaviour

Topcu and Demir (2007) reported that rubber incorporation in cement mortar did not have significant effect on the compressive strength reduction due to increase in temperature. The highest decrease was observed for rubberized mortar after treatment at 400 °C. Nayaf et al. (2010) reported that the addition of 5 % microsilica to cement and the use of fine rubber aggregates with a maximum size of 0.07 mm could improve the rubberized concrete's compressive strength behaviour at high temperature. On the other hand, microsilica does not have any effect on concrete with coarse rubber aggregates with maximum size of 20 mm. Both aggregate sizes were used to replace coarse NA by 5–30 % in volume. Their results are presented in Fig. 4.59.

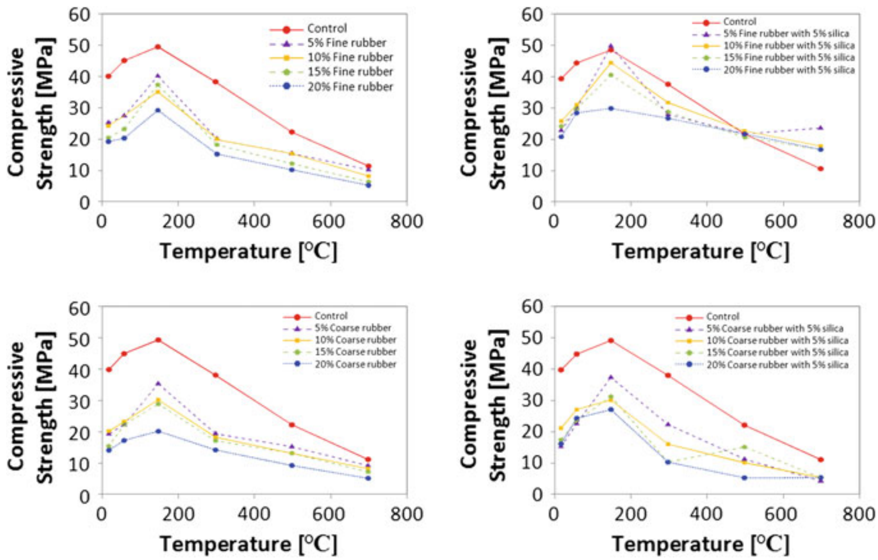


Fig. 4.59 Effect of temperature on the compressive behaviour of rubberized concrete (Nayef et al. 2010)

4.7.3.7 Damping Ratio and Base Isolation Property

Vibration damping is valuable for concrete structures because it mitigates hazards that may arouse from various factors like accidental loading, wind, ocean waves, or earthquakes. It can also increase the comfort of a person who uses the structures and enhances their reliability.

Zheng et al. (2008b) measured the damping ratio of conventional as well as rubberized concrete. These results suggest that the use of coarse and fine rubber aggregates as partial substitution of coarse NA increased the damping ratio and this effect further increased with the particles size and the rubber aggregates content. They also observed that the damping ratio increased with an increase in the maximum response amplitude. These results are presented in Fig. 4.60. From the analysis of the dynamic modulus of elasticity and the damping behaviour, the authors concluded that the rubber aggregates content in concrete should be below 30 % since higher contents dramatically reduced the modulus of elasticity of rubberized concrete.

Owing to the excellent flexibility and energy absorbency of rubberized concrete, Li et al. (1998) evaluated the base isolation capability of conventional and rubberized concrete. They determined the top dynamic response of the structure due to base excitation. Their results showed that the fundamental frequency of the structure shifted from 8 to 5 Hz when part of the base structure is replaced by rubberized concrete (Fig. 4.61). Moreover, the maximum acceleration frequency

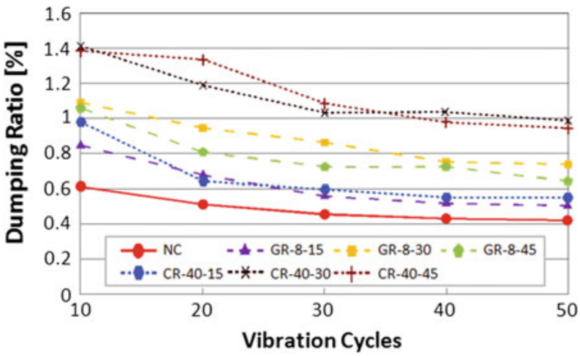


Fig. 4.60 Effect of rubber aggregates incorporation on the dumping ratio of concrete (Zheng et al. 2008b)

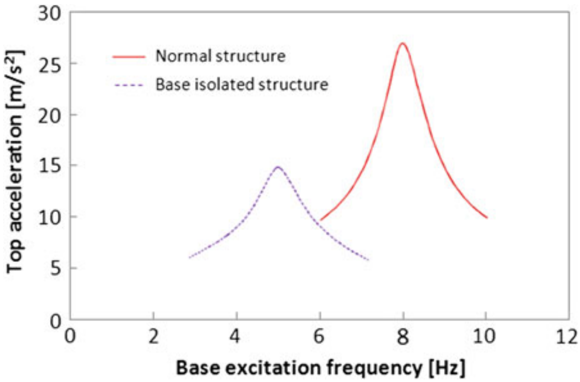


Fig. 4.61 Dynamic response of conventional and rubberized concrete (Li et al. 1998)

for rubberized concrete was significantly lower than that for conventional concrete, which indicates that rubber incorporation in concrete reduced the resonant response.

4.7.3.8 Thermal Insulation Properties

Yesilata et al. (2009) observed improvement of the thermal insulation performance of concrete due to the incorporation of rubber elements with thickness of 2 mm. This improvement was 18.52 % by adding of square rubber matrix in ordinary concrete.

4.8 Ceramic Industry Waste

In Chap. 2, the properties of some ceramic waste as aggregate were presented in detail. Here, the fresh and hardened concrete properties with different types of ceramic waste will be presented.

4.8.1 Fresh Concrete Properties

The slump of a concrete mix with ceramic waste aggregates depends on the nature of the aggregates. The majority of ceramic aggregates reported in the literature have higher porosities than normal aggregates, and therefore the incorporation of these aggregates in the concrete mix decreases slump due to their high porosity, rough surface texture and angular nature. Topcu and Canbaz (2007) observed workability problems due to the use of tile waste as partial and full replacement of coarse aggregate because of rough surface texture of the tile aggregates. Lopez et al. (2007) and Guerra et al. (2009) observed a similar workability of concrete with ceramic aggregates and natural concretes, when the latter was replaced by various amounts of fine and coarse ceramic aggregates. On the other hand, ceramic aggregates have some properties like lower water absorption than NA and smooth surface texture that can increase the slump of the resulting concrete mix (Senthamarai and Devadas 2005). Debeib and Kenai (2008) observed some segregation of concrete mix when brick waste was used as fine and coarse aggregates. The variations of concrete's slump due to the incorporation of ceramic aggregates given in various references are presented in Table 4.22.

The density of concrete with waste ceramic aggregates is generally lower than (or similar to) that of conventional concrete. Binici (2007) reported similar density and air contents for conventional concrete and concrete mixes prepared by replacing 40, 50 and 60 % by weight of sand by ceramic aggregate. Torkittikul and Chaipanich (2010) observed a decreasing trend in the density of fresh concrete and cement mortar mixes due to use of ceramic aggregates as replacement of sand (Fig. 4.62).

Brito et al. (2005) reported that the density of concrete decreased as the replacement ratio of coarse limestone aggregates by similar size ceramic aggregates increased, due to the lower density of ceramic aggregates compared to the limestone aggregates. The bulk density of fresh concrete mixes with ceramic waste with specific gravity of 2.45 as coarse aggregates with different w/c ratios in the Senthamarai et al. (2011) study was in the 2215–2281 kg/m^3 range in comparison to the equivalent range of 2383–2480 kg/m^3 for conventional concrete with granite coarse aggregates with specific gravity of 2.68. Cachim (2009) observed about a 5 and 6 % decrease in fresh density at w/c ratios of 0.45 and 0.5, respectively, when coarse NA were replaced by brick waste aggregates. Debeib and Kenai (2008) observed a reduction of up to 17 % in the fresh density of concrete with brick waste aggregates by comparison with NA concrete. The air content in concrete also increased as the content of ceramic waste aggregates rose.

Table 4.22 Slump of concrete with various types of ceramic aggregates

Torkittikul and Chaipanich (2010)	Fine aggregates	*W* (%)		0, *w*	50, *w*	100, *w*				WA: 1.25 (*C*);
		L (mm)		120	110	5				Texture: rough (*C*)
										Shape: angular (*C*)
Senthamarai and Devadas (2005)	Coarse aggregates	w/c		0.35	0.40	0.45	0.50	0.55	0.60	WA: 0.72 (*C*); 1.2 (*N*)
		L (mm)	*N*	10	18	35	48	80	148	Texture: smooth (*C*)
			C	13	24	45	64	99	155	Shape: angular (*C*)
Suzuki et al. (2009)	Coarse aggregates	*W* (%)		0, *v*	10, *v*	30, *v*				WA: 9.31 (*C*); 0.88 (*N*)
		L (mm)		600	550	530				
Binici (2007)	Fine aggregates	*W* (%)		0, *w*	40, *w*	50, *w*	60, *w*			WA: 2.44 (*C*); 2.65 (*N*)
		L (mm)		110	90	85	80			Texture: smooth (*C*)
										Shape: angular (*C*)
Lopez et al. (2007)	Fine aggregates	*W* (%)		0, *w*	10, *w*	20, *w*	30, *w*	40, *w*	50, *w*	–
		L (mm)		30	49	30	42	36	34	
Guerra et al. (2009)	Coarse aggregates	*W* (%)		0, *w*	3, *w*	5, *w*	7, *w*	9, *w*		
		L (mm)		40	42	39	41	43		

W replacement amount, *L* slump/consistency, *w* and *v* substituted by weight and by volume respectively, *WA* water absorption, *C* and *N* ceramic and natural aggregates

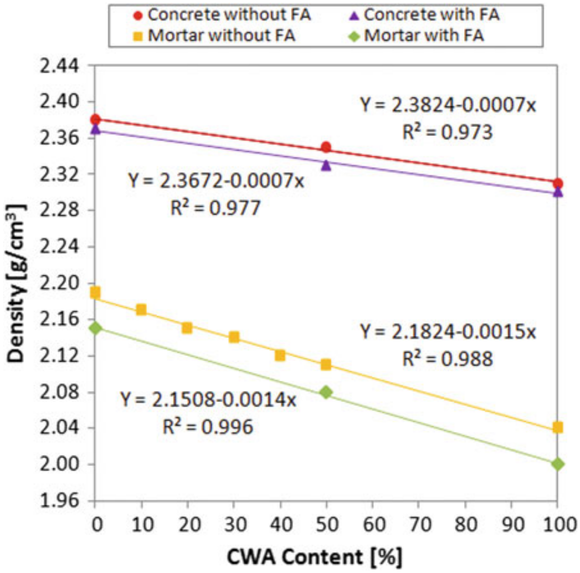


Fig. 4.62 Effect of ceramic waste aggregates incorporation in the density of fresh concrete and mortar mixes (Torkittikul and Chaipanich 2010)

4.8.2 Mechanical Properties

The compressive strength behaviour of concrete with ceramic aggregates depends on the properties of these aggregates. In several studies, it was observed that the incorporation of ceramic aggregates in concrete increased the compressive strength. This is particularly true for ceramic aggregates with low water absorption capacity like aggregates made of glazed ceramic waste. On the other hand, concrete with ceramic aggregates with very high water absorption capacity like aggregates generated from brick type ceramics exhibited lower compressive strength than conventional concrete.

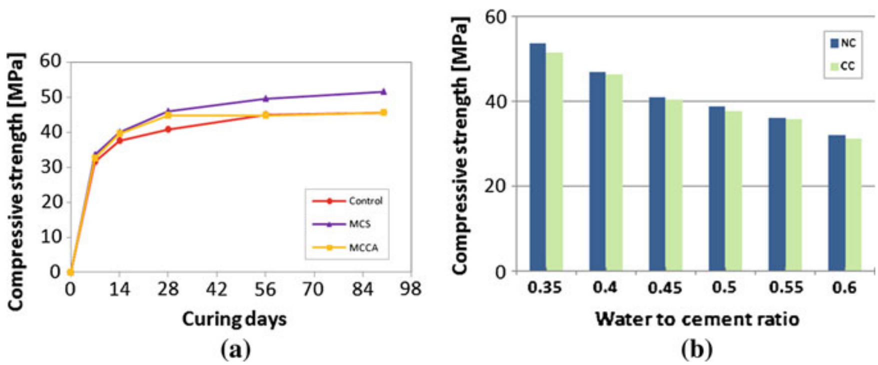


Fig. 4.63 Compressive strength of concrete with various types of ceramic aggregates: **a** Pacheco-Torgal and Jalali 2010; **b** Senthamarai and Devadas 2005

Binici (2007) observed higher compressive strength in concrete with 40, 50 and 60 % by weight of natural sand replaced by fine ceramic aggregates than in conventional concrete after 1 year of curing. In the Torkittikul and Chaipanich (2010) study, the 28-day compressive strength of concrete with ceramic earthenware waste aggregates as 50 and 100 % by weight replacement of natural fine aggregates was respectively 40.0 and 38.5 MPa that compares with 37.0 MPa for conventional concrete. This increase was attributed to improved interfacial zone due to the rough surface texture of ceramic aggregates and the presence of hard crystalline material like mullite in sintered ceramics. However, a slight drop at 100 % replacement level was observed, as the angular nature of ceramic aggregates deteriorated the workability of fresh concrete.

Pacheco-Torgal and Jalali (2010) observed higher compressive strength for two types of concrete with water-saturated white ceramic waste as complete replacement of fine and coarse NA than for conventional concrete (Fig. 4.63a). Ceramic aggregates replacing sand (MCS) were more effective than coarse ceramic aggregates (MCCA) in increasing the compressive strength of concrete after 28-day of curing. Lopez et al. (2007) also observed higher early compressive strength (up to 28 days) of concrete with ceramic aggregates content in the 10–50 % by weight range as replacement of natural sand.

Guerra et al. (2009) observed similar 28-day compressive strength for concrete with aggregate from sanitary porcelain waste replacing 3 % by weight of natural coarse aggregates; however, compressive strength increased with the content of ceramic aggregates at 5 and 7 % replacement levels but slightly decreased at 9 % replacement level even though still higher than that of the reference concrete. Compressive strength also increased with curing time. Senthamarai and Devadas (2005) observed a maximum 3.8 % reduction in 28-day compressive strength of concrete with coarse white ceramic waste aggregates with various w/c ratios when compared to concrete with NA (Fig. 4.63b).

Brito et al. (2005) observed lower compressive strength in concrete pavement blocks prepared by replacing 33, 66 and 100 % by volume of coarse limestone aggregates by aggregates from ceramic hollow bricks waste due to the lower density and lower crushing strength of ceramic aggregates than those of the limestone aggregates. The strength decreased as the content of ceramic aggregates increased. Topcu and Canbaz (2007) reported that using tile waste as replacement of coarse aggregates could decrease up to 43 % the compressive strength exhibited by the reference concrete due to the lower crushing strength of tile aggregates than that of crushed stone as well as the higher pores content in tile aggregates concrete. Debeib and Kenai (2008) observed up to 35 and 30 % reduction in compressive strength of concrete when coarse and fine NA were, respectively, replaced by coarse and fine recycled brick aggregates. Compressive strength was further decreased up to 40 % when both fine and coarse aggregates were replaced by brick aggregates. Cachim (2009) reported the compressive strength of two types of ceramic brick waste with different physical properties as partial (15 and 30 %) replacement of natural coarse aggregates in concrete. The author observed that the incorporation of brick aggregates with higher crushing strength than the natural

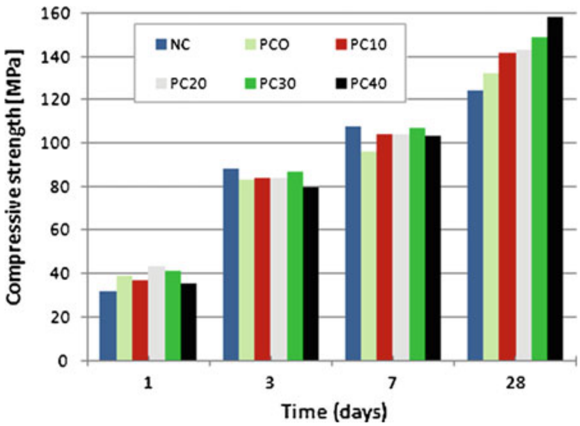


Fig. 4.64 Effect of the incorporation of porous ceramic aggregates on the compressive strength behaviour of internally cured concrete samples (Suzuki et al. 2009)

ones as well as of other type of brick aggregates having low crushing strength but with similar shape index as that of NA gave slightly higher 90-day compressive strength than for conventional concrete when 15 % by volume of natural coarse aggregates were replaced by this type of brick aggregates. On the other hand, the compressive strength of concrete with other type of brick aggregates was lower than that of conventional concrete at all curing ages. According to the author, the observed increase in strength was due to internal curing of concrete as the water absorbed by brick aggregates was used for hydration at later stages of curing.

Suzuki et al. (2009) reported the effect of internal curing on various properties including compressive strength of concrete with porous red ceramic aggregates as a 0–40 % by volume replacement of natural coarse aggregates. Incorporating ceramic aggregates did not affect the 3- and 7-day compressive strength but the 28-day strength increased with the content of ceramic aggregates. The 28-day compressive strength of concrete with 40 % by volume replacement of NA by ceramic aggregates was 20 % higher than that of NA concrete. These results are presented in Fig. 4.64.

There are few reports available on the evaluation of other strength properties (splitting tensile and flexural strength) and modulus of elasticity of concrete with ceramic aggregates. Lopez et al. (2007) and Guerra et al. (2009) did not observe any significant differences in the indirect tensile and fracture strengths of concrete where 10–50 % by weight of sand was replaced by white ceramic aggregates even though this incorporation significantly increased its compressive strength. Senthamarai and Devadas (2005) observed that the 28-day splitting tensile and flexural strengths of concrete with white ceramic waste aggregates as complete replacement of coarse NA were in the ranges of 3.2–4.5 MPa and 4.7–6.9 MPa at various w/c ratios (0.35–0.60). In this study, the corresponding ranges of splitting tensile and flexural strengths for conventional concrete were 3.9–5.5 MPa and 5–7 MPa, respectively. The authors observed lower tensile and flexural strengths to compressive strength ratios for ceramic aggregates concrete than for conventional concrete. The modulus of elasticity of conventional as well as of ceramic waste aggregates concrete varied in the ranges of 16.5–25.1 GPa and 16.1–22.2 GPa, respectively, at various w/c ratios.

Topcu and Canbaz (2007) observed a reduction of up to 40 % in the splitting tensile strength of concrete when coarse NA were replaced by tile waste aggregates. Brito et al. (2005) observed a linear decreasing trend in flexural strength of concrete pavement blocks with increasing content of aggregates from ceramic hollow brick as partial replacement of coarse limestone aggregates. However, the reduction in flexural strength of concrete was 26 % in comparison to 45 % in compressive strength when coarse NA were completely replaced by ceramic aggregates (Fig. 4.65). A considerable reduction in flexural strength and modulus of elasticity was also observed when crushed ceramic brick was used as partial and full replacement of natural fine and coarse aggregates in concrete (Debieb and Kenai 2008). On the other hand, Cachim (2009) observed slightly higher or similar flexural strength and modulus of elasticity of concrete with coarse brick aggregates at 15 and 30 % replacement level of coarse aggregates than that observed for conventional concrete.

Suzuki et al. (2009) observed a decreasing trend in 28-day splitting tensile strength of high-performance concrete with increasing content of porous ceramic aggregates (after observing a slight increase at the of 10 % by volume replacement level of coarse aggregates), due to a weaker interfacial bonding of ceramic aggregates–cement paste than for NA–cement paste. The Young‘s modulus of elasticity also decreased as the content of ceramic aggregates rose.

Cachim (2009) drew similar stress–strain curves for conventional concrete and concrete with two types of coarse brick aggregates replacing 15 and 30 % by volume of natural coarse aggregates. Topcu and Canbaz (2007) observed lower toughness of concrete with tile waste as partial and full replacements of coarse aggregates than that of conventional concrete.

Contrasting results are available on the effect of ceramic aggregates incorporation on the abrasion resistance of concrete due to changes in the properties of the aggregates. In some studies, the incorporation of fine or coarse ceramic aggregates improved the abrasion resistance of the resulting concrete due to good adhesion of porous ceramic aggregates to the cement paste (Brito et al. 2005; Binici 2007).

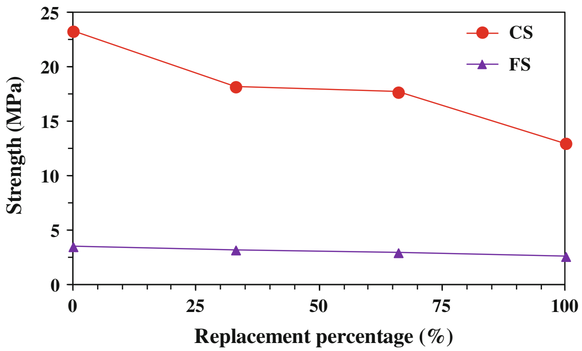


Fig. 4.65 Compressive strength (CS) and flexural strength (FS) of concrete with waste ceramic aggregate (Brito et al. 2005)

However, Topcu and Canbaz (2007) observed a significant reduction in the abrasion resistance of concrete due to the addition of ceramic tile waste as partial or full replacement of fine NA.

4.8.3 Durability Behaviour

Durability properties such as water absorption, chloride migration, gas permeation, freeze–thaw resistance and shrinkage were reported in various references even though the numbers of references for each type of ceramic aggregates is not substantial.

Debeib and Kenai (2008) observed higher drying shrinkage for concrete with crushed brick as partial or full replacement of fine and coarse aggregates than for NA concrete. Shrinkage increased with the content of both types of aggregates. The increase in shrinkage was more prominent for concrete with fine ceramic aggregates than for coarse ceramic aggregates, possibly due to the movement of water present in fine brick aggregates as progressive drying changed the moisture conditions (Fig. 4.66). The presence of porous red ceramic waste as partial replacement of coarse NA can significantly reduce the autogenous shrinkage of high-performance concrete when it is subjected to internal curing (Suzuki et al. 2009). The shrinkage reduction increased with the content of ceramic aggregates in concrete (Fig. 4.67).

The use of crushed brick as partial and full substitution of fine and coarse aggregates significantly increased the water absorption by capillarity of concrete (Debeib and Kenai 2008). The absorption was more pronounced for concrete with coarse brick aggregates. The water permeability of concrete with crushed brick aggregates was found to be 2.0–2.5 times higher than that of conventional concrete.

Use of a superplasticizer can improve the water absorption behaviour of concrete with brick aggregates (Debeib and Kenai 2008). Correia et al. (2006) also

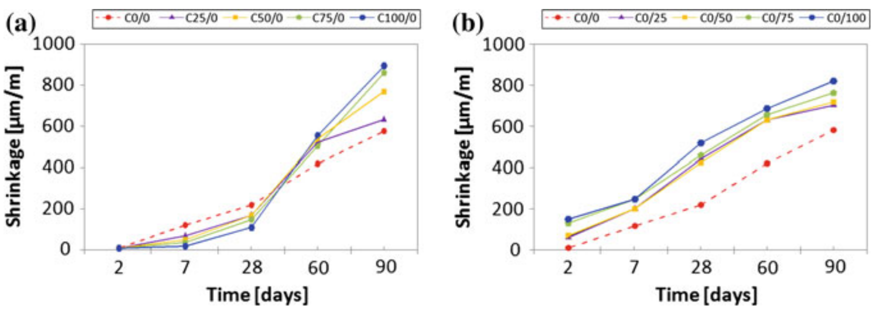


Fig. 4.66 Behaviour of drying shrinkage of concrete ceramic waste aggregates: **a** coarse; **b** fine (Debeib and Kenai 2008)

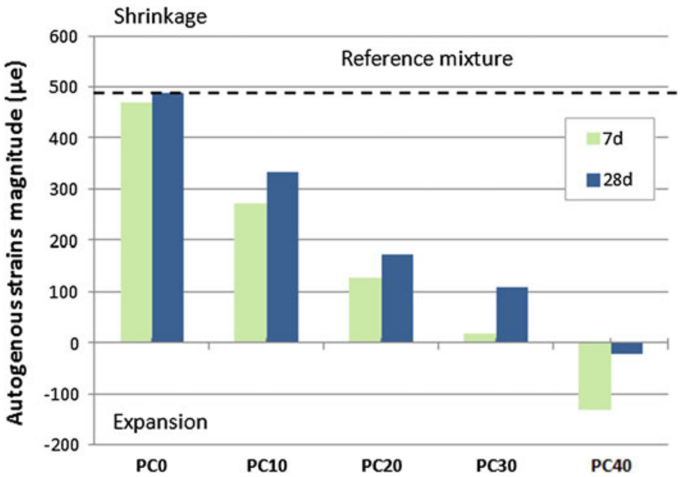


Fig. 4.67 Autogenous shrinkage behaviour of concrete with ceramic waste aggregates (Suzuki et al. 2009)

observed increasing water absorption of concrete pavement blocks as the replacement of natural coarse aggregates by ceramic hollow bricks aggregates increased. The water absorption capacity of conventional concrete blocks and blocks with 1/3, 2/3 and 3/3 (by mass) replacement of natural coarse aggregates by ceramic aggregates were 17.05, 21.11, 23.97 and 27.64 %, respectively. Senthamarai et al. (2011) observed higher water absorption by capillarity and volume of voids for concrete with coarse white porcelain waste aggregates than for conventional concrete with granite coarse aggregate. The water absorption by capillarity of concrete with waste aggregates and conventional concrete were in the ranges of 3.74–7.21 % and 3.10–6.52 %, respectively. Pacheco-Torgal and Jalali (2010) found lower water and oxygen permeability for concrete with ceramic waste as fine and coarse aggregates than for conventional concrete (Fig. 4.68a); however, the vacuum water permeability of conventional concrete waste was negligibly lower than that of ceramic aggregates concrete (Fig. 4.68b).

Binici (2007) observed lower depth of chloride permeation for concrete with ceramic waste aggregates replacing 40, 50 and 60 % by volume of fine NA than for conventional concrete. The depth of penetration for ceramic waste aggregates concrete was 10–15 mm in comparison to about 45 mm in conventional concrete. In the same study, the compressive strength of concrete with ceramic aggregates was also higher than that of conventional concrete and strength increased with the content of ceramic aggregates. Pacheco-Torgal and Jalali (2010) also observed lower chloride diffusion through concrete with fine and coarse ceramic waste aggregates than that through conventional concrete and best performance was observed for concrete with fine ceramic aggregates (Fig. 4.69a). Like in the Binici (2007) study the results can be related with the compressive strength of concrete. Senthamarai et al. (2011) observed higher electrical charge for concrete with

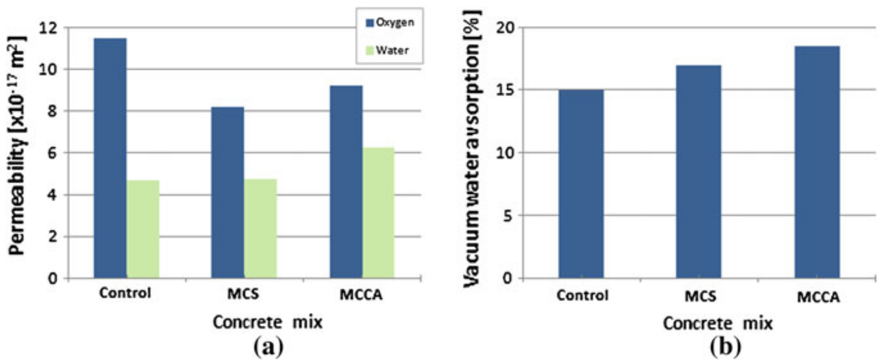


Fig. 4.68 Permeability of concrete with conventional and ceramic waste aggregates (Pacheco-Torgal and Jalali 2010)

coarse porcelain waste aggregates than for conventional concrete when both were subjected to ASTM C1292-10 specified rapid chloride penetration test (Fig. 4.69b). The increase in porosity in the ITZ of ceramic aggregates and cement paste due to their smoother surface texture than that of NA increased chloride diffusion.

Pacheco-Torgal and Jalali (2010) performed an accelerated ageing test to evaluate the effect of very harsh environmental conditions on the behaviour of concrete with ceramic waste aggregates as well as conventional concrete. The adopted procedures and results are presented in Fig. 4.70. The compressive strength after ageing of concrete with ceramic waste aggregates was higher than that of conventional concrete; however, the reduction in compressive strength of conventional concrete (14 %) was lower than that of the concrete with ceramic sand aggregate (18 %) and ceramic coarse aggregate (19 %). The incorporation of waste ceramic tile as 50 and 100 % replacement of coarse NA increased the weight loss during the freeze–thaw test due to lower hardness of tile aggregates and weaker binding of cement paste-tile aggregates than those of NA (Topcu and Canbaz 2007).

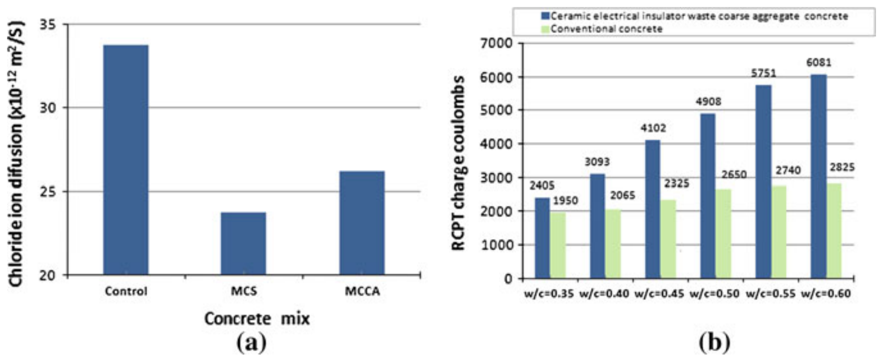


Fig. 4.69 Results of the chloride penetration test

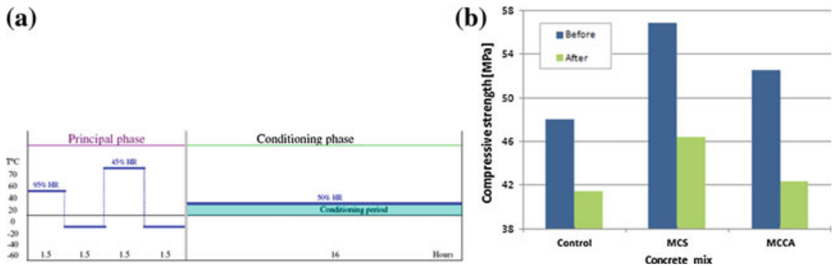


Fig. 4.70 Sequence of accelerated ageing tests and compressive strength results before and after ageing (Pacheco-Torgal and Jalali 2010)

4.9 Other Waste Materials

Other waste materials have also been used as aggregates in concrete and cement mortar production. The behaviour of these waste materials, which can be considered as industrial byproducts, will be discussed here.

4.9.1 Oil Palm Shell

The use of oil palm shell (OPS), an agricultural waste, created in palm oil producing countries as coarse aggregates in structural lightweight concrete was reported in detail (Basri et al. 1999; Jumaat et al. 2009; Mannan et al. 2006; Mannan and Ganapathy 2001a, b, 2002, 2004; Teo et al. 2006, 2007, 2010). According to Mannan and Ganapathy (2004) concrete with OPS as coarse aggregates can be used for several purposes such as road pavement, kerbs, concrete drains and flooring of buildings.

The workability of concrete with OPS as coarse aggregates was better than that of conventional concrete, due to the smooth surface texture of OPS aggregates (Basri et al. 1999; Mannan and Ganapathy 2004). The air content of concrete with OPS aggregates was also higher than that of conventional concrete due to the lower compaction of concrete with OPS aggregates. According to these authors, the obstructions in compaction due to variations in shape of OPS aggregates as well as the porous nature of OPS aggregates increased the air content of concrete. The fresh and dry densities of OPS concrete were about 20 % lower than those of conventional concrete and this mixes can be considered as structural lightweight concrete (1450–1900 kg/m^3) (Basri et al. 1999; Mannan and Ganapathy 2004).

Basri et al. (1999) observed that the compressive strength of OPS concrete was about 40–55 % lower than that of conventional concrete, when both were cured in three curing conditions. The highest compressive strength was seen in concrete cured in standard moist curing conditions. The reduction in strength was mainly due to the weaker crushing strength of OPS aggregates than that of NA. They observed a drop in compressive strength due to use of FA in OPS concrete. Mannan and Ganapathy (2001a) reported that a lightweight concrete with OPS as coarse aggregates with 28-day compressive strength of 24.2 N/mm^2 and a density of about 1900 kg/m^3 can be produced from a mixture of cement, normal sand and OPS in a proportion of 1:1.71:0.77 with free w/c ratio of 0.41. Adding 1 % $CaCl_2$ can be considered to increase concrete strength up to 29 N/mm^2. In another study, Mannan and Ganapathy (2002) reported that concrete with OPS aggregates has considerably lower compressive, flexural and splitting tensile strengths and dynamic elastic modulus than conventional concrete. However, the strength properties of concrete with coarse OPS aggregates meet the standard requirement for structural lightweight concrete. Jumaat et al. (2009) observed better shear capacity without shear reinforcement in the concrete with OPS aggregates beam than in that of conventional concrete. The concrete beam with OPS aggregates also showed better ductility behaviour with more shear and flexural cracks than the conventional concrete beam (Jumaat et al. 2009; Teo et al. 2006).

Concrete with OPS aggregates also had higher drying shrinkage than conventional concrete; however, the increase was in the normal range generally observed for lightweight concrete. The water absorption capacity of OPS concrete was also higher than that of conventional concrete due to the higher porosity of the OPS aggregates concrete. The results presented by Teo et al. (2010) indicated similar permeability properties (water and chloride) of concrete with OPS aggregates like in other lightweight aggregates concrete. However, proper curing is essential for OPS concrete to achieve better performance at later ages. Generally, normal immersion curing was best for better durability performance of OPS aggregates lightweight concrete.

Treatment of OPS by PVA (20 % solution was best) can improve mechanical and durability performance of lightweight concrete with OPS aggregates because it strengthens cement paste–OPS aggregates binding by forming a thin layer on the surfaces of OPS aggregates (Mannan et al. 2006). PVA can form a thin layer on the surface of OPS aggregates and therefore increase the aggregates–cement interaction.

4.9.2 Crushed Oyster Shell

Yang et al. (2005) studied the effect of crushed oyster shell (OS) as partial replacement of sand (10 and 20 % by weight) in concrete. From the experimental results they concluded that:

1. The slump of concrete with OS decreased with as the fineness modulus of OS (though it is insignificant) and the substitution rate of sand by OS increased, due to the dry and flaky nature of OS aggregates; however, setting time and air content were not affected by the partial substitution of sand by OS;

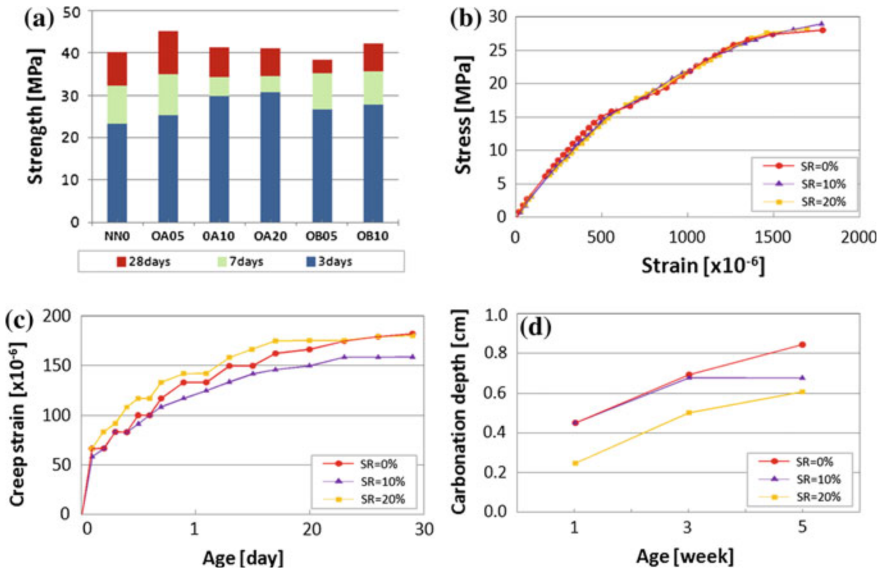


Fig. 4.71 Properties of concrete with crushed oyster shell as partial substitution of fine natural aggregates (Yang et al. 2010)

2. The early age compressive strength (up to 28 days) of concrete with OS at 10 and 20 % level was higher than that of conventional concrete due to the lower free water content in this type of concrete than in conventional concrete (Fig. 4.71a); however, the 1-year compressive strength of concrete with OS at 10 and 20 % level was already lower than that of conventional concrete; the modulus of elasticity of concrete with OS at all replacement levels was lower than that of conventional concrete at all curing period of 1 year of curing due to the lower stiffness of OS aggregates than that of fine NA; the addition of OS as aggregates in concrete did not have any effect on the stress–strain curve (Fig. 4.71b);
3. The drying shrinkage of concrete with OS at all replacement levels was higher than that of conventional concrete due to the lower stiffness and fineness modulus of OS aggregates than those of natural sand; the creep of concrete with OS aggregates at 10 and 20 % replacement levels of sand were, respectively, lower and higher or similar to that of conventional concrete (Fig. 4.71c); the permeability of concrete with OS was lower than that of conventional concrete; the use of OS as partial replacement of sand at 10 and 20 % levels improved concrete's freeze–thaw resistance as fine grains of OS filled the entrapped air voids scattered in concrete; the partial substitution of OS as fine aggregates decreased the carbonation depth of the resulting concrete and the decrease was higher at 20 % level than at 10 % level (Fig. 4.71d); the hydrochloric and sulphuric acid resistances of concrete with OS as partial substitution of sand were similar to those of conventional concrete.

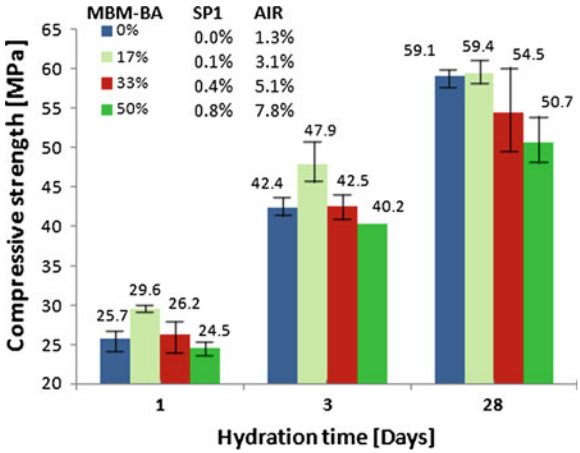


Fig. 4.72 Compressive strength at 1, 3 and 28 days of mortars with 17, 33 and 50 % of MBM-BA in replacement of sand and reference mortar (0 % MBM-BA)

4.9.3 Low-Risk Meat and Bone Meal Bottom Ash

Cyr and Ludmann (2006) reported the use of low-risk meat and bone meal bottom ash (MBM-BA) as fine aggregates in cement mortar production. This use up to a 17 % by weight level increases the early age compressive strength of cement mortar due to its high chloride content (Fig. 4.72). However, the presence of considerable amounts of chlorides (0.38 %) in MBM-BA, which restricts the use of this material as aggregates in reinforced concrete, must be considered before any cement based applications.

4.9.4 Tobacco Waste and Spent Mushroom Substrate

Ozturk and Bayrakli (2005) prepared a lightweight concrete using tobacco waste, collected from a cigarette factory, as fine aggregates. The concrete meets the specifications of lightweight concrete class that can be used as coating and dividing material in construction according to values of consistency, density (0.50–0.56 kg/dm^3), porosity, compactness, compressive strength (0.20–0.60 N/mm^2) and the thermal insulating behaviour (thermal conductivity 0.194–0.210 W/mK^{13}).

Pang et al. (2007) reported that concrete with quicklime treated spent mushroom substrate (SMS) as partial substitution of sand can be used for sidewalks, concrete curbs, concrete barricades, sound walls, and other non-structural applications. However, the durability of this kind of concrete needs further investigation.

4.9.5 Pulp and Paper Mill Waste

The use of waste generated from pulp and paper mills as fine aggregates, fillers or fibres in concrete was also reported (Ahmadi and Al-Khaja 2001; Gallardo and Adajar 2006; Naik 2002; Naik et al. 2004). The dry paper mill sludge used as fine aggregates in concrete up to 10 % replacement of fine NA can improve the compressive and splitting tensile strengths of the resulting concrete (Gallardo and Adajar 2006). On the other hand, Ahmadi and Al-Khaja (2001) reported that the incorporation of paper sludge as aggregates decreased the compressive and splitting tensile strengths of the resulting concrete. Concrete masonry blocks made with sludge at 5 % replacement level exhibit compressive strength of 8 MPa, splitting strength of 1.3 MPa, water absorption of 11.9 %, and density of 20 kg/dm^3 (Ahmadi and Al-Khaja 2001).

Fibres and waste from pulp and paper mill production as well as deinking solids from paper-recycling plants were also considered for incorporation in concrete. However, these materials should be properly dispersed in water, preferably hot water, before using such sludges in making structural-grade Portland cement concrete. Chloride-ion penetration can be reduced by adding residual solids to concrete. This type of concrete also showed higher resistance to salt scaling and freeze–thaw damage than that of control concrete (Naik 2002; Naik et al. 2004).

4.9.6 Wastes from the Shoe Industry

Baffa and Akasaki (2005) investigated the performance of lightweight concrete prepared with leather waste, mostly residue from cattle, as aggregates. Results showed that hardened mortar specimens that contain leather pieces with dimensions larger than 10 mm can be permanently deformed if the mortar is cured in humid condition. The compressive strength, splitting tensile strength and drying shrinkage are inside the standards limits and may be different with different types of leather. The expansion of concrete with leather is lower than that of control concrete and it decreases as the content of leather pieces in concrete increases (Baffa and Akasaki 2005). Santiago et al. (2009) reported the use of ethylene vinyl acetate (EVA), a waste from the shoe industry as aggregates in concrete. The EVA aggregates were obtained by cutting off the waste of EVA expanded sheets used to produce insoles and innersoles of the shoes. Two concrete mixes were prepared by replacing 50 % by volume of natural coarse aggregates by EVA aggregates, and by a 1:1 mixture of EVA and construction and demolition waste (CDW) aggregates. Their results suggests that EVA waste can be used as aggregates in the production of structural lightweight concrete by mixing it with CDW aggregate with a 28-day compressive strength of about 18 MPa. The toughness value of concrete was also increased due to the addition of EVA aggregates.

4.9.7 Different Sludges

Rao et al. (2009) reported that partial replacement of river sand by bone char sludge, a waste material generated during purification of fluoride contaminated water, did not degrade the strength or environmental integrity of dense concrete specimens and met the British standard specification (BIS 2185) for load-bearing blocks (Fig. 4.73). The concrete specimens with bone char sludge at a replacement level of 3 % of sand yielded higher compressive strength than conventional concrete specimens due to the improvement of packing of small NA and larger bone char sludge particles within concrete. The leachability of fluoride from concrete blocks was much lower than the specified limit for disposal of treated leachate into inland surface water (2 mg/l).

Kuo et al. (2007) studied the use of organic-modified reservoir sludge (OMRS) as fine aggregates in cement mortars. The sludge contained high amounts of hydrophilic smectite clay, and therefore the sludge was treated with organic surfactant to convert it into a hydrophobic material.

The results indicated that increasing OMRS use to replace sand gradually decreased the compressive strength of mortar; however, the 28-day compressive strength of mortar made replacing 1–30 % of fine NA by OMRS aggregates was more than 30 MPa, and therefore up to a 30 % replacement of fine aggregates by OMRS aggregates can be accepted for production of normal-strength cement mortars with improved water permeability (Fig. 4.74). On the other hand, the OMRS particles can be used in controlled low-strength materials if the replacement ratio for fine aggregates exceeds 80 %.

Sales and De Souza (2009) reported the possibility of recycling water treatment sludge to produce medium strength structural concrete, underlayment concrete, and block laying mortar. Their results suggest that sludge may be applied as a regulator of consistency and plasticity and, in suitable quantities (e.g. 2 % of sand) can even increase the compressive strength of medium strength concrete with NA. The axial compressive strength, modulus of elasticity and stress–strain curve of

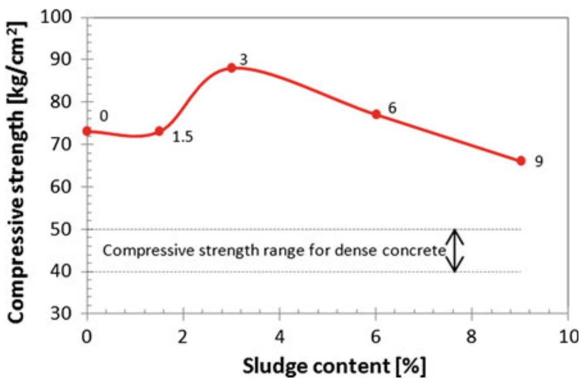


Fig. 4.73 Change in compressive strength of dense concrete blocks due to bone char sludge incorporation (Rao et al. 2009)

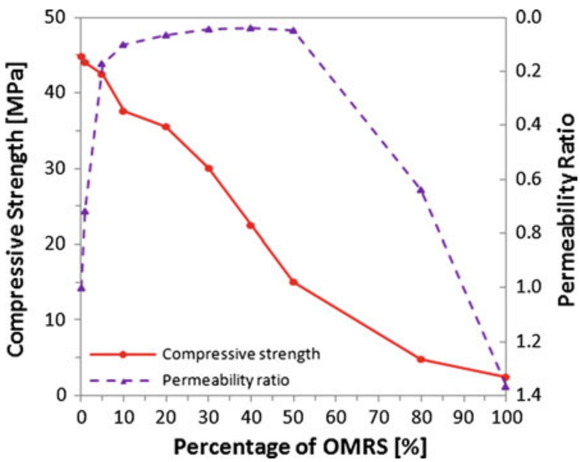


Fig. 4.74 Behaviour of cement mortar with 5 % OMRS aggregates: compressive strength (Kuo et al. 2007)

underlayment conventional concrete and concrete with 3 % replacement of sand by sludge were similar.

Aspiras and Manalo (1995) produced a concrete type composite material by using a mixture of textile waste cuttings with an average length of 2 cm (short fibre) and 6 cm (long fibre), taken from disposed trimmings of a garments producer and a textile manufacturer, cement and water. The results indicated that textile waste cuttings can be used to prepare lightweight cement composite with maximum 28-day compressive, tensile and flexural strengths of 8.48, 9.24 and 16.14 MPa, respectively. This composite has various potential uses like in ceilings, walls, or as a wooden board substitute.

Agostini et al. (2007) investigated the use of dredged polluted sediment with high concentrations of toxic elements such as Cd, Cu, Cr, Pb and Zn after treatment as partial and complete replacement of fine aggregates in cement mortar. In this work, the dredged sediment was initially treated by a well-established technique to stabilize toxic elements by forming phosphate minerals followed by a heat treatment at 650 °C to remove organic matters. The results suggest that the incorporation of sediment in mortar considerably increases its drying shrinkage. However, the incorporation of sediment in mortar does not affect water permeability and it significantly increases strength for low-to-moderate substitution levels, while high levels of incorporation of sediment lead to strength on the same order of that of the reference mortar.

4.9.8 Waste Generated from Mining Industry

Yellishetty et al. (2008) reported the use of iron ore mineral waste of 12.5 and 20 mm size ranges as coarse aggregates in concrete. Their results indicate that the 28-day uniaxial compressive strength of concrete with iron-ore mineral waste as aggregates was 21.93 MPa while the equivalent compressive strength of concrete with conventional granite aggregates was 19.91 MPa. The higher strength of concrete with mine waste aggregates than that of conventional aggregate was due to an internal curing effect whereby water from the aggregate was gradually released into the concrete to further hydrate the paste. Their results are presented in Fig. 4.75. Leaching results indicated very low amount of dissolution of toxic element into the leaching solution.

Nataraja and Nalanda (2008) reported that quarry dust generated from granite mining could be used as complete replacement of sand in the preparation of controlled low strength materials (CLSM). Bouzalakos et al. (2008) reported the use of a waste generated from bioleaching of pyrite ore (OMW) and a jarosite residue (JR) generated from zinc production as 10 % by weight substitution of silica sand in the preparation of CLSM. Their results indicated that OMW can be used as CLSM and met the strength requirements. Excessive leaching of arsenic, copper and chromium in a follow-up study, however, negate the use of this material in such purposes and therefore further study is necessary before proper application (Chan et al. 2009). On the other hand, CLSM with required strength from JR can only be prepared by mixing JR with a binder with an equal amount of Portland cement and lime mixture. The acidic nature of JR as well as its high content of Pb and Zn in JR was possibly the reason for this behaviour. Their results are presented in Fig. 4.76.

Kinuthia et al. (2009) prepared a well-graded aggregate from the colliery spoil (minestone), a byproduct of coal mining, by mixing its coarse and fine fraction. This material has several drawbacks such as excessive wear, expansive behaviour, leaching of toxic elements and radioactivity. The use of this material as fine and coarse aggregates in concrete significantly increased the w/c value and therefore reduced the resulting compressive strength. Adding an admixture could reduce water requirement, and therefore improve the compressive strength of the resulting concrete. Despite the several drawbacks, the compressive strength of the resulting concrete indicated its possibility to be used in the production of low and medium category concrete usable for blinding concrete and for use in bound granular fill or foundations.

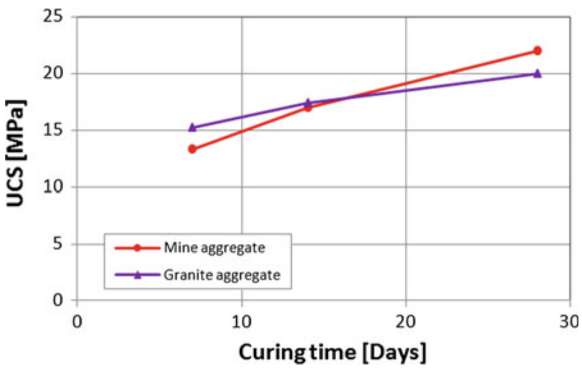


Fig. 4.75 Compressive strength of concrete with coarse aggregates of iron-ore mining waste and granite (Yellishetty et al. 2008)

Negm and Abouzeid (2008) reported that coarse solid phosphate mill tailings could be used as coarse aggregates to prepare concrete with 240 kg/cm^2, to be used in construction of small buildings.

Madany et al. (1991) reported the use of sand blasting grit waste (copper slag) as replacement of sand in the preparation of concrete blocks. The compressive strength of the concrete blocks with grit waste was 12 N/mm^2 and higher than the Bahrain specification for precast concrete blocks.

4.9.9 Waste from Metal Processing

Shinzato and Hypolito (2005) reported the use of a waste, called non-metallic product (NMP) generated during recovery of Al from aluminium black dross and salt cake by tertiary aluminium industry as aggregate in non-structural cement mortar block production. The NMP mainly contains silica, alumina and spinel along with alkali chlorides. The produced block met the requirement of size, water absorption and humidity prescribed in Brazilian standard, NBR 7173/1982. However, strength was slightly lower than the standard limit (2 MPa). However, according to the authors, the mortars were prepared using cement to aggregate ratio of 1:6 instead of the prescribed ratio of 1:3. According to the authors, NMP could be used in some construction applications after removing undesirable constituents such as chlorides and alkalis, which can reduce the usable amount.

Ismail and Al-Hashmi (2008b) reported the use of iron waste generated from an industrial workshop due to ironsmith processes. The material with maximum size of 4.75 mm and fineness modulus of 2.65 was used to replace 10, 15 and 20 % of sand in the preparation of concrete. The slump of concrete decreased 3.3, 4 and 8 % with respect to the reference concrete when 10, 15 and 20 % of sand, respectively, was replaced by iron waste aggregates, due to the heterogeneity and

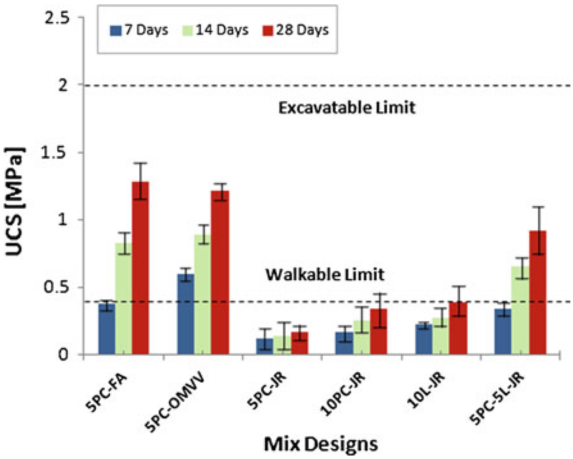


Fig. 4.76 Compressive strength of CLSM (Bouzalakos et al. 2008)

angular shape of the waste iron particles. On the other hand, fresh and dry densities of concrete increased with the incorporation of iron waste aggregates, due to their higher specific gravity by comparison with that of natural sand. The compressive and flexural strengths of concrete with various contents of iron waste aggregate were also higher than those of conventional concrete at all curing period up to 28 days (except the 14-day compressive strength) due to the higher specific gravity and strength of iron aggregate than those of natural sand (Fig. 4.77). However, after 14 days, the compressive strength of all compositions with waste iron aggregates were lower than that of the control mix, which may be due to the formation of a layer of iron compounds on the hydrating cement particles.

4.9.10 Marble Waste

Several reports are available on the use of waste generated from marble waste stored in marble quarries or sizing industries. This waste can be used either as filler material in cement or fine aggregates during preparation of cement mortar and concrete (Rai et al. 2011; Cornaldesi et al. 2010; Topcu et al. 2009; Alyamaç and Ince 2009).

Rai et al. (2011) uses marble granules with fineness modulus of 2.72 as 0–20 % by weight replacement of fine aggregates in cement mortar and concrete. The compressive strength of cement mortar with marble granule up to a 15 % replacement level was higher than that of the reference mortar; however, at 20 % level the compressive strength was lower than that of the reference mortar. The slump of concrete slightly increased with the replacement ratio of fine NA by marble aggregates. The compressive strength and flexural strengths of concrete with marble waste as partial replacement of fine NA increased with the content of marble aggregates and peaked at the 15 % replacement level. The mean compressive strength of concrete with marble aggregates was about 5–10 % higher than that of the reference concrete. Cornaldesi et al. (2010) observed a compressive strength of cement mortar with very fine marble powder as 10 % by weight replacement of sand about 10 % lower than that of the reference mortar. However,

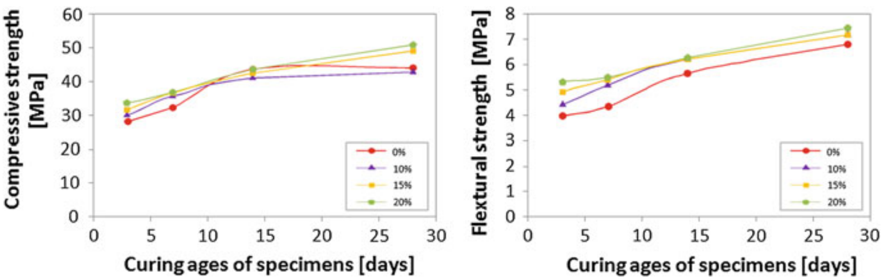


Fig. 4.77 Strength behaviour of concrete with iron waste fine aggregates

they observed similar compressive strength for concrete with marble aggregates and reference concrete when the former mix was prepared with a superplasticizer.

4.9.11 Waste from Wood Sawing

The waste generated from sawing of wood can be used as lightweight aggregates in the preparation of special types of concrete composites. Sales et al. (2010) prepared lightweight composite aggregates by mixing sawdust and water treatment sludge. The composite concrete prepared by using this composite aggregate had a lower thermal conductivity by about 23 % than conventional concrete. The authors proposed to use this concrete as non-structural sealing elements. Al Rim et al. (1999) produced a clay–cement–wood composite using wood pieces with maximum size of 22 mm as aggregates. The incorporation of wood aggregate into clayey concrete improved the thermal insulation and deformability behaviour but reduced compressive strength. Several treatments methods were proposed by Ledhem et al. (2000) to treat wood aggregate to improve the compressive strength of wood with clayey concrete.

Becchio et al. (2009) reported that the use of waste generated from wood-sawing industry as aggregates in concrete reduced the density and compressive strength but significantly improved the thermal insulation of mineralised wood concrete; furthermore, the compressive strength of this type of concrete was significantly higher than that of commercial wood concrete. Turgut (2007) prepared a composite brick using Portland cement, limestone powder waste and wood sawdust waste. The wood sawdust waste with different size fractions was used as aggregates in this composite. The compressive strength, flexural strength, density, ultrasonic pulse velocity and water absorption of the composite satisfied the relevant international standards. The incorporation of high amounts of wood aggregates with limestone powder increased the ductility of the resulting composite brick. According to the author, the product could be used for several purposes such as walls, wooden board substitute, an alternative to concrete blocks, ceiling panels, sound barrier panels, and absorption materials.

4.10 Concluding Remarks

Several types of waste material generated from various industries are currently used as aggregates in the preparation of concrete and cement mortar. The use of waste materials in concrete as aggregates significantly changes the properties of concrete.

The incorporation of ground plastics and shredded rubber can enhance some specific properties of concrete such as toughness and post-cracking behaviour, damping behaviour, thermophysical behaviour, which have several technical applications. The incorporation of rubber and plastic wastes and CBA, waste from wood sawing and some ceramic waste as aggregates can reduce the density of concrete and therefore these wastes can be used to produce lightweight concrete. On the other hand, several industrial wastes such as the majority of non-ferrous slags and some ferrous slags that have significantly higher bulk density than that of NA can be used as aggregates in the production of heavyweight concrete. CBA and some ceramic wastes can absorb high amounts of water and therefore they are used as aggregates for internal curing of concrete. FA generated from coal fired powered plant can beneficially be used as fine aggregates in concrete and cement mortar preparation, which can increase the consumption volume of these materials in the construction sector.

The incorporation of rubber and plastic wastes as well as some ceramic aggregates deteriorates the mechanical properties of the resulting concrete. On the other hand, the addition of some ferrous slags causes the expansion of concrete due to the presence of deleterious constituents in these slags, like free lime and magnesia. Therefore, treatments of these wastes before application or some innovation during the production of concrete and cement mortar are occasionally necessary to overcome these problems. The presence of toxic elements in some industrial wastes such as coal ash, some mining industries wastes and the majority of slags will increase the concentrations of these elements in cement mortar and concrete with these waste aggregates. Therefore, the fate and leachability of toxic elements from waste with cement mortar and concrete are important aspects that need to be considered for proper use of these wastes in constructions.

Some other solid waste materials generated from various industries such as mining, shoe, tobacco production, food and metal processing, pulp and paper mills, marble processing and waste sludge generated from processes such as contaminated water treatment, petroleum exploration are also reported as aggregates in various types of concrete production. The use of industrial wastes as aggregates in concrete has the potential to consume a vast amount of waste materials since aggregates are the major constituent of concrete and cement mortar. The use of waste materials in construction can solve most of the problems associated with their disposal as well as save natural resources related to aggregates mining. Therefore, the production of cheaper and more durable concrete using these waste aggregates can solve to some extent the ecological and environmental problems. However, the lack of widespread reliable data on the use of the majority of waste materials as aggregates in cement mortar and concrete can hinder their use in construction industry. Therefore, more research is required to design consistent and durable concrete with such waste aggregates.

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