

Mechanical and durability performance of concrete made using acrylonitrile butadiene styrene plastic from waste-EEE as a partial replacement of the coarse aggregate

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ARTICLE INFO

Keywords:

Acrylonitrile butadiene styrene waste plastic
Carbonation curing
Compressive strength
Durability performance
Electrical and electronic equipment waste
Microstructure

ABSTRACT

This paper presents the findings of a study focused on assessing the mechanical strength, durability performance and microstructure of concrete produced using acrylonitrile butadiene styrene (ABS) plastic derived from waste electrical and electronic equipment as aggregate. Granulated waste ABS replaced the coarse natural aggregate by volume in proportions of 5%, 10%, 25%, 50%, 75% and 100%. All concrete mixes were cured by precarbonation followed by water curing to improve early-age concrete performance in addition to conventional water curing alone. Mechanical and durability indexing tests (oxygen permeability, water sorptivity and chloride conductivity) indicate reduced compressive strength and durability index with increased waste ABS aggregate replacements. Even so, mixes made using substitutions $\leq 75\%$ waste ABS aggregate achieved the minimum compressive strength required for use in structural applications ($f_{cu} \geq 25$ MPa) and satisfactory durability performance at 28-days. Prediction models for reinforced concrete exposed to an inland environment suggest a cover depth of 60 mm and 30 mm for concrete made using 100% waste ABS subjected to water curing alone and precarbonation, respectively. In contrast, prediction models for mixes exposed to coastal regions require a cover depth of 80 mm when made using 25% waste ABS cured using water alone, while 100% waste ABS may be used when subjected to precarbonation curing. Statistical data indicate a strong correlation between the nature of the interfacial transition zone and the observed strength and durability performance.

List of abbreviations

ABS	Acrylonitrile butadiene styrene
ASTM	American Society for Testing and Materials
BSE	Backscattered Electron
CH	Calcium Hydroxide
CoV	Coefficient of Variance
CSH	Calcium Silicate Hydrate
EEE	Electrical and Electronic Equipment

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FR	Flame Retardant
HCP	Hardened Cementitious Paste
ITZ	Interfacial Transitions Zone
IZ	Interfacial Zone
OPI	Oxygen Permeability Index
PCAR	Precarbonation
PSD	Particle Size Distribution
SANS	South African National Standards
SCZ	Semi-Carbonation Zone
SEM	Scanning Electron Microscope
SD	Standard Deviation
WC	Water Cured
w:c	Water-to-cement
WEEEP	Waste Electrical and Electronic Equipment Plastic
WSI	Water Sorptivity Index

1. Introduction

Electrical and electronic equipment (EEE) refers to various devices that operate at less than 1000 V for alternating currents and 1500 V for direct currents [1]. When such equipment reaches its end-of-life, it is classified as waste-EEE (WEEE). WEEE is one of the fastest-growing waste streams globally, generating approximately 53.6 million metric tons (Mt) per annum (p/a) in 2019, and is expected to exceed 74 million Mt p/a by 2030. Based on a study by Lydall et al. [2], South Africa is one of the largest WEEE generators in Africa, producing 6.2 kg of WEEE per inhabitant per year, which is much higher than the African average of 1.9 kg per inhabitant per year (*ibid*).

A recent study by Reuters [3] showed that only 12% of South Africa's WEEE is correctly documented for recycling, while the majority is recycled under substandard conditions or landfilled. WEEE contains various valuable materials such as copper, gold and silver, which makes it vulnerable to cherry-picking. This means the valuable materials are removed, and the rest (such as glass and plastic, which have little value) are discarded in landfills [2].

Acrylonitrile butadiene styrene (ABS) is a commonly used thermoplastic in the EEE industry due to its exceptional mechanical properties [4,5]. ABS used in EEE often contain flame retardant additives (FR) to reduce the risk of a fire outbreak or to extend the time to escape during plastic-related fire incidents, when halogenated FRs such as bromine and antimony react to produce dense smog, which can smother the fire [6]. Brominated FRs are commonly incorporated into ABS applications because they require low dosages and have minimal impact on mechanical performance [7]. It is important to note that some halogenated FRs are not bonded within the polymeric matrix but instead seeded [8], which may cause the FRs to migrate into the environment, posing a potential health hazard.

Most EEE manufacturers prefer to use virgin plastic instead of WEEE plastic (WEEEP) due to the high-quality control achieved during manufacturing [9,10] since WEEEP may contain unknown additives and a history which may have degraded the plastic. Furthermore, a study by Lydall et al. [2] indicates that some non-EEE plastic recyclers have mixed WEEEP waste into plastic products that do not require protection from fire, such as toddler toys or food-contact containers. Although some technologies can revert the plastic to its monomer and collect the residual bromine, most processes are complex and expensive to implement and maintain [11]. Subsequently, much ABS WEEEP is stockpiled on company sites or dumped in landfills [5]. One solution to reducing WEEEP can be found in the construction sector by using it as an aggregate in concrete. Using WEEEP may reduce the demand for natural aggregate and promote employment within the recycling industry. Several studies have shown the potential use of waste products as a replacement for the natural aggregate using waste foundry sand [12], crumb tyre rubber [13], recycled aggregate from demolition waste [14] or WEEEP to promote a circular economy.

Kumar and Baskar [4] conducted research on the *Recycling of E-Plastic Waste as a Construction Material in Developing Countries* and showed that using a water-to-cement (w:c) ratio of 0.49 and replacing 50% of the coarse natural aggregate with WEEEP with a maximum size of 12.5 mm significantly decreased the compressive (47.41%), tensile (47.89%), and flexural (37.38%) strength of concrete. Furthermore, results indicate that only 30% of mixed WEEEP replacements achieved structural concrete (characteristic strength ≥ 25 MPa) at 28-days. Building upon this work, Liu et al. [15] examined the *Performance of Recycled Plastic-Based Concrete* by replacing 20% of the fine natural aggregate with polycarbonate/ABS (PC/ABS) WEEEP. The researchers used a w:c ratio of 0.36 and a water-reducing admixture, resulting in a 36.0% decrease in compressive strength compared to the control mix after 28 days. Even so, 20% of WEEEP replacements yielded structural concrete with a compressive strength of 28.6 MPa [15]. Manjunath [16] assessed concrete using mixed WEEEP as coarse aggregate, with a fraction size of 20 mm and a w:c ratio of 0.50. The findings revealed that a 30% replacement of mixed WEEEP led to a reduction of 53.1% compressive, 22.5% tensile and 42.5% flexural strength at 28-days. Interestingly, the percentage decrease in compressive strength was more apparent compared to the research conducted by Kumar and Baskar [4] despite using a lower WEEEP replacement. Furthermore, the results indicate that only 10% of WEEEP replacements achieved the strength requirements for use as structural concrete at 28-days. Martínez et al. [17] suspected that poor bonding between the hardened cementitious paste (HCP) and WEEEP might have caused previously observed strength reductions in concrete. They subsequently irradiated polycarbonate WEEEP with gamma radiation to increase its surface area. Results show that replacing 15% of

the fine natural aggregate with the irradiated WEEEP in concrete made using a w:c ratio of 0.78 improved the compressive strength by 12% compared to the control mix at 28-days. Their findings suggest that WEEEP can replace natural aggregate without compromising strength. Evram et al. [18] found that the use of a pozzolana, such as marble dust, at a WEEEP replacement of 40% for the natural aggregate increased the compressive tensile strength by 12% while reducing the modulus of elasticity of concrete by 29%. In a recent study, Parsons and Nwaubani [19] investigated the fresh and hardened properties of concrete that include different WEEEP types (polycarbonate/ABS, high-impact polystyrene, and ABS, respectively) as a partial replacement for the coarse and fine natural aggregate. Their results showed that a w:c ratio of 0.52 and 30% WEEEP replacements achieved the minimum strength requirement for use as structural concrete. However, the use of ABS WEEEP resulted in the lowest strength, which may be due to the formation of a phase similar to belite and calcium silicate hydrate gel between ABS WEEEP and the HCP formed during the hydration process.

Based on the reviewed literature, few researchers have replaced $\geq 50\%$ natural aggregate with granulated WEEEP in concrete. Therefore, this study aims to produce structural concrete using granulated ABS WEEEP as coarse aggregate.

2. Characterisation of materials used

2.1. Portland cement chemical, mineralogical and physical properties

A CEM I (52.5R) rapid hardening Portland cement was used as binder material to produce high-strength concrete at 28-days of curing, owing to the negative effect of plastic substitution on the mechanical and durability performance. Furthermore, chemical, mineralogical, and physical properties were measured to aid reproducibility.

The average density of the binder was measured as 3.350 g/cm^3 from three consecutive tests using a Pentrapyc 5200e gas pycnometer. Table 1 presents the binder oxide composition measured through wavelength dispersive spectroscopy using a CAMECA SX5-FE electron probe micro analyser. The results were cross-referenced to fixed mineral standards. In this study, *Mg*, *Al* and *Si* were measured on a TaP crystal, *Ca* and *S* were measured on a PET crystal, and *Fe* were measured on a LIF crystal. In addition, *Mg* was referenced on periclase, *Al* on yttrium-Al-garnet, *Ca* on Calcite, *Fe* on hematite, *S* on anhydride and *Si* on quartz.

Table 2 presents the average particle size distribution (PSD) for the binder from three measurements using an Anton-Paar Particle Size Analyser 1190, and Fig. 1 shows the average particle size and distribution. Fig. 2 shows the binder's X-ray diffraction (XRD) pattern, which was analysed using a D2 Phaser diffractometer using *CuK α* as the target material. Fig. 3 displays a scanning electron microscopy (SEM) image of the binder at 10K magnification.

2.2. Natural and ABS WEEEP aggregate physical properties

Crushed natural coarse and fine aggregate was mined from andesite lavas at the Eikenhof quarry in Roodepoort. The PSD of the coarse (stone) and fine (sand) aggregate was determined per South African National Standards (SANS) 201 [20] using a set of sieves ranging in size from 16 to 0.075 mm. The fineness modulus of the sand was measured as 3.48, with a dust content (particle size $< 75 \mu\text{m}$) of 8.3%, measured according to SANS 1083 [21], which is less than the maximum allowable dust content of 10% for fine aggregate derived from mechanical crushing of rock (*ibid*). The natural aggregate had well-graded curves, which fell within the minimum and maximum grading limits prescribed in the British Standards (BS) 882 [22], as shown in Fig. 4.

ABS WEEEP was derived primarily from telecommunication devices (phones and routers), personal computers and television sets. The quality of the ABS WEEEP will inherently vary since the end-of-life for WEEEP depends on the EEE application and user. Whole fractions of ABS WEEEP were granulated into 13, 10, 7 and 5 mm, as shown in Fig. 5. The granulation process is proprietary knowledge but follows a similar process discussed by Nwaubani and Parsons [23]. Each fraction size was mixed in proportions to replicate the coarse natural aggregate grading curve shown in Fig. 4 above. All ABS WEEEP was observed to be platy, whereby the 5 and 7 mm fractions had numerous sharp, jagged edges, which were less prominent in the 10 and 13 mm fractions.

The physical properties and standards used for the natural and ABS WEEEP aggregate are shown in Table 3.

Fig. 6 presents an SEM image of the surface topography of the rough natural and smooth ABS WEEEP aggregate. It should be noted that the surface roughness of the aggregate plays a vital role in the properties of concrete, as a rough surface would have a higher surface area than a smooth surface, which can affect the degree of adhesion between the aggregate and HCP. The average roughness of three random ABS WEEEP samples was measured as $0.247 \mu\text{m}$ using an RT-200 Surface Roughness Tester.

2.3. Superplasticiser admixture

The superplasticiser admixture used was a proprietary chemical based on an aqueous solution of modified polycarboxylate ethers. The admixture was used to maintain sufficient workability and ensure that all mix designs maintained a similar consistency.

2.4. Mixing water

Johannesburg Water provided municipal potable water for this study. The water is deemed appropriate for use in concrete, and no testing is required [27].

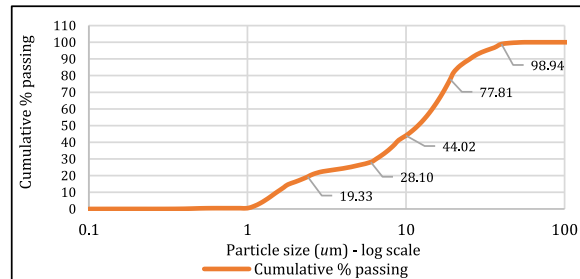
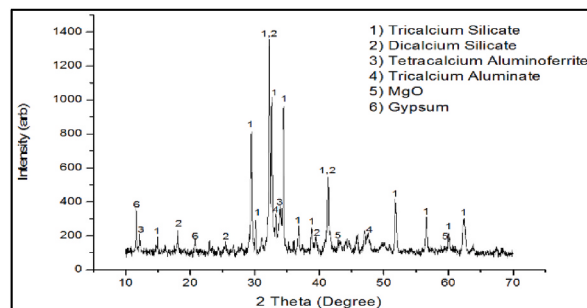
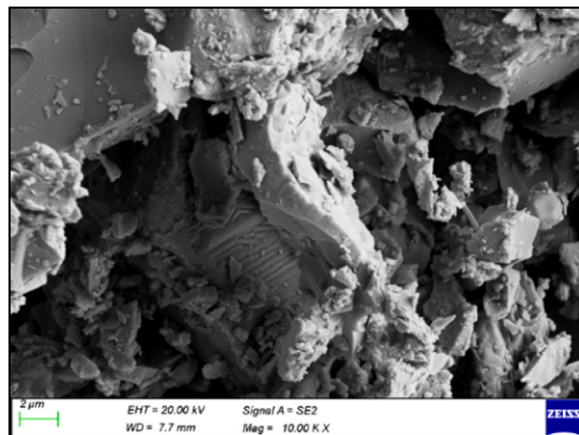
Table 1
Oxide composition of cement binder.

Oxide	CaO	SiO ₂	SO ₃	Al ₂ O ₃	Fe ₂ O ₃	MgO
%	66.33	22.57	1.58	4.82	2.72	1.81

Table 2

Particle size distribution results for cement binder.

D-sizes	D10	D50 (μm)	D90 (μm)	Mean
Particle size (μm)	1.555	10.192	26.675	12.956
Standard deviation (%)	0.003	0.183	0.172	0.124
Coefficient of variance (%)	0.210	1.790	0.650	0.960

**Fig. 1.** Particle size and distribution of the cement binder.**Fig. 2.** XRD diffractogram of cement binder.**Fig. 3.** SEM of cement binder.

3. Experimental details

3.1. Layout of experimental details and procedures

Fig. 7 summarises experimental details and procedures followed in this study.

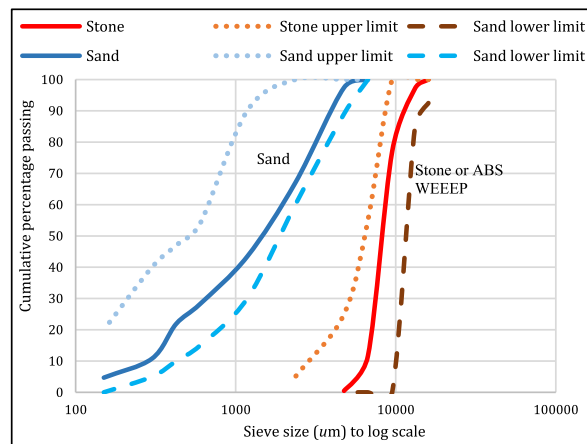


Fig. 4. Grading curve for the fine and coarse natural aggregate.

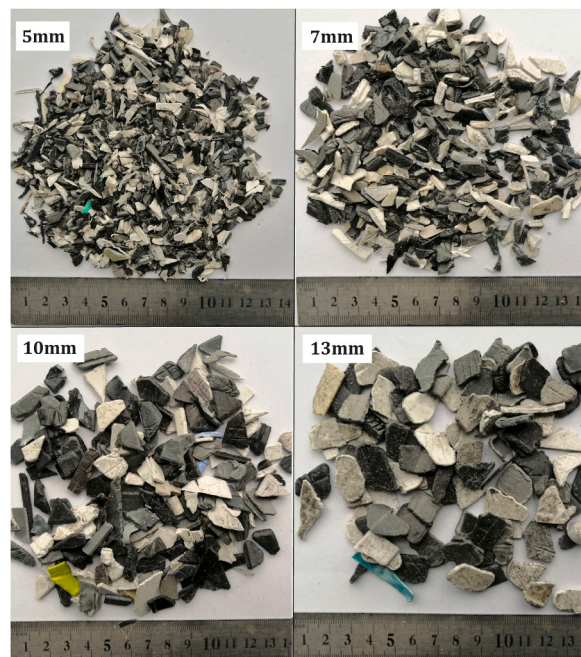


Fig. 5. Granulated ABS WEEEP sorted into various fraction sizes.

Table 3

Physical properties of the natural and ABS WEEEP aggregate.

Test	Standard	Stone (coarse)	Sand (fine)	ABS WEEEP
Particle density (kg/m^3)	SANS 5844] [24]	2630		1050
Compact bulk density (kg/m^3)	SANS 5845] [25]	1670	2050	580
Uncompacted bulk density (kg/m^3)	SANS 5845] [25]	1660	1830	
Water absorption (%)	SANS 5843] [26]	0.56		1.75

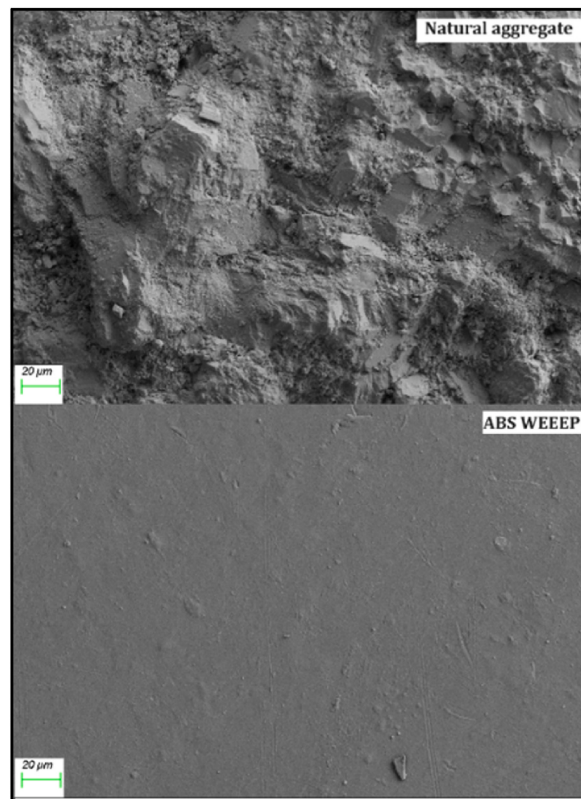


Fig. 6. SEM image of the natural and ABS WEEEP at 5K times magnification.

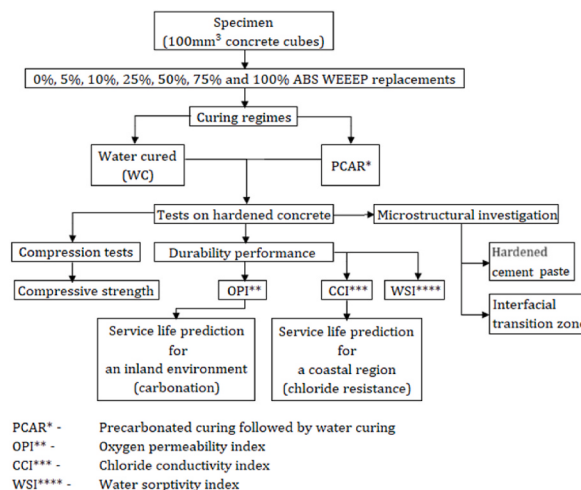


Fig. 7. Flow diagram of the experimental investigation.

3.2. Mix procedure, workability and casting of fresh concrete

The materials were weighed using an electronic scale with an accuracy of 0.1 g, and mixing was performed using a 50 L pan mixer. Likewise, the superplasticiser was volume-batched using a graduate cylinder. Mixing of the dry materials was done for approximately one to 2 min. Potable water was gradually added, and mixing was continued for two more minutes. A flow test was conducted directly after mixing per SANS 5862-2 [28] to maintain the desired spread of 405 ± 25 mm for each mix design, which was based on the assumption that the concrete may be used in applications requiring medium workability, as Neville and Brooks suggest [29]. Fig. 8 (a and b) presents the spread of concrete made using 0% and 100% ABS WEEEP, in which it was observed that all mixes maintained homogeneity.

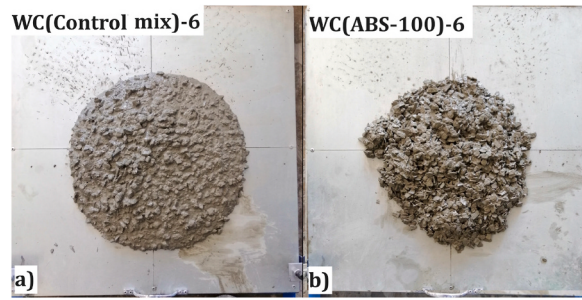


Fig. 8. Spread of concrete made using 0% (a) and 100% (b) ABS WEEEP.

Fresh concrete was compacted in intervals of 20 ± 3 s (indicated by air bubbles ceasing to rise [30]) on a vibrating table and was filled in two layers to the volume of the cube mould with an inner size of 100 mm^3 . The dimensions of the specimen were based on requirements set by SANS 5860 [31], which were dependent on the nominal size of the aggregate. As compaction affects concrete microstructure, all mixes were compacted using the same method and given similar compaction durations. The specimens were covered with a plastic sheet and demoulded after $22 \text{ h} \pm 2 \text{ h}$ per SANS 5861-3 [32].

3.3. Curing regime

Mixes were subjected to either water curing alone (WC) or precarbonation (early carbonation) curing followed by water curing (PCAR), as discussed in the following subsections.

3.3.1. Water curing (WC)

After demoulding, the specimens were transferred to a water curing tank containing a water pump and heater to maintain a uniform temperature of 23 ± 2 °C, as prescribed by SANS 5861-3 [32].

3.3.2. Precarbonation curing followed by water curing (PCAR)

Precarbonation curing of concrete involves carbon dioxide (CO_2) gas diffusion in concrete within 24 h of casting [33], which is primarily conducted on a lab scale due to the high cost associated with CO_2 sequestration [34,35].

Precarbonation curing is frequently performed on concrete with a low w:c ratio (typically 0.3) as excessive moisture content can fill concrete pores and impede the passage of CO_2 gas into concrete, thus limiting the CO_2 sequestration process [36]. Conversely, excessive drying can remove extra water from the concrete, altering the aqueous state required for carbonation [37]. Therefore, preconditioning is frequently necessary to remove surplus water from concrete with greater w:c ratios (*ibid*). A study by Zhang and Shao [35] found that concrete with a w:c ratio of 0.4 and a slump of 151 mm could be preconditioned by subjecting it to open-air fan drying after the initial set to remove free water. Similarly, Liu et al. [38] reconditioned their samples by placing them in an environmental chamber maintained at 20 °C and relative humidity (RH) of 50% for 3 h.

In this study, specimens were preconditioned using open-air fan drying, similar to Zhang and Shao [35]. The initial set of concrete took approximately 4 h, as determined by SANS 50196-3 [39], using a Vicat apparatus. The specimens were carefully removed from steel moulds at the initial set and preconditioned using an electric fan, generating a wind speed of 1.2 m/s across the surface of the specimens for about four and a half hours until 40% water loss was achieved [35]. The specimens were subsequently precarbonated, similar to Chen and Gao [36], using a free-flow carbonation chamber shown in Fig. 9. The carbonation chamber used 100% laboratory pure CO_2 gas, set at a concentration of $20\% \pm 0.1\%$ for 48 h at a temperature of $20 \text{ °C} \pm 3 \text{ °C}$ and RH of $57\% \pm 4\%$ which was controlled using saturated salt of sodium dichromate. The specimens were subsequently transported to a water curing tank, and curing

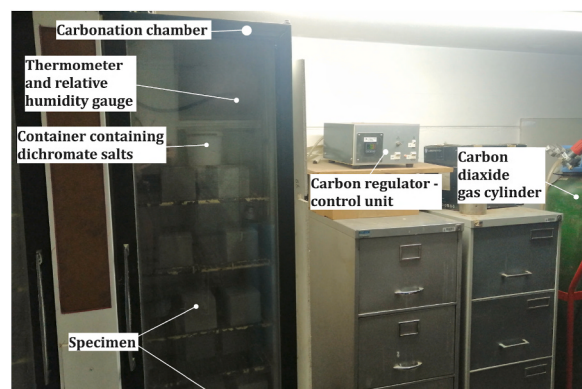


Fig. 9. Precarbonation experimental setup.

resumed until the day of testing.

3.4. Measurement of carbonation depth using phenolphthalein solution

The carbonation depth or front can be identified using a colourimetric base indicator called phenolphthalein on freshly broken concrete surfaces. When exposed to a $\text{pH} < 9$, the phenolphthalein indicator remains colourless and turns magenta at a $\text{pH} \geq 9$ [40]. It is important to note that the carbonation depth consists of two parts, namely the complete carbonation zone (CCZ), which has a pH ranging between 8.5 and 9 (indicative by a light shade of pink) and a semi-carbonation zone (SCZ), which has a pH between 9 and 12.5 [41] and that the carbonation front is not sharp but gradual [42]. This study used a phenolphthalein indicator to indicate the depth at which the CO_2 completely reduced the alkalinity of the hydrated paste and not the position at which carbonation reached. The indicator contained 1g phenolphthalein powder per 70 mL ethanol and 30 mL distilled water. The indicator was stored in a plastic container fitted with a nozzle, producing a fine spray.

3.4.1. Carbonation depth test procedure

The average carbonation depth was measured for all precarbonated mixes. The process involved splitting 100 mm³ concrete cubes equally along the side face by applying a line load in compression across opposite faces of the sample using an Amsler Universal Testing Machine with a 2000 kN capacity. The loading platens were made from mild steel with a 10 mm mild steel bar welded on the surface. The cubes were subsequently cleaned with a brush, sprayed with phenolphthalein solution and left for 1 h $\pm$ 15 min. The carbonation front does not advance constantly due to inhomogeneities in the concrete, and the indicator does not show dense aggregates. The average carbonated depth (D) was determined by taking the average of eight measurements (two measurements per side) per cube, as shown in Fig. 10. The measurements were taken perpendicular to the edges of the split face, measured to an accuracy of 0.5 mm using a Vernier calliper.

3.5. Compressive strength

The compressive strength of concrete cubes (100 mm³) was tested in its saturated surface dry condition directly after removal from the curing bath. A cube press with a capacity of 2000 kN applied a perpendicular load at a uniform load rate of 0.25 MPa/s to the casting direction across opposing cube surfaces per SANS 5863 [43] guidelines.

3.6. Durability indexing tests

The South African durability indexing tests involve measurements of the fluid transport characteristics of the cover zone (near-surface) of concrete [44] based on empirical observation under controlled test conditions, which are presented in the following subsections [45]. These tests include the oxygen permeability index (OPI), water sorptivity index (WSI), and chloride conductivity index (CCI) laboratory tests, which were performed at 7 and 28-days, per SANS 3001-CO3-2 [46], SANS 3001-CO3-3 [47] and Durability Index Manual [48]. The 7-day durability tests were carried out to compare the effect of precarbonation curing on the durability of concrete at early and later ages. The following subsections detail the preconditioning and testing processes for the durability indexing tests.

3.6.1. Preparation and preconditioning of concrete discs

The concrete discs were cored from 100 mm³ concrete cubes in their saturated surface dry condition according to the instructions in SANS 3001-CO3-1 [49], as shown in Fig. 11. Coring was done perpendicular to the casting direction with a water-cooled diamond-tipped core barrel with a nominal internal diameter of 70 mm, shown in Fig. 12. (a). Subsequently, a water-cooled diamond-tipped circular saw blade removed 5 mm from the core's outside faces. The discs in Fig. 12(b) were cut from the core to a thickness of 30 mm $\pm$ 2 mm from either side and labelled on their inside faces for reference.

For the precarbonated specimens, 1 mm was removed from the exterior face to open closed pores while maintaining a representative layer of carbonated concrete. The discs were placed in a vented oven for 7 days at 50 $\pm$ 2 °C. The discs were removed from the oven and placed in a desiccator for two to 4 h to cool to room temperature maintained at 22 $\pm$ 1 °C.

3.6.2. Oxygen permeability index (OPI)

The OPI value is the negative logarithm of the permeability coefficient derived from a falling head permeameter gas test and is

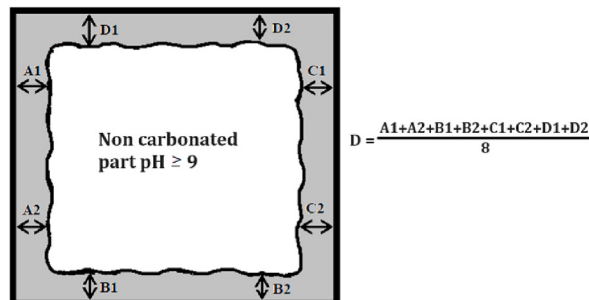


Fig. 10. Measurement of the carbonation front.

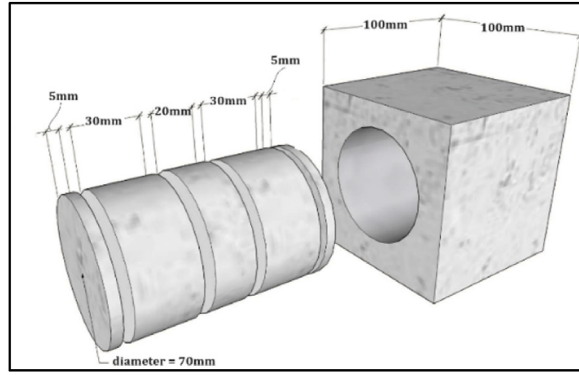


Fig. 11. Discs from a 100 mm³ concrete cube sample [48].

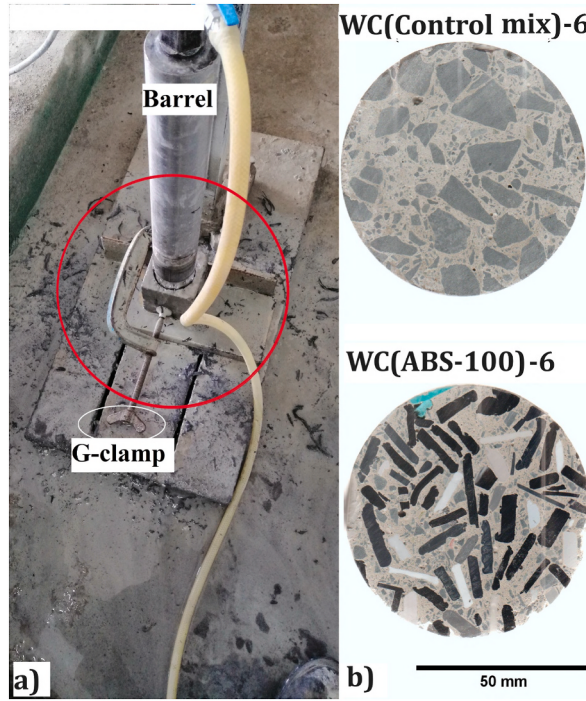


Fig. 12. Coring machine (a) and discs (b).

calculated as the negative log of the D'Arcy Permeability Coefficient (k) (m/s) [50]. The OPI test involves exposing the preconditioned concrete disc to a known internal pressure on one side while the opposing side is exposed to atmospheric pressure. As permeation occurs through the disc, the internal pressure lowers over time, and the oxygen permeability coefficient is calculated from Equation (1):

$$k_i = \frac{\omega \times V \times g \times d \times z}{R \times A \times T} \quad (1)$$

where ω is the molecular mass of oxygen (kg/mol), V is the volume of oxygen under pressure in the permeameter (m³), and g is the acceleration due to gravity (m/s²), d is the average thickness of the specimen (m), z is the slope of the linear regression line (s⁻¹), R is the universal gas constant (Nm/k mol), A is the superficial cross-section area of the sample (m²), and T is the temperature (K). As demonstrated in Equation (1), the average OPI of four discs is given as the negative log of the average coefficient of permeability, calculated using Equation (2) below.

$$OPI = -\log_{10} \left(\frac{k_1 + k_2 + k_3 + k_4}{4} \right) \quad (2)$$

The OPI test results are often similar to the Torrent permeability test and Cembureau methods [51]. After the preconditioning stage explained in Section 3.6.1, the discs were weighed to an accuracy of 0.01 g to determine the oven dry weight. After that, the discs were clamped down to the permeability cells, as illustrated in Fig. 13, while Fig. 14 presents the experimental setup.

The cells were initially pressurised with 99% laboratory pure oxygen gas at pressures ranging between 100 and 120 kPa and then opened for 5 s to eliminate other gasses to obtain a high oxygen concentration in the cell. The pressure in the cell was increased to 100 ± 5 kPa and examined to ensure no gas leaks. The test was terminated when the pressure reached 50 ± 2.5 kPa or after 6 h and 15 min from the start of the test.

3.6.3. Water sorptivity index (WSI)

The WSI test was used to evaluate the rate of water uptake of a laboratory-prepared concrete specimen under the action of capillary forces to quantify the near-surface properties of concrete. The capillary forces are highly reliant on both the orientation and connectivity of the pores, which often decrease with an increase in the depth of the material [52]. The lower half of the discs were sealed with cello tape and weighed to the closest 0.01 g after being removed from the OPI cells. This step ensured that the fluid was absorbed unidirectionally through its exposed surface, as shown in Fig. 15.

The exposed face of the discs was partially immersed in a fluid containing a calcium hydroxide solution (3 g of $\text{Ca}(\text{OH})_2$ for 1 L of potable water) to a height of no more than 2 mm up the side of the discs. After submerging, the discs were rubbed dry with a damp paper towel and weighed on an electronic scale every 3, 5, 7, 9, 12, 16, 20, and 25 min to the nearest 0.01 g. Fig. 16 presents the experimental setup, while Equation (3) was used to calculate the water sorptivity (S).

$$S = Fd / (M_{sv} - M_{so}) \quad (3)$$

F is the slope of the best-fit line, d is the average specimen thickness (mm), M_{sv} is the vacuum saturated mass (g), and M_{so} is the mass of the specimen (g) at the initial time. The average WSI is measured from four discs prepared, as detailed in Section 3.6.1.

3.6.4. Chloride conductivity index (CCI)

The CCI test relates to concrete chloride resistance and diffusion properties. The test involves measuring the disc's electrical resistance after being saturated in a highly ionic chloride solution, such as sodium chloride (NaCl) [50]. NaCl (5 M) solution was made by combining 2.93 kg chemical pure NaCl salt with 10 L potable water and storing it in a container for at least one day. Following preconditioning, the dry mass of the disc was weighed to an accuracy of 0.01g and placed in a vacuum saturation tank for 3 h at pressures of 75 kPa and 80 kPa. The NaCl (5 M) solution was allowed to flow into the tank without allowing air to enter. Once the solution level reached 40 mm above the top of the discs, the tank was closed and left under the previously mentioned pressures for 1 h. The vacuum was released, and the discs were soaked for 18 h at atmospheric pressure. The discs were removed, dried using paper towels, and weighed to an accuracy of 0.01 g. After measuring the mass, each disc was moved to the chloride conduction rig and placed in a compressible rubber collar, as illustrated in Fig. 17, while Fig. 18 show the experimental setup.

The conduction rig was filled with NaCl solution, and the DC power source was set to apply a voltage of about 10 V across the disc. The ammeter and voltmeter were then used to read the current and voltage. Equation (4) was used to calculate the chloride conductivity:

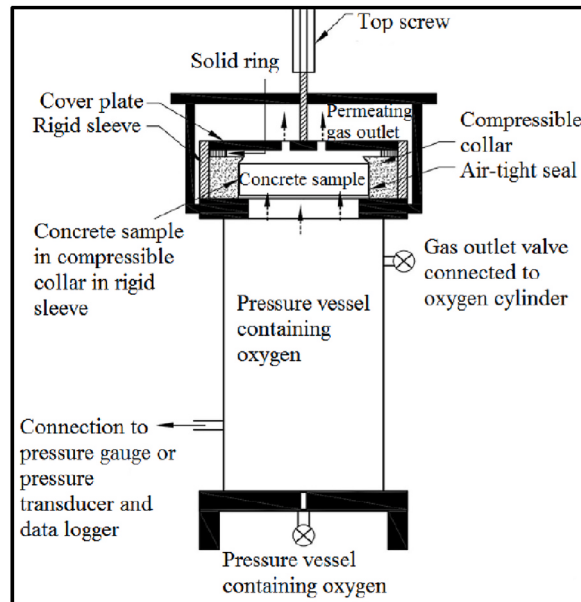


Fig. 13. Detailed schematic representation of a permeability cell [46].

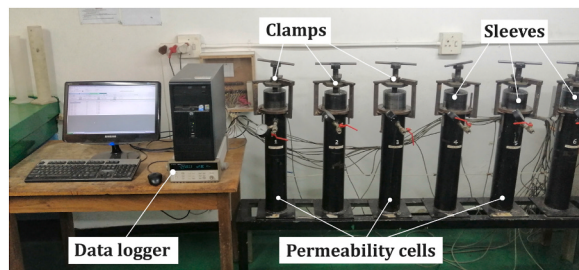


Fig. 14. Oxygen permeability indexing experimental setup.

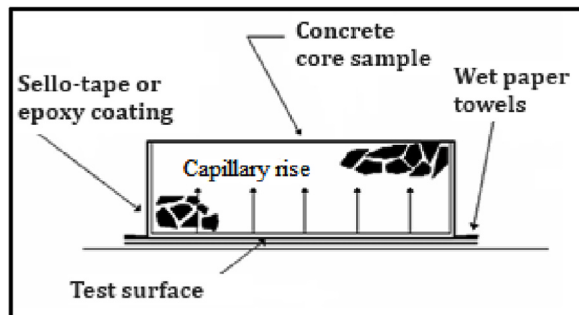


Fig. 15. Capillary rise in water sorptivity indexing test [53].

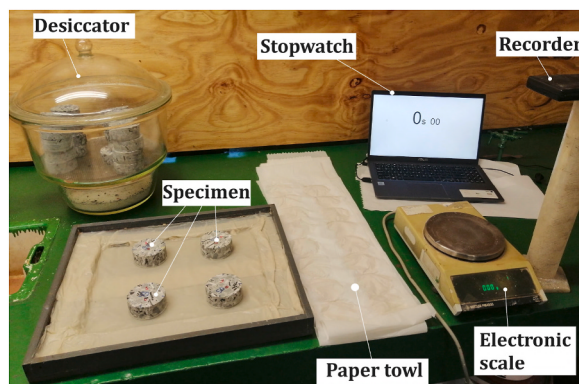


Fig. 16. Experimental setup for water sorptivity indexing test.

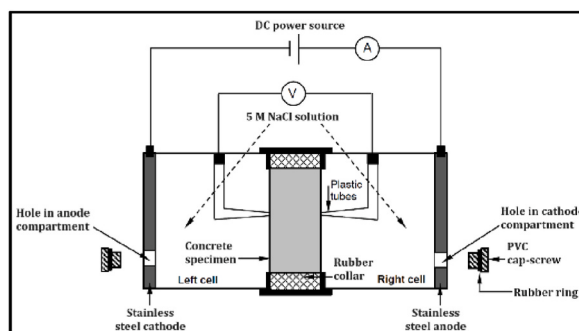


Fig. 17. Schematic representation of the chloride conduction rig [47].

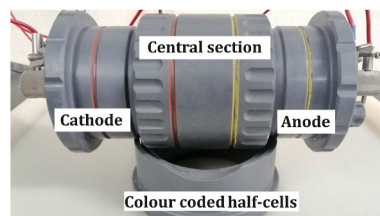


Fig. 18. Induction rig used for determining the chloride conductivity index.

$$\sigma = id/V_A \quad (4)$$

where I is the electrical current (mA), d is the average thickness of the specimen (cm), V is the voltage difference, and A is the cross-sectional area of the specimen (cm²). The average CCI is measured from four discs prepared, as detailed in Section 3.6.1.

3.7. Mix designs and proportions

The mix designs used a w:c ratio of 0.6 with a nominal cement-to-sand-stone ratio of 1 : 2.2 : 2.7, as shown in Table 4. ABS WEEEP replaced the coarse natural aggregate in percentages of 0%, 5%, 10%, 25%, 50%, 75% and 100% by volume. Fig. 19 presents the acronyms used to describe the various mix designs. Since ABS WEEEP had a lower specific gravity than the natural aggregate, ABS WEEEP replacements reduced the unit density of concrete.

3.8. Backscatter electron image acquisition for use in microstructural characterisation

The Overflow Point Method (OPM) was used to characterise the microstructure using backscattered electron microscopy and image processing. A Vega Tescan SEM was used to scan thin sections under a backscatter electron (BSE) detector. BSE image analysis directly observes the morphology of the concrete microstructure [54] using a greyscale level to distinguish different phases based on the measured atomic number. Phases with a high atomic number appear brighter (higher grey image), while phases with low atomic numbers appear dark (lower greyscale image) [55]. Therefore, BSE imaging can distinguish three primary phases in concrete: hydration products, anhydrous clinker and pores [56,57]. Since pores are epoxy-impregnated during sample preparation of the thin section, pores have a low BSE coefficient and appear dark on the BSE image. Therefore, voids can be separated from the concrete matrix

Table 4
Concrete mix proportions.

Mix design label	Sand (kg/m ³)	Stone (kg/m ³)	ABS WEEEP Coarse (kg/m ³)	Water (kg/m ³)	Cement (kg/m ³)	Superplasticiser (%)	Design density (kg/m ³)	Slump flow (mm)	Room temperature (°C)/Humidity (%)
WC(Control mix)-6	823	1020	0	224	373	0.00	2440	430	25,3/47
WC(ABS-5)-6	823	969	20	224	373	0.00	2409	425	25,3/48
WC(ABS-10)-6	823	918	41	224	373	0.00	2379	425	25,3/47
WC(ABS-25)-6	823	765	102	224	373	0.00	2287	380	24,9/48
WC(ABS-50)-6	823	510	204	224	373	0.06	2134	400	24,9/48
WC(ABS-75)-6	823	255	305	224	373	0.07	1980	420	25,1/48
WC(ABS-100)-6	823	0	407	224	373	0.10	1827	415	24,8/47
PCAR (Control mix)-6	823	1020	0	224	373	0.00	2440	425	22,2/49
PCAR(ABS-5)-6	823	969	20	224	373	0.00	2409	420	19,2/48
PCAR(ABS-10)-6	823	918	41	224	373	0.00	2379	425	23,1/52
PCAR(ABS-25)-6	823	765	102	224	373	0.00	2287	380	23,2/56
PCAR(ABS-50)-6	823	510	204	224	373	0.06	2134	405	23,5/52
PCAR(ABS-75)-6	823	255	305	224	373	0.07	1980	420	23,8/54
PCAR(ABS-100)-6	823	0	407	224	373	0.10	1827	410	20,1/49

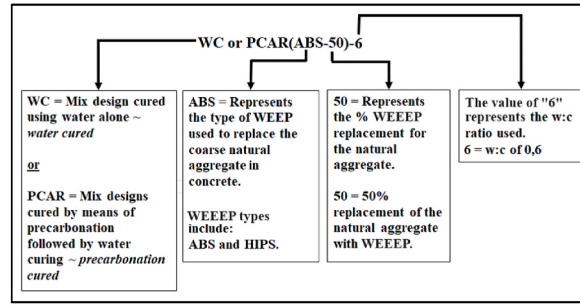


Fig. 19. Flowchart describing the acronyms used for the mix designs.

through a threshold grey filter value, similar to Korouzhdeh et al. [58].

The observations were carried out at a fixed magnification of 1K, accelerating voltage of 30 kV, and a working distance of 16.5 mm to avoid operator bias and allow comparability. To ensure the test's reproducibility, the brightness and contrast values were acquired from trials and fixed at a constant for each sample. Each image was 1024×1024 pixels, with a corresponding field size of $274.71 \mu\text{m} \times 274.71 \mu\text{m}$. As a result, the pixel size was $0.27 \mu\text{m} \times 0.27 \mu\text{m}$, with a greyscale spanning from 0 (dark) to 255 (bright). BSE images were processed and analysed using ImageJ digital imaging software. The OPM is a widely used approach for BSE image thresholding that determines the upper greyscale value of the pore by the inflexion of the cumulative percentage against the greyscale histogram curve, which is determined from the junction of two linear points, as illustrated in Fig. 20 [54,56,59].

Additionally, to compare the size and distribution of pores, the equivalent circular diameter (ECD) was calculated by equating the pixel area of a pore to that of a perfect circle, as shown in Equation (5) [56]:

$$ECD = \sqrt{4 \times A_p / \pi} \quad (5)$$

where A_p is the number of pixels included within each pore, the value was multiplied by the pixel size to convert the pixel size into an actual size.

3.9. Sample preparation for BSE image analysis

Hydration of concrete core specimens used for BSE image analysis was ceased by immersing the cores in a beaker containing isopropanol solution with a volume of 10 times the core, similar to Mitchell and Margeson [60] and Zhang et al. [37]. Isopropanol exchange is a drying process wherein isopropanol solvent replaces water in the open pores of hardened concrete in a three-stage approach. In the first stage, isopropanol gradually replaces water in large capillary and gel pores. In the second stage, the remaining water inside smaller pores gradually moves out under a concentration gradient. Meanwhile, isopropanol cannot enter the pores yet as it has a larger molecular size than water [61]. In the third stage, isopropanol solvent eventually enters the small pores when subjected to prolonged durations (six weeks), leading to secondary damage to the microstructure.

Isopropanol exchange was used in this study instead of oven drying alone to reduce microstructural damage, modification of paste structure or phase changes of CSH [62], CH and ettringite [61]. The isopropanol solvent was replaced daily for three days and then kept in the same solution for another four days. The cores were weighed and placed in an oven maintained at 50°C for 24 h to remove the solvent. The samples were subsequently weighted and placed back into the oven for 24 h until the weight remained constant and subsequently placed in a desiccator. The cores were removed from the desiccator and cut in half lengthwise.

The half-cores were partially immersed in EpoFix epoxy and placed in a vacuum chamber at 6 bars for 12 h to allow the epoxy to penetrate the concrete matrix. After curing, the samples were ground for 8 min at 300 rounds per minute using 400, 600, 800, and 1000-grit silicon carbide paper. The samples were mounted onto glass slides using a mounting press at 34°C for 12 h, resulting in a

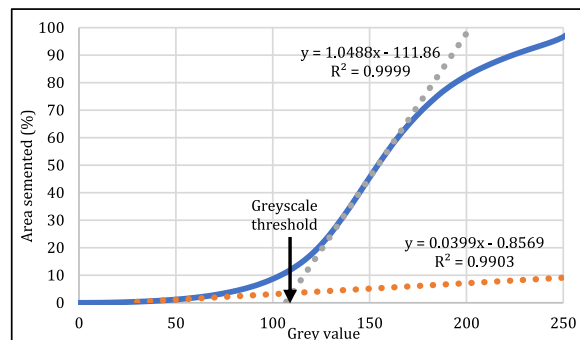


Fig. 20. Determination of the upper threshold value for pores using the OPM for WC(ABS-25)-6.

smooth surface. The samples were ground again using the same grit range as before, but this time, silicon carbide powder was used. The samples were subsequently polished using a diamond polishing paste of particle sizes 15 μm , 6 and 3 μm . Since concrete and WEEP materials are non-conductive, thin sections were coated with 30 nm gold-palladium before SEM and BSE analysis.

4. Results and discussions

4.1. Mineralogy of concrete

Figs. 21 and 22 on the next page present the XRD spectrums for WC(Control-mix)-6 and PCAR(Control mix)-6, indicating various crystals of tricalcium/dicalcium silicate ($\text{C}_2\text{S}/\text{C}_3\text{S}$), calcium silicate hydrate (CSH), calcium carbonate (CaCO_3), ettringite (AFm), silicon dioxide (SiO_2) and calcium hydroxide (Ca(OH)_2 or CH). A comparison of the two XRD results shows the presence of Ca(OH)_2 peaks and little or no calcium carbonate (CaCO_3) in Fig. 21.

Conversely, Fig. 22 shows the appearance of CaCO_3 peaks and the reduction in the Ca(OH)_2 peaks as the process of carbonation progressed, similar to results by Chen and Gao [36] and Lui et al. [38]. It was noted that CaCO_3 shared its dominant peak with calcium-silicate-hydrate (CSH), centred around 2θ (29.2°), while other peaks of calcite are observed at 2θ (23.0° , 35.9° , 39.3°), while CSH can be noted at 2θ (16.9° , 32.0° , 42.7° , 49.8° , 55.3°) [63] and CH was observed at 2θ (18.1° , 34.1° , 47.1°). Table 5 compares the intensities of carbonated and uncarbonated XRD at relevant 2θ diffraction angles.

Figs. 21 and 22 and Table 5 clearly show that CH peaks decrease as the calcite peaks increase due to carbonation [36]. Furthermore, the presence of CH in carbonated mixes suggests that further hydration of calcium silicates occurred after PCAR curing [33]. It was observed that PCAR-cured concrete had higher CSH peaks at 2θ (42.7°) compared to water-only cured mixes made using the same w:c ratio. This can be attributed to the additional CSH created by PCAR curing (*ibid*).

4.2. Concrete carbonation depth

This section presents the average carbonation depth for ABS WEEEP concrete, as shown in Table 6. Fig. 23 presents freshly broken concrete at 28-days of curing sprayed with a phenolphthalein indicator. Fig. 23 (ai and aii) present the left-hand side (LHS) and right-hand side (RHS) of PCAR(Control mix)-6, while Fig. 23 (bi and bii) present the LHS and RHS of PCAR(ABS-75)-6. The colourless region observed on the outer perimeter of the cube presents the complete carbonation zone (CCZ) (with a $\text{pH} < 9$). By contrast, the inner magenta-coloured region (having a $\text{pH} \geq 9$) may contain some carbonated material, which often increases (higher pH) as a front towards the interior regions of the concrete, which is similar to results observed by Liu and Meng [64], Zhang et al. [65] and Siddique et al. [66].

4.3. Pore size and distribution of the bulk hardened cementitious paste and interfacial transition zone

The pore size and distribution of the hardened cementitious paste (HCP) and the interfacial transition zone (ITZ) of concrete made using 0%, 25% and 100% ABS WEEEP replacements, subjected to WC and PCAR curing regimes, were obtained through BSE image analysis and presented in the subsections below.

4.3.1. Pore size and distribution of the bulk hardened cementitious paste

Figs. 24 and 25 show the raw (LHS) and segmented (RHS) BSE images of mixes made using 0% and 100% ABS WEEEP replacements, respectively, at a magnification of 1K using a Tescan SEM.

A visual comparison between the two mixes suggests that concrete made using natural aggregate has smaller and fewer interconnected pores compared to concrete made using only ABS WEEEP as aggregate. Table 7 presents the average pore size and percentage area covered by pores measured for two thin sections from the same mix.

The data acquired from the segmented images were plotted onto pore size and distribution curves, as shown in Figs. 26 and 27. Fig. 26 suggests the Control Mix had more pores $< 1.5 \mu\text{m}$ and fewer pores $> 3.5 \mu\text{m}$, while Fig. 27 shows that concrete made using 100% ABS WEEEP replacements had the highest number of pores for most pore sizes. Furthermore, the Control Mix had the least pores $> 3.5 \mu\text{m}$.

Concrete subjected to the PCAR curing regime had a smaller average pore size compared to WC curing alone. This could be due to the formation of calcite precipitation during precarbonation curing [67], which reduced the total capillary porosity [65].

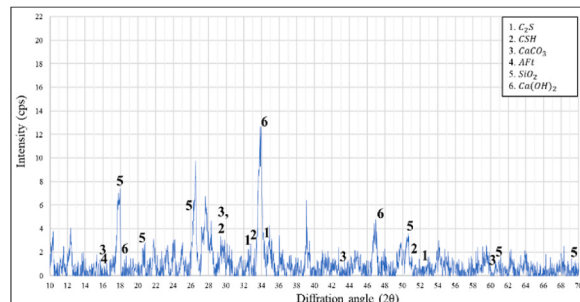


Fig. 21. XRD results for WC(Control mix)-6 at 28-days.

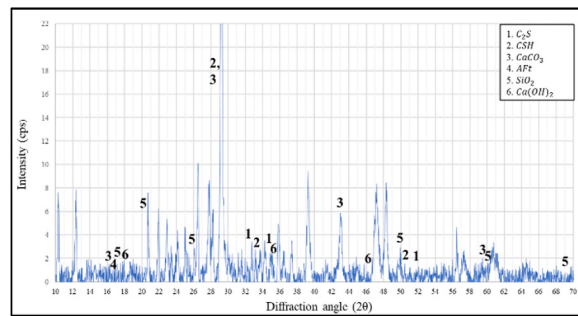


Fig. 22. XRD results for PCAR(Control mix)-4 at 28-days.

Table 5

Intensities of crystals for concrete subjected to WC and PCAR at 28-days.

Name of mix design	$Ca(OH)_2$ or CH Angle: 2θ (34.1°) <i>Intensity (cps)</i>	$CaCO_3$ or calcite Angle: 2θ (29.2°) <i>Intensity (cps)</i>	CSH Angle: 2θ (42.7°) <i>Intensity (cps)</i>
WC(Control mix)-6	12.18	0.67	0.32
PCAR(Control mix)-6	0.73	46.63	1.06

Table 6

Average carbonation depth for mixes subjected to PCAR curing regime.

Mix design label	3 Day			7 Day			28 Day		
	Carbonation depth (mm)	StD	CoV (%)	Carbonation depth (mm)	StD	CoV (%)	Carbonation depth (mm)	StD	CoV (%)
PCAR(Control mix)-6	12.78	0.59	4.65	12.54	1.07	8.51	12.41	0.69	5.56
PCAR(ABS-5)-6	12.34	0.35	2.81	12.76	0.42	3.32	11.38	0.73	6.44
PCAR(ABS-10)-6	12.56	0.35	2.79	12.73	0.77	60.6	12.12	1.39	11.43
PCAR(ABS-25)-6	12.39	0.47	3.80	13.15	0.44	3.08	11.62	0.81	7.00
PCAR(ABS-50)-6	12.14	0.51	4.18	13.07	0.86	6.59	13.44	0.77	5.76
PCAR(ABS-75)-6	12.51	0.57	4.52	12.34	0.96	7.78	12.08	0.77	6.34
PCAR(ABS-100)-6	12.48	0.43	3.48	12.96	0.50	3.87	12.42	0.36	2.91

4.3.2. Pore size and distribution of the interfacial transition zone

The interfacial transition zone (ITZ) is often considered the third phase in concrete and the weak link between the two phases of the aggregate and bulk paste [68]. It is structurally inferior to the bulk paste as it contains a higher concentration of pores, fewer cement particles and more CH [69]. The pore size and distribution in the ITZ are dependent on the property of the aggregate, w:c ratio and, binder type and particle size distribution. Cement particle sizes often range from 1 to 100 μm , while the aggregate is several magnitudes larger. Blending the cement and aggregates can disrupt the particle size distribution of cement and affect the packing arrangement around the larger aggregate [70], giving rise to the “Wall Effect”.

The Wall Effect occurs when finer cement particles concentrate near the aggregate surface, and larger particles accumulate further away, as seen in Fig. 28 (*ibid*). The Wall Effect results in much cement being excluded from the region of the aggregate, resulting in increased porosities around the aggregate face [71]. This suggests that the ITZ is not a fixed zone with a fixed porosity but rather a transition from a highly porous microstructure to the bulk HCP that normalises outwards from the aggregate face.

Because the ITZ is generally more porous than the bulk HCP, it can potentially convey aggressive ingress into the concrete microstructure. This is particularly true where nearby ITZs overlap, resulting in continuous porous routes [71], which may significantly impact the strength and durability of hardened concrete [68]. The same thin sections, equipment (including the settings) and software used to determine the average pore size and distribution for mixes analysed in this paper were used to study the average pore size in the ITZ. The highest porosities are found in the first 20 μm from the aggregate, while zones $> 40 \mu\text{m}$ from the aggregate are similar to the bulk HCP. This study analysed the microstructure between the aggregate and the ITZ with a size of approximately 30 μm , similar to Diamond and Huang [71].

Fig. 29 presents the BSE images for PCAR mixes made with 0% (a) and 100% ABS WEEEP (c) replacements and segmented and processed images (b and d) at 28-days. ABS WEEEP and pores have a low atomic number; they appear dark, while heavier elements, such as the aggregate, appear bright [71,72].

Comparing the pore size and distribution in Fig. 29, images (b-2) and (d-2), it is clear that concrete containing no ABS WEEEP had fewer, smaller and less interconnected pores compared to the same mix using ABS WEEEP. Fig. 30 shows the average pore size in the

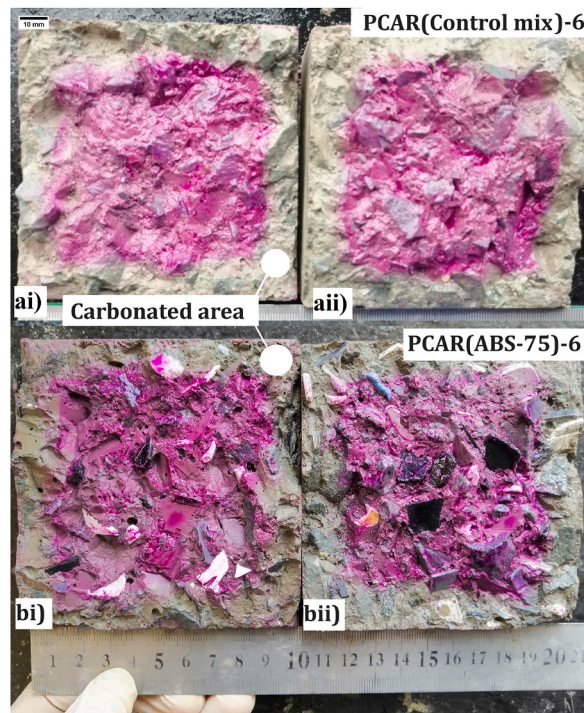


Fig. 23. Carbonation depth of mixes containing 0% and 75% ABS WEEEP.

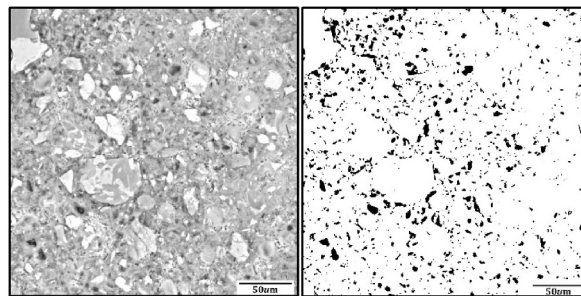


Fig. 24. BSE image of the HCP (LHS) and pore size and distribution in the bulk HCP (RHS) for WC(Control mix)-6.

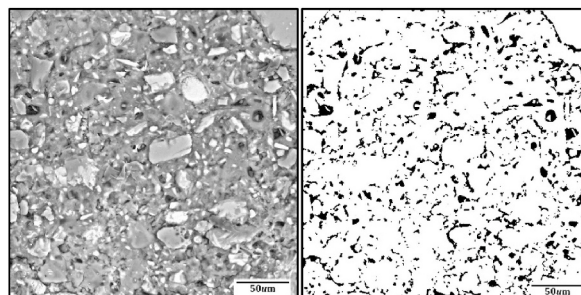


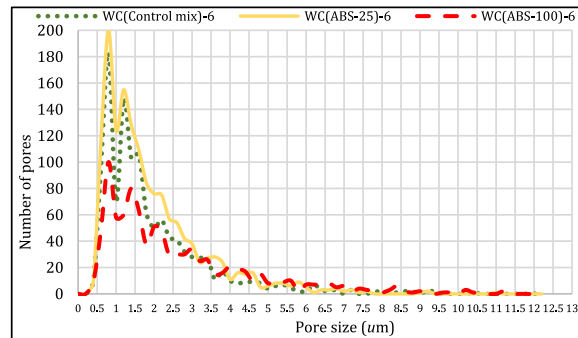
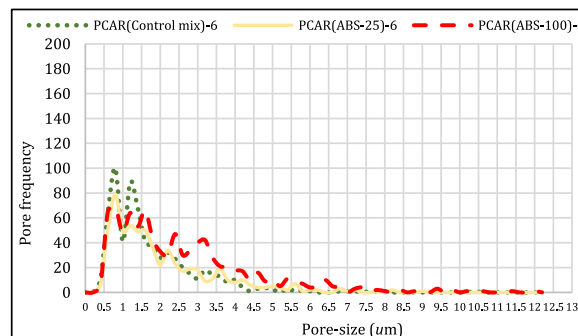
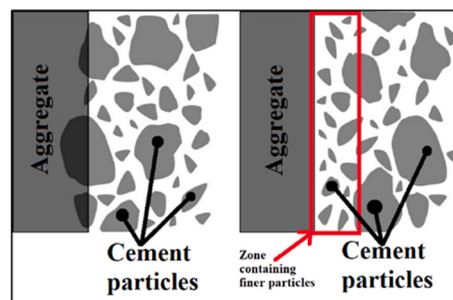
Fig. 25. BSE image of the HCP (LHS) and pore size and distribution in HCP (RHS) for WC(ABS-100)-6.

ITZ for concrete made using 0%, 25% and 100% ABS WEEEP replacements for WC and PCAR mixes measured from two thin sections made from the same mix design at 28-days. It was observed that concrete with 0% ABS WEEEP replacements have an average pore size of $4.453\ \mu\text{m}$, while 100% ABS WEEEP replacements have an average pore size of $12.107\ \mu\text{m}$.

Table 7

Summary of the average pore size and % area of the map covered by pores.

Mix design label	Average pore size (μm)	% Area of the map covered by pores
WC(Control mix)-6	4.078	7.089
WC(ABS-25)-6	5.579	8.153
WC(ABS-100)-6	8.169	10.442
PCAR(Control mix)-6	3.549	3.193
PCAR(ABS-25)-6	4.750	4.261
PCAR(ABS-100)-6	7.275	8.765

**Fig. 26.** Pore size and distribution for WC concrete containing 0%, 25% and 100% ABS WEEEP.**Fig. 27.** Pore size and distribution for PCAR concrete containing 0%, 25% and 100% ABS WEEEP.**Fig. 28.** Wall effect [70].

4.4. Compressive strength

The average compressive strength (f_{cu}) from three specimens per mix is shown in Table 8. It should be noted that the compressive strength observed may be different compared to mixes made onsite as they are often subjected to different curing conditions, compaction and stresses. Figs. 31 and 32 compare WC and PCAR mixes at 7 and 28-days, respectively.

It was noted that the average compressive strength decreased with increased WEEEP replacements, regardless of the curing regime

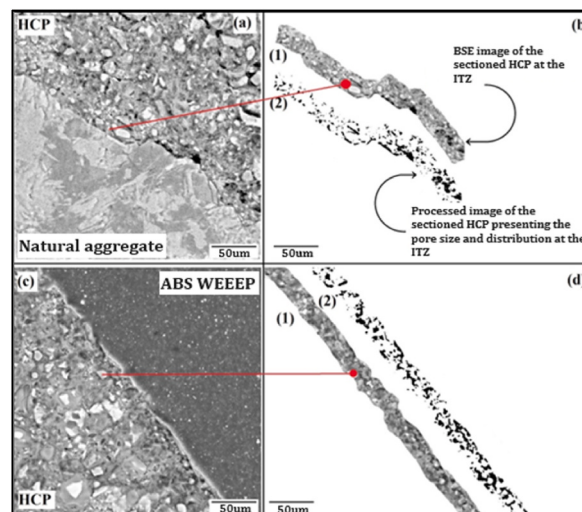


Fig. 29. ITZ between PCAR(Control mix)-6 and PCAR(ABS-100)-6 at 28-days.

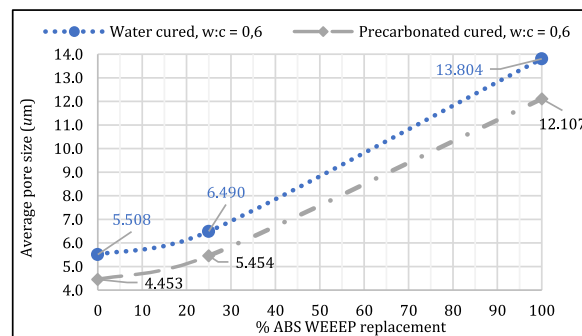


Fig. 30. Average pore size for WC and PCAR 0%, 25% and 100% ABS WEEEP replacements at 28-days.

Table 8

Average compressive strength.

Concrete mix label	3 Day			7 Day			28 Day			90 Day		
	fcu (MPa)	StD	CoV (%)	fcu (MPa)	StD	CoV (%)	fcu (MPa)	StD	CoV (%)	fcu (MPa)	StD	CoV (%)
WC(Control mix)-6	33.2	1.71	5.16	39.8	1.26	3.16	47.8	0.87	1.83	52.1	0.23	0.44
WC(ABS-5)-6	32.2	0.76	2.37	38.8	0.18	0.47	45.7	0.32	0.70	49.5	0.80	1.61
WC(ABS-10)-6	30.2	2.52	8.34	35.5	0.41	1.15	43.4	0.83	1.92	46.1	0.57	1.24
WC(ABS-25)-6	25.1	0.18	0.71	28.6	0.85	2.98	38.8	0.29	0.75	40.8	0.40	0.98
WC(ABS-50)-6	20.5	0.47	2.29	26.9	1.53	5.68	32.2	1.67	5.19	33.9	1.94	5.73
WC(ABS-75)-6	17.2	0.27	1.59	21.3	0.28	1.30	26.6	1.36	5.13	28.5	0.25	0.88
WC(ABS-100)-6	17.4	0.91	5.25	16.4	0.65	3.94	17.7	1.18	6.68	27.7	0.84	3.02
PCAR(Control mix)-6	37.1	1.97	5.31	43.2	1.89	4.36	48.7	0.88	1.81	52.6	1.27	2.41
PCAR(ABS-5)-6	36.1	0.62	1.73	42.4	1.01	2.39	46.6	1.03	2.20	49.1	1.30	2.64
PCAR(ABS-10)-6	33.4	0.34	1.00	38.4	0.37	0.97	43.9	1.29	2.94	45.7	0.96	2.09
PCAR(ABS-25)-6	28.8	3.29	11.41	32.5	0.18	0.55	38.5	0.63	1.62	40.1	4.11	10.27
PCAR(ABS-50)-6	24.2	0.93	3.84	28.5	1.33	4.67	32.2	0.53	1.65	32.6	0.26	0.79
PCAR(ABS-75)-6	20.9	0.32	1.52	23.6	0.34	1.45	25.7	0.40	1.57	29.5	0.70	2.36
PCAR(ABS-100)-6	20.7	1.12	5.44	18.7	0.57	3.04	18.7	0.23	1.20	28.7	0.85	2.98

used. At 7-days, mixes containing $\leq 50\%$ ABS WEEEP replacements achieved the minimum requirements for use as structural concrete. It was observed that PCAR mixes made using 25% ABS WEEEP increased by 3.9 MPa, while 75% and 100% replacements increased by 2.3 MPa.

Fig. 32 shows that all mixes but 100% ABS WEEEP replacements achieved the minimum strength requirement for structural concrete at 28-days. It was noted that there were few differences in the compressive strength of PCAR compared to WC. The largest increase in strength occurred at 100% ABS replacements, increasing by 1.02 MPa, while the smallest was 0.82 MPa, which occurred at

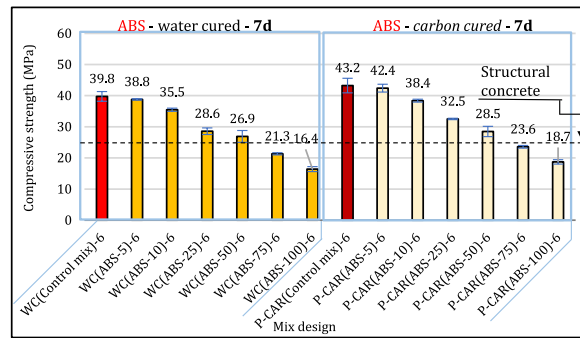


Fig. 31. Compressive strength of WC and PCAR cured concrete at 7-days.

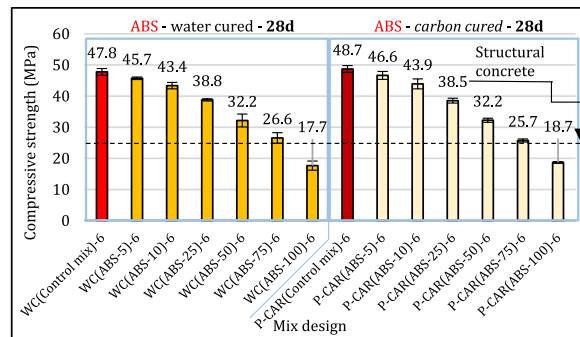


Fig. 32. Compressive strength of WC vs PCAR cured concrete at 28-days.

75% replacements with little effect on the compressive strength at 28-days. It was noted that there was a gradual decrease in compressive strength with an increase in ABS WEEEP replacements. Results indicate that 50% ABS WEEEP replacements reduced the compressive strength by 32.3% for WC concrete and 34.1% for PCAR concrete, which is lower than other studies, which replaced $\leq 30\%$ WEEEP [4,16,73].

Fig. 33 relates average compressive strength to the average pore size in the bulk HCP. The graphs show a strong negative linear correlation between the compressive strength and the average pore size in the bulk HCP.

4.5. Failure mode

The strength reduction associated with increased average pore size in the bulk HCP may be due to high localised straining around the larger pores. One reason for the development of larger pores associated with higher replacements of ABS WEEEP could be due to incomplete compaction due to the platy nature of WEEEP (as shown in Section 2.2 (Fig. 5)). Another cause for the reduced strength may be due to the bonding between the relatively smooth ABS WEEEP and HCP. The phenomenon is termed mechanical interlacing, whereby hydration products of cement penetrate the cavities of the aggregate, which, in turn, acts as several hooks to which the paste can bond [74,75].

Cores with a diameter of 25 mm were drilled from 100 mm³ concrete cubes at 28 days and dried similarly to processes described in Section 3.9 for the mixes shown in Table 7 previously shown. The cores were placed in a mechanical vice and tapped with a hammer to

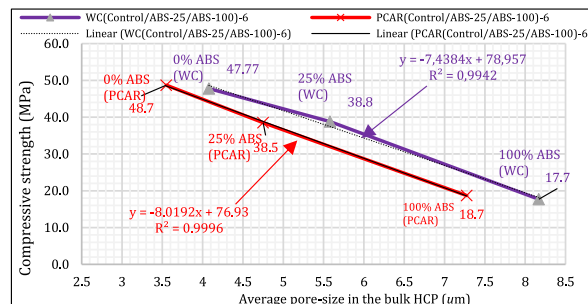


Fig. 33. Correlation between the average compressive strength average and pore size in the bulk HCP.

expose the aggregate and HCP interface. Fig. 34 shows the negative (A and C at the top) and positive (B and D at the bottom) faces for WC(ABS-25)-6 and PCAR(ABS-25)-6. The HCP at A and E was previously joined to the natural aggregate, whereas the HCP at B, C and D were previously connected to ABS WEEP. Based on these photographs, the HCP behind the natural aggregate appears rougher than the HCP behind ABS WEEP.

4.6. Durability index of concrete

The durability indexing tests are subdivided into three main categories, namely: oxygen permeability index (OPI), water sorptivity index (WSI) and chloride conductivity index (CCI). Each test has a different indexing value indicating concrete performance, as shown in Table 9.

4.6.1. Oxygen permeability index (OPI)

Table 10 presents the average (from four discs) OPI values for WC and PCAR cured mixes using ABS WEEEP as aggregate replacement material at 7 and 28-days. Results show a decrease in OPI with an increase in ABS WEEEP.

Fig. 35 indicates that mixes subjected to WC curing containing 25% and fewer ABS WEEEP replacements achieved excellent durability, while 75% ABS WEEEP replacement had good durability. The graph shows a considerable increase of 0.57 in OPI for PCAR mixes containing 100% ABS WEEEP replacements. However, since the error bars for PCAR containing 50% and higher replacements are large and overlap, these values should be studied with care.

Fig. 36 shows PCAR initially had a higher rate of strength loss for mixes containing 50% and fewer ABS WEEEP replacements, while WC mixes had higher rates of OPI loss at higher ($\geq 75\%$) replacements. Similar results have been found by Faraj et al. [77], which showed that concrete made using a w:c ratio of 0.32 and replacing the coarse aggregate with recycled polypropylene increased by 67.08% from $3.22 \times 10^{-16} \text{ m}^2$ to $5.38 \times 10^{-16} \text{ m}^2$ at 40% replacements.

Fig. 37 suggests a negative correlation between the average pore size observed in the bulk HCP and the OPI value of concrete. Mixes containing no ABS WEEEP replacements had the smallest average pore size and highest OPI value, while mixes containing large pores tended to have the lowest OPI values. The larger size and number of pores may allow the oxygen gas to pass through the concrete at a higher rate. A similar observation can be made by referring to the pore size and distribution curves in Figs. 26 and 27 (Section 4.3.1). Mixes containing no ABS WEEEP often had few large pores, while mixes containing 100% ABS WEEEP replacements tend to have the largest and highest number of pores.

4.6.2. Water sorptivity index (WSI)

Table 11 presents the average WSI values (from four discs) for mixes subjected to WC and PCAR curing using ABS WEEEP as aggregate replacement material at 7 and 28-day curing.

Fig. 38 indicates that all mixes made using 5% and fewer ABS WEEEP replacements showed excellent durability performance, while those made using $\geq 25\%$ had good durability performance, with the WC mix containing 100% being the exception. Fig. 39 showed a higher rate of sorptivity for PCAR mixes at 25% and higher ABS WEEP replacements, which are similar to the results by Abu-Saleem et al. [78] and Hannawi et al. [79].

Fig. 40 shows a strong positive linear correlation between the sorptivity and average pore size in the bulk HCP, which suggests that the sorptivity of mixes increased with increased average pore size due to increased ABS WEEEP replacements. These larger pores have a higher capacity to allow fluid to be absorbed into the concrete matrix, resulting in higher fluid absorption rates.

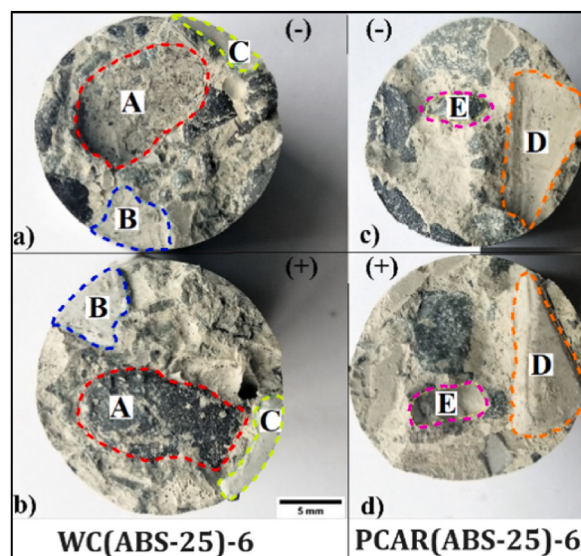


Fig. 34. HCP at the aggregate interface.

Table 9
Suggested durability index range [76].

Index test	Estimate of concrete performance			
	Excellent	Good	Poor	Very poor
OPI	> 10	9.5	9.0	< 9.0
WSI	< 6.0	6.0	10.0	> 15.0
CCI	< 0.75	0.75	1.50	> 2.50
		1.50	2.50	

Table 10
Average OPI for WC and PCAR mixes containing ABS WEEEP.

Concrete mix label	OPI data at 7-days			OPI data at 28-days		
	Average (index)	StD	CoV (%)	Average (index)	StD	CoV (%)
WC(Control mix)-6	10.47	0.12	1.12	10.72	0.12	1.09
WC(ABS-5)-6	10.03	0.15	1.46	10.42	0.23	2.20
WC(ABS-10)-6	10.08	0.10	0.98	10.49	0.16	1.52
WC(ABS-25)-6	9.82	0.06	0.57	10.00	0.37	3.75
WC(ABS-50)-6	9.51	0.17	1.75	9.96	0.13	1.26
WC(ABS-75)-6	9.36	0.24	2.59	9.57	0.29	3.01
WC(ABS-100)-6	8.88	0.19	2.15	9.30	0.17	1.80
PCAR(Control mix)-6	10.87	0.18	1.65	11.11	0.09	0.84
PCAR(ABS-5)-6	10.26	0.35	3.38	10.61	0.06	0.59
PCAR(ABS-10)-6	10.32	0.09	0.90	10.52	0.21	1.97
PCAR(ABS-25)-6	10.32	0.40	4.03	10.15	0.08	0.78
PCAR(ABS-50)-6	9.72	0.16	1.68	10.15	0.66	6.47
PCAR(ABS-75)-6	9.49	0.13	1.32	10.00	0.75	7.48
PCAR(ABS-100)-6	9.15	0.11	1.16	9.87	0.84	8.48

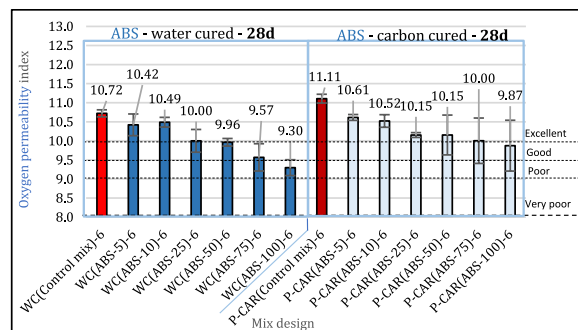


Fig. 35. OPI values of WC and PCAR cured concrete at 28-days.

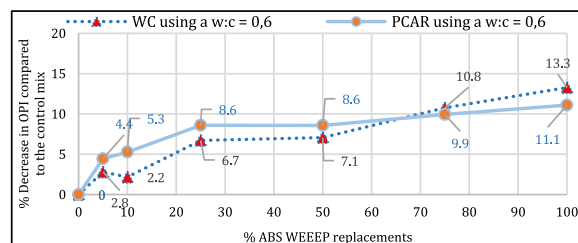


Fig. 36. Comparison of the OPI (% decrease) for WC and PCAR mixes at 28-days.

4.6.3. Chloride conductivity index

Table 12 presents the average (from four discs) CCI values for mixes subjected to WC and PCAR curing using ABS WEEEP as aggregate replacement material at 7 and 28-day curing.

Fig. 41 indicates that all mixes made using $\leq 10\%$ ABS WEEEP had good durability performance. Furthermore, WC mixes made

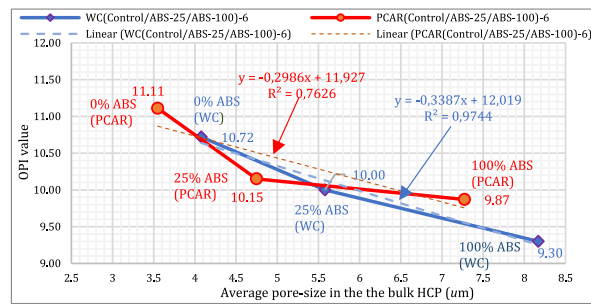


Fig. 37. Correlation between OPI and pore size in the bulk HCP.

Table 11

Average WSI at 7 and 28-day curing for concrete containing ABS WEEEP.

Concrete mix label	WSI data at 7-days			WSI data at 28-days		
	Mean (mm/ $\sqrt{h}$)	StD	CoV (%)	Mean (mm/ $\sqrt{h}$)	StD	CoV (%)
WC(Control mix)-6	7.48	0.64	8.50	4.31	0.55	12.86
WC(ABS-5)-6	8.78	0.63	7.14	5.58	0.56	10.02
WC(ABS-10)-6	10.94	0.36	3.31	7.14	0.24	3.31
WC(ABS-25)-6	11.28	0.80	7.07	8.52	1.00	11.76
WC(ABS-50)-6	11.72	0.36	3.08	9.82	0.23	2.30
WC(ABS-75)-6	12.07	0.68	5.65	9.98	0.40	4.00
WC(ABS-100)-6	12.68	0.75	5.88	10.01	1.39	13.86
PCAR(Control mix)-6	3.47	0.30	8.53	3.16	0.26	8.32
PCAR(ABS-5)-6	4.80	0.62	12.96	4.69	0.63	13.34
PCAR(ABS-10)-6	5.29	0.61	11.63	6.01	0.26	4.37
PCAR(ABS-25)-6	8.71	0.26	2.96	7.40	0.13	1.71
PCAR(ABS-50)-6	11.82	1.29	10.94	8.85	0.35	3.97
PCAR(ABS-75)-6	11.53	2.34	20.30	9.20	0.54	5.89
PCAR(ABS-100)-6	12.09	1.01	8.38	9.46	0.35	3.74

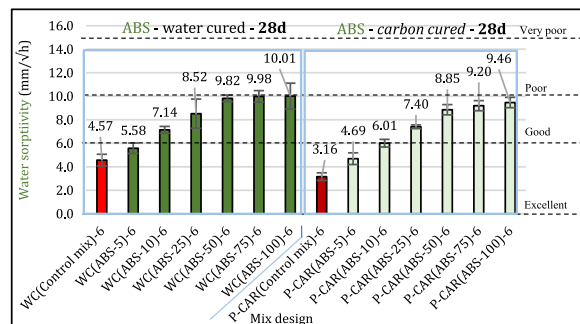


Fig. 38. WSI values of WC vs PCAR cured concrete at 28-days.

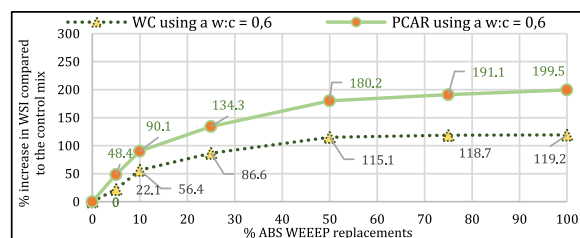


Fig. 39. Comparison of the WSI (% increase) for WC and PCAR mixes.

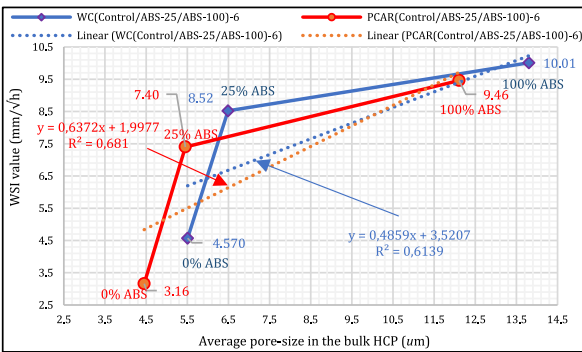


Fig. 40. Correlation between WSI and pore size in the bulk HCP.

Table 12
Average CCI at 7 and 28-days curing for concrete containing ABS WEEEP.

Concrete mix label	CCI data at 7-days			CCI data at 28-days		
	Mean (mS/cm)	StD	CoV (%)	Mean (mS/cm)	StD	CoV (%)
WC(Control mix)-6	1.36	0.11	8.10	1.03	0.04	3.88
WC(ABS-5)-6	1.82	0.08	4.51	10.7	0.06	6.01
WC(ABS-10)-6	1.95	0.14	7.05	1.37	0.07	4.76
WC(ABS-25)-6	2.04	0.15	7.54	1.85	0.14	7.38
WC(ABS-50)-6	2.69	0.16	5.83	2.23	0.08	3.70
WC(ABS-75)-6	3.52	0.26	7.48	2.75	0.21	7.47
WC(ABS-100)-6	3.71	0.12	3.26	2.96	0.07	2.50
PCAR(Control mix)-6	1.11	0.05	4.06	0.77	0.04	5.86
PCAR(ABS-5)-6	1.61	0.05	3.14	0.85	0.07	8.10
PCAR(ABS-10)-6	1.77	0.10	5.58	0.92	0.09	10.11
PCAR(ABS-25)-6	1.89	0.15	7.95	1.49	0.14	9.19
PCAR(ABS-50)-6	2.37	0.13	5.28	1.76	0.09	5.21
PCAR(ABS-75)-6	2.52	0.11	4.42	2.09	0.09	4.13
PCAR(ABS-100)-6	2.84	0.05	1.81	2.14	0.09	4.24

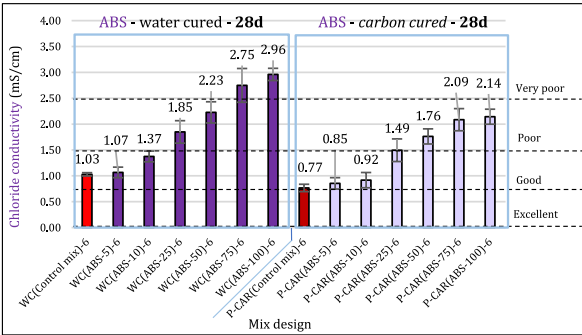


Fig. 41. CCI values of WC vs PCAR cured concrete at 28-days.

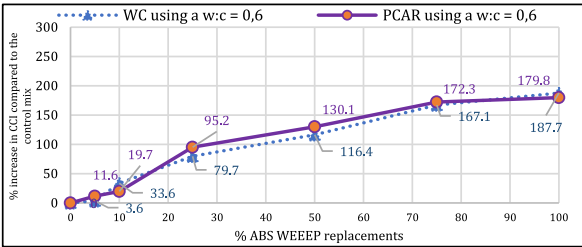


Fig. 42. Comparison of the CCI (% increase) for WC and PCAR mixes.

using 25% and 50% ABS WEEEP had poor durability characteristics, while higher replacements suggest very poor durability. PCAR mixes containing 50% and higher replacements showed poor durability. It was noted that PCAR rapidly increased the CCI between 10% and 25% ABS WEEEP replacements. The graphs showed that all mixes subjected to PCAR curing had lower CCI than WC curing.

Fig. 42 shows that the largest rate of CCI loss occurred at 100% ABS WEEEP replacements in WC and PCAR mixes. It also suggests that the largest difference between the two curing regimes occurred at 50% replacements. These increases are low compared to a study by Balasubramanian et al., who performed a rapid chloride permeability test of concrete made using mix WEEEP as a replacement for the coarse aggregate and a w:c ratio of 0.50 [80]. Their results showed 3500 Coulombs (C) for the control mix, and 3968 C and 4527 C for 10% and 20% replacement for the natural aggregate, which relates to an increase of 13.37% and 29.34%. Then again, Faraj et al. [77] showed that concrete made using a w:c ratio of 0.32 and replacing the coarse aggregate with recycled polypropylene increased by 36.25% from 3070 C to 4183 C at 40% plastic replacements. It should be noted that the increase observed from the rapid chloride permeability is comparable to the CCI, as the two tests have a good linear correlation [51].

The data in Fig. 43 indicate a strong positive correlation between the CCI and average pore size in the HCP, with increased ABS WEEEP replacements. It should be pointed out that the average pore size does not indicate the number of pores in the HCP, as shown in Figs. 26 and 27 (Section 4.3.1). The increased number of larger pores measured may have allowed more chloride ions to pass through the specimen when a voltage was applied across the disc, which could account for the higher CCI observed.

4.7. Service life prediction

4.7.1. Carbonation depth for an inland environment

Weather carbonation is a gradual process in which atmospheric CO_2 (concentrations at around 0.035%) penetrates concrete and reacts with CH to produce CaCO_3 . This process lowers the alkalinity of concrete from about 12 to nine [81]. This reaction process advances as a front into the concrete matrix.

The carbonation depth is often expressed by the relationship between the carbonation coefficient (K) ($\text{mm}/\sqrt{\text{yr}}$) (is a function of concrete properties and environmental exposure conditions) and the time to initiation (t) (years) (time required for the carbonation front to reach the steel reinforcement at which corrosion will initiate) as shown below in Equation (6) [81].

$$D = K\sqrt{t} \quad (6)$$

The above equation assumes a uniform pore structure across the concrete depth and constant environmental exposure. It also assumes that the quality of the concrete remains constant across the measured depth. The $\sqrt{t}$ is often believed to overestimate the time to de-passivate the concrete structure (*ibid*). Thus, an alternative expression shown in Equation (7) uses a power series constant ' n ' depending on the exposed environment.

$$D = Kt^n \quad (7)$$

The n -value is often taken as < 5 if the concrete is subjected to wet and dry cycles, as the carbonation rate will be lower as water in the pores reduces carbon dioxide gas from entering the concrete matrix. The n -value is often taken as four in coastal regions in South Africa, while a more conservative n -value of five is used in inland environments such as Johannesburg due to the drier climate [50].

This study used the carbonation prediction model proposed by Ikotun [82] for use in an inland environment (Johannesburg, South Africa) exposed to a sheltered outdoor setting for a 50-year service life (where the 50-year is often designated for buildings and other common structures [52]), as shown in Equation (8), using the OPI values from Table 10 to estimate the required cover depth.

$$K_{PC} = 81.50 - 8.08(OPI) + 1.77 \quad (8)$$

Fig. 44 on the next page shows that no significant de-passivation of steel reinforcement should occur in a 50-year service life for concrete containing $\leq 50\%$ ABS WEEEP replacements. Greater cover depths are recommended for higher replacements, as the carbon front may have passed the reinforcing steel at 50 years or less. Fig. 45 shows that all PCAR mixes should provide adequate protection for a 50-year service life with a minimum cover depth of 30 mm (cover depth recommended for most steel-reinforced concrete structures subjected to moderate exposure conditions [83]). Even so, it is clear from Fig. 44 that WC(ABS-100)-6 may be used in a

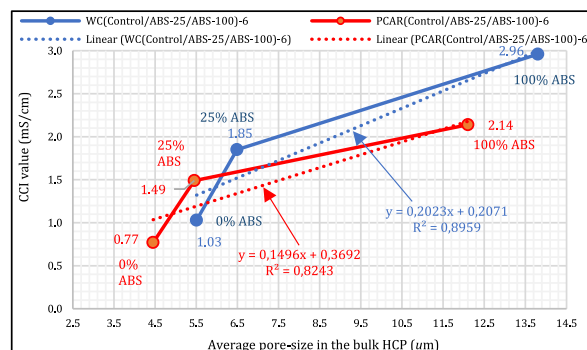


Fig. 43. Correlation between CCI and pore size in the ITZ.

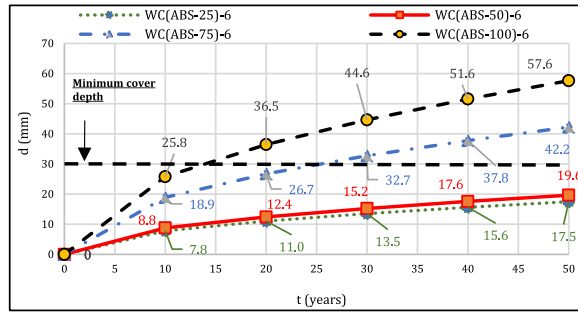


Fig. 44. Depth to the carbonation front for the inland environment for WC mixes.

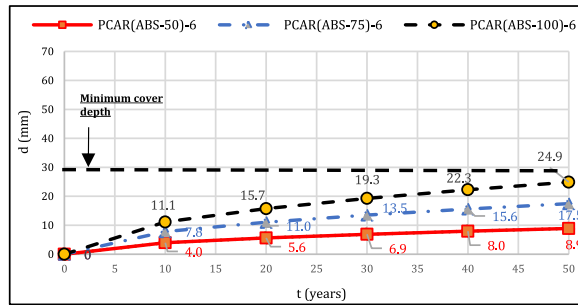


Fig. 45. Depth to the carbonation front for the inland environment for PCAR mixes.

reinforced concrete structure and should have adequate durability performance with a recommended concrete cover depth ≥ 57.6 mm (but preferably 60 mm for ease of workmanship on-site) for a 50-year service life.

Thus, Regarding the durability of reinforced concrete structures and, in particular, the resistance to carbonation in an inland environment, using a cover depth of 30 mm over a service life of 50-years, concrete using a w:c ratio = 0.6, containing $\leq 50\%$ WEEP should provide satisfactory protection.

4.7.2. Chloride resistance for a coastal region

The South African ingress model was used to predict the long-term performance of mixes subjected to a marine environment. The chloride conductivity is related to steady-state diffusivity D_s through a relationship shown in Equation (9) [84]:

$$Q = D_s / D_0 = \sigma / \sigma_0 \quad (9)$$

where Q is the diffusivity ratio, D_s is the steady state of the chloride ions through the concrete (m^2/s), D_0 is the diffusivity of the chloride ions through the concrete (m^2/s), σ is the conductivity of concrete (S/m), and σ_0 is the conductivity of the pore solution. However, a non-steady state exists, and the diffusion coefficient can be determined using Fick's second law of diffusion, which relates the rate of change of concentration with concerning and spatial position, as shown in Equation (10) below [85]:

$$\partial C / \partial t = D \left(\partial^2 C / \partial x^2 \right) \quad (10)$$

where D is the diffusion coefficient. A modified version of Fick's second law is shown in Equation (11), which uses a Gaussian error function to allow for a change in surface concentration and diffusion coefficient with time (*ibid*).

$$C_x = C_s(t) \left(1 - \text{erf} \left[\frac{x}{2\sqrt{D_c(t)t}} \right] \right) \quad (11)$$

Here, C_x is the chloride concentration at a depth x at a given time t , $C_s(t)$ is the surface chloride concentration (%), D_c is the decreasing chloride diffusion with time t (seconds), x is the depth into the concrete (cm), and the *erf* is the mathematical error function. Using Fick's modified second law of diffusion, the predicted service life was estimated by determining the chloride concentration at the surface of the steel reinforcing for a structure exposed for 50 years in a marine environment. The D_c value was determined using a nomogram, which links the 28-day CCI value to a respective diffusion coefficient depending on the binder type and exposure condition. This nomogram was developed for the Western Cape environmental conditions and materials and allows for long-term effects such as chloride binding and continued hydration [86]. There are three main types of exposure conditions for marine structures: extreme, very severe and severe. This study evaluated the service life for a structure exposed to severe conditions, as this should provide a general

overview of the performance of ABS WEEEP in concrete. Furthermore, this study used a conservative chloride threshold of 0.4% by mass of total cementitious content, while a surface concentration C_s of 3.0% by mass of total cementitious content was used, as recommended by Mackechnie [86].

Fig. 46 shows the chloride profiles for mixes containing ABS WEEEP. The graph indicates that a minimum cover depth of 60 mm for the control mix should provide adequate protection as it is below the 0.4% threshold. Similarly, an 80 mm cover should provide sufficient protection for mixes using 25% ABS WEEEP replacements. On the other hand, mixes containing 100% ABS WEEEP may not provide adequate protection to reinforcing steel, even with a cover of 100 mm. Then again, Fig. 47 indicates that all mixes subjected to a PCAR curing regime and a cover depth of ≤ 80 mm should provide sufficient protection for a 50-year service life.

Thus, the results indicate that all mixes made using a w:c ratio = 0.6 and a cover depth of 80 mm should provide satisfactory performance for mixes containing $\leq 25\%$ WEEEP replacements.

5. Conclusions

The following conclusions are based on the experimental results conducted under laboratory conditions:

- The replacement of the natural aggregate with ABS WEEEP strongly affects the unit density of concrete.
- BSE image analysis can be used to visualise the pore size and distribution at the ITZ and bulk HCP of hardened concrete made using ABS WEEEP. Additionally, OPM image thresholding from BSE images and ImageJ software can indicate concrete pore size and distribution using ABS WEEEP as aggregate.
- The average pore size in the bulk HCP and ITZ increased with an increase in ABS WEEEP replacements. All mixes subjected to the PCAR curing regime showed reduced average pore size and distribution in the bulk HCP and ITZ.
- The average compressive strength of concrete reduced with the increase in ABS WEEEP replacements at all ages and curing regimes.
- All ABS WEEEP replacements $\leq 75\%$ in concrete achieved minimum strength requirement for use as structural concrete ($f_{cu} \geq 25$ MPa) at 28-days.
- Mixes subjected to the PCAR curing regime had higher compressive strengths than WC mixes at 3 and 7 days but similar strengths at 28 and 90 days.
- Results showed a strong negative linear correlation between the reduction in compressive strength and an increase in average pore size in the ITZ with the rise in WEEEP replacements.
- Incremental replacements of ABS WEEEP for the natural aggregate gradually reduced the durability indexes of concrete at 7 and 28-days.
- All mixes subjected to PCAR curing had higher durability performance than those subjected to WC alone at 7 and 28-days.
- Strong negative linear correlations were obtained between the decrease in durability performance and the increase in the average pore size in the ITZ with an increase in ABS WEEEP replacements.
- Concrete used in steel-reinforced concrete structures (with a predicted service life of 50-years) exposed to an inland environment made with 100% ABS WEEEP as coarse aggregate should have a cover depth of ≥ 60 mm when subjected to WC curing regime or ≥ 30 mm when subjected to PCAR curing regime. It should be noted that a smaller cover depth can be used for mixes containing lower replacements of ABS WEEEP.
- Concrete used in steel-reinforced concrete structures (with a predicted service life of 50-years) subjected to a marine environment and exposed to “severe” conditions should provide adequate durability for mixes made using $\leq 25\%$ ABS WEEEP replacements for WC cured mixes, while all PCAR cured mixes could be used.

6. Use of ABS WEEEP as aggregate in the construction industry

One of the main reasons for choosing lightweight structural concrete is to reduce the structure's dead load for either economic gain, increase the span between two or more supports, or keep the weight of the building down due to foundation limitations. It also allows the designer to design freely, creating all types of infrastructure and futuristic building shapes [87]. Most lightweight structural concrete is derived from natural or processed material, such as expanded shale. At the same time, lightweight waste material such as WEEEP is dumped chiefly due to the presence of halogenated flame-retardant additives. The results from this study suggest that 75%

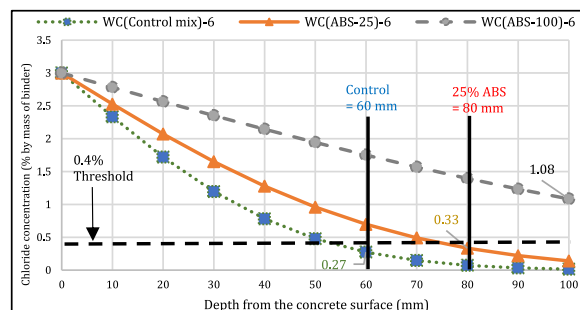


Fig. 46. Minimum cover depths required for the various ABS WEEEP replacements for a 50-year service life for WC mixes.

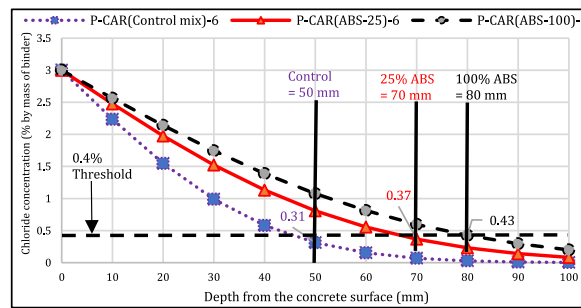


Fig. 47. Minimum cover depths required for the various ABS WEEEP replacements for a 50-year service life for PCAR mixes.

ABS WEEEP replacements for the natural aggregate can produce lightweight concrete ($\leq 1980 \text{ kg/m}^3$) for use in structural applications with a minimum characteristic strength of 25 MPa at 28-days of curing.

These are the minimum strength requirements set by BS 5400 and Australian Standards (AS) 3600 for structural lightweight aggregate concrete (SLWAC) [88]. The codes indicate no maximum strength but are limited to 50 MPa for reinforced concrete and 60 MPa for prestressed concrete (*ibid*). Results in Section 4 showed that replacing 75% of the natural aggregate with ABS WEEEP achieved the minimum strength requirements ($\geq 25 \text{ MPa}$) for use as lightweight aggregate concrete set in BS 5400 and AS 3600 at 28-days [88]. SLWAC is becoming more widely accepted by clients, consultants and contractors for use in floor systems, columns, ramps and large monolithic castings in bridges, car parks, in-situ buildings and precast sections (*ibid*). For example, Japan's 109 m tall Commercial Centre Tower used lionite and expanded shale to produce SLWAC from the 3rd to the 26th floor. The Standard Bank in Johannesburg used a combination of 20 mm aglite and 10 mm expanded clay to produce SLWAC to construct the double-T precast floor slabs [89]. A four-level parking garage in Dubuque, Iowa, used SLWAC in the floor slab to reduce the unit weight of concrete to increase the spacing between columns. This, in turn, increased the parking capacity of the structure. In terms of structural concrete, WEEEP could be used in any of the scenarios mentioned above where the self-weight of the structure becomes the critical factor in design.

7. Recommended future research

ABS WEEEP plastics are well known for their ability to deform plastically under an applied load. Therefore, future research must investigate the magnitude and rate of creep strain and concrete modulus of elasticity to prevent excessive deflections of structural members under load throughout their service life. Additionally, due to the low glass transition temperature associated with ABS, additional research on the fire resistance of concrete is required to ensure the structure and occupants safety during and after exposure to fire. Lastly, It is suggested that future research study the effect WEEEP substitutions may have on the behaviour of reinforced concrete elements such as slabs, walls and beams. It should be noted that due to the limitation of the relatively small number of microstructural experimental tests on the HCP and ITZ, the effect of replacing the natural coarse aggregate with ABS WEEEP needs further study.

Funding

This work was partially supported by the Post Graduate Merit Award of the University of the Witwatersrand, Johannesburg, South Africa.

CRediT authorship contribution statement

Lewis A. Parsons: Conceptualization, Data curation, Formal analysis, Funding acquisition, Investigation, Methodology, Project administration, Software, Validation, Visualization, Writing – original draft, Writing – review & editing. **Sunday O. Nwaubani:** Conceptualization, Data curation, Formal analysis, Funding acquisition, Investigation, Methodology, Project administration, Supervision, Validation, Visualization, Writing – review & editing.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Data will be made available on request.

Acknowledgements

The authors are grateful for the donations of cement and natural aggregate from AfriSam and superplasticiser from Sika. The authors are thankful for the assistance and use of the XRD, SEM and EPMA instruments at the Microscopy and Microanalysis Unit at the

University of the Witwatersrand. The authors are grateful for the plastic donation from Computer Scrap Recycling and AST Recycling.

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