Scientific paper

Assessing the Performance and Transport Properties of Concrete using Electrical Property Measurements

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Received 21 March 2020, accepted 5 July 2020

doi:10.3151/jact.18.437

Abstract

The electrical properties of porous systems are intimately linked to mass transport and flow processes such as diffusion and permeability and offer a simple testing methodology for assessing those properties which are responsible for the durability and long-term performance of construction materials. In the current study, electrical impedance spectra for concretes containing both plain and blended Portland cement binders were obtained over a period of 360 days. In-situ impedance measurements were used to accurately identify the bulk resistance (hence evaluation of resistivity) of the concretes and the optimum frequency range for bulk resistance measurements. The bulk resistivity was normalised by that of the pore-fluid resistivity obtained from computer simulations and the results indicated that the pore-fluid resistivity decreased only marginally with time once the hydration process had advanced beyond 28 days. It is shown that the normalised resistivity – termed the Formation Factor – displayed a continual increase with time, highlighting on-going hydration/pozzolanic reaction and pore structure refinement over the entire test period. This was particularly evident for the slag concretes. Using the normalisation process, a simple approach is presented to evaluate the effective diffusion coefficient of the concretes and a durability/performance classification system, based on the Formation Factor, is presented.

1. Introduction

The durability and long-term performance of reinforced concrete structures depend on the microporous nature of the cementitious binder and the environment to which the structure is exposed (Jennings *et al.* 1996; McCarter *et al.* 2001; Maekawa *et al.* 2008; Song *et al.* 2008; Kim *et al.* 2016). For reinforced concrete structures located in, or near, the marine environment, or where de-icing salt is used for winter maintenance purposes, it is the protective qualities of concrete against chloride-induced corrosion of the steel reinforcement which are of considerable interest to engineers (Streicher and Alexander 1995; Yokota *et al.* 2009; Ueda *et al.* 2020). In practice, the ability of concrete to protect the embedded steel (hence chloride ingress) is based on a prescriptive or deem-to-satisfy

approach (Andrade *et al.* 2013; McCarter *et al.* 2017) and often equated with strength. In BS EN206-1:2013 (BSI 2016) and BS 8500-1:2015 (BSI 2019a), for example, a list of deterioration mechanisms, each containing a prescriptive measure of minimum grade, maximum water-binder ratio, and minimum binder content, is specified to ensure durability and is used in conjunction with a series of well-defined environmental classifications.

With increasing importance placed on producing durable concrete, there is now a growing interest and, indeed, an inevitable need to relate the desired performance of concrete to an easily measurable parameter through an appropriate performance-based test (hence performance-based approach) (Baroghel-Bouny *et al.* 2014; Alexander and Thomas 2015; McCarter *et al.* 2017; Beushausen *et al.* 2019). In this case, the focus is on those performance requirements which have a direct influence on the corrosion of the steel reinforcement e.g. chloride ion diffusivity, resistivity of concrete and corrosion potential (Tang and Utgenannt 2007; Nanukuttan *et al.* 2008; Moradi-Marani *et al.* 2010; Pack *et al.* 2010; Pang and Li 2016; Wattanapornprom and Ishida 2017; Kim *et al.* 2018; Chlayon *et al.* 2020; Kim *et al.* 2020; Tran *et al.* 2020). It should be noted that although BS EN206-1:2013 (BSI 2016) still defines prescriptive design methods for durability, Section 5.3.3 of this code allows for performance-related methods and defines concrete on the basis of an equivalent durability procedure (EDP); more details on the EDP are available in PD CEN/TR 16563 (BSI 2013). To fully implement a performance-based approach, the following need to be considered.

(i) Long-term experience of local materials and practices,

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Table 1 Summary of concrete mixes (w/b = 0.5).

Mix Designation	CEM I 52.5 N kg/m ³	GGBS kg/m ³	20 mm kg/m ³	10 mm kg/m ³	Fine (< 4 mm) kg/m ³	HRWR kg/m ³
PC	380	0	620	410	840	1.9
GGBS/35	247	133	617	407	835	1.9
GGBS/50	190	190	616	406	833	1.9
GGBS/65	133	247	614	406	832	1.9

Note: HRWR means high range water reducer.

Table 2 Oxide analysis of cementitious materials.

Material	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	CaO	MnO	MgO	TiO ₂	SO ₃	K ₂ O	Na ₂ O
CEM I	19.9	4.94	3.07	62.02	Not determined	2.08	n.d.	3.0	0.62	0.15
GGBS	33.27	13.38	0.56	41.21	0.3	8.49	0.9	n.d.	0.50	0.00

- and detailed knowledge of the local environment;
- (ii) Test methods based on approved and proven tests that are representative of actual conditions and have approved performance criteria; and,
- (iii) Analytical models that are developed based on the physics underlying the problem and are corroborated against test-data representative of actual conditions in practice.

With regard to the durability of concrete, it is the permeation properties of concrete which are of importance, and terms such as diffusivity, permeability and sorptivity are used in this respect (Guimarães *et al.* 2011; Patel *et al.* 2016; McCarter *et al.* 2000, 2018; Alaswad *et al.* 2018). As the permeation properties of concrete are intimately linked to microstructural features of concrete such as capillary pore size, pore-size distribution, pore tortuosity and the connectedness of the capillary pore network (McCarter *et al.* 2013b; Ishida *et al.* 2009), a testing technique that is capable of quantifying the hierarchical porous microstructure of the cement system and relating this information to the desired performance would address (ii) above. Given that the underlying mechanism associated with permeability and diffusion is analogous to the flow of current under a potential gradient (hence, electrical conductivity, or its reciprocal, electrical resistivity), it is understandable that there is now increasing attention directed towards utilisation of the electrical properties of concrete as a durability indicator/index (Sengul 2014; Van Noort *et al.* 2016; McCarter *et al.* 2017; Riding *et al.* 2018; Torres-Acosta *et al.* 2019). This recent initiative builds upon continued developments in the application of electrical property measurements that started to emerge around three decades ago, when the multi-frequency approach was first applied to cementitious systems (McCarter *et al.* 1988) and subsequently triggered widespread interests from the research community (Coverdale *et al.* 1993; Liu and Beaudoin 1999; Neithalath *et al.* 2006; Nokken and Hooton 2008). Electrical property measurements, as applied to cement systems, continued to evolve until today and are increasingly finding their way beyond research laboratories.

In this paper, the electrical properties of concrete – both the bulk concrete and the pore fluid within the cap-

Table 3 Physical properties of cementitious materials.

Material	Specific gravity	Blaine surface area (m ² /kg)
CEM I	3.15	400
GGBS	2.90	not determined

illary pore network – are developed to highlight the testing methodology as a potential performance-based testing technique. A concrete durability/performance classification system/criteria, based on intrinsic electrical properties, is presented with emphasis placed on assessing the relative performance of Portland cement (PC) concrete and concretes containing ground granulated blast furnace slag (GGBS) which is now routinely specified in concreting operations (Kanda *et al.* 2015; Hashimoto *et al.* 2016; Lye *et al.* 2016; Ishida *et al.* 2018; Matthes *et al.* 2018). Data are presented for concrete specimens undergoing continuous moist curing, and as such, results are directly relevant to concrete that remains saturated (little/no time to dry out). It is envisaged that the testing methodology could be developed and exploited as a pre-qualification test at the design stage, prior to concreting operations or during construction to ensure that the concrete, as delivered, is substantially the same as the pre-qualified concrete.

2. Experimental programme

2.1 Materials and mix proportions

The cementitious materials used in this experimental programme were CEM I 52.5 N Portland cement, conforming to BS EN197-1:2011 (BSI 2019b), and GGBS (Regen, supplied by Hanson UK) conforming to BS EN15167-1:2006 (BSI 2006). Four concrete mixes with a water/binder ratio of 0.5 were produced and summarised in **Table 1**: one mix contained plain Portland cement (CEM I), whereas the other three contained varying levels of GGBS with replacement levels of 35%, 50%, and 65%, which can all be categorised as CEM III/A (BSI 2019b). Typical oxide analysis and physical properties of the materials are presented in **Tables 2** and **3**. A high-range water-reducing (HRWR) admixture (MasterGlenium ACE499) was used in all mixes at a dosage of 0.5% by mass of binder.

2.2 Sample dimensions, fabrication and curing

A total of 21 cubes were produced for each mix; three cubes with dimensions of $150 \times 150 \times 150$ mm were used for electrical testing and the remaining eighteen cubes with dimensions of $100 \times 100 \times 100$ mm were used for mechanical testing. A pair of 2.4 mm diameter stainless steel (Grade 316L) pin-electrodes were embedded inside each 150 mm cube (see Fig. 1) during casting at a centre-to-centre spacing of 50 mm (i.e. $> 2 \times$ maximum aggregate size). Both pins were inserted into the centre of each cube at a depth of 100 mm from the top surface. They were secured in position by means of an acrylic former placed over the top centreline of each cube. Each pin was sleeved with heat shrink tubing apart from the 50 mm tip inside the cube (and a short tip outside of the cube) to facilitate electrical measurements (see Fig. 1). Embedding the electrodes in this fashion not only ensures intimate contact between the electrodes and concrete, but also negates any electrode/concrete interfacial effects on the measured electrical response [see, for example, McCarter *et al.* (2015a, 2015b)]. It should be noted this type of embedded electrode system is ideally suited for saturated concrete (i.e. concrete undergoing continuous moist curing). For partially saturated concrete (i.e. concrete exposed to wetting/drying cycles), the multi-electrode arrays, with pairs of electrodes placed at discrete depths from the exposed surface [see, for example, McCarter *et al.* (2012)], could be used.

Each concrete mix was manufactured at one-week intervals using a 200-litre pan mixer in accordance with BS EN 12390-2:2019 (BSI 2009). After mixing, the fresh concrete was cast into respective moulds (polystyrene moulds for the 150 mm cubes and steel moulds for the 100 mm cubes) before being compacted in two equal layers on a vibrating table; all moulds were treated prior to casting with a proprietary release agent. Immediately after casting, the cubes were covered with cling film to prevent moisture loss and demoulded at 24 hours before being placed in a curing tank located in a temperature-controlled laboratory ($20 \pm 2^\circ\text{C}$). The cubes remained in the curing tank until they were required for testing.

Fig. 1 Schematic diagram showing concrete sample and electrode configuration used in the experimental programme.

2.3 Electrical and mechanical testing

Once the appropriate curing age was achieved, each cube was removed from the curing tank, surface dried with a paper towel, and electrical measurement taken using a Solartron 1260 frequency response analyser (FRA). The analyser was operated in voltage drive mode at a signal amplitude of 350 mV rms, with logarithmic sweep measurements taken over the 1 Hz to 10 MHz frequency range at 20 points per decade. The measurements were taken using four individually screened short coaxial leads in a two-point mode configuration i.e. the current-generator and potential-high leads were coupled to one electrode, by means of an alligator clip, and the current-input and potential-low leads to the other electrode though another alligator clip. The cubes were returned to the curing tank and measurements repeated on a regular basis over a 360-day period.

The strength development of the concrete cubes was determined using a 3000 kN Avery-Denison testing machine over a 180-day period. The load was applied at a rate of 240 kN/min, representing the lower bound of the recommended rates in BS EN 12390-3:2019 (BSI 2019c).

2.4 Preliminaries and data processing

When a cement-based sample is placed between a pair of electrodes and subjected to a small amplitude alternating current extending over several decades of frequency, the complex impedance response at any frequency, $Z^*(\omega)$, is given in Nyquist format as,

$$Z^*(\omega) = Z'(\omega) - iZ''(\omega) \quad (1)$$

where $Z'(\omega)$ is the resistive or in-phase component; $Z''(\omega)$ is the quadrature or out-of-phase component; ω is the applied (angular) frequency and $i = \sqrt{-1}$. Both components can be measured as a function of frequency of applied electrical field and are commonly presented on a Nyquist form, where $-Z''(\omega)$ is plotted against $Z'(\omega)$ over several decades of frequency. A typical impedance response obtained using the two-point method described in Section 2.3 is presented in Fig. 2,

Fig. 2 Schematic showing the typical impedance response of a cementitious system displaying the definitions of electrode and bulk responses, and the bulk resistance, R .

with frequency increasing from right-to-left across the curve. The Nyquist plot comprises two distinct regions: a spur at the right-hand (low-frequency) side of the plot, which is representative of the polarisation at the electrode/sample interface, and an arc whose centre is depressed below the real axis at the left-hand (high-frequency) side, which can be associated with the bulk response of the sample (McCarter and Brousseau 1990; Christensen *et al.* 1994; Suryanto *et al.* 2016; Kim *et al.* 2019). The junction between the electrode spur and the arc is commonly referred to as the cusp point. The bulk resistance of the test sample, R (Ω), which will be related to ionic conduction processes (Whittington *et al.* 1981), can be obtained from the intercept of the low-frequency end of the arc with the real axis and is easily identifiable. This value is then multiplied by the electrode geometrical factor, k (m), to give the bulk resistivity, ρ (Ωm), as given by:

$$\rho = kR \quad \Omega\text{m} \quad (2)$$

The electrical field between the pin-electrodes in the two-point method is ill-defined and the geometrical factor for this spatial arrangement cannot be readily calculated; however, this can be evaluated through a calibration procedure using the four-point method (McCarter *et al.* 2013a). This was conducted by placing a stainless-steel plate-electrodes at opposite (external) faces of each cube, with a saturated synthetic sponge inserted between the plate and the specimen interface to provide intimate contact (McCarter *et al.* 2015a; Suryanto *et al.* 2018b). These plates were then connected to the current generator and input terminals in the FRA, whereas the embedded pin-electrodes were connected to the potential high and low terminals.

As the electrical field between the current electrodes is uniform and electrode polarisation problems are, effectively, eliminated using the four-point method, the measured concrete resistance can be converted to resistivity through the geometrical factor, $k = A/L$ (McCarter *et al.* 2015b), where, for the prismatic sample, A (m^2) is

the cross-sectional area of the test sample ($0.15 \times 0.15 \text{ m} = 0.0225 \text{ m}^2$) and L (m) the distance between the two potential electrodes ($= 0.05 \text{ m}$). The frequency in the four-point measurement was taken at 1 kHz, following the findings in a previous study (McCarter *et al.* 2015b).

Figure 3 presents concrete resistivity calculated using the four-point method, plotted against concrete resistance measured between the embedded pin-electrodes using the two-point method. Both measurements were taken at 28 days curing, with the markers representing the measured data (mean value) and the x and y error bars representing ± 1 standard deviation (note that in cases where the error bar appears missing, the data markers are larger than the error bar). The slope of the best linear-fit relation gives the geometrical constant ($= 0.065 \text{ m}$) for the embedded electrode pair. Measured resistance readings at any curing time could, therefore, be simply converted to resistivity through the relationship:

$$\rho = 0.065R \quad \Omega\text{m} \quad (3)$$

where R is the low-frequency intercept obtained from the impedance response discussed above.

In the discussion below, the two-point method is used as, from a practical point of view, it has the added advantage of being much simpler to perform than the four-point method [and other methods such as the time domain reflectometry (TDR) (Woo *et al.* 2007; Rangelov and Nassiri 2019)] and hence lends itself to in-situ monitoring and remote interrogation (McCarter *et al.* 2012).

3. Results and Discussion

3.1 Strength development

Although the focus of this paper is to assess the durability performance of concrete based on its electrical properties, for completeness, it was considered appropriate to discuss the compressive strength development of the concretes. Figure 4 presents the mean compressive strengths versus curing time for all mixes over the initial 180-day

Fig. 3 Calibration curve for the embedded electrode pair presented in Fig. 1.

Fig. 4 Compressive strength development over the initial 180-day curing period.

Table 4 Parameter values used for calculating concrete strengths of all mixes.

Parameter	PC	GGBS/35	GGBS/50	GGBS/65
f_{ref} (MPa)	45.0	44.9	42.5	38.0
S	0.25	0.38	0.38	0.38

period, with the markers representing the test data and the error bars representing ± 1 standard deviation. Also, as before, in cases where the error bar appears to be missing, the data marker is larger than the error bar. In general terms, it is evident that as the curing period increases, the compressive strength of the concrete also increases. The increase in strength with time is represented by best fit curves in **Fig. 4** using the BS EN 1992-1-1:2004 expression (BSI 2015):

$$f(t) = f_{\text{ref}} e^{s(1-(t_{\text{ref}}/t)^{0.5})} \quad \text{MPa} \quad (4)$$

where $f(t)$ is the predicted compressive strength at age t (day); f_{ref} is the compressive strength at reference age t_{ref} ($= 28$ days) and the exponent s is a constant that indicates the rate of strength development. Best-fit curves for each mix are plotted on this figure in solid lines through the measurement points using the parameters presented in **Table 4**.

With reference to **Fig. 4**, in the short term, the PC mix gains strength more rapidly than the GGBS mixes, whereas in the longer term (i.e. > 60 days) the PC mix only develops a marginal strength increase. This is as expected as the PC mix undergoes hydration reaction, whereas the GGBS mixes undergo both hydration and pozzolanic reaction, the latter occurring at a reduced rate (Hinrichs and Odler 1989; Luan *et al.* 2012). This is in agreement with the magnitude of the s parameter used to fit the data, which was evaluated as 0.25 for the PC mix and 0.38 for the GGBS mixes (i.e. the higher the s value the lower the rate of strength development).

It is also apparent from **Fig. 4** that although the replacement of PC with GGBS reduces the short-term compressive strength of the concrete, it is not detrimental

in the long-term due to the pozzolanic reaction of the GGBS which continues over an extended period of time. At 90 and 180 days, for example, the GGBS/35 and GGBS/50 mixes both demonstrate a comparable, if not higher, compressive strength than the PC mix highlighting the beneficial effect of GGBS at a longer time scale.

3.2 Complex impedance

The complex impedance spectra for PC and GGBS/50 mixes at selected times during the initial 180 days are presented in Nyquist format in **Figs. 5(a)** and **(b)**, with the frequency increasing from right-to-left across the curve and only the response from one of the three notionally identical specimens (variations within $\pm 10\%$) presented. The open data marker represents the measured response while the solid marker highlights the response at every decade of frequency. It is immediately evident from the figures that over the 180-day period, both mixes display the classic impedance response (McCarter and Brousseau 1990), comprising an electrode spur on the right-hand side and a bulk arc on the left-hand side, which resembles the diagram presented in **Fig. 2**. As the curing age increases, there is a progressive displacement of the curves to the right-hand side of the plot indicating an increase in impedance, although this is not as pronounced in the case of the PC mix. The more noticeable displacement exhibited by the GGBS/50 mix clearly reflects the ongoing pozzolanic activity in the GGBS/50 mix and resulting pore refinement, which has the effect of reducing the connectivity of the capillary pore network with time (Hooton 1986; Shattaf *et al.* 2001; McCarter *et al.* 2013b).

3.3 Optimal frequency range

In a regular/frequent testing regime, it would be desirable to undertake a single frequency measurement rather than multi-frequency measurement as described above, owing to its simplicity in terms of measurement time and instrumentation required. However, the bulk resistance, hence resistivity, obtained from a single frequency

Fig. 5 Typical complex impedance response over the initial 180 days curing period for (a) PC mix, and (b) GGBS/50 mix.

measurement can be affected by spurious electrode polarisation effects if the frequency is too low or relaxation processes if the frequency is too high, which can lead to significant errors. A priori evaluation of the optimal frequency/frequency range (using multi-frequency measurements) over which the electrode effects are reduced to minimal proportions is discussed below.

Figure 6 displays the frequency obtained from the cusp point for all mixes using the electrode geometry and spatial arrangement described above. It is clear that the cusp-point frequency is affected by binder type and curing time. As curing time increases, there is an overall reduction in cusp-point frequency, particularly over the initial 60 days suggesting that the cusp-point frequency is related not only to the electrode configuration, but also to the rate of hydration and microstructural changes within the cementitious binder which are, in turn, affected by the GGBS replacement level. It is also apparent that the cusp-point frequency attains an almost constant value in the longer term. To obtain the optimal frequency range which could then be used in a single frequency measurement, resistivity was de-embedded from the impedance data obtained over the entire frequency range of investigation through the relationship (Suryanto *et al.*

2016; 2018a):

$$\rho(\omega) = \frac{Z'(\omega)^2 + Z''(\omega)^2}{Z'(\omega)} k \quad \Omega\text{m} \quad (5)$$

where $Z'(\omega)$ and $Z''(\omega)$ are as in Eq. (1) and k is the geometrical constant for the embedded electrode pair ($= 0.065 \text{ m}$).

Figures 7(a) and (b) display the resistivity of the PC and GGBS/50 mixes calculated using Eq. (5) and plotted in the frequency domain, with the frequency at the cusp-point highlighted using circular data markers. It is evident that the resistivity decreases with frequency across the entire frequency range of the investigation and increases with increasing hydration (or curing time). The overall increase in resistivity with time is due to the continual refinement of pore-structure as a result of the on-going hydration and pozzolanic activity (further discussed in the section below), whereas the decrease in resistivity with an increase in frequency is due to the reducing electrode polarisation and relaxation/dispersion of the bulk polarisation processes within the concrete. In general, the frequency-domain response can be divided into three distinct frequency regions: low ($< 100 \text{ Hz}$), intermediate (100 Hz to 100 kHz) and high frequency ($> 100 \text{ kHz}$) range. Over the low frequency range, electrode polarisation operates and results in an increased resistivity with decreasing frequency, increasing to anomalously high values at 1 Hz . For example, the PC mix, resistivity increases to $> 300 \Omega\text{m}$ at 1 Hz . Over the intermediate frequency range, there is a very gradual decrease in resistivity with increasing frequency and is a direct result of relaxation of the polarisation processes (e.g. double layer). Over the higher frequency range, a greater reduction in resistivity is evident and this can be associated with the relaxation of double-layer polarisation relaxing within the kHz region (Schwan *et al.* 1962) and a stronger contribution resulting from relaxation of Maxwell-Wagner interfacial polarisation processes at frequencies $> 1 \text{ MHz}$ (Hasted 1973).

Overall, based on the range of the cusp-point fre-

Fig. 6 Cusp-point frequency for all mixes.

Fig. 7 Frequency domain response of resistivity for the (a) PC and (b) GGBS/50 mixes.

quencies obtained from all mixes using the testing arrangement described above, the work indicates that the optimal frequencies which can be used in a single frequency measurement are in the 400 Hz to 8 kHz range; this frequency range falls within the plateau region of the frequency domain responses presented in **Figs. 7(a)** and **(b)**. This frequency range is large enough to minimise errors due to electrode polarisation effects and low enough to ensure that bulk polarisation processes do not influence the resistance. The bulk electrical property of cement-based systems can therefore be determined within this frequency range (i.e. via a single-frequency measurement) without the need for plotting and locating the cusp point (as described earlier in **Figs. 2** and **5**). This approach would still maintain a high level of accuracy and due to its ease of use and practicality, it can be applied to a regular/frequent testing regime more effectively. It is, therefore, recommended that a-priori (multi-frequency) measurements, as described above, should, ideally, be undertaken, particularly when different electrode configurations are used.

3.4 Bulk resistivity

Figure 8 presents the bulk resistivity obtained from the cusp-point frequency for all mixes presented on a semi logarithmic scale, with the data markers displaying the measured data obtained using Eq. (6) and the error bar representing ± 1 standard deviation. The solid lines are the best-fit curves for each mix and plotted through the data points using the relationship,

$$\rho(t) = \rho_{\text{ref}} \left(\frac{t}{t_{\text{ref}}} \right)^n \quad \Omega\text{m} \quad (6)$$

where $\rho(t)$ (Ωm) is the predicted resistivity at time t (day); ρ_{ref} and t_{ref} are, respectively, the reference resistivity (Ωm) and reference time (day); and n , is the ageing exponent which is related to both hydration and pozzolanic activity. In this work, the reference time is taken as 28 days. Overall, all mixes display a continual increase in resistivity mimicking the strength develop-

ment curve. Mixes containing GGBS, generally, exhibit a higher resistivity than the PC mix except during the initial period of about 10 days when the PC mix displays higher values than the GGBS/35 mix. There are several competing factors that influence the resistivity of the concretes, which include the following:

- The release of ions during hydration which will increase ionic concentrations within the pore fluid (Taylor 1987; Larbi *et al.* 1990). Although some of these ions would be captured in the hydrate formation, dissolution will generally have a more dominant influence. This will therefore have the effect of decreasing the pore-fluid resistivity and hence the bulk resistivity;
- Pore structure refinement resulting from on-going hydration and pozzolanic activity which will decrease porosity (Zhou *et al.* 2010; Berodier and Scrivener 2015; Kurumisawa and Nawa 2016) and hence increase the bulk resistivity; and
- Increasing constriction and tortuosity and decreasing connectedness of the pore network, which is as a direct result of pore structure refinement (Gallucci *et al.* 2007; Promentilla *et al.* 2009; Yio *et al.* 2019; Song *et al.* 2020; Zhang *et al.* 2020). This will, in turn, increase the bulk resistivity.

Figure 8 would suggest that (b) and (c) must exert a greater influence on bulk resistivity than (a) – viz, microstructural changes have a greater influence than changes in pore-fluid chemistry. Furthermore, it is interesting to note at the end of the test period that whilst the four different concrete mixes display comparable compressive strength (to within $\pm 10\%$, see **Fig. 4**), they display a more distinguishable variation in bulk resistivity (i.e. up to an order of magnitude in **Fig. 8**). This clearly reflects the role of GGBS in producing concrete with a much more disconnected and tortuous pore structure although not necessarily of a lower porosity than the plain PC mix (Kim *et al.* 2018).

3.5 Formation factor and pore-fluid resistivity

The resistivity of concrete alone cannot be used as a measure of its ability to resist chloride ingress as it is dependent not only upon the interconnectivity of the capillary pore network, but also on the resistivity of the pore fluid. To investigate the influence of changes in the concrete microstructure alone, the effect of the pore fluid resistivity must be taken into consideration.

In relation to saturated rock formations (Archie 1942), the ratio of the bulk resistivity of the saturated rock, ρ (Ωm), to the resistivity of the saturating fluid (i.e. pore fluid), ρ_f (Ωm), is related to the rock porosity, ϕ , through the relationship,

$$F = \frac{\rho}{\rho_f} = a\phi^{-m} \quad (7)$$

where F is defined as the formation factor; a is a correction factor, which is valid over a specified range of porosities ϕ ; and the exponent m is the cementation factor

Fig. 8 Bulk resistivity versus time response for all mixes and fitting equations.

which is related to the tortuosity and connectivity of the pore network within the rock. A wide range of values have been reported for m and a for different rock and sediment formations, with a in the range 0.4 to 2.5 and m in the range 1.2 to 2.5 (Worthington 1993; Khalil and Monterio Santos 2009). Whilst the pore fluid resistivity can be determined experimentally through using a pore-fluid expression technique [see, for example, Barneyback and Diamond (1981), Lothenbach and Winnefeld (2006) and Tsui-Chang *et al.* (2019)], this method is extremely difficult to execute in concretes especially at

longer timescales.

In this work, the Virtual Cement and Concrete Testing Laboratory (VCCTL) software (NIST 2019) was used to evaluate the pore fluid resistivity, ρ_f (Ωm). VCCTL is a virtual platform that can be used to model the hydration, microstructural development, and physical properties of cementitious systems. The pore-fluid resistivity is calculated based on the pore structure generated by the software [see, for example, Fig. 9(a)] and pore-fluid chemistry which is calculated automatically in the software from the concentration of primary ions (i.e. Na^+ , K^+ , SO_4^{2-} and OH^-) in the pore-water and their equivalent conductivity, using the NIST model (Snyder *et al.* 2003; Bentz 2007). The ionic concentrations are calculated using the binder composition and the total and soluble alkali contents in the cementitious materials. Regarding the latter, it was assumed that 35% of the Na_2O and 55% of the K_2O are released into the pore-water upon gauging, following the recommendations in Brouwers and vanEijk (2003). The accuracy of the core platforms implemented in the VCCTL for determining the pore-fluid chemistry and resulting conductivity was previously studied by several investigators [see, for example, Chen and Brouwers (2010), Tsui Chang (2017) and Riding *et al.* (2018)] and varying degrees of success were obtained.

Figure 9(b) presents the computed pore-fluid resistivity for each mix over the entire test period. It is apparent that there is a rapid reduction in pore fluid resistivity over the initial few weeks of curing which is then followed by a continual reduction over the remainder of the test period, albeit at a much decreased rate when hydration has already advanced beyond the 28-day curing period, which reaffirms statement (a) in Section 3.4. The plots also show that the pore-fluid resistivity increases with an increasing level of GGBS replacement (due to decreasing ionic concentrations in the pore fluid) and that the difference in value remains almost constant and envisaged as an offset. This is due to the assumption that it is only alkalis in the cement that contribute to the pore solution chemistry as those in the slag are absorbed on the slag products of hydration (Bentz 2007; Schäfer and Meng 2001). As such, it should be possible to calculate the pore solution resistivity of the GGBS mixes based on the magnitude of replacement levels without the need to perform laborious and complex pore-water extraction or computer simulation. This aspect is currently being investigated and not presented in the current work.

Having evaluated the pore-fluid resistivity, it is now possible to determine the formation factor using Eq. (8). Figure 10 presents the computed formation factor over the 360-day period for each concrete mix. The data markers represent the test data while the lines are the best-fit curves plotted through the data points using the relationship:

$$F(t) = F_{\text{ref}} \left(\frac{t}{t_{\text{ref}}} \right)^n \quad (8)$$

Fig. 9 (a) Pore structure generated at 6 hours and 90 days of hydration for the matrix in the PC and GGBS/50 mixes. (b) Estimated pore solution resistivity at various GGBS replacement levels.

where $F(t)$ is the predicted formation factor at time t (day); F_{ref} and t_{ref} are, respectively, the reference formation factor and time (day); and n , is the ageing exponent which is related to hydration and pozzolanic activity. As before, the reference time was taken as 28 days. The trend is similar to that observed in the bulk resistivity and it is evident from Fig. 10 that all mixes display a continual increase in formation factor over the entire test period, indicating ongoing pore structure refinement, which reaffirms statements (b) and (c) in Section 3.4. It is clearly observed that the GGBS mixes exhibit a higher formation factor than the PC mix, apart from the initial stage up to about 28 days where the opposite trend is observed. The higher formation factor exhibited by the GGBS mixes indicates that the replacement of Portland cement with GGBS results in further pore structure refinement in the longer term by producing a capillary pore network which is more tortuous and disconnected in nature. This is as a direct result of gradual infilling of the capillary pore space with further hydrate formation from the reaction of the GGBS with cement hydrates. The reduced pore interconnectivity will reduce ionic mobility and, therefore, increase the resistance of the concrete to ionic (i.e. chloride) ingress (Takahashi and Ishida 2016). At the end of the test period, the formation factor (± 5) increases in the order PC (315); GGBS/35 (545); GGBS/50 (835) and GGBS/65 (1040). It is accepted that these values are based on the evaluation of the resistivity of the pore-water which is currently at best approximate and work is continuing in this respect. Finally, it is noteworthy that while the formation factor can give a measure of the resistance of concrete to chloride ingress, the resistivity of concrete surrounding the embedded steel reinforcement has a key role to play in corrosion dynamics as it governs the magnitude of the corrosion current, hence the corrosion rate of steel after depassivation. The higher the concrete resistivity, the lower will be the corrosion current flow between the anodic and cathodic areas along the surface of the steel reinforcement and therefore a lower corrosion rate will result.

Fig. 10 Estimated Formation Factor for all mixes.

Table 5 Concrete resistivity thresholds adapted from Broomfield (1997) and estimated formation factor classification.

Resistance to corrosion	Resistivity (Ωm) (Broomfield 1997)	Estimated Formation Factor*
Low	< 50	< 350
Low/Moderate	50 to 100	350 to 700
High	100 to 200	700 to 1400
Very High	> 200	> 1400

Note: *Based on pore-fluid resistivity of 0.15 Ωm .

Table 6 Concrete resistivity values for marine concrete adapted from Balestra *et al.* (2019) and estimated formation factor classification.

Probable corrosion risk	Resistivity (Ωm) (Balestra <i>et al.</i> 2019)	Estimated Formation Factor*
Very High	< 150	< 1000
High	150 to 200	1000 to 1350
Medium	200 to 550	1350 to 3700
Low	550 to 800	3700 to 5350
Negligible	> 800	> 5350

Note: *Based on pore-fluid resistivity of 0.15 Ωm .

3.6 A performance-based durability index

Given that concrete durability and the formation factor are both linked to the hierarchical porous nature of the cementitious binder, the formation factor parameter described above could be used to rank the relative performance of the concretes. In this work, the previously reported empirical concrete durability classification systems by Broomfield and his co-workers (Broomfield 1997; ACI Committee 222 2001) and by Balestra *et al.* (2019) are used to establish performance (formation-factor) based durability criteria. Based on the assumption that the pore-fluid resistivity in the previous studies is comparable to this study, the empirical resistivity thresholds in the two classification systems are normalised against the pore-fluid resistivity of the PC mix at 28 days ($\rho_f = 0.15 \Omega\text{m}$), as its value changes only marginally beyond this point of time. The results are further compared with the durability classification system suggested in Alexander *et al.* (1999); in this classification, however, as the concrete is pre-conditioned by vacuum saturation with 5 M NaCl solution, the pore-fluid resistivity in the analysis is taken as 0.035 Ωm (McCarter *et al.* 1996) assuming that the concrete is fully saturated with the NaCl solution despite its very high molarity. The computed formation factor (± 50) for each empirical concrete resistivity threshold is presented in Tables 5, 6 and 7. Overall, it is evident that the computed formation factor thresholds presented in Table 6 are the most stringent, whereas those presented in Table 7 are the most lenient.

Table 8 provides the performance of the concrete mixes investigated in this study at selected curing ages over the 360-day period based on the classifications presented in Tables 5, 6 and 7. With reference to classifications in Table 5, it is apparent that the performance of the concretes to corrosion varies from “Low” to “High” depending on the binder type and time. The PC mix,

Table 7 Durability classification system adapted from Alexander *et al.* (1999) and estimated formation factor classification.

Durability class	Chloride Resistivity Index*(Ωm) (Alexander <i>et al.</i> 1999)	Estimated Formation Factor**
Very Poor	< 4	< 100
Poor	4 to 6.5	100 to 200
Good	6.5 to 13.5	200 to 400
Excellent	> 13.5	> 400

Notes: *Specimen vacuum saturated with 5 M NaCl solution;

**Based on pore-fluid resistivity of 0.035 Ωm .

Table 8 Relative performance of concrete mixes investigated in this study based on the classifications presented in Tables 5, 6 and 7.

PC					GGBS/35			
Time	F	Resistance to corrosion	Probable corrosion risk	Durability class	F	Resistance to corrosion	Probable corrosion risk	Durability class
Days		(Table 5)	(Table 6)	(Table 7)		(Table 5)	(Table 6)	(Table 7)
7	203	Low	Very High	Good	139	Low	Very High	Poor
28	250	Low	Very High	Good	256	Low	Very High	Good
90	269	Low	Very High	Good	372	Moderate	Very High	Good
180	289	Low	Very High	Good	469	Moderate	Very High	Excellent
360	315	Low	Very High	Good	546	Moderate	Very High	Excellent

GGBS/50					GGBS/65			
Time	F	Resistance to corrosion	Probable corrosion risk	Durability class	F	Resistance to corrosion	Probable corrosion risk	Durability class
Days		(Table 5)	(Table 6)	(Table 7)		(Table 5)	(Table 6)	(Table 7)
7	176	Low	Very High	Poor	201	Low	Very High	Good
28	342	Low	Very High	Good	434	Moderate	Very High	Excellent
90	509	Moderate	Very High	Excellent	633	Moderate	Very High	Excellent
180	662	Moderate	Very High	Excellent	827	High	Very High	Excellent
360	836	High	Very High	Excellent	1038	High	High	Excellent

regardless of time, exhibits Low performance throughout as would be expected from a plain PC mix with a moderate w/c ratio. The GGBS mixes consistently outrank the PC mix at longer timescales, in particular GGBS/50 and GGBS/65, which show a transition from Low to High resistance against corrosion with increasing time. This result would suggest that as electrical measurements are technically simple and straightforward to perform, multiple measurements could be performed during the normal curing period (and beyond) to evaluate the likelihood of future performance of a concrete mix in terms of its resistance to chloride ingress. This is especially useful in assessing the performance of concretes with supplementary cementitious materials such as the GGBS concretes investigated in this study which normally undergo a more progressive pore structure development than plain PC concretes.

Based on the performance classification in **Table 6**, it is surprising that all the concretes are now classified as a “Very High” risk of corrosion, although they all satisfy the maximum w/b ratio and minimum binder content requirements specified in BS 8500-1:2015 (BSI 2019a) to resist environmental exposure classes XD1 and XD2 (exposure to chlorides other than from sea water) for an intended working life of at least 100 years with a 50 mm cover to steel, and requirements specified by the Japan Society of Civil Engineers (JSCE 2007b) to resist envi-

ronmental exposure classification “Undersea” and other classifications (i.e. “Offshore air” and “Splash zone”) if supplemented with test data. It is also noteworthy that the GGBS concretes investigated in this study also meet the minimum w/b ratio and binder content requirements in BS 8500-1:2015 to resist environmental exposure classes XD3 (cyclic wet and dry from water containing chlorides other than from sea water); XS1 (airborne salt from sea water) and XS2 (permanently submerged in sea water) for the same intended working life (> 100 years) with a 60 mm cover; and XS3 (tidal, splash and spray zones) with a thicker (70 mm) cover. This would indicate that the performance criteria adopted in this classification are too stringent. In contrast, based on the durability performance indices presented in **Table 7**, it is evident that all mixes now fall primarily under either “Good” or “Excellent” durability category, including the PC mix. Referring to the requirements of BS 8500-1:2015 (BSI 2019a) and the JSCE specification (2007b) above, there is evidence to suggest that the performance criteria in this classification are too lenient and more stringent criteria would be required to disqualify mixes that potentially have low performance in terms of its resistance to chloride ingress in the longer term. The large discrepancies amongst the estimated performance index thresholds clearly warrant further investigation and will be the direction of future work.

3.7 Formation factor and diffusion

To provide a further direct measure of the ability of the concrete mixes to resist chloride ingress, the diffusivity of the concretes is discussed. As the solid phase in the concrete can be regarded as non-conductive in comparison to that of the pore-fluid, diffusivity and ionic conductivity of a saturated porous material are connected through the Nernst-Einstein relationship (Lu 1997; Tumidajski *et al.* 1996; McCarter *et al.* 2015c);

$$F = \frac{\rho}{\rho_t} = \frac{D_o}{D_i} \quad (9)$$

where D_i (m^2/s) is the instantaneous diffusion coefficient of the ion within the porous material; D_o (m^2/s) is the free-solution diffusion coefficient of the ion in an infinitely dilute bulk solution [in this work, the chloride ion is chosen and $D_o = 1.84 \times 10^{-9} \text{ m}^2/\text{s}$ at 20°C (Li and Gregory 1974)] and other parameters are as before. It is noteworthy that Nernst-Einstein relation can be expected to always overestimate the conductivity (therefore, underestimate the resistivity) of an electrolyte as it disregards the formation of ion complexes in the pore solution

(such as ions with opposite charges in contact that do not produce a net charge, therefore not contributing for the dc-conductivity), as discussed for instance in the work of Harris and Kanakubo (2015). For the ionic concentration typically found in the pore solution of cement systems, however, the Nernst-Einstein relation could be considered as a good approximation. Equations (6), (8) and (9) can, therefore, be combined to determine the instantaneous diffusion coefficient at time t , as given by:

$$D_i(t) = \frac{D_o}{F_{\text{ref}}} \left(\frac{t_{\text{ref}}}{t} \right)^n \text{ m}^2/\text{s} \quad (10)$$

Figure 11 presents the instantaneous diffusion coefficient versus curing time over the entire 360-day test period, evaluated using Eq. (10). In this figure, the data markers represent the test data while the lines represent the best-fit curves. In general terms, it is apparent that all mixes display a similar trend, each exhibiting a rapid decrease in instantaneous diffusion which is then followed by a continuous gradual reduction over the remainder of the test period, indicating an enhanced resistance to chloride ingress with curing time. It is also apparent from the figure that the instantaneous diffusion coefficient decreases with increasing GGBS replacement levels, which at the end of the test period are in the order PC ($5.8 \times 10^{-12} \text{ m}^2/\text{s}$); GGBS/35 ($3.2 \times 10^{-12} \text{ m}^2/\text{s}$); GGBS/50 ($2.2 \times 10^{-12} \text{ m}^2/\text{s}$); and GGBS/65 ($1.7 \times 10^{-12} \text{ m}^2/\text{s}$).

It should be noted that the instantaneous diffusion coefficient discussed above is different to the diffusion coefficient normally obtained from the solution of the error function complement (erfc) to a one-dimensional chloride profile obtained from dust drillings of concrete exposed to chloride. This is generally referred to as the effective diffusion coefficient, D_{eff} (m^2/s), (Delagrave *et al.* 1996; Kato and Uomoto 2005; Patel *et al.* 2018) or when chloride binding is included, referred to as the apparent diffusion coefficient. For clarity, Fig. 12(a)

Fig. 11 Estimated instantaneous diffusion coefficient, D_i .

Fig. 12 (a) Schematic showing the definition of D_i and D_{eff} (McCarter *et al.* 2015), and (b) Estimated D_{eff} assuming $t_{\text{ex}} = 28$ days plotted against exposure time ($t - t_{\text{ex}}$).

displays the relationship between the instantaneous and the effective diffusion coefficient, D_i and D_{eff} , respectively, which can be related through the following relationship (Pack *et al.* 2010; McCarter *et al.* 2015c),

$$D_{\text{eff}}(t) = \frac{1}{t - t_{\text{ex}}} \int_{t_{\text{ex}}}^t D_i(t) \, dt \quad \text{m}^2/\text{s} \quad (11)$$

where $D_{\text{eff}}(t)$ is effective diffusion coefficient of the concrete at time t after casting; $D_i(t)$ is the corresponding instantaneous diffusion coefficient of the concrete at time t , with t being time at which the effective diffusion coefficient is required and t_{ex} being time at which the exposure to chloride starts. **Figure 12(b)** displays the predicted effective diffusion coefficient, D_{eff} , over a period of one year, with zero time for exposure, t_{ex} , taken as 28 days after casting. There is clearly a general reduction in D_{eff} with time, with the GGBS mixes displaying a more significant reduction in D_{eff} than the PC mix.

Figure 13 presents the comparison of the computed effective diffusion coefficients for the PC and GGBS/50 mixes over the 360-day period with some of the diffusion coefficients available in the literature (Collepari *et al.* 1973; Diab *et al.* 1988; Dhir *et al.* 1990; Page *et al.* 1991; Tang and Nilsson 1992a, 1992b; Bamforth 1993; Mangat and Molloy 1994; Zhang and Gjorv 1995; Tang 1995; Arsenault *et al.* 1995; Higgins 1995; Polder 1995; Baroghel Bouny *et al.* 1995; Bentz *et al.* 1996; Johnson *et al.* 1996; Hooton *et al.* 1997; Kanaya *et al.* 1998; Pedersen and Arntsen 1998; Bleszynski *et al.* 2002; Yeau and Kim 2005; Thomas *et al.* 2008; Li *et al.* 2011; Chen *et al.* 2012; Maes *et al.* 2013; Shin *et al.* 2016), which are limited to an exposure time of one year, w/b ratios from 0.3 to 0.7 and slag replacement ratio of about 50%. Also presented on the figure is the apparent diffusion coefficients of PC and GGBS cement concretes based on the Japan Society of Civil Engineers models (JSCE 2007a;

JSCE 2010):

$$\text{Log}_{10} D_{\text{eff,PC}} = -3.9(w/c)^2 + 7.2(w/c) - 13.99 \quad \text{m}^2/\text{s} \quad (12)$$

$$\text{Log}_{10} D_{\text{eff,GGBS}} = -3.0(w/c)^2 + 5.4(w/c) - 13.69 \quad \text{m}^2/\text{s} \quad (13)$$

where w/c is the water/cement ratio. It is worth noting that in these empirical equations, the effect of time is not considered. Equation (13) was proposed for GGBS cement Type B to JIS R 5211 (JSA 2009) which corresponds to a replacement range of 30 to 60% (or, on average, close to GGBS/50). For the mixes used in the work presented in this paper ($w/c = 0.5$), $D_{\text{eff,PC}}$ is evaluated as $4.3 \times 10^{-12} \text{ m}^2/\text{s}$ and $D_{\text{eff,GGBS}}$ as $1.8 \times 10^{-12} \text{ m}^2/\text{s}$ using, respectively, Eqs. (12) and (13).

It is apparent from **Fig. 13**, that there is clearly a large scatter in published values for diffusion coefficients of similar concretes and the effective diffusion coefficients computed for the PC and GGBS/50 mixes are within the range of the anticipated values. When compared with the JSCE predictions, the method of determination of the diffusion coefficient using the formation factor tends to overestimate the actual values (i.e. $4.3 \times 10^{-12} \text{ m}^2/\text{s}$ vs $6.3 \times 10^{-12} \text{ m}^2/\text{s}$ at 1 year, and $1.8 \times 10^{-12} \text{ m}^2/\text{s}$ vs $2.9 \times 10^{-12} \text{ m}^2/\text{s}$ at 1 year). This could possibly be attributed to the longer timescale from which the empirical equations were derived and possibly due to the fact that in the formation factor approach, no consideration on chloride binding and associated physical changes has been given. Accordingly, when the D_{eff} derived from formation factor is used to make predictions as to the time to activation of corrosion, they would result in conservative estimates, i.e. they will underestimate the time to activation of corrosion.

The estimated diffusion coefficients are based on the accuracy of the VCCTL model in predicting the resistivity of the pore-water which has not been experimentally validated; however, considering the extremely wide variation in published values for diffusion coefficients [see, for example, **Fig. 13** or Bamforth *et al.* (1997)], the method of determination of the diffusion coefficient using the formation factor could be regarded as a good alternative and further developed as a means of indexing concrete performance based on its ability to resist chloride ion penetration. The technique could, potentially, be exploited as a rapid, non-destructive method in a pre-qualification testing programme and further developed to allow early estimation of the long-term diffusion coefficient.

4. Conclusions and concluding remarks

The work presented highlights the application of electrical property measurements to study microstructural changes and the ability of Portland cement concrete and concretes containing various levels of GGBS to resist chloride ingress. Testing was conducted over a 360-day period using a combination of bulk resistivity measurements and virtual pore-fluid simulations in the computer.

Fig. 13 Comparison of the predicted and measured values of diffusion coefficient reported in previous studies for PC and GGBS concretes with w/b ratios ranging from 0.3 to 0.7 and exposure time of less than 1 year.

The role of multi-frequency measurements (over the 1 Hz to 10 MHz range) to minimise errors resulting from polarisation effects is highlighted and used to identify the optimum range for single-frequency resistance measurements. The bulk resistivity was normalised against the pore fluid resistivity following the Nernst-Einstein equation and termed the formation factor. Using this presentation formalism, it was possible to account for the pore fluid effect and differences in pore-solution chemistry between the different binders. As such, changes in the formation factor can be related solely to microstructural changes. The results were then used to assess the diffusivity of the concretes.

The following conclusions can be drawn from this study:

- 1) Over the test period, all mixes are shown to exhibit an increase in resistivity and formation factor which reflects on-going hydration and pozzolanic reaction with resulting pore-structure refinement.
- 2) GGBS exerts a more significant influence on bulk resistivity and formation factor than compressive strength, indicating that further hydrate accretion into the capillary pore space has a greater influence on the interconnectivity of the capillary pore network (i.e. reduction in both pore size and pore connectivity as well as enhancement in both pore tortuosity and pore constrictivity) than the matrix strength.
- 3) No detectable decrease in bulk resistivity resulting from an increase in ionic concentration within the pore fluid was observed over the entire test duration. This indicates that during this period, microstructural changes resulting from on-going hydration and pozzolanic reaction exerts a more dominant influence than changes in ionic concentration within the capillary pore fluid.
- 4) The GGBS concretes exhibited a more marked increase in resistivity relative to both their initial and 28-day values than the plain PC concrete. This was attributed primarily to the pozzolanic reaction, which continued over the longer term, and resulting physical influence on the pore network. At the end of the test period, the resistivity of GGBS/65 mix was one order of magnitude higher than the PC mix.
- 5) Using the NIST model, hydration is shown to influence the pore-fluid resistivity during the first two weeks of hydration resulting mainly from increasing ionic concentration within the pore fluid. Once the degree of hydration was in the advanced stage, the pore fluid resistivity remained virtually constant. The pore-fluid resistivity is shown to increase with an increase in GGBS replacement levels. It is also shown that within the GGBS systems, the influence of pozzolanic reaction on the electrical properties of the pore-fluid was limited.
- 6) Concretes containing GGBS had a higher formation factor than the plain PC concrete at time-scales > 28 days. The formation factor at 360 days increased in the order PC (315) < GGBS/35 (545) < GGBS/50

(835) < GGBS/65 (1040) highlighting microstructural differences in the capillary pore network.

- 7) Based on the durability classifications using the formation factor, it is shown the GGBS mixes outrank the PC mix at longer timescales (i.e. beyond 28 days), particularly at higher replacement levels (i.e. 50% and 65%).
- 8) The instantaneous and effective diffusion coefficients of the concretes were found to decrease with an increase in curing time, indicating an enhanced ability to resist chloride ingress with increasing time; the GGBS mix was found to outperform the PC mix in the longer term (i.e. beyond the normal 28 days curing period).
- 9) Almost the same aging factor n was obtained from the functional forms that were used to describe the temporal changes in resistivity, formation factor and diffusion for a given cement system. The formation factor and arguments based on Nernst-Einstein equation are approximations and the results in this work corroborate that these approximations are sufficiently good to describe the aging of a complex system such as concrete and to define cross-property correlations that are useful for durability assessment.

Overall, it is shown that the resistivity of concrete is relatively straightforward to evaluate and the method of determining the diffusion coefficient using the formation factor could be further exploited as a pre-qualification test for evaluating concrete performance or in-situ evaluation of the as-delivered concrete.

Acknowledgements

Aspects of the work presented form part of the concrete durability programme investigating improvements into the performance of reinforced concrete in chloride environments and in the development of performance-based testing procedures. The financial support of the Scottish Funding Council through a Global Challenges Research Fund is gratefully acknowledged. The Authors wish to thank Hanson UK for supplies of ground granulated blast furnace slag and Professor Malcolm Chrisp, Head of the School of Energy, Geoscience, Infrastructure and Society, for placing the facilities of the School at their disposal. Thanks are also expressed to Dr A Tambusay, Mr R Arthur and Ms R Wilson for their assistance in the experimental work.

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