

Parameters Affecting the Measurements of Embedded Electrical Sensors for Concrete Health Monitoring Applications

by F. Rajabipour and J. Weiss

Synopsis: Traditional methods of condition assessment frequently rely on manual inspections of a structure to locate signs of aging and deterioration. In comparison, the sensor-based health monitoring shows several advantages including increased accuracy and frequency of the measurements, possibility of remote sensing, and ability to determine the rate of damage and remaining life of a structure. Several sensors have been developed to monitor the durability of concrete. Among these, a series of electrical/electro-chemical sensors have been recently introduced that enable measuring the transport properties of concrete as well as monitoring the corrosion process of the reinforcing steel. In this paper, several types of such electrical sensors are introduced and the theoretical background behind each measurement is discussed. For example, it is shown that measurement of the electrical conductivity of concrete can be used to estimate the permeability, ion diffusivity, or moisture content of concrete.

Although the embedded electrical sensors provide valuable measurements enabling prediction of concrete durability, proper calibration of the measurements is essential. As an example, the electrical conductivity measurements of concrete are shown to be simultaneously influenced by four parameters: (a) concrete microstructure, (b) pore fluid composition, (c) moisture content, and (d) temperature. Experimental evidence is provided suggesting that the sensor measurements must be calibrated to account for these parameters; otherwise, the results can be misleading. A combination of several sensors can provide the information needed for proper calibration of the measurements.

Keywords: corrosion; durability; electrical measurement; health monitoring; sensor

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1.0 INTRODUCTION

Reinforced concrete structures are subjected to aging and gradual deterioration. If symptoms of durability problems are diagnosed early, preventative measures can be taken to avoid or at least slow further damage development. On the other hand, undiscovered deterioration leads to accelerated damage which often requires costly repairs and replacements.

Traditionally, to diagnose potential durability problems, routine manual inspections have been adopted for large-scale structures^[1]. These manual inspections, performed every few years (e.g., every 2-5 years), often include visual inspections of the structure to locate any sign of damage development (e.g., cracking, spalling, rust stain). In addition, inspections may include some destructive or non-destructive measurements (e.g., taking cores to determine concrete strength or half-cell potential mapping to assess the risk of corrosion).

The manual inspections provide a general and qualitative assessment of the condition of a structure. However, the measurements are subjected to human error and their interpretations depend often solely on the engineering judgment of the inspector. The results provide a “snapshot in time” assessment and may not provide information on the rate of damage development or the remaining life of the structure. Also, reliable condition assessment of some hard-to-reach areas such as underwater or underground members may not be possible.

In recent years, advancement in sensing technology has enabled engineers to use embedded sensors for continuous condition assessment and health monitoring of structures^[2-4]. The embedded sensors provide several advantages over the traditional inspections including the increased accuracy and frequency of the measurements as well as the possibility of remote sensing. When calibrated properly, the sensor measurements can provide a quantitative assessment of the extent and rate of damage development. Further, the results can be input into life-cycle prediction models to assess the remaining life of the structure and to examine different scenarios for repair and rehabilitations^[5]. Finally, the sensing system can provide the necessary warning before acute damage development pose a safety hazard to the users.

For reinforced concrete structures, an area of particular interest has been sensors that diagnose the corrosion of reinforcing steel. Steel corrosion is known as one of the most significant sources of concrete deterioration and imposes large maintenance costs for shoreline structures, bridges, and parking garages. In recent years, several types of electrical/electro-chemical sensors have been developed to monitor the transport of moisture and ions inside concrete and to provide information on the initiation and propagation of corrosion reactions^[6-14]. These sensors provide valuable measurements enabling prediction of the durability of concrete.

While the measurements from these sensors have been repeatable and reliable in controlled laboratory environments, in uncontrolled field conditions, the results can show scatter which makes their interpretation challenging^[15]. This stems from the fact that the electrical properties of reinforced concrete is simultaneously influenced by several parameters including the internal temperature and moisture content of concrete^[10, 15]. Internal temperature and moisture are time dependent and can vary from point to point inside concrete. As such, the sensors' measurements should be properly calibrated to account for these influencing parameters.

In this paper, several types of electrical/electro-chemical sensors are introduced. For each sensor, a brief discussion is provided in Section 3.0 on the theoretical background behind the measurements and on the parameters that influence the results. Particularly, the measurement of electrical conductivity of concrete is discussed that is used for estimating the permeability, ion diffusivity, and moisture content of concrete. A set of experimental measurements is provided in Section 4.0 showing the dependence of electrical conductivity on four main parameters: (a) concrete microstructure, (b) composition of pore fluid, (c) moisture content, and (d) temperature. Some insight into the type of sensors that needs to be developed to improve the health monitoring of concrete is given in Section 5.0.

2.0 RESEARCH SIGNIFICANCE

Embedded electrical sensors are valuable for health monitoring of reinforced concrete structures. To obtain reliable information, it is important to understand the parameters that influence the sensor measurements and to properly calibrate the results to account for such influences. As an example, this paper discusses the measurements of electrical conductivity of concrete that are used for determining the transport properties and the moisture content of the material. It is shown that concrete conductivity is significantly influenced by four concurrently changing parameters and as such, without proper calibrations, the sensor measurements can be misleading.

3.0 ELECTRICAL SENSORS FOR HEALTH MONITORING OF REINFORCED CONCRETE

3.1 Concrete Conductivity

This sensor is composed of two embedded and non-corroding electrodes (e.g., stainless steel). The electrical resistance of concrete is measured by applying a known voltage between the electrodes and measuring the resulting current (Figure 1a) (or by applying a known current and measuring the voltage difference between electrodes). Alternatively, a four-point measurement setup can be used in which the signal electrodes used to apply the electrical current are different than the sensing electrodes used to measure the voltage difference (Figure 1b). Measurements can be performed using direct (DC) or alternating (AC) currents. AC measurements provide advantages such as faster and more stable and repeatable readings, eliminating the electrical polarization, and the use of smaller voltages that cause less perturbation to surrounding concrete.

The measured electrical resistance is converted to electrical conductivity by applying a geometry factor to normalize the influence of electrodes' and specimen's geometry:

$$\sigma = \frac{k_g}{R} \quad (1)$$

where σ (S/m) [S/in] is the conductivity of concrete, R (Ω) [Ω] is the measured resistance, and k_g (1/m) [1/in] is the geometry factor. It has been shown that the conductivity of concrete is a function of the composition, volume fraction, and tortuosity of the pore fluid and can be described as ^[6, 16]:

$$\sigma = \sigma_o \phi \beta \quad (2)$$

where σ_o (S/m) [S/in] is the pore fluid conductivity, ϕ (unitless) is the volume fraction of pore fluid (i.e., liquid filled porosity), and β (unitless) is the pore fluid connectivity (i.e., it accounts for the effective length and constrictions of the liquid-filled pores ^[17]). Since several transport properties of concrete (such as ion diffusivity and water permeability) are similarly dependent on the volume and tortuosity of the liquid-filled pores, theoretical correlations have been established between the electrical conductivity and these transport properties ^[18,19]:

$$D = D_o (\sigma / \sigma_o) \quad (3)$$

$$k = \frac{1}{226} d_c^2 (\sigma / \sigma_o) \quad (4)$$

where D (m²/s) [in²/s] is the bulk ionic diffusivity of concrete (for a known penetrating substance such as NaCl), D_o (m²/s) [in²/s] is the diffusivity of that substance inside the pore fluid (for NaCl, $D_o = 1.484 \times 10^{-9}$ m²/s [2.300×10^{-6} in²/s]).

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in^2/s) can be assumed ^[20], k (m^2) [in^2] is concrete's intrinsic permeability, and d_c (m) [in] is a characteristic pore size that controls permeability ^[19]. These expressions have been validated experimentally and good agreement between theory and experiment was observed ^[18, 21].

Equations (3) and (4) show that for calculation of the transport properties, in addition to concrete conductivity (σ), knowledge of the pore solution conductivity (σ_o) is required. This stems from the fact that the electrical conductivity by itself cannot quantify concrete's microstructure; rather, the ratio σ/σ_o can be related to the volume (ϕ) and connectivity (β) of the liquid-filled pores (Eq. (2)). This is further discussed in Section 4.0.

In addition, the conductivity of concrete is dependent on its moisture content and temperature ^[16]. This dependence has encouraged the use of insitu electrical conductivity measurements to estimate the moisture content of field concrete ^[7, 8, 22]. In addition to sensing applications, it is common to manually measure the electrical conductivity of concrete using a four-point setup applied to concrete's surface ^[23] or using the RCPT (ASTM C1202) method ^[24, 25]. The results of such measurements are used similarly for estimating the transport properties or the moisture content of concrete.

3.2 Half-cell Potential

The corrosion of steel inside concrete includes formation of anodic and cathodic areas at the surface of steel bars (Figure 2a). At an anodic site, iron is oxidized ($\text{Fe} \rightarrow \text{Fe}^{2+} + 2\text{e}^-$) to release electrons which will be consumed by the reduction of oxygen at the cathode ($\text{O}_2 + 2\text{H}_2\text{O} + 4\text{e}^- \rightarrow 4\text{OH}^-$). The electrons flow through the steel rebar from anodic to cathodic sites. Meanwhile, to maintain charge equilibrium, ions (e.g., Fe^{2+} , OH^-) move inside concrete's pore fluid and generate an electrical current flowing from anode to cathode. The current flow is the result of a potential difference (i.e., an electrical field) that forms between the anodic and cathodic areas (Figure 2b). The magnitude of the current, which is analogous to the rate of corrosion, depends on the potential gradient as well as the resistivity (i.e., inverse conductivity) of concrete ^[26, 27].

The magnitude of electrode potential (E) at anodic and cathodic sites (also known as the half-cell potential of anode or cathode) is dictated by the electro-chemical reactions occurring at each site. The half-cell potential of anode is lower which causes the current to flow from anode to cathode. As a result, an electrical field is formed and the potential increases gradually by moving from anode towards cathode (as shown by the equi-potential lines in Figure 2b) ^[28].

The magnitude of half-cell potentials can be measured with the aid of a voltmeter and a reference electrode (e.g., copper-sulfate electrode, CSE) that is embedded inside concrete or placed on its surface. It must be noted that such measurements do not reveal the true half-cell potentials of anode or cathode. Rather, they report the potential at a particular location inside concrete at which the reference electrode has been placed (for example, in Figure 2b, a potential of approximately -500mV is measured by the voltmeter) ^[28]. This indicates that lower potentials correspond to areas in the vicinity of a local anode. As such, by measuring the half-cell potential at several locations, possible anodic areas can be recognized. This is a common practice known as potential mapping and has been standardized by ASTM C876.

The half-cell potential mapping is a useful qualitative method to locate possible areas undergoing active corrosion. However, the measurements cannot be used quantitatively to determine the corrosion rate ^[29]. The readings are directly influenced by the distance between the reference electrode and a local anode as well as the electrical conductivity of concrete ^[28]. Other parameters influencing the measurements include the temperature and moisture content as well as the ionic composition of concrete's pore fluid ^[23, 27]. An additional complication exists for sensing applications which make use of embedded reference electrodes at known locations inside concrete. Unfortunately, the stability of such embedded sensors is a cause for concern as the potential of the embedded reference electrode may show considerable drift with time ^[23].

3.3 Chloride Concentration

The idea of using embedded reference electrodes has been expanded to measure the chloride ions concentration inside concrete's pore fluid. For this purpose, the potential of an embedded silver/silver-chloride (Ag/AgCl) electrode is measured against a separate reference electrode (e.g., MnO_2 electrode). The potential of the silver

chloride electrode is a function of the activity ($\approx$ concentration) of chloride ions in the pore fluid of surrounding concrete according to the Nernst equation ^[23]:

$$E = E^0 - \frac{RT}{F} \ln a_{Cl^-} \quad (5)$$

where E (mV) is the measured electrode potential, E^0 (mV) is the potential at standard conditions ($T = 298$ °K [536.4 °R], $P = 1$ atm, Cl^- concentration = 1mol/lit [0.016 mol/in³]), R (=8.314 J/mol°K [40.881 lbs.in/mol°R]) is the universal gas constant, T (°K [°R]) is the temperature, F (=96485 col/mol) is the Faraday constant, and a_{Cl^-} (mol/lit [mol/in³]) is the activity ($\approx$ concentration) of chloride ions in the vicinity of the sensor.

Several chloride concentration sensors have been developed based on this principle ^[11, 22, 30]. The measurements show dependence on the moisture content and electrical conductivity of concrete and as such, proper calibration of the results is required.

3.4 Macrocell Measurements

In this setup, the galvanic corrosion current passing between an active (i.e., corroding) and a passive (i.e., non-corroding) electrode is measured (Figure 3a). Several embedded macrocell sensors have been developed including the anode ladder system developed by Raupach and Schießl (Figure 3b) ^[12]. This sensor is composed of a stainless steel cathode and a ladder of black steel anodes mounted at different depths from the surface of concrete. The anodes are electrically connected to the cathode and the corrosion current flowing between each anode and the cathode is monitored. As long as the black steel bars are in the passive condition, the galvanic corrosion current is minimal. When an anode is depassivated (due to carbonation or chloride penetration), the exchanged current increases rapidly. As a result, the sensor is capable of monitoring the depth of steel depassivation in concrete ^[12].

The measured corrosion current exclusively represents the rate of galvanic corrosion between active and passive electrodes. In addition, microcell (i.e., pitting) corrosion occurs at the surface of the active electrode whose magnitude cannot be monitored using the macrocell sensor. As such, the corrosion rate measured by macrocell techniques is only a fraction of the total rate of corrosion of the reinforcing steel in concrete ^[31]. The galvanic current is inversely proportional to the electrical resistance of concrete between the two electrodes and as such is dependent on the conductivity, moisture content, and temperature of concrete.

3.5 Corrosion Rate Measurements

Several methods are available for measuring the rate of corrosion of steel in concrete. The two most common techniques are linear polarization (LP) and electrical impedance spectroscopy (EIS). Both of these methods work based on the principle of polarizing the corroding electrode away from its equilibrium half-cell potential (E_{corr}). The measurement setup is shown in Figure 4 and includes the corroding electrode (also known as working electrode; WE), a non-corroding counter electrode (CE), and a reference electrode (RE) for potential measurements. A small polarizing potential ($\Delta E = \pm 10$ mV) is applied to the working electrode and the resulting corrosion current (ΔI) flowing between the working and counter electrodes are measured. The polarization resistance ($R_p = \Delta E / \Delta I$) is calculated and used to determine the corrosion current density (i.e., corrosion rate) of the working electrode using:

$$i_{corr} = \frac{B}{R_p A} \quad (6)$$

where i_{corr} (mA/m² [mA/in²]) is the corrosion rate, B (≈ 26 mV) is a constant (relating to anodic and cathodic Tafel slopes ^[23, 26]), R_p (Ω) is the polarization resistance, and A (m² [in²]) is the surface area of the working electrode polarized by the experiment. The main difference between the LP and the EIS methods is the use of alternating currents in the EIS method to calculate the polarization resistance ^[32]. A comprehensive description of the proper measurement practices for the LP method has been published recently ^[33]. For both LP and EIS methods, the ohmic resistance of concrete between the working and counter electrodes must be subtracted from the polarization resistance measured before determining the actual corrosion rate.

The biggest challenge in interpreting the measurements is identifying the true surface area of the steel polarized during the experiment. As shown in Figure 4, the area of the working electrode polarized cannot be known with

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certainty. To improve the accuracy of the results, a guard ring is often added surrounding the counter electrode. This guard ring confines, to some extent, the exchanged current to the area directly below the counter electrode. Nevertheless, the exact surface area of the polarized steel remains uncertain and as such, the results cannot provide the actual mass loss of the corroding rebar^[33].

For sensing applications, miniature versions of the measurement setup shown in Figure 4 have been developed and are available commercially^[13, 14, 22, 30]. These sensors enable continuous measurement of the corrosion rate over a period of time which can be advantageous in comparison to instantaneous corrosion rate measurements performed manually^[15]. Nevertheless, the embedded sensors are susceptible to problems originating from the uncertainty in the actual surface area of the polarized steel and the potential drift of the embedded reference electrodes. In addition, the measurements are influenced by the electrical conductivity of concrete as well as its moisture content and temperature^[15, 33]. Proper interpretation of the measurements requires consideration of these parameters.

4.0 DISCUSSION ON THE PARAMETERS INFLUENCING THE ELECTRICAL CONDUCTIVITY OF CONCRETE

In Section 3.0, it was discussed that the measurements of embedded electrical sensors used for monitoring the durability of reinforced concrete can be influenced by parameters such as the pore fluid composition, moisture content, and temperature of concrete. In this section, we specifically focus on the interpretation of the sensors measuring the electrical conductivity of concrete. It is shown that the transport properties of concrete (e.g., its chloride diffusion coefficient) can be determined based on the measured conductivity. However, a sole measurement of concrete conductivity is not sufficient to provide a reliable estimation of the diffusion coefficient. Rather, an accurate measurement of both concrete conductivity and pore fluid conductivity is needed. It is also discussed that the conductivity measurements are influenced by the moisture content and temperature of concrete and as such, proper calibrations are required.

4.1 Materials and Experiments

To use electrical conductivity for estimation of the transport properties of concrete and to study the dependence on the composition and volume of pore fluid, a series of five mortar mixtures were prepared using a w/c of 0.40 and aggregate volume fraction of 55%. Type I portland cement was used with Blaine fineness = $360 \text{ m}^2/\text{kg}$ [$2.54 \times 10^5 \text{ in}^2/\text{lbs}$] and the Bogue phase composition $\text{C}_3\text{S}=60\%$, $\text{C}_2\text{S}=12\%$, $\text{C}_3\text{A}=12\%$, $\text{C}_4\text{AF}=7\%$, and Na_2O equivalent = 0.72% . Natural river sand with fineness modulus 3.13 was used in these mixtures. The five mixtures had the same proportions, except they contained different dosages of chemical or mineral admixtures.

Table 1 summarizes the mixtures tested in this study. M1 was a plain mortar mixture. M2 was a mortar mixture in which the mixing water was replaced entirely by a 0.5 molar potassium hydroxide (KOH) solution. The alkaline solution can serve as an accelerator^[34]. M3 was a mortar mixture in which 7.5% of the mixing water (by weight) was replaced with a commercially available accelerator (Pozzolith[®] NC 534). M4 was a mortar mixture in which 5.0% of the mixing water (by weight) was replaced with a shrinkage reducing admixture (Tetraguard[®] AS20). M5 was a mortar mixture in which 10.0% of the cement (by weight) was replaced with densified silica fume.

In addition, to study the dependence of electrical conductivity on the moisture content (or internal relative humidity) of concrete, two cement paste mixtures (M6 and M7) were prepared. M6 was a plain paste mixture with w/c = 0.50 and M7 was a paste mixture with w/c = 0.35 in which 5.0% of cement (by weight) was replaced with densified silica fume.

All mixtures were prepared according to the procedure of ASTM C305. Cylindrical mortar specimens (50mm [1.97in] diameter $\times$ 65mm [2.56in] height) were prepared and seal cured inside airtight plastic containers of similar size. Electrical conductivity was measured with the aid of two stainless steel electrodes (2.4mm [0.095in] diameter distanced 10mm [0.39in] center to center) embedded inside mortar at the time of casting (i.e., 2-point measurement setup). Electrical resistance of mortar between the two electrodes was measured using electrical impedance spectroscopy following the procedure described in^[16]. Conductivity was obtained from the electrical resistance using Eq. (1). The measurements were performed at the laboratory temperature of $23 \pm 1^\circ\text{C}$ [$73.4 \pm 1.8^\circ\text{F}$].

Companion mortar specimens of similar geometry were prepared for pore solution extraction. After 9 days of curing, pore fluid from each specimen was extracted using a high pressure steel die according to the procedure of [35]. The extracted solutions were stored in 2.5ml [0.15in³] plastic vials and sealed immediately to prevent carbonation. The conductivity of the extracted pore solutions was measured using a conductivity meter connected to the impedance analyzer.

To obtain the correlation between cement paste's conductivity and its moisture content, thin cylindrical specimens (12.5mm [0.50in] diameter by 100 mm [3.94in] length) were cast in acrylic molds. After 24 hours, the specimens were demolded and transferred to a moist room (23°C [73.4°F] and 100%RH) for 18 months of moist curing. The extended curing period was used to minimize the effect of microstructure changes on the measured conductivity of cement paste. Each paste cylinder was then cut into small prisms (8×5×2mm [0.31×0.20×0.08in]). A pair of stainless steel electrodes was attached to each prism (on the opposing 5×2mm [0.20×0.08in] faces) using conductive silver epoxy (conductivity = 3.3×10^5 S/m [8250 S/in]) manufactured by Resin Technology Group.

Complete desorption-absorption conductivity isotherms (i.e., conductivity versus relative humidity) were obtained at 23°C [73.4°F] and the following relative humidities: 100% (i.e., saturated surface dried), 93%, 90%, 80%, 70%, 60%, 50%, 40%, 30%, 20%, and 10%. The desorption-absorption experiment was performed inside environmental chambers under controlled temperature (23°C [73.4°F]) and relative humidity (RH). The chambers operated under a nitrogen purge to reduce the potential for carbonation. After reaching equilibrium at a particular relative humidity, the electrical conductivity of each specimen was measured using impedance spectroscopy. Also, the moisture content was determined using mass measurements. For each cement paste, three duplicate specimens were tested. Further experimental details are provided in [16].

4.2 Results and Discussion

Figure 5 shows the electrical conductivity as a function of age for the 5 mortars studied in this work. The electrical conductivity decreases as the mortars hydrate. This observation is consistent with similar experiments performed on cement paste and concrete [6, 7]. The reduction in conductivity can be explained by equation (2) as the volume fraction (ϕ) and connectivity (β) of the pore fluid decreases significantly as hydration proceeds. As cement hydrates, water is consumed by chemical reactions, which causes the conduction paths (i.e., the liquid-filled pores) to become smaller and more tortuous. Previous work [6, 10] shows that the pore fluid conductivity (σ_o) increases with age; however, the reduction in ϕ and β outweighs the increase in σ_o and as such, the mortar conductivity (σ) decreases [36].

To estimate the transport properties, equations (3) and (4) can be employed. For this purpose, however, knowledge of the pore solution conductivity (σ_o) is also required. Unfortunately, this is often ignored by quality control engineers and even researchers. It is a public misconception that concrete conductivity can be determined, for example using the RCPT method (ASTM C1202), and used directly to evaluate concrete quality and determine its transport properties. We argue that this practice can lead to significant errors as it does not consider the important effect of pore fluid conductivity.

This can be illustrated by our measurements from the five mortar mixtures studied in this work. Figure 6 shows the mortar conductivity (σ) and the pore fluid conductivity (σ_o) measured after 9 days of hydration. If quality evaluation is based solely on the mortar conductivity, it will be concluded that mixture M5 (10%SF) has the lowest conductivity (and as such the best quality) and mixture M3 (7.5%Pozz) has the highest conductivity and therefore the lowest quality. Further, the chloride diffusivity can be calculated from equation (3) by assuming an average pore fluid conductivity (for example $\sigma_o = 9$ S/m [0.225 S/in]) for all five mixtures ($D_o = 1.484 \times 10^{-9}$ m²/s [2.300×10^{-6} in²/s] was used for diffusivity of NaCl in pore fluid [20]). These results are provided in Figure 7 (solid points) confirming that M3 and M5 has the highest and lowest diffusivity values.

However, the measured pore fluid conductivities shown in Figure 6 suggest that ignoring the value of σ_o (or assuming that σ_o is similar for all five mixtures) may not be appropriate. Although the five mortar mixtures have similar w/c and aggregate content, and have been cured similarly for 9 days, their measured pore fluid conductivities vary widely in the range 2.44 S/m [0.061 S/in] (for mixture M5) to 17.57 S/m [0.439 S/in] (for mixture M2) based on the type and dosage of the admixture used. When the actual σ_o values are used in equation (3) to calculate chloride diffusivity, the results are considerably different than the diffusivities obtained by assuming a similar σ_o value for all the mixtures. Figure 7 provides a comparison between these two sets of diffusion coefficients. When

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chloride diffusivity is calculated based on actual measurements of both σ and σ_o , mixture M5 (10%SF) shows to have the highest diffusivity (and therefore the lowest quality). In this case, the calculated diffusion coefficient is 14.8×10^{-12} S/m [22.94×10^{-9} S/in] which is 3.7 times larger than when an average $\sigma_o = 9$ S/m [0.225 S/in] was used for all mortar mixtures.

In summary, to evaluate the quality of concrete and especially to obtain its transport properties using electrical measurements, both concrete conductivity (σ) and pore fluid conductivity (σ_o) must be measured. The influence of pore fluid conductivity cannot be assumed to be minor. The magnitude of pore fluid conductivity can vary over a wide range based on concrete's mixture proportions (w/c and admixtures), age, and exposure history. As such, using a constant or typical value for σ_o will lead to errors in calculating concrete's transport properties.

In addition to the pore fluid conductivity, concrete conductivity can be influenced significantly by changes in its moisture content. To study the dependence of electrical conductivity on the internal humidity and moisture content, two cement paste mixtures (M6 and M7) were tested. Mature (18 months old) specimens were prepared and dried to equilibrium at known relative humidities inside environmental chambers. The equilibrium electrical conductivity and moisture content of each paste specimen were measured as a function of the relative humidity. Figure 8 shows the electrical conductivity as a function of relative humidity and moisture content. In general, conductivity is observed to decrease as the material dries to lower relative humidities. This signifies the influence of moisture on the measured conductivity value.

The σ -RH curves show significant hysteresis. This hysteresis is mainly due to the constrictions in the pore network of the cement paste and has been discussed in details in a previous paper^[16]. As a result of this hysteresis, the correlation between conductivity and relative humidity is not unique and depends on the drying history of the specimen (as shown in Figure 8a for the case of RH = 50%). Consequently, insitu conductivity measurements cannot be used directly to estimate the internal relative humidity of concrete. Alternatively, a porous material with low hysteresis can be used inside concrete as an embedded sensor for insitu measurements of relative humidity^[10, 16].

On the other hand, the σ -MC curves do not show hysteresis. This means that a one-to-one correlation can be established between the moisture content and the electrical conductivity of concrete irrespective of the moisture history of the material. This observation further suggests that it is the moisture content and not the relative humidity which determines the electrical conductivity of concrete exposed to drying. As such, it may be possible to use insitu electrical conductivity measurements to estimate the moisture content of concrete. Unfortunately, the σ -MC correlation is not similar for different concrete mixtures (Figure 8b) which prevents using a "master curve" to relate insitu conductivity measurements to the moisture content of concrete. Further research is needed in this area to study the dependence of the σ -MC correlation on the microstructure of concrete and the composition of pore solution.

Finally, the electrical conductivity is influenced by concrete's temperature. The temperature dependence of conductivity has been studied by several researchers^[7, 37, 38] and procedures have been proposed to temperature calibrate the conductivity using an activation energy approach (i.e., using Arrhenius equation). We will not discuss the temperature effect further in this paper.

5.0 FUTURE NEEDS FOR DEVELOPMENT AND IMPLEMENTATION OF NEW SENSORS

The experimental results and discussions presented in this paper emphasize the importance of measuring concrete conductivity, pore fluid conductivity, moisture content, and temperature. These parameters have a significant influence on the measurements of embedded electrical sensors currently used for health monitoring of concrete. At present, concrete conductivity and its temperature can be measured with commercially available sensors. On the other hand, no insitu sensor is available for measuring the pore fluid conductivity or the moisture content of concrete (There are sensors that measure the relative humidity of concrete^[23, 39] although their field performance is questionable. Direct measurement of concrete's moisture content in the field is not currently possible).

It was discussed earlier that measurement of pore fluid conductivity is important for accurate estimation of the transport properties of concrete. Such measurement also provides a means to monitor changes in the internal chemistry of concrete due to ion penetration, carbonation, or deteriorative reactions such as ASR. Moisture content of concrete influences its transport properties as well as the corrosion rate of reinforcing steel. In addition, moisture content is known to be significant with regards to other deterioration mechanisms such as freezing and thawing and shrinkage^[40, 41].

It is important to develop and implement sensors capable of insitu monitoring of pore fluid conductivity and moisture content of concrete. Recently, a prototype sensor has been developed at Purdue University for the measurement of pore fluid conductivity^[10, 36]. This sensor makes use of a porous material (e.g., natural porous stone) as an inclusion inside concrete. The sensor is initially saturated with a synthetic pore solution. After embedding inside concrete, the sensor's pore fluid gradually reaches equilibrium with the pore solution of the surrounding concrete. Since the sensor's pore structure is known, by measuring the sensor's electrical conductivity, the conductivity of pore fluid can be determined. This sensor has shown promising results in the laboratory^[36] and is currently being tested at field environments.

While several sensors have been developed for insitu measurement of concrete's relative humidity^[10, 23, 36, 39], none can be used directly to determine concrete's moisture content. Since electrical conductivity is directly related to concrete's moisture content, it could be possible to develop a sensor that determines the moisture content based on concrete conductivity. Further research is needed to develop the σ -MC correlations that can be applicable to field concrete.

6.0 CONCLUSIONS

Based on the observations made in this paper, the following conclusions can be drawn:

- Several types of electrical sensors are available for health monitoring of reinforced concrete structures. When calibrated properly, these sensors can measure the transport properties of concrete and monitor the initiation and propagation of steel corrosion inside concrete.
- The measurements of the electrical sensors are dependent on several factors including concrete's porous microstructure, pore fluid conductivity, concrete's moisture content, and temperature. The sensors' readings must be calibrated and normalized to account for these parameters; otherwise, the results can be misleading.
- The electrical conductivity of concrete cannot by itself represent the quality of concrete, nor can it be used to determine concrete's transport properties. The ratio of concrete to pore fluid conductivity (σ/σ_o) must be used to evaluate concrete's quality and its transport properties.
- The electrical conductivity shows a one-to-one correlation with concrete's moisture content. As such, it is potentially possible to use insitu conductivity measurements to determine the moisture content of concrete in the field.
- Correlation between the conductivity and the internal relative humidity of concrete shows significant hysteresis. As such, conductivity cannot be used directly to estimate concrete's internal humidity. Porous materials with low hysteresis may be used as embedded relative humidity sensors.
- It is important to develop and implement additional sensors to monitor concrete's pore fluid conductivity and its moisture content. These measurements provide necessary information to calibrate the readings of other electrical sensors. In addition, the proposed sensors improve health monitoring by providing information regarding changes in the internal chemistry of concrete as well as the risk of freezing and thawing and shrinkage damage.

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Table 1 — Summary of the mixtures tested

Mixture	Type	w/c	Admixture	Admixture Dosage	Age Studied	Purpose
M1	Mortar	0.40	Plain	---	up to 10 days old	Estimate transport properties
M2	Mortar	0.40	0.5 molar KOH	Repl entire mix water		
M3	Mortar	0.40	Accelerator	7.5%wt. repl water		
M4	Mortar	0.40	SRA	5%wt. repl water		
M5	Mortar	0.40	Silica Fume	10%wt. repl cement		
M6	Paste	0.50	Plain	---	at 18 months old	Conductivity-moisture content correlation
M7	Paste	0.35	Silica Fume	5%wt. repl cement		

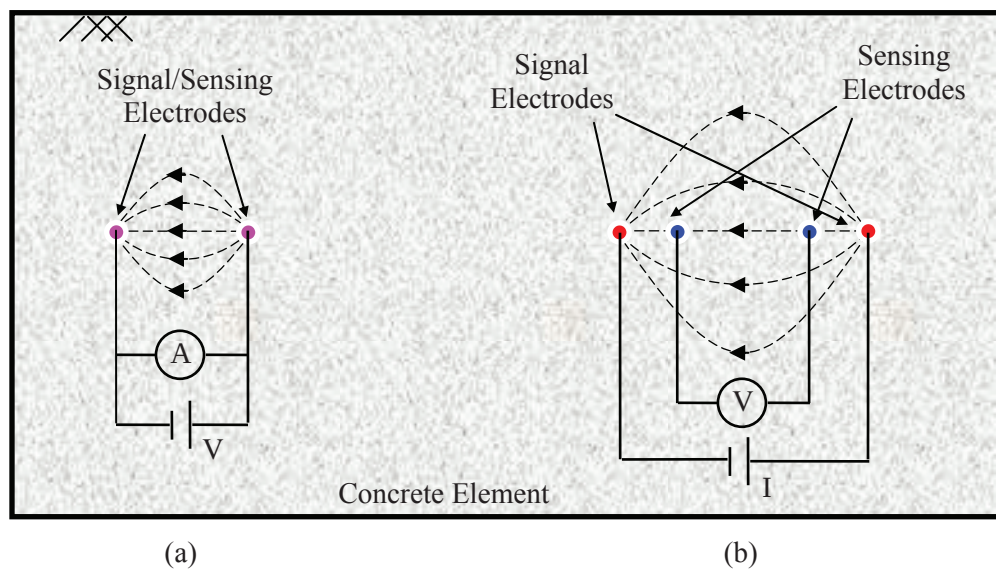


Figure 1 — Schematic representation of the electrical conductivity sensor showing (a) two-point and (b) four-point measurement setup

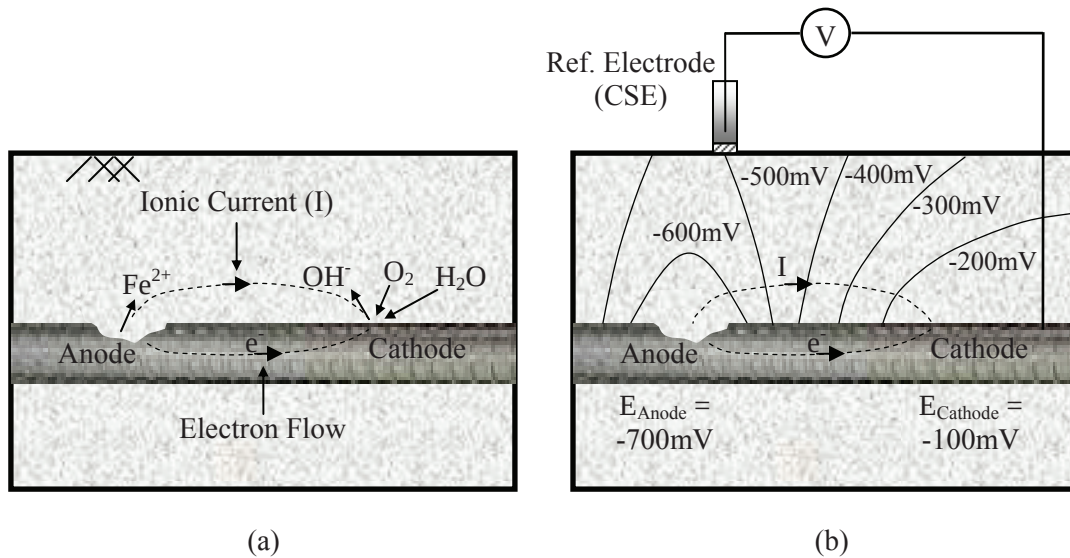


Figure 2 — (a) Anodic and cathodic areas formed during the corrosion process; (b) Formation of electrical field that causes current flow from anode to cathode

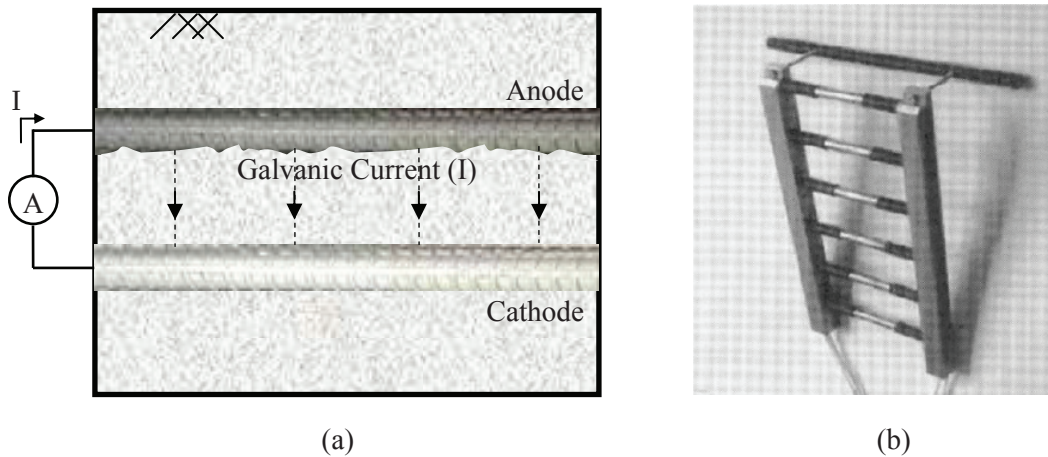


Figure 3 — (a) Schematic representation of the macrocell measurements; (b) The macrocell anodic ladder sensor developed by Raupach and Schießl^[12]

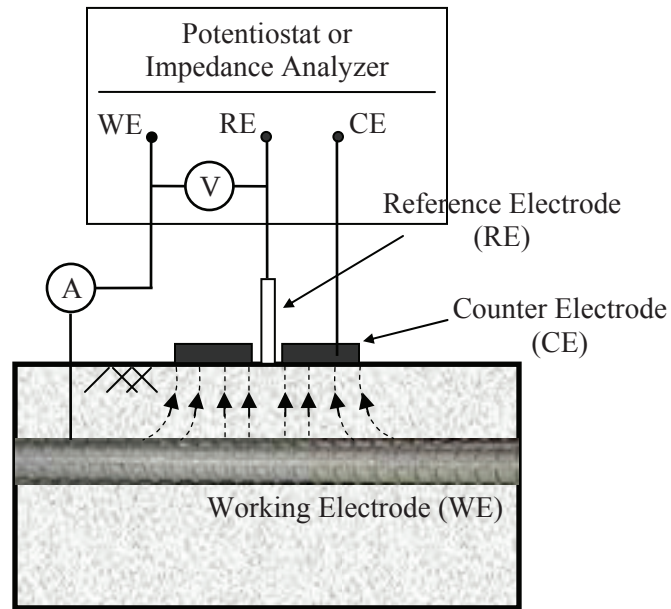


Figure 4 — Corrosion rate measurement using linear polarization (LP) or electrical impedance spectroscopy (EIS)

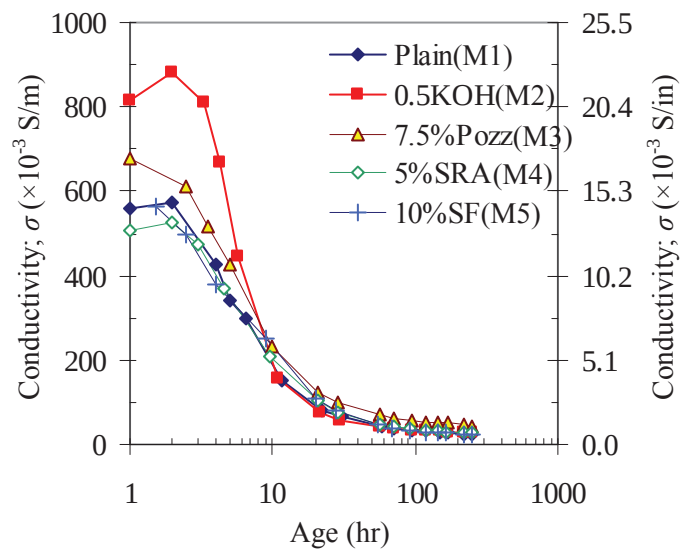


Figure 5 — Electrical conductivity of the mortar mixtures as a function of age

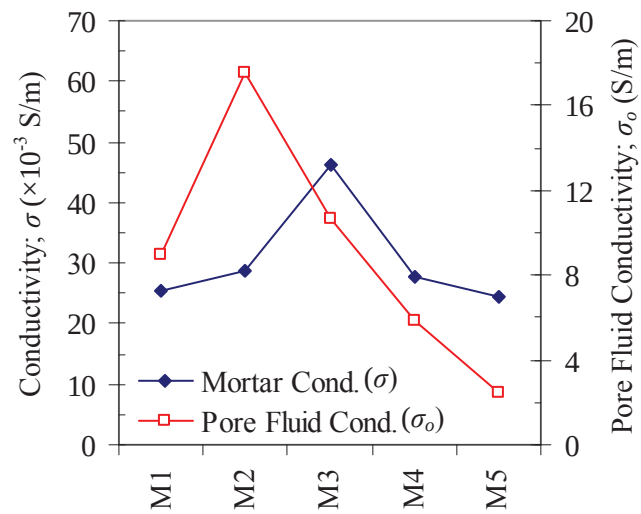


Figure 6 — Mortar conductivity and pore fluid conductivity after 9 days of hydration (1 S/m = 0.025 S/in)

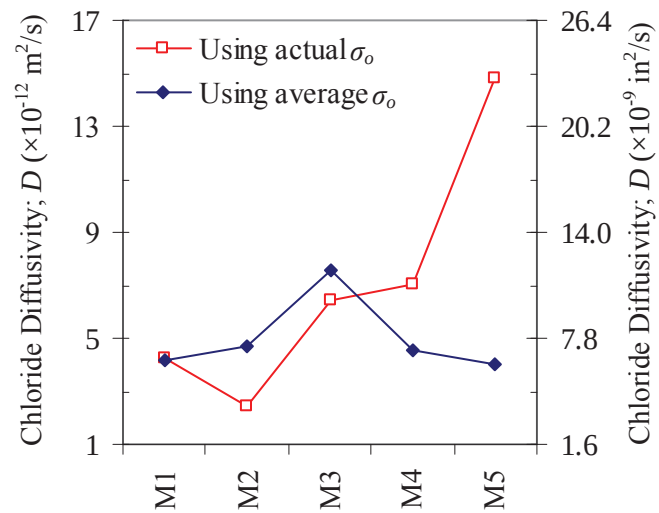


Figure 7 — The 9-day chloride diffusion coefficient of mortar calculated based on the actual and average values for pore fluid conductivity (σ_o)

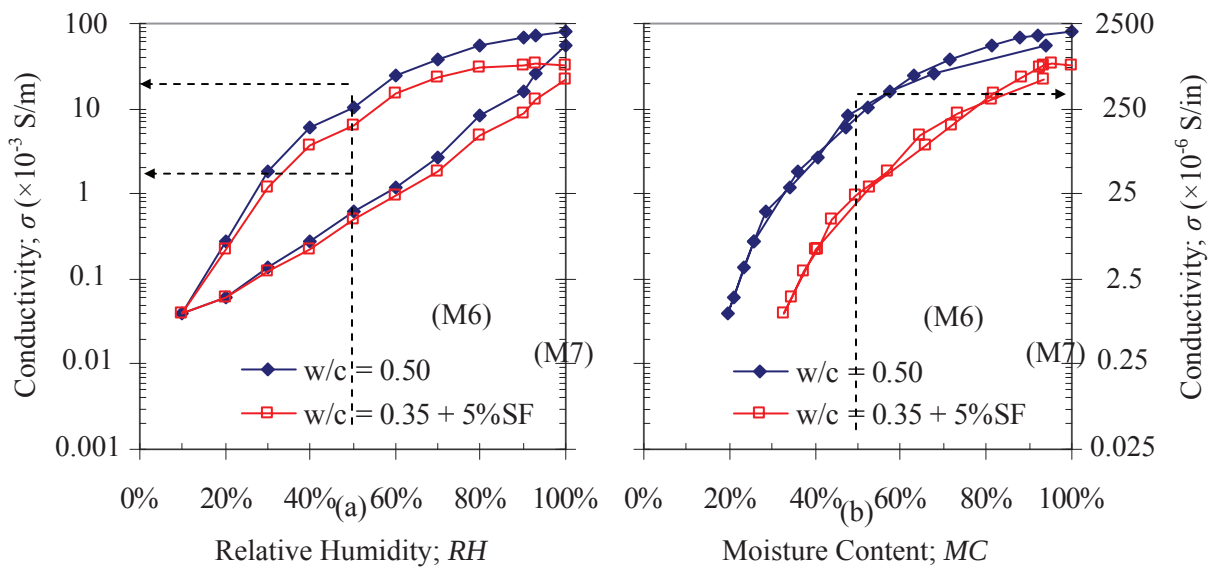


Figure 8 — Relationship between the electrical conductivity and the relative humidity (a) or the moisture content (b)

Electrical Property-Based Sensing of Concrete: Influence of Material Parameters on Dielectric Response

by R.K. Manchiryal and N. Neithalath

Synopsis: Electrical property based sensing of the properties of cement based materials is becoming increasingly popular. This paper reports the results of a systematic investigation carried out to understand the influence of material parameters on the dielectric response of cement pastes and concretes. A simple mathematical model that links the effective conductivity and time has been proposed, which can provide the setting time of the pastes. The variation in conductivity between plain and fly ash modified pastes is brought out. A fractional factorial experimental design coupled with range analysis is used to extract the significant parameters that influence the electrical conductivity of concretes. It is found that the aggregate-to-cementing materials ratio (a/cm) influences the very early age conductivity of concretes while the combinations of a/cm and water-to-cementing materials ratio (w/cm), and fly ash content and curing condition influences the final conductivity significantly. The changes in dielectric constant and conductivity spectra are attributed to the polarization phenomena. The dispersion in electrical conductivity starts at lower frequencies for fly ash modified pastes. There is an observed dielectric enhancement also for fly ash modified pastes. The dielectric response of concrete is very similar to that of pastes, and the effect of dilution by the aggregates is evident. The relaxation times of the cement pastes were determined from a geometric analysis of the bulk arc in the Nyquist plot. This method facilitates the determination of relaxation times and depression angle of the bulk arc using limited frequency sweep. The relaxation times decrease with increase in hydration time and conform to a power law. Though only a single relaxation time is considered in this study, the relaxation time distributions as represented by the depression angle, indicate increasing material heterogeneity with increasing hydration. Representing the dielectric constant at particular frequencies as a function of the capillary porosity in the cement pastes, it is possible to detect the percolation threshold of cement pastes. The observed percolation thresholds agree well with those reported for cement based materials.

Keywords: conductivity; dielectric response; electrical impedance spectroscopy; percolation threshold; relaxation time

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INTRODUCTION

Portland cement based materials change from a fluid phase (when it is placed) to a semi-solid phase and then to a solid phase when it is finally set. Monitoring the changes occurring in concrete with time is very important to estimate the performance and durability of concrete at later ages. Electrical impedance spectroscopy (EIS) has emerged as one of the promising techniques to non-invasively probe the microstructure and property development in cement based materials especially at early ages.¹⁻⁸ The dielectric properties of cement based materials and the microstructure changes with time as cement hydrates. The electrical properties of porous media such as conductivity and dielectric permittivity are extremely sensitive to physical conditions, the chemical composition of solid and liquid phases, and microstructural features like porosity, its distribution, and connectivity. EIS has also been proposed as a method to assess the transport properties as well as changes in pore solution and microstructure of cement based materials at later ages.⁹⁻¹² The application of electrical impedance techniques for evaluation of other similar porous materials like rocks have also been well reported.¹³⁻¹⁸

This paper reports on detailed investigations conducted on cement pastes and concrete mixtures to understand the influence of material parameters like water-cementing materials ratio, fly ash content, aggregate-cementing ratio, and aggregate size on the dielectric response of the material at early and relatively later ages. Understanding the influence of these parameters on the dielectric response is important in ensuring proper data interpretation from field deployed electrical impedance based sensing systems. This paper also presents preliminary results that attempt to fill a void that exists in relating the dielectric response to characteristic microstructural features like porosity and percolation threshold.

EXPERIMENTAL PROGRAM

Materials and mixture proportions

Type I/II ordinary Portland cement conforming to ASTM C 150 is used for both the cement pastes and concrete mixtures investigated in this study. A Class F fly ash is used as a replacement for some amount of cement in a few mixtures. Cement pastes were prepared with water-cementitious materials (cement + fly ash) ratios (w/cm) of 0.32 and 0.42. The replacement rates of cement with fly ash were 5, 10, and 20% by mass. For the concrete mixtures investigated in this study, emphasis is placed on understanding the influence of the factors listed below on the electrical response of concretes. The factor levels used for the experiments are also shown. (i) water-to-cementing materials ratio (w/cm) - levels of 0.40 and 0.50, (ii) percentage of fly ash replacing cement - levels of 0 and 20 %, (iii) aggregate-cementing materials ratio (a/cm) - levels of 3 and 5, (iv) aggregate size - levels of 10 mm and 20 mm, and (v) curing condition - saturated and sealed conditions. These factors were chosen because they are believed to exert a significant influence on the electrical performance of concrete.

With 5 factors and two levels for each factor, 32 (2^5) experiments are required to understand the influence of each of the parameters on the electrical response of concrete. Since these experiments are time consuming, it was decided to use a half fractional factorial experiment, resulting in 16 runs. Table 1 shows the 16 run experimental design with the factors and factor levels.

Measurement of electrical conductivity of cement pastes and concrete

The cement pastes were filled in acrylic molds (50 mm (2 in) x 50 mm (2 in) x 150 mm (6 in)) immediately after they were mixed. Two stainless steel plates (50 mm (2 in) x 75 mm (3 in) x 0.75 mm (0.03 in)) were placed at the ends of the molds to be used as electrodes. Alligator plugs from the impedance analyzer were attached to the electrodes. The impedance measurements (using Solartron 1260™ gain phase analyzer) were carried out over a frequency range of 1 Hz to 1 MHz using a 250 mV AC signal. 10 measurements per decade of frequency were carried out. From the bulk resistance (R_b) of the sample (the real coordinate of the meeting point of bulk and electrode arcs in a Nyquist plot), the effective conductivity (σ_{eff}) is calculated as:

$$\sigma_{eff} = \frac{l}{R_b A} \quad (1)$$

where, 'l' is the spacing between the steel electrodes, and A is the cross sectional area.

The first electrical conductivity measurements were carried out at 15 minutes after the cement and water were mixed. For the first 6 hours, the measurements were carried out at every 15 minutes and thereafter at every 30 minutes for the next 3 hours. After the conductivity changes become gradual, the measurement intervals were increased to about once every 6 hours.

The concrete mixture was poured in acrylic molds (75 mm (3 in) x 75 mm (3 in) x 250 mm (10 in)). Steel plates (75 mm (3 in) x 90 mm (3.54 in) x 0.75mm (0.03 in)) were used as the electrodes. The steel plates were inserted into the specimen at a distance of 50 mm from the edges of the mold. The impedance measurements carried out are similar to those for cement pastes, as explained earlier. The impedance spectra were generated at frequent intervals for the first 24 hours. After that, the specimens were removed from the molds. The specimens that needed to be cured under saturated conditions were transferred to plastic containers filled with saturated limewater, where as the specimens that needed to be cured under sealed conditions were covered with non-conducting water resistant tape and stored in an environment with 70 % RH and at $23 \pm 2^\circ$ C. The impedance spectra of these specimens were also obtained at frequent intervals.

ANALYSIS OF RESULTS AND DISCUSSIONS

Electrical conductivity behavior of cement pastes at early ages

The Nyquist plots for the cement pastes were generated at each of the measurement times. Figure 1(a) depicts the variation of the effective conductivity with time for plain cement pastes of w/cm 0.32 and 0.42, while Figure 1(b) shows the relationship for fly ash modified pastes. The effective conductivity decreases with time (as more solid hydration products are formed in the system) as can be seen from these figures. There is a certain period during which the conductivity drop is drastic and the behavior in this region could be exploited to sense the physical processes in the system at relatively early ages. The conductivity-time curve can be divided into five distinct regions. During phase I, the ions are starting to be released from the cement to the surrounding pore solution. Higher w/cm mixtures show similar or slightly lower conductivities at this time due to a lower amount of conducting ions (because of lower cement content in high w/cm mixtures). The conductivity slowly starts to drop with time during phase II. The induction phase in cement hydration ends at this time. A faster rate of conductivity decrease is noticed in phase III (3 to 12 hours). The initial and final setting times of the pastes occur in this phase and the solid hydration products start to form at this stage. In phase IV, the conductivity decrease becomes more gradual. The hydration products are formed around the cement grains and further hydration is facilitated by diffusion. A slow and steady reduction in conductivity is noticed in phase V, and eventually the conductivity becomes almost constant. Once the entire capillary pore space is depercolated, further reduction in conductivity is minimal and the time scale shifts from hours to days.

From Figures 1(a) and (b), it could be seen that the plain cement paste has a higher conductivity at very early times (in Phase I) whereas the 0.32 w/cm paste with 20% fly ash show the lowest conductivity. This could be attributed to (i) the reduced amount of mobile ions in the pore solution for fly ash paste because of reduced cement content, (ii) lower w/cm, and (iii) lower specific gravity of fly ash than the cement it replaces, leading to higher volume of fly ash for the same mass replacement of cement and consequently lower effective pore volume at early times.¹⁹ With an increase in fly ash content, the conductivities at the end of measurement duration (7 days) are slightly higher.

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This is because, at the end of 7 days, secondary reaction of fly ash to form hydration products is far from complete, resulting in a higher pore volume than that of the plain pastes. It is anticipated that the conductivity of fly ash pastes after very long time periods (56 or 90 days) will be lower than those of plain pastes because of secondary reaction.

Based on the experimental data relating effective conductivity (σ_{eff}) of plain and modified pastes with time (t), a mathematical model has been developed in this study. The general trend in the σ_{eff} - t relation can be expressed using the following equation.

$$\sigma_{\text{eff}} = \sigma_f + \frac{\sigma_0 - \sigma_f}{1 + \left(\frac{t}{t_s}\right)^b} \quad (2)$$

where σ_f is the conductivity at the point where Phase V begins (when the conductivity-time curve starts to plateau), σ_0 is the conductivity corresponding to the Phase I (initial conductivity), t_s is a characteristic time, (which is found to correlate fairly well to the final setting time), and b is a constant. The parameters σ_f , σ_0 , t_s and b could be obtained from a least-squares fit of the experimental data to Equation 2.

As an example, the model fit and data points for a 0.32 w/cm paste with 5% fly ash is shown in Figure 2. It is found that the model agrees very well with the experimental data (R^2 value of 0.99). The values of the parameters and the constant in the Equation 2 for all the pastes studied are shown in Table 2. It can be observed that the difference between the initial and final conductivities of pastes ($\sigma_0 - \sigma_f$) decreases with increase in fly ash content. This points to an increased sensitivity requirement for electrical conductivity sensing in mixtures modified with fly ash. An increase in setting time as predicted by the fit is observed for both increasing w/cm and fly ash content, as expected. The constant b also shows a trend similar to that of the difference between initial and final conductivities. The final conductivity values also are found to increase with increasing fly ash contents, the reason for which has been explained earlier.

Using the experimental data of conductivity as a function of time shown in Figure 3, the time derivatives of conductivity ($\frac{d\sigma_{\text{eff}}}{dt}$) were determined. Figure 3 shows the experimental values of σ_{eff} and $\frac{d\sigma_{\text{eff}}}{dt}$ plotted against time for a 0.32 w/cm paste modified with 5% fly ash. It can be noticed from Figure 3 that the derivative plot has certain distinct peaks and troughs which can be used to infer information on material properties at early ages. Detailed analysis of time-derivative of conductivity plots and their relationship to the progress of cement hydration has been described elsewhere.^{19, 20}

Figure 4 shows the comparison of initial and final setting times from Vicat's apparatus test and the time derivative of conductivity plot. The values are very close to each other, validating the use of minima from time derivative of conductivity as a viable method to predict the setting time of cement pastes.

Electrical conductivity – time relationships for concrete mixtures

As mentioned earlier, σ_{eff} - t relationships were extracted for all 16 concrete mixtures. The same mathematical model used for cement pastes (Equation 2) was used for concretes also. As an example, the model fit for concrete mixture with a w/cm of 0.50, a/cm of 3, 20 mm (0.8 in) maximum size aggregates, and cured under saturated condition is shown in Figure 5. The fit provides the values of $\sigma_0 - \sigma_f$, σ_f , and the constants t_s and b from the least squares fit similar to that of cement pastes. The values obtained from the fit for these parameters are shown in Table 3. The values of t_s are in the range of setting times of concretes, measured using pin penetration tests on the separated matrix phase of concretes.²¹ The conductivity-time curves of concretes essentially show the effect of dilution, when compared with the curves for cement pastes. Also noticeable from Table 3 are the differences in initial and final conductivity values between mixtures made with a/cm of 3 and 5. The mixtures with a/cm of 3 show higher σ_0 and σ_f values owing to higher cement content. The effect of dilution can be clearly observed for mixtures with a/cm of 5.

Since a fractional factorial (one half of 2^5) experimental design was adopted on concrete mixtures, all individual effects are not obtained. Hence, a range analysis was carried out to obtain the most significant factors and the interactions. Range is defined as the difference between the mean values of levels of test results for a factor.²² The sensitivity of the factor depends on the extent of range. Negative and positive values of the range indicate whether

to minimize or maximize the level. Range analysis was carried out for initial and final conductivities, and the parameter t_s from the fit of Equation 2 to the experimental data. The salient observations from range analysis are summarized below. These pertain to the factors and factor levels considered in this study.

- i. a/cm has the highest influence on initial conductivity;
- ii. A non-negligible effect is noticed for the fly ash content, while w/cm is found to have no effect on initial conductivity. This is expected because the material is fluid at early times and the conduction paths are continuous;
- iii. Fly ash content, aggregate size and curing method exert significant influence on the final conductivity values;
- iv. Fly ash content and w/cm show the highest effect on the parameter t_s . The setting times are dependent on paste parameters and it is not surprising that aggregate size and content are insignificant; and
- v. The factor combinations of: (i) a/cm and w/cm , and (ii) fly ash content and curing condition are the influential factor combinations for final conductivity and the parameter t_s .

A brief description on two of the influential factor or factor combinations are given below:

Influence of aggregate size--Figure 6 shows the effect of aggregate size (10 mm (0.4 in) and 20 mm (0.8 in) maximum size) on effective electrical conductivity for concrete specimens made with a w/cm of 0.40, and a/cm of 3 for: (a) specimens cured under saturated condition and no fly ash content and (b) specimens cured under sealed condition, with 20 % fly ash as replacement for cement. In the proportioned mixtures, a change in aggregate size is made in such a way that the total aggregate and paste volumes do not change. By replacing 20 % cement with fly ash, it is observed from Figure 6(b) that there is a slight decrease in early and later age conductivities when compared to the plain concrete mixtures in Figure 6(a). It needs to be noted that the early age conductivities (less than 24 hrs) are not influenced by the curing conditions since the specimens are in the molds during that period. The reduction in conductivity for the fly ash mixture is because of the reduced amounts of ions released into the pore solution because of reduced cement content.

Influence of w/cm , a/cm and fly ash content--Figure 7 shows the effect of w/cm , a/cm and fly ash content on effective electrical conductivity of concretes made with 10 mm (0.4 in) aggregates and: (a) cured under saturated condition and with no fly ash, and (b) cured under sealed condition, with 20 % fly ash. As stated earlier, whether sealed or saturated, the behavior is similar for the first 24 hours. It is observed that the mixture with higher a/cm has a lower effective electrical conductivity irrespective of the curing condition and the presence of fly ash. This is because the higher a/cm mixture has more volume of aggregates and consequently a lower volume of paste. This results in the release of fewer ions into the pore solution, and thus lower electrical conductivity. From Figure 7(a), it can be seen that even when the w/cm is higher, the early age effective conductivity of 0.50 w/cm mixture is lower. This indicates the increased significance of a/cm in determining the early age conductivity of concretes, a fact that was evident from the range analysis results as stated earlier. The influence of fly ash in reducing early age conductivity is much more dominant for 0.40 w/cm mixture than for 0.50 w/cm mixtures as could be observed from a comparison between Figures 7(a) and (b). This interrelationship between w/cm and a/cm is to be considered when electrical property based sensing is used to determine water contents in fresh concrete.

Dielectric spectra of cement pastes and concrete

The dielectric permittivity (ϵ) and conductivity (σ) as a function of frequency (ω) were determined from the impedance data obtained from the experimental results using the following Equations (3) and (4) given below:²³

$$\epsilon = \left[\frac{1}{(j\omega\epsilon_0 Z(\omega))(A/d)} \right] \quad (3)$$

$$\sigma(\omega) = \epsilon_r''(\omega)\omega\epsilon_r \quad (4)$$

where A is the electrode surface area, d is the electrode spacing, and ϵ_0 is the permittivity of free space (8.854×10^{-12} F/m). The real part of ϵ (ϵ') is the dielectric constant ϵ_r . The dielectric response spectra were generated at all ages

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(starting at 15 minutes until 7 days for pastes and 28 days for concrete) but only representative data will be discussed in this section.

Dielectric spectra for cement pastes--The variation of dielectric constant and conductivity with frequency for cement pastes is described in this section. Figure 8(a) shows the dielectric constant as a function of frequency for plain cement pastes with w/cm of 0.32 and 0.42 at different times (60 minutes, 24 hours, and 7 days). At very early ages, there is no appreciable difference in ϵ_r between pastes of both w/cm. A general trend of rapidly decreasing dielectric constant from a very large value at low frequency and a tendency to stabilize at higher frequencies can be observed in this figure. The extremely high (of the order of 10^9) dielectric constants at low frequencies are attributed to interfacial or electrode polarization.^{23,24} Also, at low frequencies, the particles of the dielectric medium might screen the external electric field, thus contributing to a strong polarization effect and effectively giving the water molecules a very large dielectric constant.¹⁷ The plateau which starts to appear at high frequency could be attributed to the relaxation of weakly bound water molecules.²⁴ The dielectric constant, especially at lower frequencies, increases with increase in w/cm for all ages as can be seen from Figure 8(a), directly pointing to the increased amount of weakly bound water in the system. At early ages, the dielectric constant dispersion occurs at higher frequencies (about 100 kHz) and the dispersion shifts towards lower frequencies as the paste ages.

Figure 8(b) shows the dielectric spectra for pastes with w/cm of 0.32, with or without fly ash. Similar trends as explained above can be found in this case also. The plain paste shows lower dielectric constant values at lower frequencies than that of pastes modified with fly ash, at all times. It is observed that the dispersion for the modified pastes is extended to higher frequencies when compared to the plain pastes. This can be attributed to the effect of fly ash in the enhancement of the bulk polarization process.²³ From Figure 8(c) it is observed that, as the quantity of fly ash is increased, there is a marginal increase in the magnitude of the dielectric constant. This is in line with the dielectric enhancement reported for fly ash modified pastes.²³

Figure 9 shows the conductivity spectra at different times for pastes with: (a) w/cm 0.32 and 0.42, and (b) 0 and 20% fly ash as replacement for cement for the paste with w/cm of 0.32. It can be observed that the conductivity spectra, especially the early age one, can be divided into two regions at approximately 1 kHz frequency. At later ages also, similar behavior is seen, though not explicitly discernible in the graph due to the scale of the ordinate axis. Plotting the conductivity in the range of 0 to 0.5 S/m is seen to capture this behavior better. Below 1 kHz, the significant decrease in conductivity with decreasing frequency is the result of the increasing contribution of electrode effects and interfacial polarization. At frequencies greater than 1 kHz, the conductivity is almost independent of the frequency. In this region, the ionic conductance dominates. A very high frequency dispersion has been noticed for conductivity but in this study, only a maximum frequency of 1 MHz was used, which is below the high frequency range for dispersion. With time, the conductivity decreases and then becomes almost constant, as was observed in Figure 1. The plateauing of conductivity after about 1 kHz indicates the presence of weakly bound water which follows a low frequency electric field but not a high frequency one. In pastes with fly ash as shown in Figure 9(b), the conductivity dispersion starts at a higher frequency, the reason for which has been elaborated earlier. Also worth mentioning from these figures are the changes in conductivity with time. The explanation for this could be found in an earlier section.

Dielectric spectra for concretes--Figure 10 shows the dielectric constant spectra for: (a) concrete mixtures with 10 mm (0.4 in) and 20 mm (0.8 in) aggregates, and (b) concrete mixtures with a/cm of 3 and 5. The results are for mixtures cured under saturated condition for the entire duration. It is observed that the magnitude of the dielectric constant for the concrete mixtures is lower when compared to that of pastes of similar w/cm, which can be attributed to the effect of dilution by aggregates. The dielectric dispersion for concretes also occurs in the same frequency range as those for the pastes. The characteristic features in the dielectric constant spectra are very similar to those of pastes explained earlier.

Figure 11 shows the conductivity spectra of concretes at 6 hours, 24 hours, and 28 days after mixing. The same mixtures as in Figure 10 are compared. A lower conductivity as compared to the pastes is observed because of the dilution effect of aggregates. The other features of the spectra remain unchanged. For both pastes and concretes discussed in this section, no high frequency dispersion in conductivity is observed as is found for rocks and highly porous concretes.^{17, 25} The reason could be that, at the chosen w/cm and under saturated curing conditions, the samples do not lose enough moisture either by hydration or evaporation to lower the ionic conductivity to a level where this dispersion would become dominant. For specimens cured under sealed conditions, or for those allowed to dry, high frequency dispersion is to be expected at later ages. This will be explored in a later study.

Dielectric Relaxation Times and its Relationship to Property Development in Cement Pastes

The frequency dependent complex permittivity ($\epsilon(\omega) = \epsilon'(\omega) - i\epsilon''(\omega)$) and conductivity ($\sigma(\omega) = \sigma(0) + \sigma_d(\omega)$ where $\sigma(0)$ is the d.c conductivity and σ_d is the dielectric conductivity) described in the previous section can be used to characterize the amplitude and time-scale of the charge-density fluctuations in the specimen. The time scale of these fluctuations (or, the relaxation times) depend on the sample composition, age, the state of water molecules in the material (free or bound), and the relaxation mechanism.^{17, 18, 26-28} Low viscosity liquids typically show very low relaxation times (of the order of 10^{-7} and below). The relaxation mechanism in cement based materials could either be due to the fluctuations arising from the re-orientation of the water molecules under the influence of a variable frequency electric field, and the presence of interfacial charges. These mechanisms and their influence on the dielectric constant and conductivity response have been detailed in the previous section. In this section, an initial approach to determine the characteristic relaxation times of the cement pastes at various times is outlined. Though the relaxation time for heterogeneous materials is distributed around a mean value,^{28, 29} no attempt is made in this study to extract the distribution functions. However, an indication of the relaxation time distribution functions could be obtained from the analysis presented below.

There are several methods of predicting the relaxation time (τ_R). They include: (i) using the simple relationship between dielectric constant, conductivity and relaxation time ($\tau_R = \frac{\epsilon_r \epsilon_0}{\sigma}$), (ii) using the relation between the frequency at the peak of the bulk arc and relaxation time ($\tau_R = \frac{1}{\omega_{peak}} = R_{peak} C_{peak}$), or (iii) using the plot of ϵ'' or ϵ'' with frequency and fitting the classical Debye expression to extract τ_R . The determination of frequencies corresponding to the relaxation time might be complicated and some times beyond the capability of many measuring instruments, limiting the applicability of the first two methods, while for the third method, an adequate fit to the experimental data of ϵ' or ϵ'' with frequency depends on the measurement range. Hence, a method that can provide τ_R from a smaller frequency range is explained here.

The Nyquist plots of cement based materials show two arcs – the bulk arc at high frequencies, and the electrode arc at lower frequencies. The bulk arc is typically depressed from the Z' axis (i.e., its center does not lie on the Z' axis, rather it is depressed by an angle θ ($=\pi\alpha/2$, where α is the exponent in the Cole-Cole distribution function). Using any point on the bulk arc with a specific value of Z ($Z^* = |Z|$), and the points R_b (corresponding to the bulk resistance – the point on the bulk arc where the frequency is the lowest), and R_∞ (the point on the bulk arc where the frequency is maximum, or the point where the bulk arc crosses the Z' axis; $R_b > R_\infty$), it can be shown that [28]:

$$\ln \left[\frac{R_b - Z^*}{Z^* - R_\infty} \right] = \psi_{ZC} [\ln(\omega) + \ln(\tau_R)] \quad (5)$$

If a number of Z^* points belonging to the bulk arc corresponding to different frequencies are used, and the left hand side of the Equation 5 is plotted against $\ln(\omega)$, a straight line with a slope of ψ_{ZC} and an intercept of $\psi_{ZC}(\ln(\tau_R))$ can be obtained. The relaxation time can be calculated from the intercept. The value of ψ_{ZC} is related to the depression angle such that:

$$\psi_{ZC} = 1 - \frac{2\theta}{\pi} \quad (6)$$

The main advantage with this method is that only a few points in the bulk arc are necessary to predict the relaxation times. This becomes important considering that sweeping up to extremely high frequencies might be necessary to obtain the entire bulk arc, which is not always feasible in sensing systems that needs to be field deployed. However, the use of this method necessitates the determination of R_b and R_∞ . R_b can be easily obtained from the intersection of the bulk and electrode arcs in the Nyquist plot. High frequencies to the range required to experimentally provide

the value of R_∞ was not adopted in this study. Hence R_∞ has been determined from equivalent circuit models of the Nyquist plots.

There are different ways in which the impedance plots could be modeled as equivalent circuits.^{1,30-33} The circuit modeling adopted in this paper is a simple version which retains the physical qualities of the system, intended to facilitate extraction of the value of R_b , and thus be able to compare between mixtures of different w/cm, composition, and ages. The bulk response of the plain cement paste is modeled as a resistor (R_1) in series with a combination of a resistor (R_2) and capacitor (C_1) in parallel. R_1 from the circuit model gives the value of R_∞ and R_1+R_2 gives the value of R_b . The measured and modeled values of R_b are very close to each other, which increases the confidence in using this circuit model for the prediction of the resistance values needed for τ_R determination. Since fly ash pastes exhibit a plateau in between the bulk and electrode arcs, similar to what has been reported earlier,³⁴ the plateau part was accounted for in the circuit model by another parallel combination of a resistor and a constant phase element (CPE). This combination was placed in series with the model for the bulk response of the plain paste to model the fly ash modified pastes.

Figures 12 (a) and (b) shows the relaxation times determined using Equation 5 for plain cement pastes as well as fly ash modified pastes with hydration time. The relaxation times are seen to decrease with increase in age of the specimens. The reduction can be expressed as a power law function. The higher relaxation times observed in pastes at early ages is due to the free water present during those times. This is similar to the trend reported for dry and wet composite mixtures.³⁵ As hydration progresses and the volume of free water and its thermodynamic state changes, the relaxation time decreases. Also, with time, the pore structure becomes more tortuous. Thus the relaxation times (and their distributions) could be viewed as representing the degree of heterogeneity of the material.¹⁷ From Figure 12(a), it could be seen that the relaxation times are higher at all times for the high w/cm paste. There is more free water early on in high w/cm systems, and more adsorbed or bound water at later times (as evidenced from non-evaporable water content results). For the fly ash modified paste in Figure 12(b), it is seen that the relaxation time is lower than that of the plain pastes after about 24 hours. While one would expect higher relaxation times for fly ash modified pastes at early ages, it is believed that this observation is due to the features of equivalent circuits (especially the CPE) used to model the Nyquist plots of fly ash modified pastes. More studies are necessary to understand this effect. From these figures, it could also be noticed that the multiplicative constant in the power law function for relaxation time as a function of hydration time might be close to the relaxation time of the specimen at very early ages, possibly at the start of setting. Experiments are being carried out to explore this aspect also. It can be observed From Figure 13 that the depression angle calculated from Equation 6 generally increases with increase in hydration time. This is an indication of the spread of the dielectric relaxation time distribution function, and hence increased material heterogeneity.

Dielectric Constant – Porosity – Depercolation Threshold Relationships

Percolation theory predicts the transition of a porous material from a conductor to an insulator.¹⁸ The critical porosity ϕ_c (or the concentration of conducting inclusions) is termed the percolation threshold. In cement based materials, the depercolation threshold of the pore phase is reached when the hydration products interfere with the ionic conduction paths so that they are no longer continuous. Percolation phenomena cause strong effects in conductivity and polarization in systems where the conductivities of the components are widely different. Hence it is anticipated that either the dielectric constant or conductivity should be able to detect the depercolation threshold of the pore phase. The conductivity increases dramatically beyond the depercolation threshold because the dominant mechanism changes to ionic conduction through connected paths rather than the surface conduction.

The capillary porosity of the cement pastes was computed from the measured degrees of hydration (from non-evaporable water contents) using the equations given in.³⁶ In Figure 14, the porosity calculated as above is plotted against the dielectric constant at two different frequencies - 10 kHz and 5 kHz - for plain and 10% fly ash modified pastes. It can be seen that the dielectric constant rapidly increases beyond a certain porosity. When the paste is young (higher porosity), the ionic pore solution is present in larger clusters, and the thickness of the low conductivity solid products is low. This is the reason for the large dielectric constant at high porosities, or at earlier times. For all the pastes investigated in this study, the critical value of porosity beyond which the dielectric constant increases rapidly is in the range of 0.19-0.22. The observed depercolation threshold agrees well with the depercolation threshold reported for cement pastes, which is about 0.18.^{37,38} For fly ash modified pastes, the depercolation threshold of the pore phase moves slightly towards lower values within this range.

CONCLUSIONS

This paper has dealt with the influence of material parameters on the dielectric response of cement pastes and concretes. The features that could be deduced from the dielectric response were also related to the property development of pastes. The salient conclusions from this study are stated in this section.

The setting times of the pastes could be obtained from characteristic features in the time derivative plot of effective conductivity. It was observed that the fly ash modified pastes exhibit lower conductivity than the plain pastes because of the reduction in the ionic content of the pore solution caused by the cement replacement. A mathematical model that relates the measured effective conductivity to hydration time was developed. This model gives reasonably good estimates of the setting times of the pastes. Studies on concrete mixtures showed that the trend in the conductivity-time plots were very similar to that of the pastes, except for reduction in the values, owing to the dilution with aggregates. The range analysis methodology helped to identify the significant parameters that have influence on the initial conductivity, final conductivity and setting times of concretes. While a/cm has a greater influence on initial conductivity, the combinations of a/cm and w/cm , and fly ash content and curing condition exert influence on final conductivity. The fly ash content and w/cm were found to be the major factors that dictate setting time. The dielectric constant spectra show extremely high values at low frequencies due to electrode or interfacial polarization. The high frequency plateau of dielectric constant is due to the relaxation of weakly bound water molecules. It is observed that there is a marginal increase in the dielectric constant values with fly ash replacement.

The second part of this paper has dealt with the dielectric relaxation times, its relationship with property development, and predicting the depercolation threshold of cement pastes from dielectric constant measurements. The relaxation times of the cement pastes at different times were determined using the frequency dependent impedance values and characteristic resistance as obtained from equivalent circuit models of the experimental data. The relaxation times decrease with increase in hydration time and conform to a power law. It has been shown that the relaxation times can represent the degree of heterogeneity of the material. The depercolation threshold of the pore phase for the cement pastes can be detected from the plot of dielectric constant as a function of porosity at different frequencies. The dielectric constant values increase drastically beyond the depercolation threshold, thereby making the dielectric response of cement pastes useful in depercolation threshold prediction.

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Table 1: Factor values for the fractional factorial experiments

Mix	w/cm	% Fly ash replacing cement	a/cm	Agg Size (mm) (in)	Curing condition
1	0.4	0	3	10 (0.4)	Saturated
2	0.5	20	3	10 (0.4)	Saturated
3	0.4	0	3	20 (0.8)	Saturated
4	0.5	20	3	20 (0.8)	Saturated
5	0.5	0	5	10 (0.4)	Saturated
6	0.5	0	5	20 (0.8)	Saturated
7	0.4	20	5	10 (0.4)	Saturated
8	0.4	20	5	20 (0.8)	Saturated
9	0.4	20	3	10 (0.4)	Sealed
10	0.4	20	3	20 (0.8)	Sealed
11	0.5	0	3	10 (0.4)	Sealed
12	0.5	0	3	20 (0.8)	Sealed
13	0.5	20	5	10 (0.4)	Sealed
14	0.5	20	5	20 (0.8)	Sealed
15	0.4	0	5	10 (0.4)	Sealed
16	0.4	0	5	20 (0.8)	Sealed

Table 2: Values of the parameters and constants in Equation 2 for modified cement pastes

w/cm	% Fly Ash Replacing Cement	Values from fit of equation 2			
		$(\sigma_0 - \sigma_f)$ (S/m)	σ_f (S/m)	b	t_s (min)
0.32	0	2.1112	0.0832	2.0798	312.87
	5	2.0585	0.0835	2.0292	342.94
	10	1.9600	0.0920	2.0083	351.05
	20	1.7908	0.1016	1.8603	357.56
0.42	0	2.0533	0.1236	1.8078	389.81
	5	1.9532	0.1124	1.6495	386.88
	10	1.9099	0.1251	1.7663	453.97
	20	1.8102	0.1299	1.5403	418.85

Table 3: Values of the parameters and constants in Equation 2 for concrete mixtures

Mix	Values from fit of Equation 2			
	$\sigma_0 - \sigma_f$ (S/m)	σ_f (S/m)	t_s (min)	b
1	0.5982	0.0478	310.75	2.0234
2	0.5955	0.0537	382.68	1.4372
3	0.6065	0.0427	365.15	1.8134
4	0.5474	0.0599	479.39	1.8151
5	0.3711	0.0352	413.49	2.0883
6	0.3132	0.0361	379.44	1.9062
7	0.3990	0.0415	418.50	1.6296
8	0.3641	0.0294	337.10	1.2378
9	0.4711	0.0265	403.94	1.7350
10	0.5640	0.0401	422.33	1.9516
11	0.6179	0.0247	365.55	1.7533
12	0.7201	0.0313	373.18	1.7162
13	0.4003	0.0247	484.84	1.3891
14	0.3817	0.0212	540.16	1.4961
15	0.4544	0.0225	364.94	2.0754
16	0.4266	0.0220	358.47	1.9074

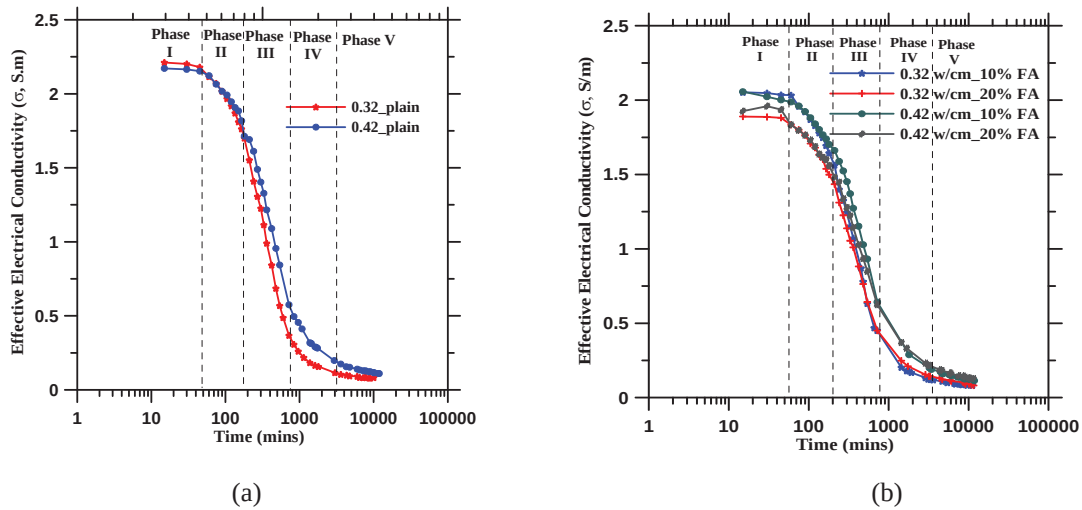


Figure 1--Effective conductivity variation with time for: (a) plain cement pastes, and (b) for pastes with fly ash replacements

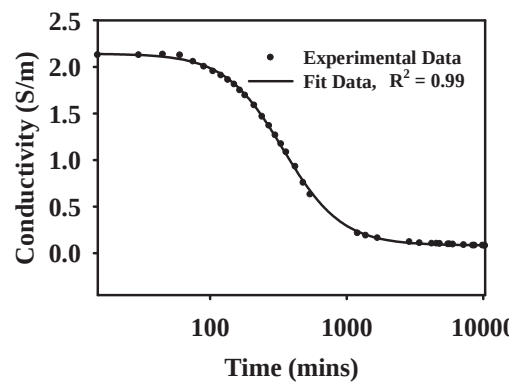


Figure 2-- Experimental data and model fit for conductivity-time relationship for 0.32 w/cm paste with 5% fly ash

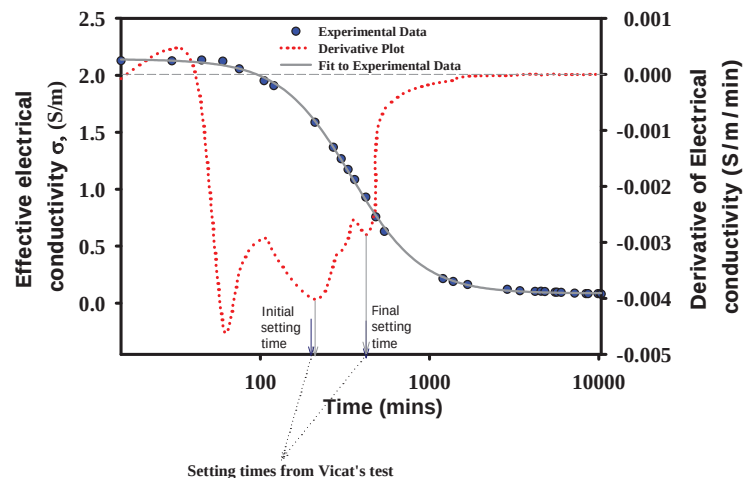


Figure 3--Effective conductivity and derivative of effective conductivity variation with time for plain paste.

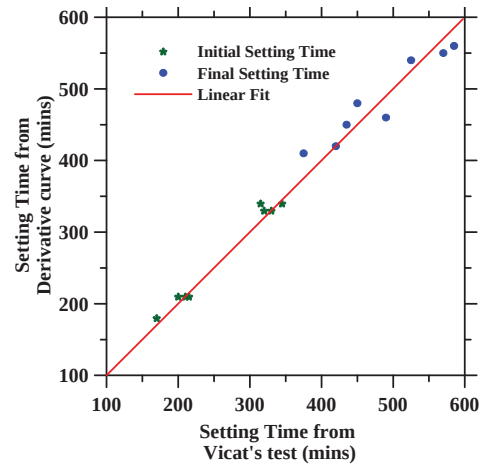


Figure 4--Comparison of setting times obtained from Vicat's test and derivative plot

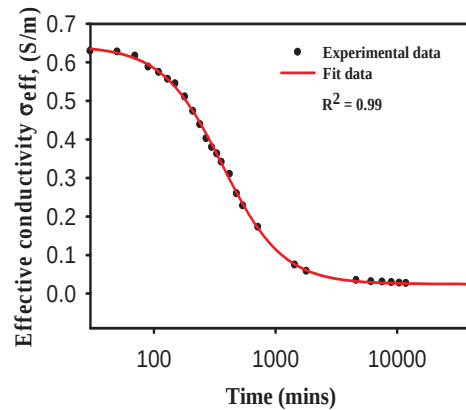


Figure 5--Experimental data and model fit for conductivity-time relationship for concrete mixture without fly ash and with w/cm of 0.5, a/cm of 3 and 20 mm (0.8 in) aggregate size.

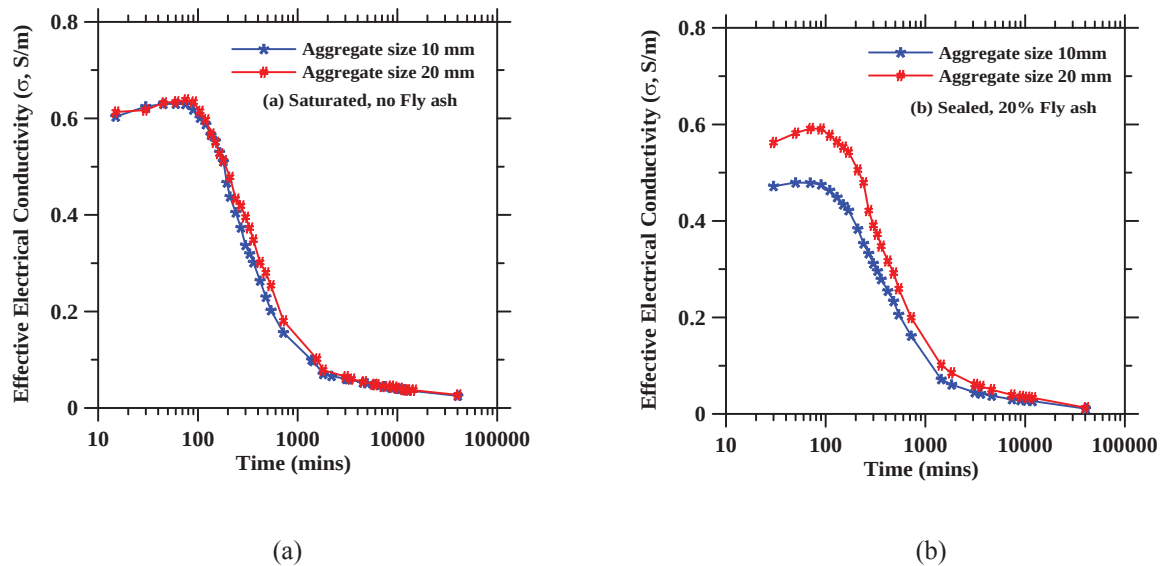


Figure 6--Effect of aggregate size on effective electrical conductivity for specimens made with w/cm of 0.4 and a/cm of 3 cured under: (a) saturated condition with no fly ash, and (b) sealed condition with 20 % fly ash.

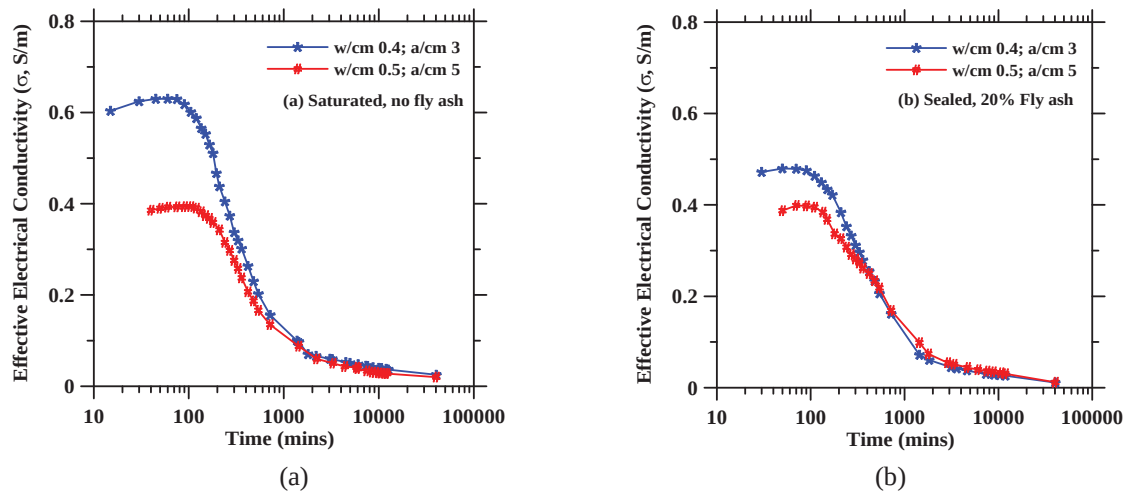


Figure 7--Effect of w/cm and a/cm on effective electrical conductivity of specimens made with 10 mm (0.4 in) aggregate cured under: (a) saturated condition and with no fly ash, and (b) sealed condition and with 20 % fly ash.

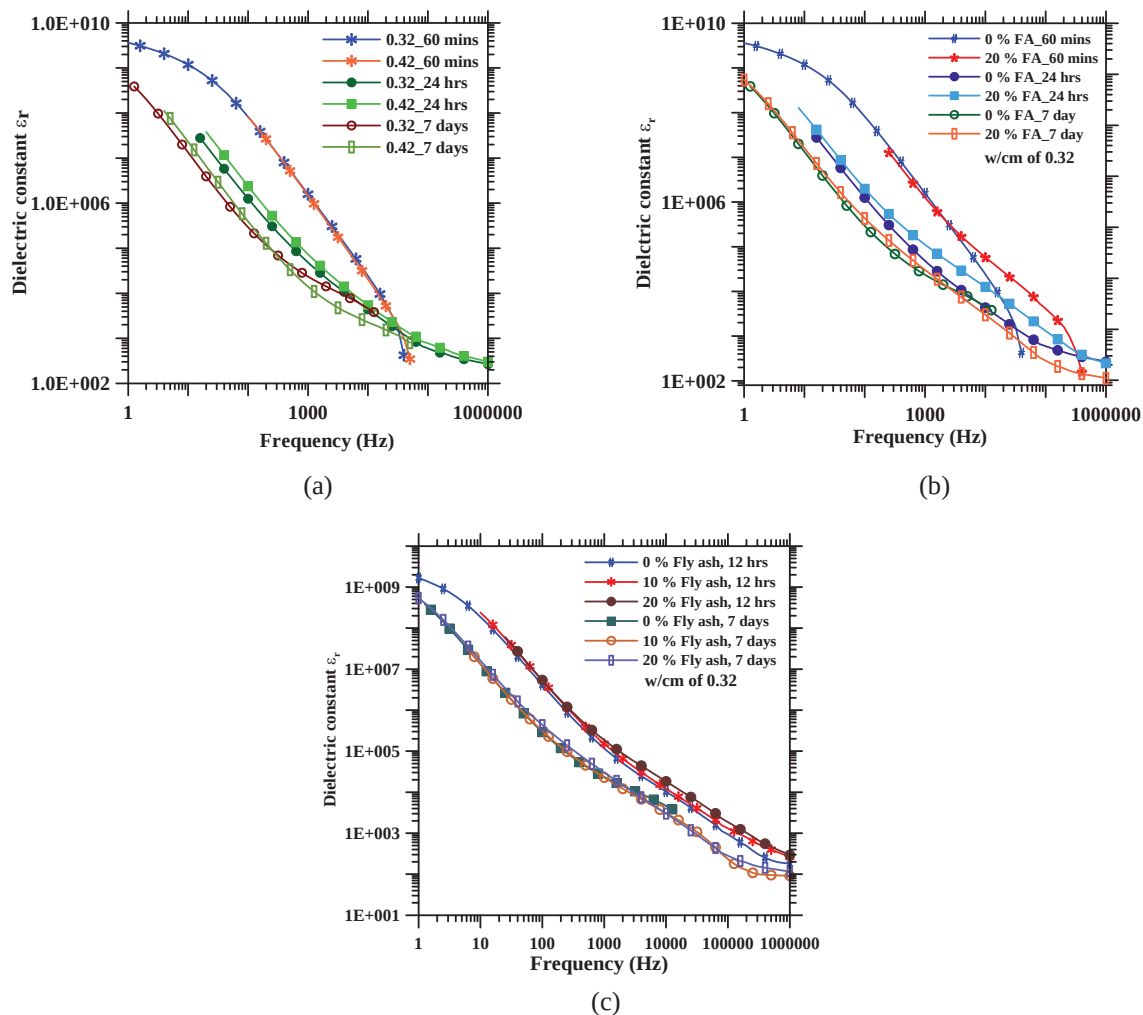


Figure 8--Dielectric dispersion with frequency at different times for cement pastes: (a) with w/cm of 0.32 and 0.42 with no fly ash, (b) with 0 and 20 % fly ash for w/cm of 0.32, and (c) with 0, 10 and 20 % fly ash.

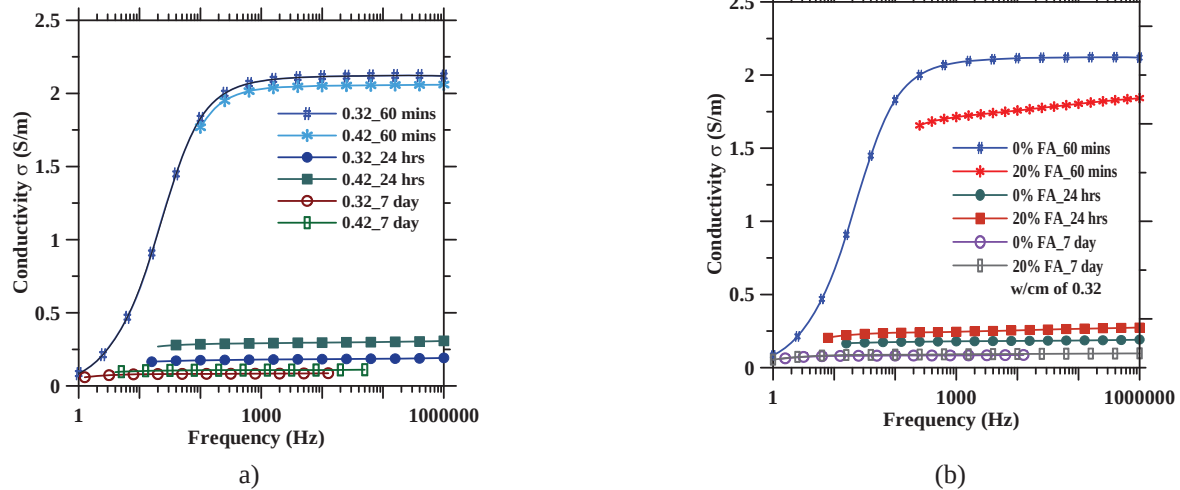


Figure 9--Conductivity dispersion with frequency at different times for cement pastes with: (a) w/cm of 0.32 and 0.42 with no fly ash, and (b) 0 and 20 % fly ash for w/cm of 0.32.

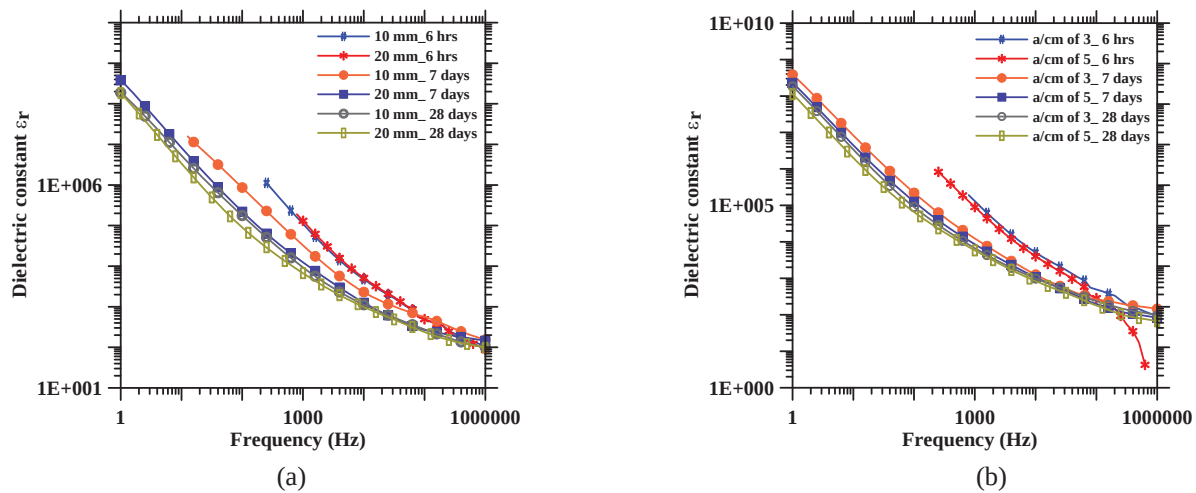


Figure 10--Dielectric dispersion with frequency at different times for concrete mixtures with: (a) aggregate size 10 mm (0.4 in) and 20 mm (0.8 in) , and (b) a/cm of 3 and 5.

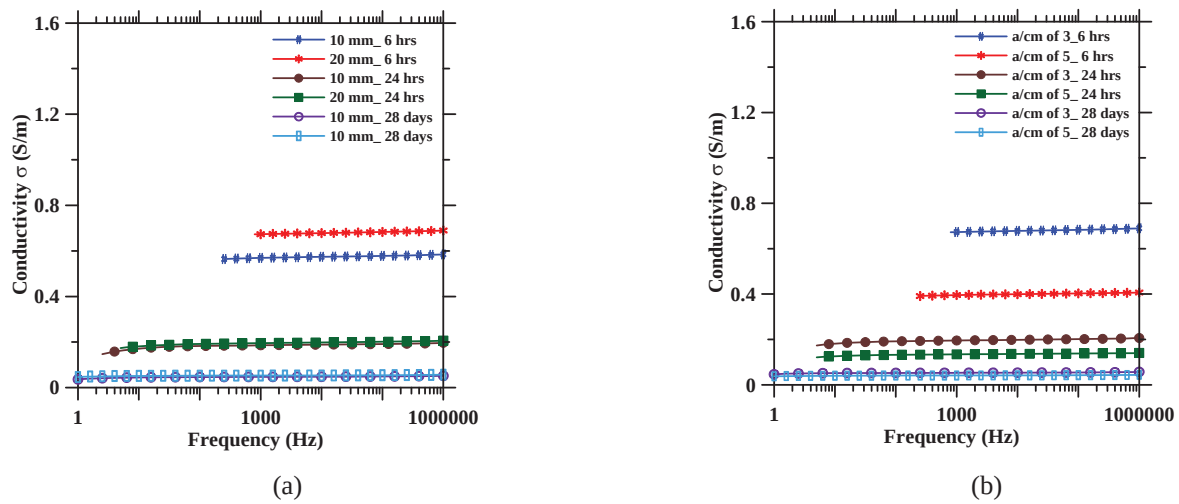


Figure 11--Conductivity dispersion with frequency at different times for concrete mixtures with: (a) aggregate size 10 mm (0.4 in) and 20 mm (0.8 in), and (b) a/c of 3 and 5.

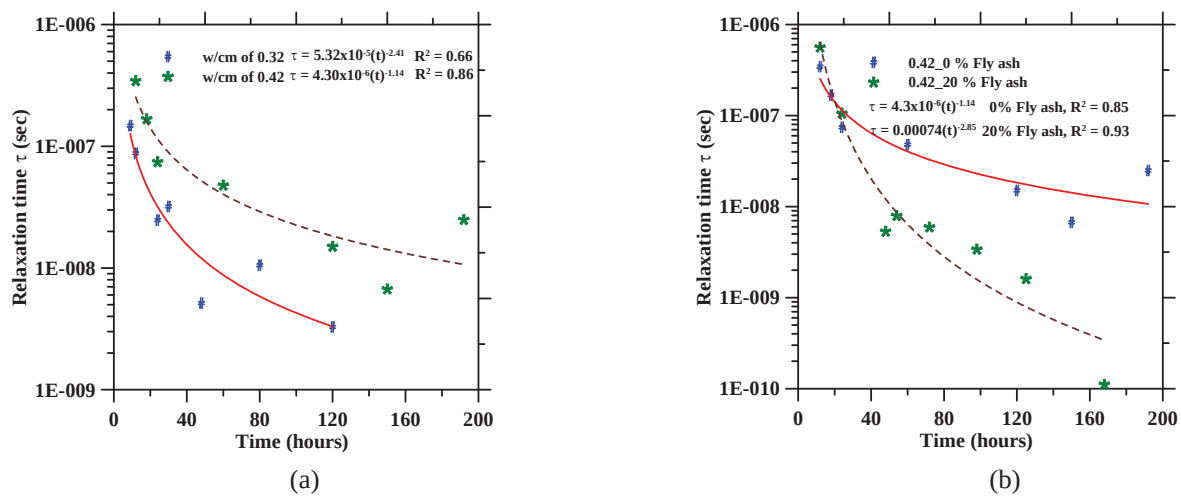


Figure 12--Variation of relaxation time with hydration time for: (a) plain pastes with w/cm of 0.32 and 0.42, and (b) 0.42 w/cm pastes with and without fly ash

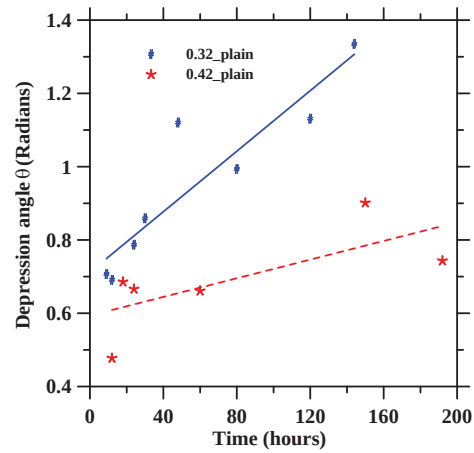


Figure 13--Variation of depression angle with time for plain cement pastes of w/cm of 0.32 and 0.42.

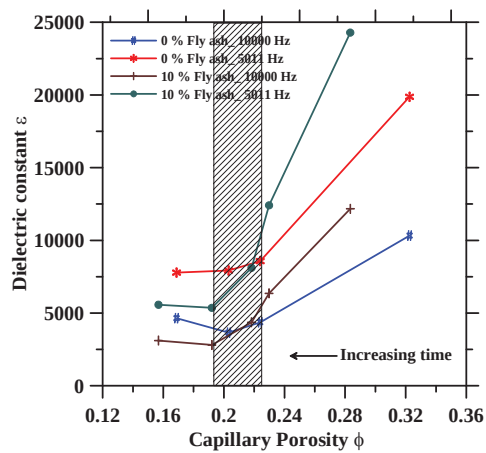


Figure 14--Variation of dielectric constant with capillary porosity for plain and modified cement pastes showing the percolation threshold

Embedded Sensors for Measuring Environmental Parameters in Concrete

by D.A. Lange, Z.C. Grasley, and R. Rodden

Synopsis: Environmental factors are major considerations of health monitoring of structures. A new system has been developed for measuring the internal relative humidity and temperature in concrete laboratory specimens and field structures. Internal relative humidity measurements in concrete are useful for evaluating shrinkage stress development and assessing structural susceptibility to durability problems. This paper describes the system, presents the essential elements of analysis that relates RH to internal stress in porous materials, and presents field applications that shows how environment data can facilitate analysis of stress and deformation of pavement. The new system has been used in laboratory and field experiments. Advantages of the new system are summarized. Some problems encountered in measuring relative humidity, especially in the field, are discussed.

Keywords: cracking; creep; embedded sensors; environment; instrumentation; relative humidity; shrinkage; slab curling; temperature

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INTRODUCTION

Temperature and moisture gradients can cause significant tensile strains and resultant stresses in concrete pavement, possibly leading to cracking without the occurrence of mechanical loading. Traditional formulation of environmental stresses in jointed plain concrete pavement (JPCP) assumes linear distribution of thermal and hygrothermal strains through the slab depth, usually underestimating the critical tensile stress. To fully define the relative humidity (RH) and temperature profiles of a concrete pavement, a field-ready measurement system capable of measuring RH and temperature through the depth of a concrete pavement by means of small, permanently embedded digital sensors was designed and constructed at the University of Illinois at Urbana-Champaign (UIUC).

Analysis of the relationship between RH and stress in a partially saturated porous microstructure (i.e. concrete under drying conditions) has been an ongoing area of work at UIUC. A material model based on water physics has been developed. The loss of moisture from the micropore structure creates a negative pressure in the concrete capillary pores which leads to local bulk concrete shrinkage. Thus, the model uses the RH parameter and temperature as parameters for computing hygrothermal stress.

The material model has proven immediately useful for modeling concrete pavement slabs. The material model can be integrated into pavement slab analysis that quantifies moisture curling when thermal and moisture gradients exist in the slab. In the field, daily cycling of thermal and hygrothermal stresses due to varying ambient temperature and moisture conditions occurs, the effect of which is much smaller at the base of the slab than the surface. If a slab were weightless and unrestrained, then the deflections due to temperature and RH could occur without generation of internal stresses. The self-weight of the slab, however, provides restraint causing any environmentally-induced strain to produce a corresponding stress. To complete the problem, it is necessary to account for creep and friction stresses at the slab/base interface. Creep relaxation is a unique component since it counteracts the net stress, regardless of whether the stress is tensile or compressive in origin. The cyclical ambient conditions of concrete pavements further complicate creep calculations since the stress or strain history is not well defined. The complexity behind implementing a creep model has caused most researchers to neglect creep for concrete pavements. However, in this research, simplifications have been made in order to include the effect of creep relaxation. In addition to the material and structural models, an investigation of an anomaly in early age above surface RH and temperature profiles and an investigation on micro- and macro-climates are presented.

The primary purpose of this research was to provide a concise solution for the development of linear and non-linear hygrothermal strains/stresses in concrete pavements based on a micromechanical model based on fundamental driving forces—RH and temperature. In addition, linear and nonlinear thermal strains/stresses were included with a creep relaxation and slab-base friction stress to build a complete analytical model that could potentially be used in design codes and/or programs. To accomplish this, a computer datalogging system capable of measuring RH and temperature in a concrete pavement was developed. This field-ready measurement system incorporates digital RH/temperature sensors that can be embedded in concrete at casting.

This paper presents elements of graduate theses [1,2], and for brevity, provides only an overview of analysis that is fully developed in the original documents.

NEW MEASUREMENT SYSTEM

Traditional RH measurement systems for concrete utilize an RH sensor probe which must be inserted into each individual sampling location. This can be time consuming and prone to error. Such systems are bulky, take a substantial amount of time for equilibration for each data point, and cannot be embedded in concrete at casting. For these reasons, traditional laboratory systems are not fit for field testing. A field-ready measurement system should consist of inexpensive, permanently embedded sensors that communicate to a small, battery powered, and reusable data acquisition system. A new system that satisfied all of these requirements was developed.

Sensor and packaging

The system uses a commercially available sensor module [3] to measure RH and temperature. The sensor has a range of 0-100% RH with an accuracy of $\pm 2\%$ from 0-90% RH and $\pm 3\%$ above 90% RH and below 10% RH. Hysteresis is reported to be $\pm 1\%$ while drift is less than 0.5% per year. Experiments at UIUC have successfully exceeded one year in length. The sensor packaging is an important part of achieving reliable and accurate results. The sensor differs from traditional capacitive RH sensors in that circuitry is included on the module and a digital signal is provided. This solves many thorny problems in dealing with long lead wires with capacitive sensors. The small size of the sensor allows it to be easily encased in a small tube (i.e. the plastic housing from a ball point pen). On one end of the tube a circle of Gore-Tex fabric is glued on to prevent water from entering the tube while allowing vapor to pass, thus allowing the RH and temperature in the cavity of the tube to equilibrate with the surrounding concrete. Once the wired sensor is placed in the tube, the lead wire is wrapped with rubber electrical tape and then sealed with silicone glue to prevent liquid moisture from entering the lead wire end. This packaging procedure is illustrated in Figure 1. To mount the packaged sensors in a pavement, PVC pipe is assembled as a “tree bracket” with sensors at heights of interest. The packaging procedure for sensors placed at the surface (troweled into the surface during finishing) is to simply encase the sensor completely with Gore-Tex fabric and glue all seams. Sensors below but near the surface may be oriented as shown in Figure 2. Sensors were also used to capture ambient environmental data, and these sensors were also packaged using the same tube approach described above for embedded sensors.

Data acquisition for field applications

The field-ready data acquisition equipment is shown in Figure 3. The RH and temperature measurement and logging system designed at the UIUC is based on a previous generation that utilized a personal computer [4]. As with the previous system, the new system utilizes a microprocessor to execute a program to gather the RH and temperature data.

The two major developments over the previous system are the integration of an external nonvolatile electronically erasable programmable read-only-memory (EEPROM), and a portable power supply. Detailed information on construction of the field-ready measurement system is found elsewhere [5]. The system interfaces with a PC for initial setup and download of data.

The field-ready system has numerous advantages over probe type RH sensors, including:

- Relatively low cost
- Small sensor allows placement in small cavities, decreasing equilibration time
- Size and cost-effectiveness allow embedment/measurement at casting for a complete RH/temperature history
- Digital signal solves long leadwire capacitance issues
- Both RH and temperature measured on same sensor, making temperature corrections more accurate; Sensors come factory calibrated, and exhibit low drift
- Easy connection to laptop computer via a serial port or USB to serial adapter
- Small system size and battery operated, allowing placement of system near construction area

LABORATORY EXPERIMENTS ON DRYING STRESS GRADIENTS

The RH and temperature system has made possible a wide variety of laboratory experiments. Drying behavior of concrete is of special interest because it impacts field performance of concrete slabs. Drying shrinkage in concrete can be induced by external drying or by internal drying. Internal drying occurs when hydration consumes the capillary pore water. Shrinkage induced by internal drying is referred to as autogenous shrinkage. As concrete dries, the larger pores empty first, followed by successively smaller pores. Once the pores have emptied to a diameter of about 50 nm [6], the surface tension of water causes capillary menisci to develop. As a result of this “tension” in the pore fluid, a balancing compressive stress develops in the solid microstructure, inducing microscale shrinkage. Thus, RH is the primary driving force behind drying shrinkage, and the mechanism of shrinkage is compressive stress on the solid microstructure of hydrated cement paste.

Laboratory experiments were conducted to investigate drying gradients. The concrete mixture used for laboratory experiments and field trials is presented in Table 1. Internal RH sensors were embedded in concrete prisms. The concrete form produced a test section with dimensions 76 x 76 x 330 mm (3” x 3” x 13”). The tubes were inserted to various depths such that measurements were being taken at depths of 6.35 mm (0.25”), 12.7 mm (0.5”), 19.0 mm (0.75”), 25.4 mm (1”), and 37.1 mm (1.5”) from the drying surfaces. The prisms were cast and immediately the surface was sealed using plastic wrap. At an age of 23 hours, the specimens were demolded. The top and bottom surfaces were sealed to prevent drying using an adhesive-backed aluminum tape. The two symmetric sides were allowed to dry. Each test lasted a total of 8 days (7 days drying time), and the data was acquired every 3 minutes. Throughout the duration of the test, the specimen was kept in a climate-controlled chamber maintained at 23° C and 50% RH. The uniaxial test used for this research was developed at UIUC by Altoubat [7]. An in-depth description of the uniaxial restrained shrinkage test procedure can be found elsewhere [8].

A detailed analytical model [9] was developed to use the RH data to predict the stress distribution in the prism. The model incorporates creep due to drying stresses as well as the applied tensile load when under restrained conditions. The tensile strength of the concrete is known, and there is an assumption that the surface material will crack when stresses exceed the tensile strength.

The experiments were performed under free shrinkage conditions and fully restrained conditions. Figure 4 shows the free shrinkage drying stress evolution for one of the concrete mixtures. Drying initiated at an age of 1 day. The figure indicates that at three days there was a zone of microcracking that extended into the material about 3-5 mm. At 5 days, the depth of this zone had been reduced to about 1-3 mm. At an age of 7 days, the microcracking zone was almost nonexistent. As time passes, there is a reduction in the stress gradient as the material approaches uniform moisture equilibrium, and the material also gains strength. Unlike the free shrinkage case where the tensile stresses on the surface relax over time, when concrete is fully restrained the depth of the microcracking zone increases with time as the average stress across the cross-section increases. Figure 5 shows the evolution of stresses in a fully restrained drying concrete specimen. The zone of microcracking progresses from 4-5 mm deep at 3 days of age to about 5-6 mm deep at 7 days.

Some findings of laboratory study of gradients

The laboratory studies provided insight into the behavior of drying concrete:

- In drying concrete, the tensile stresses in the surface layer routinely exceed the tensile strength of the material.
- In free shrinkage specimens, the tensile stresses are highest soon after exposure to drying, but gradually decrease as the material approaches equilibrium with the ambient RH.
- When concrete shrinkage specimens are fully restrained, the zone of microcracking continues to increase as the average stress across the cross-section increases.
- The severity of the drying gradient has an effect on the time to cracking in fully restrained concrete.
- Concrete with lower w/cm exhibit a higher stress gradient slope near the surface layer, which may increase the width of surface cracks.

FIELD IMPLEMENTATION

The RH and temperature system was used to monitor concrete slabs cast in November 2005. The purpose of the test program was to understand how changing ambient environment impacts RH and temperature inside concrete slabs. A second purpose was to understand how widely available weather data can be used to infer conditions inside concrete slabs.

The RH/temperature sensors were embedded at various depths in three different concrete slabs and companion cubes. The slabs were subjected to cyclical ambient conditions in the field and the cubes were kept in a climate-controlled chamber, maintained at 23°C and 50% RH. One slab and cube set contained plain PCC with no curing procedure (STD), one slab and cube set contained plain PCC but was properly cured with a polyethylene sheet cover (CUR), and one slab and cube set contained a shrinkage reducing admixture at a dosage of 1.5gal/yd³ (7.5 L/m³) with no curing procedure (SRA). The three slabs measured approximately 4' x 4' with a thickness of 15" and were cast with RH/temperature sensors placed in the center of the slab. Each slab contained two RH/temperature sensors placed at the surface, ½", 1", 11", and 14" and one RH/temperature sensor placed at 3", 5", and 7", for a complete profile of the pavement.

Figures 6 and 7 show how temperature and RH vary as a function of depth in the concrete slab over the first month after construction. The data illustrates how concrete near the surface is more readily affected by surface drying, and RH remains higher in the lower sections of the concrete. The RH was observed to return to near saturation on several occasions, and these were associated with rain events that applied water to the surface of the concrete. The sensors near the surface were highly affected by the rain events while the lower sensors were almost unaffected by the rain events.

Analysis of field results

Once a concrete slab has begun to set, a battle between the concrete strength gain and internal stress development commences. The stress profile through a concrete slab at any time is a combination of the thermal stresses, hygrothermal stresses, creep relaxation, and friction stresses. If the tensile stress (thermal, hygrothermal less the creep relaxation, and friction) in a concrete slab is equal to the tensile strength of the concrete, a crack will initiate. It should be noted that the initiation of cracking does not mean that the slab has failed as, when cracking is initiated, the stress conditions change and a crack might be arrested depending on the loading condition and fracture properties of the concrete. The two obvious points of interest are the top and bottom surface because cracking is expected to initiate at either of these locations. In this study, the bottom of the slab was found to be remain sufficiently wet (RH > 97%) so the primary interest is the top surface.

A detailed micromechanical model was developed to quantify the total linear and nonlinear hygrothermal stresses in jointed plain concrete pavement. When this model was combined with thermal stresses (linear and nonlinear) with consideration of a simple creep function and slab/subbase friction, a complete analytical model capable of calculating total environmental stress at any given point in the pavement life, given just RH and temperature data for the pavement cross-section, was developed [1, 10].

Figure 8 shows how stress in the STD slab is predicted to vary as a function of time. The tensile strength curve for the concrete is shown to illustrate the risk of cracking. The slab was externally unrestrained except by friction, and the computed stresses remain much lower than the tensile strength of the concrete. Indeed, no slab cracks were observed in the field tests. The daily temperature variation affects both internal temperature and internal RH.

Figure 9 shows how stress in the STD slab might be expected to vary as function of time if the slab were horizontally restrained, such as might be assumed if the slab was very large, continuous, and without joints. The analysis helps illustrate the role of restraint in generating stresses that lead to slab cracking. The analysis predicts cracking to occur at about four days.

Findings from field experiments

By measuring the internal RH and temperature through the depth concrete slabs in the field, representative of a full-depth concrete runway pavement, conclusions can be made about the nature of laboratory measurements versus field measurements with regard to concrete pavements.

- Cyclical thermal strain measurements, as obtained in the field, are crucial in predicting early age cracking of concrete pavements.
- The RH at locations other than the surface were higher in field specimens than in laboratory specimens due to the availability of moisture in the field.
- Properly curing a concrete pavement offsets the development of hygrothermal strains/stresses which can prevent early age cracking because the concrete strength and fracture toughness develops before high stresses occur.
- Thermal strains, both in the laboratory and in the field, are largely independent of curing procedure and/or the inclusion of a shrinkage reducing admixture.
- Our micromechanical model predicts that no cracking occurs in a 15" concrete pavement with joint spacing of 20', but cracking occurs in a fully restrained scenario.
- Consideration of both hygrothermal and thermal strains—as well as creep relaxation and slab/subbase friction stress—is important in predicting early age cracking.
- Conservative, yet accurate estimates of the maximum hygrothermal strain at the surface of a slab can be achieved using thinner and smaller laboratory specimens.
- Polyethylene sheeting and wet burlap are effective in delaying the onset of hygrothermal strains. Rewetting a surface temporarily suppresses development of high hygrothermal strains, thus aiding in early age crack mitigation.

CONCLUSIONS

A new system has been developed for measuring the internal relative humidity and temperature in concrete laboratory specimens and field structures. The system has been used in laboratory experiments which make possible the careful control of ambient temperature and relative humidity. The system has also been used to monitor concrete slabs in the field, making it possible to learn about the impact of daily temperature and humidity cycles, and the impact of rain events.

New analytical approaches developed at UIUC allow the use of internal RH to directly predict the state of stress in concrete materials. These analytical advances open the door to greater insight into how environment affects stress, deformation, and cracking of concrete pavements and structural components.

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Table 1. Concrete mixture design

Description	Value
w/c ratio	0.38
Cement (lb/yd ³)	588
Fly Ash (lb/yd ³)	100
Coarse aggregate (lb/yd ³)	1850
Fine Aggregate (lb/yd ³)	1103
Water (lb/yd ³)	262
Air Entraining Agent (oz/yd ³)	14

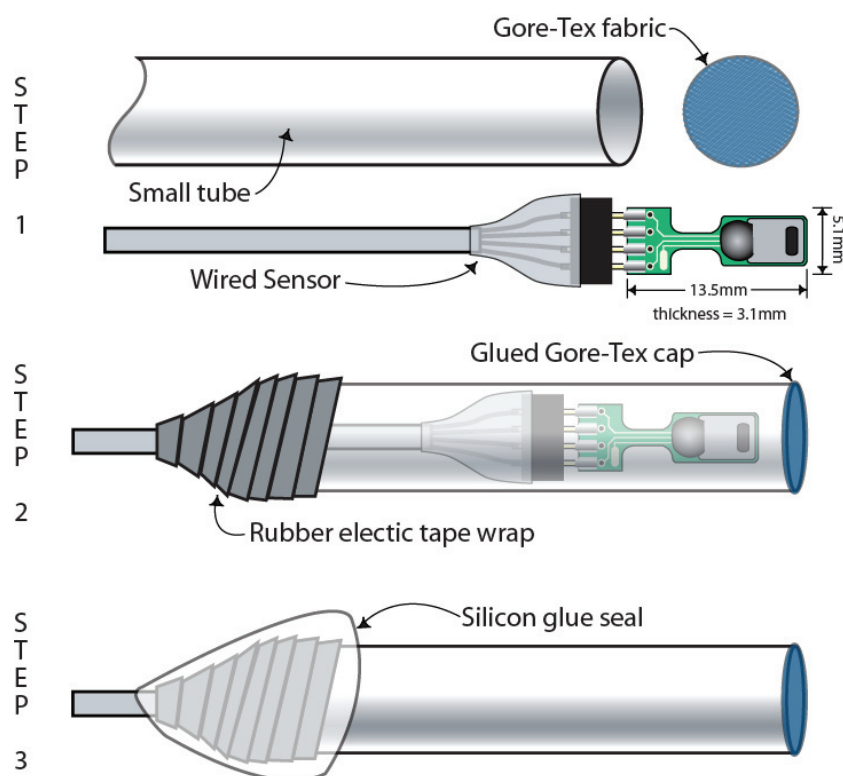


Figure 1. RH/temperature sensor packing procedure

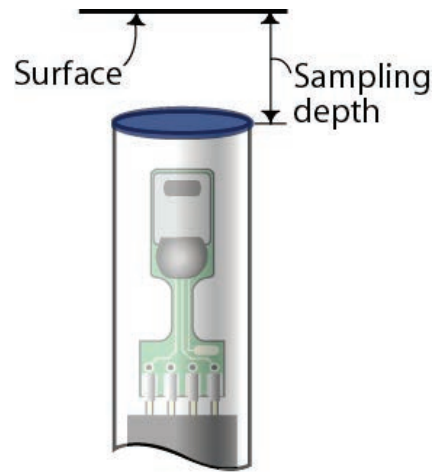


Figure 2. Method for near surface sampling



Figure 3. Field-ready system mounted in all-weather case

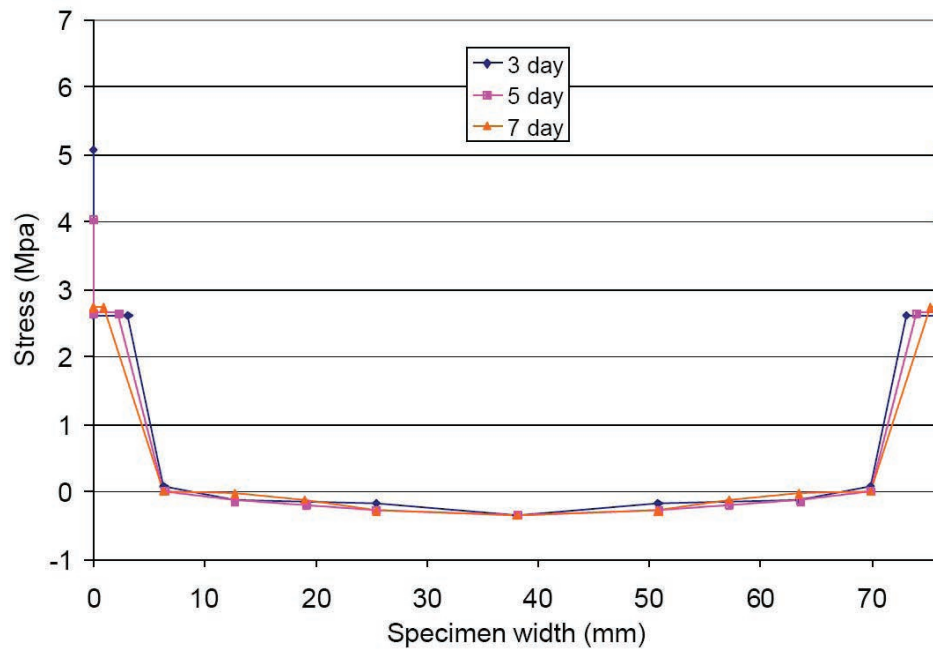


Figure 4. Evolution of stresses in free shrinkage concrete

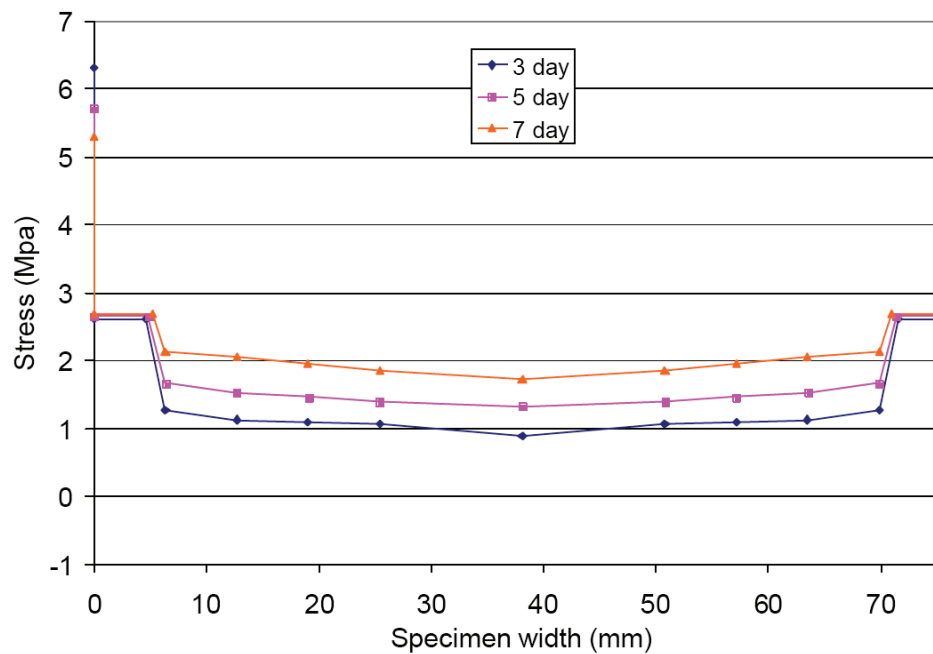


Figure 5. Evolution of stresses in fully restrained concrete

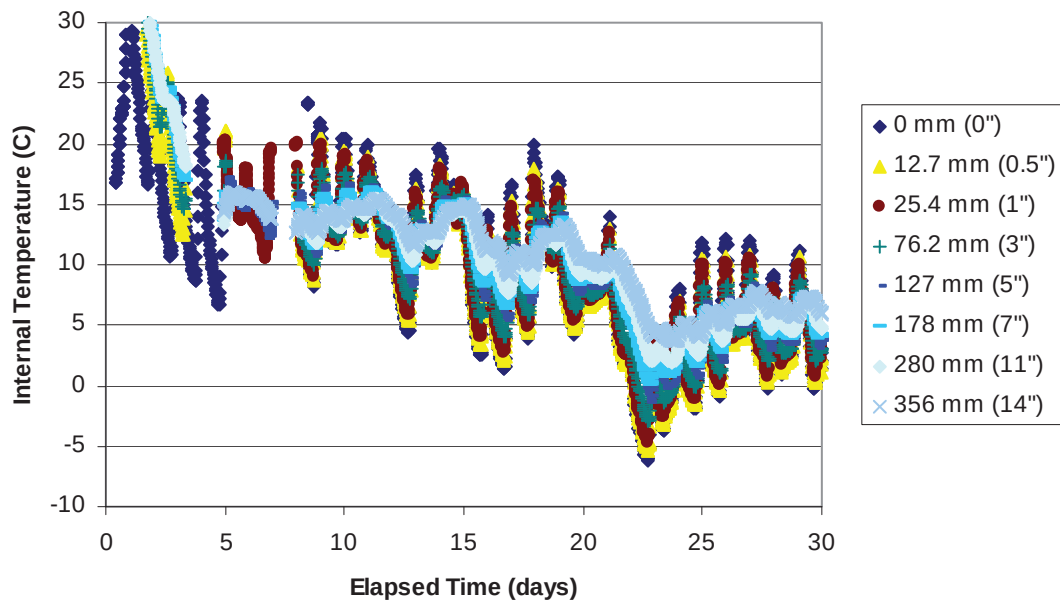


Figure 6. Internal temperature data for the STD slab

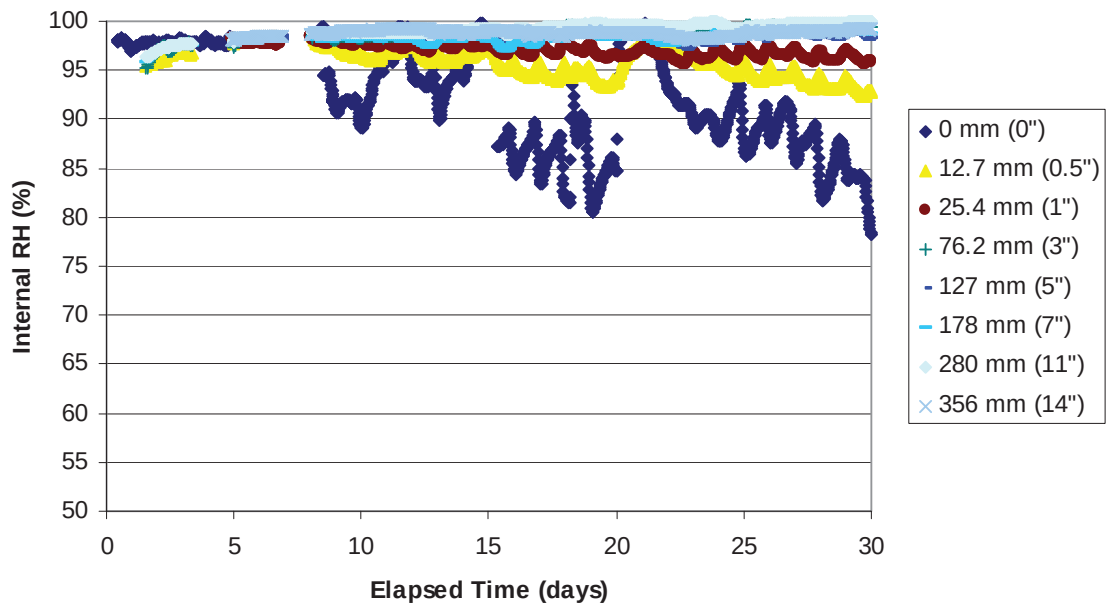


Figure 7. Internal RH data for the STD slab

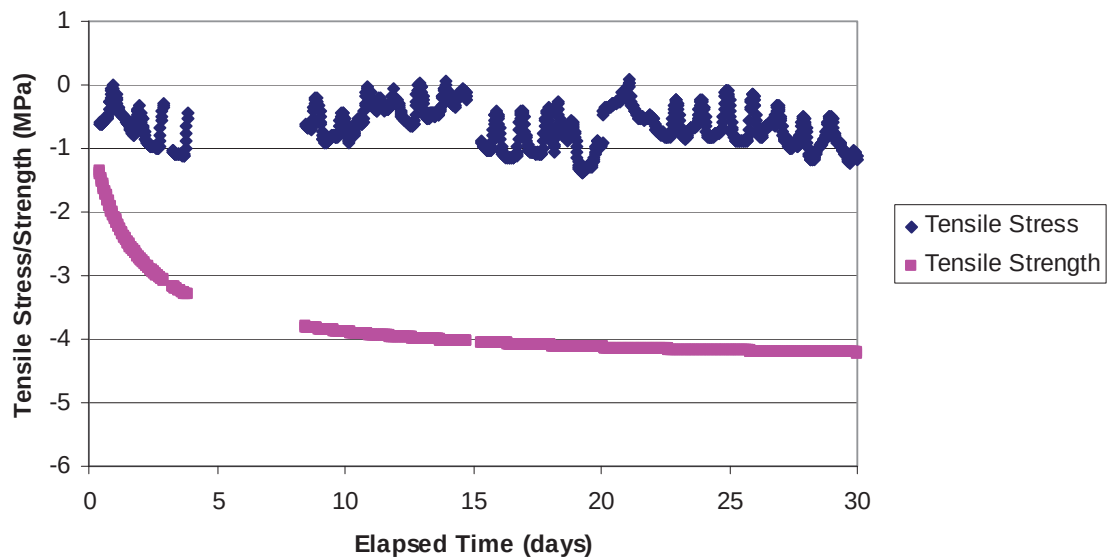


Figure 8. Tensile stress and strength for the STD slab

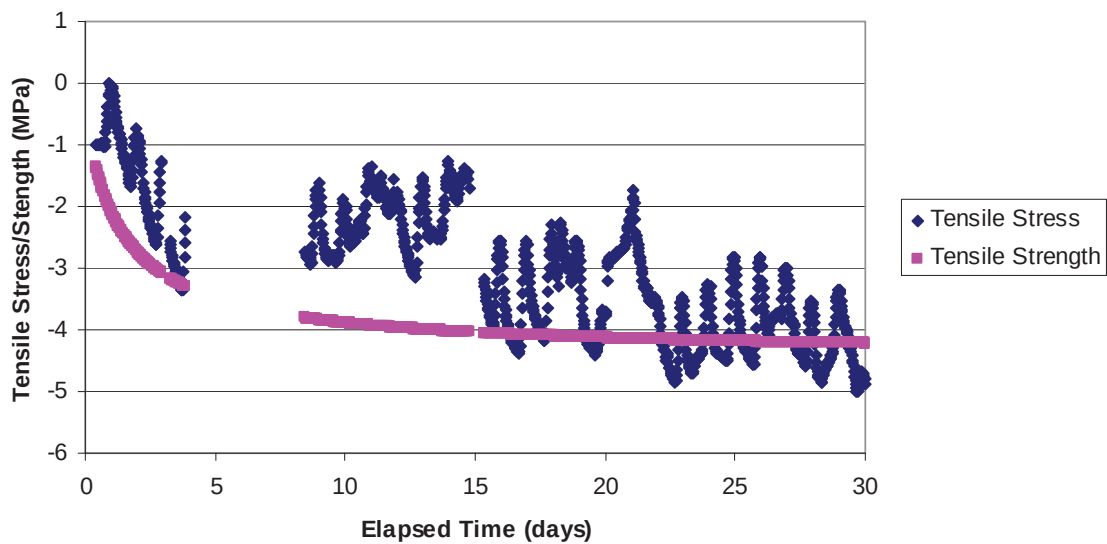


Figure 9. Tensile stress and strength for the STD slab if fully restrained