The Effect of Cement Type on the Potential and Corrosion Behaviour of Steel Reinforcement

By

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ABSTRACT

The corrosion of reinforcing steel has emerged as the most prevalent factor causing deterioration of reinforced concrete structures in aggressive exposures, such as the Middle East and Arabian Gulf States. Recently, the effect of using embedded steel bars passing through different types of concrete has been found to exacerbate the corrosion activity. This may occur with a steel bar passing from a foundation manufactured with sulphate resistance Portland cement (SRPC) into a beam or column manufactured with ordinary Portland cement (OPC).

This thesis aims at elucidating the behaviour of reinforcing steel (RC) embedded in concrete mixes of different types by incorporating both OPC and SRPC and blends with ground granulated blast furnace slag and pulverised fuel ash. Two different types of RC specimens were used: Single bars in a single mix and electrically connected bars in same or different mix types sharing a common electrolyte in the absence and presence of a specific interface. The RC specimens were exposed to wet-dry cycles of chloride and sulfate solutions.

The results of steel bars that were electrically connected, and were embedded in concrete cubes manufactured with different cement types and without a direct interface exhibited reduced times to onset of corrosion when subjected to chloride ion ingress. In the worst case, the connection of the a bar in OPC (0.45) concrete and one in SRPC (0.45) concrete caused the bar in the SRPC (0.45) mix to start corroding after only 3 cycles (6 weeks) compared to 15 cycles (30 weeks) for a single unconnected bar in the same SRPC (0.45) concrete. Moreover, measurements of the half-cell potential, polarisation resistance and corrosion current, all confirm that a single bar passing the interface between two different concretes manufactured with different cement types can suffer the formation of anode and cathode regions that depend on the local environment. For example, a steel bar passing from an OPC (0.45) concrete to a SRPC (0.45) concrete will preferentially form an active anode within the SRPC concrete. This has implications for concrete structures manufactured using SRPC foundations and OPC superstructure, as often used in the Arabian Gulf States.
DEDICATION

This humble work is dedicated to my lovely relatives:

My father, Mr. Tajeddin, and my mother, Mrs. Bahitha,

my wife, Mrs. Naila, and my sons,

Abdul-Rahman, Aasem, Abdul-Qaddoos,

Salahedin, Isa and Taj,

my brothers, Dr. Dhiya, Mr. Baraa, and Mr. Majd

for their help, patience, prayers and encouragement

throughout my study.
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Praise be to ALLAH Almighty who gave me the power, persistence and patience to accomplish and write this document. Prayers and peace be upon his last messenger who guided us to the right path.

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LIST OF ABBREVIATIONS

ACI The American Concrete Institute
ASTM The American Society for Testing and Materials
C Calcium oxide (CaO)
CaCO₃ Calcium carbonate
CH Portlandite or hydrated lime [calcium hydroxide Ca(OH)₂]
CO₂ Carbon dioxide
Curing Fog room curing, maintained at a constant temperature of 20°C ± 2°C and normally 99% RH
C-A-H Calcium aluminate hydrate
C-S-H Calcium silicate hydrate
C₂S Dicalcium silicate (2CaO·SiO₂)
C₃S Tricalcium silicate (3CaO·SiO₂)
C₃A Tricalcium aluminate (3CaO·Al₂O₃)
C₄AF Tetracalcium aluminoferrite (4CaO·Al₂O₃·F₂O₃)
GGBS Ground granulated blast furnace slag
M Mole
OPC Ordinary Portland cement
OPC (0.45) 100% OPC concrete mix with 0.45 w/c
OPC (0.6) 100% OPC concrete mix with 0.60 w/c
OPC/GGBS (0.4) Concrete mix containing 70% GGBS as an OPC replacement with a w/b of 0.40
OPC/PFA (0.4) Concrete mix containing 30% PFA as an OPC replacement with a w/b of 0.40
S Silicon dioxide (SiO₂)
SRPC (0.45) 100% SRPC mix with 0.45 w/c
S10 Sikament 10 superplasticiser.
<table>
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<tr>
<th></th>
<th>Description</th>
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<tr>
<td>w/c</td>
<td>Free water to cement ratio</td>
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<tr>
<td>w/b</td>
<td>Free water to cementitious materials (binder) ratio</td>
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CHAPTER 1. INTRODUCTION

It is often used to be assumed by designers, contractors and owners of reinforced concrete structures that reinforced concrete is a maintenance free composite. This assumption has been proven to be false due to the increasing number of deterioration case studies reported in the literature in recent years. It is becoming increasingly apparent that the reinforced concrete infrastructures of United States, Canada, United Kingdom, Arabian Gulf States and many other regions of the world are suffering large-scale degradation due to rebar corrosion and the economic implications of this problem are alarming (Tullmin et al 1996). National growth and economical prosperity are closely related to the adequacy of the transportation infrastructure. Bridges, in particular, are considered one of the critical components of the transportation infrastructure, since severe corrosion will eventually weaken structural members to the point where replacement of reinforcement is often required in order to sustain design loads (NASA 1994).

The National Research Board reports that, in the United States, there are approximately 590,000 structures in National Bridge Inventory database (Small 1998). About 33% of these bridges are either structurally deficient or functionally obsolete, or both (Alkhrdaji et al 1999). Yet, many of those bridges suffering corrosion damage could be saved with minimum repair and the use of permanent corrosion control techniques (Krause 1995). According to recent surveys, $80 to 150 billions are the estimated cost of bringing those bridges to an acceptable level of repair (Chaix et al 1995 and Report Card 1998). Inattention to corrosion control as part of an overall maintenance program for infrastructure facilities has been reported to cost the USA more than $ 250 billion (Nadel 1994).

Similar problems have been observed in reinforced concrete bridges and structures in the UK. Generally, the corrosion of reinforcing steel in concrete structures such as motorways bridges, buildings and marine installations was estimated in 1999 to be costing the UK £ 550 million per annum (Trend2000 1999).

There are two main causes of reinforcement corrosion, both of which involve the loss of the passivating layer of iron oxide around the reinforcement. The first is known as
carbonation which occurs when carbon dioxide from the atmosphere combines with moisture in the pore structure of the concrete to produce a mild carbonic acid. This acid reduces the general pH level of the concrete. If the level of alkalinity around the reinforcing steel falls below a level of approximately 10.5, the passive film cannot be maintained. As a result, the steel is left in an environment where general corrosion can occur. The subsequent rate of corrosion will be controlled mostly by the local supply of $O_2$ and $H_2O$. The second cause of reinforcement corrosion is the presence of chloride ions. The chloride ions may be environmentally sourced or have been mixed in the constituents of the fresh concrete. Chloride ions, being known as the specific and unique destroyer of reinforcing steel in concrete, are particularly damaging since they promote corrosion of the steel after penetrating the concrete by disrupting the passive oxide film and encouraging iron ions to leave the metal lattice and go into solution as iron ions that will easily promote dissolution and pitting of the steel (Rasheeduzzafar 1992).

The ongoing problem caused by chlorides ions in reinforcement has led the construction industry to the increased usage use of cement replacement materials such as silica fume, ground granulated blast furnace slag (GGBS) and pulverised fuel ash (PFA). These are reported both to reduce the mobility of chloride ions and the permeability of concretes provided that they are adequately cured. However, the use of different cement types and cement blends within a single structure gives rise to other risks.

This thesis aims to elucidate the corrosion behaviour of steel reinforcement which is, either through design or as a consequence of repair, embedded in concrete mixes of different composition, such as may occur with a steel bar passing from a foundation manufactured with Sulphate-Resisting Portland Cement (SRPC) into a beam or column manufactured with Ordinary Portland Cement (OPC). It is a common practice in the Arabian Gulf States to use SRPC for almost all types of concrete structure below the ground level (Novokshchenov 1995 and Rasheeduzzafar 1992).

Theory, as embodied in the Nernst equation, predicts that under such conditions the electrode potential of the steel will be different in the two materials and so localised corrosion may occur. With this problem in mind, this study proposes answers for the following questions:
(i) Is this effect real? That is to say does a steel bar in a mixed cement environment suffer the creation of stable anode and cathode by the local composition of the concrete?

(ii) How big is this effect compared to the other factors?

(iii) What are the practical implications of this effect for real structures?

This thesis examines the corrosion behaviour of reinforcing steel bars by measuring the electrode potentials of steel bars embedded in mortars or concrete incorporating both OPC and SRPC and blends with PFA and GGBS for both single bars in a single mix and electrically connected bars in different concrete mixes sharing a common electrolyte in the absence and presence of a specific interface. The implication of these results for the durability of real structures is addressed for reinforced steel embedded in more than one concrete mix type.
CHAPTER 2. LITERATURE REVIEW

This literature review describes the main types of cement used in normal concrete, their compositions and their properties. Thereafter, the use of blended cements with replacement materials for Ordinary Portland cement (OPC), are describe focusing on two types: Ground Granulated Blast Furnace Slag (GGBS) and Pulverised Fuel Ash (PFA), their production, chemistry and their properties. The main factors affecting the durability of concrete are then outlined followed by the electrochemical nature of the corrosion process in relation to the Pourbaix Diagrams.

The durability of reinforced concrete, the causes of corrosion and some of the recommended solutions are reviewed. Finally, the implications of the Nernst equation on the effect of the local environment on reinforcement corrosion are considered and used to predict the effect of the cement type on the corrosion potentials of the reinforcing bars.

2.1. PORTLAND CEMENTS AND CONCRETE

Concrete is a very versatile material, which can be freely moulded and its constituents can be readily obtained (Rasheeduzzafar 1992). Concrete itself is a composite material comprising of coarse aggregate particles dispersed in a cement matrix, which acts as a binder for the sand and gravel aggregate. It is the properties and behaviour of the cement paste that is responsible for many of the good and bad qualities of concrete. For a given aggregate, the physical and chemical characteristics of the cement paste determine the most important property of fresh concrete—workability and also the most significance engineering characteristics of hardened concrete—strength and dimensional stability (Neville 1995).

2.1.1 PORTLAND CEMENT

By far the biggest of tonnage of cements produced worldwide are OPC; about 90% of all the cement used in the United States and United Kingdom is Ordinary Portland cement (Neville 1995). For 1998, figures in the UK Ready Mixed Concrete Production are of the order 23,000,000 m$^3$, which at a cost £50/ m$^3$, is equivalent to £1.15 billion /annum (Housing and Construction Statistics 1998). However a range of
cement types exist* that have been developed for different applications. These include:

a) Low Heat Portland cement (LHPC),

b) Rapid Hardening Portland cement (RHPC), and

c) Sulphate Resisting Portland cement (SRPC).

Other types of cement that are not based on an OPC precursor include Super-Sulphated Portland cement (SSPC) and High Alumina cement (HAC).

*(Cement specifications for cement are covered by EN 197:2000)

It is important to note that although one cement may provide superior durability performance compared to another in one environment; it may well be inferior if exposed to a different environment (Neville 1995). For example, OPC with a high C\textsubscript{3}A content will have a greater resistance to chloride induced reinforcement corrosion than SRPC, while OPC will have less resistance to sulphate environments than SRPC (Rasheeduzzafar 1992).

2.1.1.1 ORDINARY PORTLAND CEMENT

Ordinary Portland Cement complies with BS 12 1996 that was withdrawn and replaced by BS EN 197-1:2000. It is called CEM I in BS EN 197 and is called Type I Portland cement in the USA (ASTM C 150 1999).

Portland cement is a general-purpose cement suitable for all uses when the special properties of other types are not required. It is used where cement or concrete is not subject to specific exposures, such as sulphate attack from soil or water (Neville 1995), or to an objectionable temperature rise due to heat generated by hydration. Its uses include pavements and sidewalks, reinforced concrete buildings, bridges, railway structures, tanks, reservoirs, culverts, sewers, water pipes and masonry units. Different types of Portland cement are manufactured to meet different physical and chemical requirements for specific purposes. The basic raw materials used in the production of Portland cement are calcium carbonate found in limestone or chalk, and aluminium silicate found in clay or shale (Jackson 1983). These materials are heated
in a rotary kiln to form a clinker rich in calcium silicates with relative small quantities of calcium aluminates (Table 2.1). The cooled clinker is ground to a fine powder with a small proportion (< 3%) of added gypsum (calcium sulphate), which helps regulate the rate of setting when the cement is mixed with water, and helps eliminate the 'flash' set of C3A (Neville 1995). Portland cement is typically ground to produce a specific surface area of around 350 m²/kg.

<table>
<thead>
<tr>
<th>Compound</th>
<th>Chemical formula</th>
<th>Abbreviated formula</th>
<th>Typical % by weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tricalcium silicate</td>
<td>3CaO·SiO₂</td>
<td>C₃S</td>
<td>55</td>
</tr>
<tr>
<td>Dicalcium silicate</td>
<td>2CaO·SiO₂</td>
<td>C₂S</td>
<td>20</td>
</tr>
<tr>
<td>Tricalcium aluminate</td>
<td>3CaO·Al₂O₃</td>
<td>C₃A</td>
<td>10</td>
</tr>
<tr>
<td>Tetracalcium aluminoferrite</td>
<td>4CaO·Al₂O₃·Fe₂O₃</td>
<td>C₄AF</td>
<td>10</td>
</tr>
</tbody>
</table>

Table 2.1 Main compounds in Portland cement, after Harrison and Spooner 1986 and Neville 1995.

The calcium silicate and aluminate compounds react with water to produce calcium silicate and aluminate hydrates act as binding materials and also releases large quantities of Ca(OH)₂ which is precipitated as hexagonal crystals. In reality, the silicates in cement are not pure compounds, but contain minor oxides in solid solution such as MgO, K₂O, and Na₂O, etc. These oxides represent only a few percent of the cement weight but have significant effects on the atomic arrangement, crystal form and hydraulic properties of the silicates and their hydrated products (Neville 1995). The tri- and di-calcium silicate and tri-calcium aluminate react with water at significantly different rates, and contribute different amounts to the final strength of the hardened cement (Neville 1995). It is by varying these compounds that different Portland cements such as LHPC, RHPC and SRPC can be obtained.

In the presence of water this reaction continues for months after initial hydration (Neville 1995). Gel pores that have a pore size in the range of 10-30Å can be regarded as part of the C-S-H as they occupy about 30% of the volume of the gel solids. Capillary pores are formed by the presence of free water in excess of that required to hydrate the cement. This excess free water is necessary for the production
of a workable mix. Capillary pores are much larger than the gel pores being of the order of 10 to 10,000 nm. When interconnected, the capillary pores provide an easy path for the movement of water and water-borne ions, into and through the hardened material, and are mainly responsible for the permeability of hardened cement paste.

Figure 2.1 Micro-structural developments in cement paste, after Neville 1995.

A schematic sequence of micro-structural development in a cement paste as hydration proceeds is shown in Figure 2.1. Water separating individual cement grains in the plastic paste is replaced with solid hydration products that form a continuous matrix and bind the residual cement grains together over a period of time. The binding occurs since the hydration products occupy a greater volume than the original cement compounds; one ml of dry cement becoming 2.1 ml of hydration product (Neville 1995). The individual grains of calcium silicate are initially covered with a coating of C-S-H radiating outwards into the pore space. As the hydrate layer grows, the cement grain effectively increase in size and the regions of C-S-H begin to intermesh, bonding the cement grains together, and the points of contact growing in both area and number as hydration proceeds.

As hydrated cement paste comprises of calcium silicate hydrate, calcium hydroxide and calcium aluminate/sulphoaluminate hydrates, it is the stability of these individual compounds that determines the overall durability of the cement paste (Neville 1995). However, if we consider the main components as C₂S, C₃S, C₃A and C₄AF, by
changing the ratios of these components, it is possible to alter the characteristics of the
cement (Neville 1995). For example: -

a) Rapid Hardening Portland Cement

It is called type III Portland cement in the USA (ASTM C 150 1999). This cement is
similar to OPC (BSI 12 1996), however, higher levels of C₃S achieve rapid hardening,
often as high as 70% (Neville 1995). Since C₃S hydrates more rapidly than C₂S, by
increasing the amount of C₃S and reducing the amount of C₂S, it is possible to
increase the hydration rate at early ages. This may be augmented by decreasing the
average particle size and hence increasing the specific surface area from 350 for OPC
to 440m²/kg for RHPC. This cement develops higher strength at an early age making
it useful in fast-track construction where the formwork has to be removed early
(Jackson 1983).

It can also be used to advantage in cold weather condition (Neville 1995), since its use
permits a reduction in the controlled curing period. However, the higher rate of heat
development prevents its use in large mass concrete pours where thermal cracking can
occur.

b) Low Heat Portland Cement

It is called Type IV cement in the USA. This cement is similar to OPC (BSI 1370
1979), however lowering the amounts of C₃A and C₃S in this cement, and increasing
the proportion of C₂S will certainly reduce the rate of hydration and mitigate the
overall rate of increase in strength is decreased (Neville 1995). This effect is further
coupled with a slightly coarser grading that reduce the rate of heat development,
minimising the risk of cracking due to thermal stresses. Low heat Portland cement is
particularly useful in mass concrete pours such as concrete dams, where it is
important to keep the thermal gradient as small as practicable (Jackson 1983).

c) Sulphate Resisting Cement

It is called a CEM I cement with special resistance to sulphate (BS 4027 1996) and it
is called Type V cement in the USA (ASTM C 150 1999).
Sulphate-resisting cement is used only in concrete exposed to severe sulphate action principally where soils or groundwater have a high sulphate content.

BS 4027 (1996) stipulates a maximum C$_3$A content of 3.5% and SO$_3$ content is limited to 2.5%. In other respects, it is similar to ordinary Portland cement but it is not separately recognized in ENV 197-1:2000. Sulphate resisting cement is typically ground to produce a specific surface area of around (380 m$^2$/kg).

The low tri-calcium aluminate (C$_3$A) content is required for high sulphate resistance (Jackson 1983). However, SRPC has a lower binding capacity for free chloride ions because of its low C$_3$A, so it is a disadvantage to use SRPC for reinforced concrete in a chloride rich environment (Neville 1995). Findings of a case study by Novokshchenov (1995) for the deterioration of reinforced concrete in the marine industrial environment of the Arabian Gulf that occurred shortly after completion of the construction showed that the onset and rate of corrosion of the reinforcement have been enhanced by the use of Sulphate Resisting Portland cement.

2.2. BLENDED CEMENTS

Recently, due to a growing number of cases of chloride-induced corrosion, there has been a general trend to use blended cements, such as GGBS and PFA cements. These cements, while providing resistance against chemical and sulphate attack also give better protection against chloride-induced corrosion because of combining the pozzolana additions with the calcium hydroxide produced during the hydration reaction (Page et al 1981; Hussain and Rasheeduzzafar 1993; Algahtani et al 1994 and Rasheeduzzafar et al 1992a). It reduces the expansion due to alkali-silica reaction, and minimizes the risk of cracking due to reducing the heat of hydration gradients (Rasheeduzzafar 1992 and Alsaadoun et al 1993). Partial cement replacement reduces the permeability to water and chloride ions, increases electrical resistivity and reduces unbound chlorides (Hussain and Rasheeduzzafar 1994). Partial cement replacement of GGBS and PFA cements enhances concrete performance by including cost saving and higher long-term strength, that depend on % blends replacements.
Concern about the poor durability of OPC-based concretes in recent years has seen an increase in the use of so called cement 'blends' incorporating a range of pozzolanic materials including PFA, GGBS and silica fume. Pozzolans and slag can play a useful role in enhancing the properties of hydraulic cement concrete. These blended cements, therefore, are being projected as potential materials for usage in harsh environments (Hussain and Rasheeduzzafar 1993). The Middle East, and especially the Arabian Gulf States, is the prime example of truly extreme conditions. Here, the temperature of concrete is high and the geomorphic conditions are severe (Neville 1998 and Al-Amoudi et al 1992b). Thus, the usage of such blends in concrete construction is becoming more universal (Haque and Kawamura 1992 and Neville 1995), but is limited to regions where suitable supplies of these materials are available.

EN 197-1:2000 lists 25 discrete cement types of which 24 are cement blends (Hobbs 1998). The advantages of blended cements are twofold; firstly they offer the designer, the contractor and the ready-mixed concrete supplier new options for ordinary concreting purposes and also alternatives to solving particular problems. In addition since GGBS and PFA are waste products of other industries (by-products of iron production and electricity generation in coal-fired power stations) (Dewar 1988), it should follow that a blended cement will prove to be cheaper and more environmentally friendly. Many countries are actively encouraging the use of these products as cement replacements. This adoptive process is likely to continue since not only do these materials give improvements to the concrete made with them in the fresh state, but more often in the hardened state. Furthermore they have an ecological contribution by reducing CO₂ emissions resulting from Portland cement manufacture (Neville 1995; Malhotra and Mehta 1997 and Turner-Fairbank 2002).

Blended cements always display superior performance in terms of time to corrosion initiation and reduced corrosion current density (Alsaadoun et al 1993; Hussain and Rasheeduzzafar 1993; Rasheeduzzafar et al 1992a; Rasheeduzzafar 1992 and 1994) due to the generally lower chloride penetration. In these cements, the porosity was deemed to be reduced and the pore size to be refined, which thus hinders the movement of aggressive substances in concrete (ACI 222R 1996).
A well-cured blended mix can be expected to hydrate more slowly than an equivalent Portland cement concrete but for a longer period than a Portland cement, and will eventually have a lower permeability due to the extra cement paste produced. This will help increase the physical barrier afforded by the concrete. However, if proper curing in not achieved on site, the benefits of blended cements will not be achieved in practice (Parrott 1991; Harrison and Spooner 1986). Below are the chemical compositions of typical OPC, GGBS and PFA cements, shown in Table 2.2.

<table>
<thead>
<tr>
<th>Compound</th>
<th>OPC (Wt %)</th>
<th>GGBS (Wt %)</th>
<th>PFA (Wt %)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>(17-25)</td>
<td>(30-40)</td>
<td>48</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>(3-8)</td>
<td>(8-18)</td>
<td>26</td>
</tr>
<tr>
<td>CaO</td>
<td>(60-67)</td>
<td>(40-50)</td>
<td>2.5</td>
</tr>
<tr>
<td>MgO</td>
<td>(0.5-4.0)</td>
<td>(0-8)</td>
<td>1.5</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>(0.5-0.6)</td>
<td>0.5</td>
<td>9</td>
</tr>
<tr>
<td>SO₃</td>
<td>(2-3.5)</td>
<td>0.2</td>
<td>1</td>
</tr>
<tr>
<td>Na₂O</td>
<td>(0.3-1.2)</td>
<td>0.4</td>
<td>1</td>
</tr>
<tr>
<td>K₂O</td>
<td>-</td>
<td>0.7</td>
<td>3</td>
</tr>
<tr>
<td>TiO₂</td>
<td>-</td>
<td>0.6</td>
<td>1</td>
</tr>
<tr>
<td>P₂O₅</td>
<td>-</td>
<td>&lt;0.1</td>
<td>0.3</td>
</tr>
<tr>
<td>Mn₃O₄</td>
<td>-</td>
<td>-</td>
<td>0.1</td>
</tr>
<tr>
<td>C</td>
<td>-</td>
<td>-</td>
<td>5</td>
</tr>
</tbody>
</table>

Table 2.2. Typical composition of OPC, GGBS and PFA, after Neville 1995 and Concrete Society 1991.

2.2.1 GROUND GRANULATED BLAST FURNACE SLAG (GGBS)

2.2.1.1 PRODUCTION

Blast furnace slag is produced as a by-product of the manufacture of iron in a blast furnace. It results from the fusion of a limestone flux with ash from coke and the siliceous and aluminous residue remaining after reduction and separation of the iron from the ore. The operation of the blast furnace, and hence the production of both iron
and slag, is a continuous rather than batch process, with both materials being in the molten state. When the molten slag is subjected to either fine water jets maintained at high pressure and at high water/slag ratios, or to cold air/water jets, it cools rapidly into a glassy granulate with a consistent particle size range and chemical composition. The product is dried and ground to cement fineness in conventional cement clinker grinding mills with no further additions. Immediately after granulation, blast furnace slag is a light brown colour, however, after milling, it becomes very pale, almost white. Freshly made GGBS concrete has a bluish tint due to the formation of complex iron sulphides, however this colour fades rapidly as the sulphides are oxidised (Concrete Society 1991).

2.2.1.2 PHYSICAL PROPERTIES

Granulated blast furnace slag is ground by the slag processor that has control over its fineness, although BS 6699 (1992) specifies a minimum surface area of 275 m$^2$/kg. In Australia and USA, it is a common practice to grind GGBS more finely than that currently produced in the UK. This leads to more reactive material. A fineness of 500-600 m$^2$/kg is common. This increased fineness influences the properties with respect to workability, bleeding, heat, evolution and strength development (Harrison and Spooner 1986). In the USA this admixture is added to concrete before, or during, mixing to improve the plastic or hardened properties of Portland cement concrete (ASTM C 989 1999). It is referred to as a cementitious constituent in concrete (ACI 226-1R 1987).

Particles of GGBS are irregular in shape and have a relative density of 2.9, compared to 3.15 for Portland cement. Therefore, exchanging GGBS for cement on an equal mass basis will increase the total volume of cementitious materials in the mix (Concrete Society 1991).

2.2.1.3 CHEMICAL COMPOSITION

The chemical composition of blast furnace slag may vary from works to works depending on the nature of the limestone flux and the blast furnace conditions. The major oxide components of slag are lime, magnesia, silica and alumina and if slowly cooled in air these would form an assemblage of crystalline minerals. However, the
rapid quenching in water to produce granulated slag results in the formation of a slag glass consisting of a network of calcium, silicon, aluminium and magnesium ions in disordered combination with oxygen. Minor components such as sodium, potassium and titanium are also integral parts of this structure and further modify or disorder the glass network. Blast furnace slag also contains small quantities of sulphides, present usually as calcium sulphide, however, in the presence of air, they decompose to form hydrogen sulphide (Concrete Society 1991). In addition to this glass and depending upon the slag composition and quenching conditions, small quantities of crystalline phases may also be present. The typical chemical composition of slag is shown in Table 2.2 (Concrete Society 1991).

2.2.1.4 CHEMISTRY IN CONCRETE

GGBS reacts with water thereby increasing the pH, generating heat and developing a particle-to-particle cementitious bond similar to Portland cement. This intrinsic hydraulicity distinguishes GGBS from other pozzolans. The intrinsic reaction of GGBS with water is very slow compared to Portland cement, therefore, for practical purposes, the hydraulicity is activated by the hydroxides and sulphates present in Portland cement. These activators react with the GGBS disturbing the structure of the glass; releasing reaction products for hydrate formation and continuation of the hydration process (Concrete Society 1991). Apart from the primary reactions between OPC and water, and GGBS and water, secondary "pozzolanic" reactions also occur. These occur between the reaction products of OPC with GGBS, and the reaction products of OPC with the reaction products of GGBS. The latter reactions include the pozzolanic reaction. The hydrates formed in these combinations are similar to those produced during the hydration of Portland cement, i.e. calcium silicate and aluminate hydrates (Neville 1995).

In summary, the hydration of C$_3$S and C$_2$S phases in cement follow the following reactions:

$$C_3S / C_2S + H \ \Rightarrow \ C-S-H + CH$$

The portlandite (CH) so produced has one primary advantage of increasing the reserved alkalinity (pH) of cement. However, this material is somewhat leachable in
water and, when being leached out, it leaves voids in its place; thereby increasing the porosity of the cement pastes. Furthermore, CH is known to enhance the gypsum type of sulphate attack upon exposure to sulphate-bearing media. Moreover, ettringite formation is reported to be more expansive with the increase in alkalinity provided by the portlandite produced by cement hydration. Therefore, to avoid these disadvantages, it is judicious to consume this portlandite through the pozzolanic reaction reported above.

The pozzolanic reaction between the glass phase in the pozzolan (i.e. GGBS, PFA, etc.), and the portlandite produced during the hydration of OPC cement will form the more binding calcium silicate hydrate (C-S-H) as simplified in the following equation:

\[ \text{CH} + \text{S} + \text{H} \Rightarrow \text{C-S-H} \]

Calcium hydroxide + reactive silica + water \( \Rightarrow \) Calcium silicate hydrate

The pozzolanic reaction brings certain changes in the chemical environment of concrete and brings about a very significant change in the physical structure of the hardened cement paste (Algahtani et al 1994).

It is interesting to note that blended cements have often been observed to increase the measured \( \text{Cl}^-/\text{OH}^- \) ratio in concrete with a given chloride ion content making them potentially at risk of reinforcement corrosion. However, this increase is always offset by the distinct improvement in the physical characteristics of the cement paste matrix and the substantial reduction in permeability of blended cement concrete to chloride ingress, thereby reducing the corrosion risk (Rasheeduzzafar 1992, Rasheeduzzafar et al 1992a and 1992b).

2.2.2 PULVERISED FUEL ASH CEMENT (PFA)

2.2.2.1 PRODUCTION

Pulverized-fuel ashes (PFA) are materials extracted by electrostatic and mechanical means from the flue gases of power-station furnaces fired with pulverized bituminous coal. PFA produced in the UK has high glassy silica but low lime content and thus
will not react on its own with water, but needs a source of calcium hydroxide before hydrates can be formed (i.e. it is pozzolanic rather than hydraulic). Ordinary Portland cement is usually used to provide the necessary calcium hydroxide. The suitability of PFA depends on a number of factors, two important ones being its fineness to ensure reactivity and low non-reactive carbon content. A further requirement is that the water demand of a standard mortar Portland/PFA cement mix shall not exceed 95% of that for the Portland cement alone, which partly compensates for the lower cementing efficiency of PFA at early ages (Harrison and Spooner 1986; Concrete Society 1991).

PFA is produced at power stations by the combustion of bituminous coal, which consists of carbonaceous matter and a mixture of various minerals. At the power stations in UK, the coal is pulverised in grinding mills and after this it is blown into the boiler at high speed. It burns at a temperature exceeding 1500°C, which is above the melting point of most metals present; therefore, the minerals undergo various physical changes. Most of the ash particles are carried through the furnace with the flue gases and are then extracted by being passed through a cyclone collector, which removes most of the coarser particles and, thereafter, passed over an electrostatic precipitator in series to remove the smaller particles. The fine ash precipitated from the exhaust gases is known as pulverised fuel ash (PFA). The properties of PFA will vary based on the type of coal burnt and the furnace conditions, thus the PFA will vary from plant to plant and will also vary in one plant over a period of time. The PFA has relatively consistent chemical composition as compared with the GGBS (Concrete Society 1991).

2.2.2.2 CHEMICAL COMPOSITION

PFA consists principally alumino-silicate glass containing some iron, calcium, magnesium and alkali metals in the glass structure. The ash particles are coated with calcium sulphate or potassium sulphate. The PFA will also contain some residual-burnt coal, which appears as coarse, porous particles. The typical composition is shown in Table 2.2 (Concrete Society 1991).
2.2.2.3 PHYSICAL PROPERTIES

BS 3892-1 (1997) had limitations to impurities and size of particles. But most of Europe and the US have used it in a board range as Fly Ash (ACI 226-3R 1987 and ASTM C 618 1999). PFA particles are predominantly spherical in shape with about 5% by mass being hollow cenospheres, however, due to the unburnt coal there will be some irregularly shaped particles. This spherical shape is able to lubricate a concrete mix and thus increase the workability; thereby leading to a smaller requirement of water for a specified workability. The relative density of PFA is between 2.0 and 2.3, being influenced by the amount of porous carbon particles, the relative density of Portland cement is 3.15. This means that if PFA is used to replace Portland cement on an equal mass basis, the volume of cementitious materials in the mix will rise. The colour of PFA is usually grey but will be darkened by either the amount of residual carbon or by its iron content. A PFA with high carbon content may be very dark in colour; this means that the colour of the ashes may form part of quality control of the PFA (Concrete Society 1991).

2.3. FACTORS AFFECTING DURABILITY OF CONCRETE

There is a number of important factors, which can influence the durability of plain concrete. These include:

(i) The mix design, including the cement content, w/c, etc.

(ii) The workability of fresh concrete and hence its ease of compaction

(iii) The degree of curing achieved on site, and

(vi) The materials used in the mix, such as type of cement, admixtures, etc.

2.3.1 MIX DESIGN

The main parameters in the design of a concrete mix of a given strength are (Neville 1995):

- The cement content
- The aggregate type
- The water in cement
- The free water/cementitious materials (binder) ratio.

2.3.1.1 CEMENT CONTENT

Increasing the cement content at a fixed w/c ratio increases the proportion of free water in the fresh concrete and results in a more workable mix. The resulting increase in cement paste volume can usually be expected to result in a more cohesive mix that will often improve the resistance to chemical attack once hardened (ACI 201-2R 1992).

2.3.1.2 AGGREGATE

Although aggregate usually makes up at least 75% of the volume of concrete, if unreactive, they are, by and large, a passive cheap filler material (Rasheeduzzafar 1992). Therefore, it is easy to view aggregate as just an inert material dispersed in the cement paste. However, the aggregate confers considerable technical advantages on the concrete, such as introducing a higher volume stability and better durability than cement paste alone. The maximum size of aggregate used in concrete varies from centimetres to tenths of a millimetre, and the most common method of mixing aggregate into cement is to use at least two different particle grading. These would normally be a fine aggregate, or sand, with a particle size smaller than 5mm and a coarser aggregate with particle size around 10mm or more. This grading reduces the space between the particles and thus the amount of cement pastes required (Neville 1995). The aggregate properties will depend on two different sources. Firstly, the type of the parent rock, from which the aggregate was derived, e.g. its chemical properties, specific gravity, hardness, strength, pore structure, colour, etc. Secondly, there are some properties not found in the parent rock, e.g. particle shape and size, surface texture and absorption. All these properties will have an influence on the strength and durability of the finished concrete. Aggregates can be classified into two main groups; natural aggregates formed from naturally occurring materials and manufactured aggregates which are produced from industrial products. Natural
aggregates can be further classified into crushed aggregate and aggregate which has been naturally reduced in size (Neville 1995 and ACI 201-2R 1992).

Whatever type of aggregate is used, if it has any impurities such as organic matter, they will interfere with the hydration process. Also any weak particles (e.g. recycled aggregates) or coating (e.g. clay) can greatly affect the overall strength of the concrete, therefore, both these problems must be avoided (Neville 1995).

2.3.1.3 WATER CONTENT

The amount of water used in any mix will affect the properties of both the fresh and hardened concrete and its subsequent durability. Water is held in concrete in the following three ways: As free water in the capillaries, as free water in the gel pores, and chemically-combined water with the hydrated products.

The set paste contains capillary pores, which form a pipe network throughout the concrete; this network is permeable to both gases and liquid that contains the free water. This water is not bound to the cement paste and is thus removed when the paste is heated. Chemically combined water at the other extreme is water that cannot be evaporated since it is part of the cement’s chemical composition and in fact accounts for 25% by weight of fully-reacted cement. Between these two categories of water in cement is the third class which is the water contained in the gel pores, this is held in a variety of different ways such as hydrogen bonding and surface forces of the gel particles. This water is held between the gel fibres and forms a micro-network, which contributes to the concrete porosity. The majority of this water can be removed by evaporation, although no technique is available for determining exactly what type of water remains in oven dried cement. From these three different categories of water in cement mentioned above, the water can be further classified as non-evaporable and evaporable (Neville 1995). The amount of non-evaporable water increases as the hydration process continues. In well-hydrated cement, the non-evaporable water is about 18% by weight of the anhydrous material; this proportion rises to about 23% in fully hydrated cement (Lea 1983).

It is important to distinguish between the free (known as effective) and total (known as gross) water in a concrete mix. The water, which occupies space outside the
aggregate particles when the gross volume of concrete becomes stabilised, i.e. at the time of setting, is called free or effective water. This type of water will be involved in the chemical hydration of cement. On the other hand, the total water in concrete consists of: (i) that added to the mix (effective water), and (ii) that held by the aggregate before or during the mixing time. In the mix proportioning stage, the data are usually based on the water in excess of that absorbed by the aggregate that is the free water (Neville 1995; ACI 201-2R 1992).

2.3.1.4 FREE W/C RATIO

The free w/c ratio in any given mix is of great importance in determining the strength, durability and properties of concrete. The w/c ratio determines the porosity of the hardened cement paste at any stage of hydration (Neville 1995). A low w/c ratio does not by itself assure concrete of low permeability since the concrete must be sufficiently workable to achieve good compaction and subsequently consolidated and cured to produce concrete of low permeability. The strength of a concrete at a given age and cured in water at a prescribed temperature is assumed to depend primarily on two factors only: the w/c ratio and the degree of compaction (Neville 1995).

2.3.2 WORKABILITY AND COMPACTION

Workability can be defined as the amount of useful internal work necessary to produce full compaction without segregation (Glanville et al 1947) or that property of freshly-mixed concrete or mortar, which determines the homogeneity of the mix and ease with which it can be mixed, placed, consolidated and finished (ACI 116R-90 1994). Poor compaction leads to increased voids and thus increased porosity and a reduction of strength in the hardened concrete. Therefore, the fresh concrete must be sufficiently workable to achieve a good compaction. The most important factor affecting the workability is the free water content of the mix, although the cement content, aggregate/cement ratio and aggregate surface, shape and texture also play a role in controlling the degree of workability of a particular mix (Neville 1995). If high quality concrete is demanded, low w/b should be used and a plasticizing admixture (i.e. superplasticizer) will be added to the mix to improve the workability.
2.3.3 CURING

The term ‘curing conditions’ is used to describe the moisture and temperature states in which concrete is kept after it has been cast. Water is essential to all hydration processes and to the development of concrete strength of the cements being used (Harrison and Spooner 1986). Curing of concrete at early ages is very important to obtain satisfactory long-term mechanical properties and durability characteristics. The quality of concrete cured, especially of the surface layers, is of interest when durability of reinforced concrete is concerned (Zhang et al 1999 and Neville 1995).

The permeability of concrete depends on the volume of pores in the cement paste and their size and continuity (Neville 1995). It has been found that for different w/c ratios, the time required to block continuous capillary pores with gel varies, as shown in Table 2.3.

<table>
<thead>
<tr>
<th>w/c Ratio</th>
<th>Curing Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.4</td>
<td>3 days</td>
</tr>
<tr>
<td>0.45</td>
<td>7 days</td>
</tr>
<tr>
<td>0.5</td>
<td>14 days</td>
</tr>
<tr>
<td>0.6</td>
<td>6 months</td>
</tr>
<tr>
<td>0.7</td>
<td>1 year</td>
</tr>
<tr>
<td>&gt;0.7</td>
<td>Impossible</td>
</tr>
</tbody>
</table>

Table 2.3 Recommended water curing times to block continuous capillaries of cement paste with gel, after Powers 1959.

Thus, proper curing, coupled with a low w/c ratio, is required to produce a low permeability and hence a durable concrete. It is important to note that proper curing can be even more critical when using blended cements due to their slower hydration reaction (Neville 1995). It has been documented that, concretes containing GGBS are more sensitive to poor curing than OPC/PFA (Harrison and Spooner 1986). This is made more crucial due to the tendency of cement blends to dehydrate more rapidly than pure Portland cement concretes (Parrott 1991).
The results of total chloride concentration profiles for reinforced concrete specimens, with different PFA contents designed to achieve equal strength grade and workability and placed in seawater for periods of 1 and 2 years following various periods of initial curing, indicated that in both PFA concrete and OPC concrete, the rate of penetration of chloride ions and subsequent corrosion of steel reinforcement were increased as the duration of initial curing was reduced. However, these detrimental effects became less pronounced as the length of exposure was extended (Thomas et al 1990).

It is clear that increasing the period of water curing to 28 days, prior to exposure, always produces the lowest coefficient of chloride diffusion values for OPC, PFA and GGBS concretes. However, the coefficients for PFA concrete were lower than those for OPC concrete by a factor of 3, while the largest reduction was observed with high GGBS replacement levels. The coefficient of chloride diffusion of GGBS concrete has been shown to be dependent upon initial curing, GGBS replacement level, design strength and exposure temperature (Jones 1994).

2.3.4 MATERIALS

The type of cement will play a dominant role in the durability of concrete, particularly in terms of corrosion of reinforcing steel and sulphate attack on concrete. The various types of Portland cements and their effect on durability has been reported in Section 2.1. Similarly, the usage of supplementary cementing materials (PFA, GGBS, silica fume, etc.) has been addressed in Section 2.2.

A long range of admixtures can be used in concrete to improve its durability performance (Kosmatka and Panarese 1995). Typical admixtures that are often used in concrete to improve its quality include, but not limited to, the following: superplasticisers, corrosion inhibitors, retarding and accelerating admixtures, etc.

2.4. THE CORROSION PROCESS

Four conditions must exist before aqueous corrosion can proceed: (i) there must be something that corrodes (the metal anode), (ii) there must be a cathode, (iii) there must be continuous conductive liquid path (electrolyte, usually condensate and salt or
other contaminations), and (iv) there must be a conductor to carry the flow of electrons from the anode to the cathode. This conductor is usually in the form of metal-to-metal contact such as in bolted or riveted joints. The elimination of any one of these four conditions will stop aqueous corrosion. An unbroken (perfect) coating on the surface of the metal (i.e. such as the usage of epoxy-coated rebars) will prevent the electrolyte from connecting the cathode and anode so the current cannot flow (NASA 1994).

2.4.1 ELECTROCHEMICAL ASPECTS OF AQUEOUS CORROSION

Metallic corrosion can be broadly classified into two categories;

(i) Direct Oxidation: This type refers to the chemical reaction of a dry metal surface with an oxidising gas. Such corrosion occurs in the absence of a liquid phase and the reaction occurs directly at the gas-solid interface. At temperatures above 200°C, there is usually significant reaction of most metals in dry air and the rate and extent of reaction increase as the temperature is raised, or other gases contaminate the air. The degree of oxidation is largely governed by the properties of the corrosion product and by its mechanical strength and adherence to the underlying metal (ACI 222R 1996).

(ii) Aqueous Corrosion: When water comes into contact with a metal, the predominant corrosion process is electrochemical and loss of metal section occurs by anodic dissolution. The presence of at least a thin film of water is required as this provides the solvent and connecting electrolyte needed for electrochemical corrosion. The basic corrosion mechanism is dissolution of the solid metal as metal ions with solid products (such as rust) forming by subsequent reaction. At room temperature, the progress of electrochemical corrosion is determined by several factors, notably the nature of the oxidising agent and also kinetic factors such as bulk diffusion rates and electron transfer reactions (ACI 222R 1996).

Of particular interest to civil and structural engineers is the rusting of iron and steel components and the deterioration of steel reinforcement in concrete. At anodic areas, loss of metal occurs and ferrous metal ions are released into solution. While at cathodic areas, of more positive potential, hydroxyl ions are formed. The soluble ion
products diffuse through the solution and react to precipitate ferrous hydroxide \( \text{Fe(OH)}_2 \). This subsequently oxidises to form hydrated ferric oxide \( (\text{Fe}_2\text{O}_3.3\text{H}_2\text{O}) \), which is commonly known as rust.

2.4.1.1 METALS IN AQUEOUS SOLUTION

Any reaction, which can be divided into two (or more) partial reactions of oxidation and reduction, is termed "corrosion". In every case of metallic corrosion, the anodic reaction is the oxidation of a metal to its ion, thus:

\[
M \Leftrightarrow M^{n+} + n \text{ electrons}
\]

In each case, the number of electrons produced equals the valence of the ion. It is a characteristic of all anodic reactions that electrons are lost and thus become available for reaction with other species. Where there is no mechanism by which these extra electrons can be consumed, i.e. no cathodic reaction, then the anodic reaction comes to equilibrium and no further corrosion can take place. However, in most real environments, there are essentially five common cathodic reactions encountered.

\[
2\text{H}^+ + 2e \Rightarrow \text{H}_2 \quad \text{Hydrogen evolution (acid conditions)}
\]

\[
\text{O}_2 + 4\text{H}^+ + 4e \Rightarrow 2\text{H}_2\text{O} \quad \text{Oxygen reduction (acid conditions)}
\]

\[
\text{O}_2 + 2\text{H}_2\text{O} + 4e \Rightarrow 4\text{OH} \quad \text{Oxygen reduction (alkaline conditions)}
\]

\[
M^{n+} + e \Rightarrow M^{(n-1)+} \quad \text{Metal ion reduction}
\]
Each one will occur under different conditions although it is perfectly possible for more than one of these reactions to occur at the same time. The common denominator among these five reaction is that they all consume free electrons. Indeed, it is because the cathodic reaction removes electrons produced by the anodic reaction that corrosion of the anode continues.

2.4.1.2 CORROSION AS AN ELECTROCHEMICAL REACTION

Such partial reactions can be used to interpret all corrosion problems as can be illustrated by considering the rusting of iron in neutral water, i.e. pH = 7. The anodic reaction is the production of ferrous ions (Fe$^{2+}$) accompanied by the reduction of oxygen at the cathode, according to the following reactions:

Fe $\Rightarrow$ Fe$^{2+}$ + 2e (Anodic reaction)

O$_2$ + 2H$_2$O + 4e $\Rightarrow$ 4OH$^-$ (Cathodic reaction)

Or more fully,

2Fe + 2H$_2$O + O$_2$ $\Rightarrow$ 2Fe$^{2+}$ + 4OH$^-$ $\Rightarrow$ 2Fe(OH)$_2$

The hydroxide initially formed subsequently oxidizes to form red rust.

4Fe(OH)$_2$ + O$_2$ $\Rightarrow$ 2Fe$_2$O$_3$.3H$_2$O + H$_2$O

The fact that rust is powdery, porous and non-adherent ensures that the rusting of iron and steel can occur rapidly since fresh metal is continually exposed. Under alkaline conditions, however, ferrous hydroxide forms as a thin layer on the surface and acts to exclude oxygen preventing further corrosion of the iron. This passive layer is stable provided the conditions remain substantially unchanged.
2.4.1.3 HALF-CELLS

In the general case, when a metal is placed in a solution of its own ions, metal atoms will tend to pass into solution giving up electrons in order to form ions, Figure 2.2. This reaction also occurs in reverse where a metal ion in solution accepts electrons and is deposited on the metal surface. This will continue until the rate at which the metal atoms pass into solution as ions equals the rate at which metal ions from the solution deposit on the metal surface. Since electrons left in the metal cannot be conducted away, an electrical potential is set up between the interior of metal (electrode) and the bulk of the solution (electrolyte). It is to be noted that in this reaction n moles of electrons are transferred for every mole of M ionised (Brophy et al 1964).

![Figure 2.2. Metal/metal-ion half-cell, after Brophy et al 1964.](image)

The 'electrode potential' for such a metal/metal-ion single-electrode (or 'half-cell') reaction cannot be measured directly without affecting its value and, therefore, it must be measured relative to some reference electrode. The universally accepted standard reference point is the Standard Hydrogen Electrode (SHE), which has been arbitrarily defined as having a potential of exactly 0.000 volts under equilibrium conditions at 25°C. Values of the Standard Electrode Potential ($E_0$) shown below are for some of the more common metal/metal ion systems.

The half-cell potential, $E_{cell}$, can be determined relative to the Standard Hydrogen Electrode although it is more common to use other electrodes such as:
Volts relative to SHE

(i) Calomel electrode (mercury, mercuric chloride): +0.25

(ii) Copper/copper sulphate electrode: +0.32

<table>
<thead>
<tr>
<th>Electrode Reaction</th>
<th>Standard Electrode Potential (volts relative to SHE)</th>
<th>Standard Electrode Potential (volts relative to Cu/CuSO₄)</th>
</tr>
</thead>
<tbody>
<tr>
<td>O₂ + 4H⁺ + 4e⁻ = 2H₂O</td>
<td>+1.23</td>
<td>+0.91</td>
</tr>
<tr>
<td>Fe²⁺ = Fe³⁺ + e⁻</td>
<td>+0.77</td>
<td>+0.45</td>
</tr>
<tr>
<td>O₂ + 2H₂O + 4e⁻ = 4OH⁻</td>
<td>+0.40</td>
<td>+0.08</td>
</tr>
<tr>
<td>Fe = Fe²⁺ + 2e⁻</td>
<td>-0.44</td>
<td>-0.76</td>
</tr>
<tr>
<td>Cu = Cu²⁺ + 2e⁻</td>
<td>+0.337</td>
<td>0.017</td>
</tr>
<tr>
<td>Zn = Zn²⁺ + 2e⁻</td>
<td>-0.763</td>
<td>-1.083</td>
</tr>
</tbody>
</table>

Table 2.4 Electromotive Force Series - Standard electrode potentials for some common metals, after Brophy et al 1964.

The values of $E_0$ listed in Table 2.4 are for unit concentration of metal ions in solution (i.e. 1 mole metal ion per litre), at 25°C and one atmosphere pressure. For other concentrations and temperatures, the electrode potential can be obtained from the Nernst equation (Uhlig 1971).

$$E_{cell} = E_0 + \frac{RT}{nF} \ln \left( \frac{P_{ox}}{P_{red}} \right)$$

Where:

$E_{cell}$ = Half-cell potential,

$E_0$ = Standard half-cell potential,

$F$ = Faraday constant

$T$ = Absolute temperature,

$R$ = Gas constant,
\[ n = \text{No. of electrons transferred}, \]
\[ \ln = \text{Natural logarithm, base e} \]
\[ P_{\text{ox}} = \text{Concentration of oxidised form, and} \]
\[ P_{\text{red}} = \text{Concentration of reduced form.} \]

2.4.1.4 IMPLICATION FOR DISSIMILAR METALS

An important feature of galvanic (or bi-metal) corrosion is that one metal, the anode, dissolve in preference to an adjacent metal as positive ions. The electrons generated in this way flow through the electrical contact to the other metal (called the cathode), where they are used to reduce hydrogen ions, create hydroxyl ions, or participate in a similar reduction reaction. Consequently, the corrosion rate is proportional to, and associated with, an electrical current. Whether a metal is anodic or cathodic depends on its propensity to ionise. It is common to express the propensity to ionise in terms of voltage. A tabulation of such voltages is called an electromotive force (EMF) series. The voltages depend on the concentrations of ions in solution, and the shielding of the cathode by reaction products (polarisation). For any given environment, a table, which places the materials in order of their anodic tendencies, may be constructed. Such a table is called a Galvanic Series, Table 2.5. It is important to realise that the Galvanic Series for other environments may differ radically from Table 2.5. Also, the farther away two metals on Table 2.5 lie from each other, the larger is the tendency to corrode (Brophy et al 1964).

Dissimilarity may arise from concentrations of the ions in the environment, the electrolyte velocity, the microstructure of the metals, stress, surface films, and from many other causes. Differences in microstructure, plastic strain, and access to oxygen, ion concentration, or anything else affecting the free energy of ionisation, can also lead to corrosion (Brophy et al 1964).
Noble
(least active)
Stainless steel, type 304 (passive)
Nickel (passive)
Silver solder
70-30 copper-nickel
Copper
Nickel (active)
Manganese bronze
Tin
Cast iron
Mild steel
Aluminium
Galvanized steel
Zinc
Anodic (most active)

Table 2.5 Galvanic Series in Sea Water, after NASA 1994.

2.4.1.5 IMPLICATION OF GENERAL TYPES OF CONCENTRATION CELL CORROSION

In addition to problems arising from dissimilar metals in contact, concentration cell corrosion can occur when two, or more, areas of a metal surface are in contact with different concentrations of the same solution. These may give rise to electrochemical potential differences, which cause the anode to corrode. There are three general types of concentration cell corrosion (NASA 1994):

(i) Metal ion concentration cells,

(ii) Oxygen concentration cells, and

(iii) Active-passive cells.

Thus, corrosion can also be induced if the concentration of oxygen, water, or chloride differs at various locations along a steel bar or electrically connected steel system.
2.4.2 POURBAIX DIAGRAMS (POTENTIAL - PH DIAGRAMS)

In order to decide whether corrosion can occur under a given set of conditions, a Belgian corrosion scientist, Dr Pourbaix, developed the concept of potential - pH diagrams. These are effectively corrosion 'maps' which indicate the tendency of a metal to corrode in aqueous solutions at a given temperature and show how changes in potential (brought about by changes in such things as concentration and coupling with other metals) and pH (brought about by altering the environment) affect this tendency. In constructing these diagrams, Pourbaix was careful to define three different regions of stability:

(i) Immune: This term is used for those regions where the dissolution of metal ions is effectively impossible since the solid metal represents the thermo-dynamically stable phase and so no corrosion can occur.

(ii) Corrosion: This term is used for those regions where the metal is unstable and goes into solution as metal ions. Thus, the reaction \( M \leftrightarrow Mn^{+} + ne \) is the preferred reaction and extensive corrosion can occur.

(iii) Passivation: This term is used to describe regions whereby the dissolution of metal ions is the preferred reaction (i.e. the metal is unstable with respect to its environment), however, the corrosion process produces some protective layer, such as an insoluble hydroxide, which acts to prevent or slow further corrosion of the underlying metal. Passivation depends on the physical nature of the passivating film and corrosion will cease only if the film is completely impervious - a porous film will slow down the rate of corrosion but will not stop it altogether. It is important to differentiate this passivity from 'immunity' where corrosion of the metal is thermodynamically impossible.

Figure 2.3 shows a simplified version of the Pourbaix diagram for the Iron - Water system at 25° C assuming that the only solid substances are Fe, Fe(OH)\(_2\), and Fe(OH)\(_3\). Using this diagram, it is possible to delineate areas in which iron, ferrous ions, and hydroxides are the most stable phase thermodynamically.
Pourbaix diagrams have three main uses:

(i) Predicting the spontaneous direction of a reaction.

(ii) Estimating the composition of corrosion products, and most importantly

(iii) Predicting environmental and other changes which will prevent or reduce corrosive attack.

2.4.3 EVAN’S DIAGRAMS

The limitations of Pourbaix diagrams are that they are theoretical representations of equilibrium conditions for pure metals in pure water. Since they are derived from thermodynamic considerations, they only indicate what reactions are possible but do not indicate the rate at which such reactions will occur. Thus, thermodynamics alone cannot provide a complete solution to the problem of metallic corrosion but must be coupled with an understanding of reaction kinetics, which can be assessed by using Evan’s diagram.

These diagrams require the plotting of corrosion current vs. potential with time. In these diagrams, the anodic and cathodic potentials are plotted versus the current applied. At the intersection of these potentials with the current, the corrosion current
density is determined (see Figures 2.4 to 2.7). They can be used to visualise the main parameters limiting the rate of any given corrosion reaction. Evan's Diagrams are shown below (Figure 2.4-2.7).

Figure 2.4 Cathodic control, after ACI 222R 1996.

Figure 2.5 Anodic control, after ACI 222R 1996.

Figure 2.6 Cathodic diffusion control, after ACI 222R 1996.

Figure 2.7 Resistance control, after ACI 222R 1996.

The behaviour of a single electrode process in terms of polarisation curves is very popular in the studies of corrosion rates of steel reinforcement. However, Page (1988) has pointed out that in this approach, the electrodes are made to function as net anodes or cathodes in cells in which the driving force is supplied by an external power source (potentiostat). In practice, separate anodic and cathodic reactions occur simultaneously on the same electrode surface. The driving force is the difference between the electrode potentials at anodic and cathodic sites. Coupling of a polarised
anodic reaction and a polarised cathodic reaction forms a corrosion cell. For such a simple corrosion cell, an Evan's diagram can be used to estimate the corrosion rate in terms of cell current. Evan's diagrams are constructed from separate anodic and cathodic polarisation curves. The cell current is estimated from the intersection of the cathodic and anodic curves. In the case of steel reinforcement, the surrounding concrete serves as the ion-conducting medium.

If the cathodic process is the slower process (the one with the larger polarisation), the corrosion rate was considered to be cathodically controlled (Figure 2.4). Conversely, if the anodic process is slower, the corrosion rate is said to be anodically controlled (Figure 2.5). In concrete, one or two types of corrosion-rate-controlling mechanisms normally dominate. One is cathodic diffusion, where the rate of oxygen diffusion through the concrete determines the rate of corrosion. In Figure 2.6, cathodic diffusion control is shown for two different rates of oxygen diffusion. The other type of controlling mechanism involves the development of a high resistance path. When steel corrodes in concrete, anodic and cathodic areas may be as much as several feet apart; therefore, the resistance of the concrete may be of great importance. Figure 2.7 illustrates two cases of resistance control where the potential available for corrosion is the difference in the potential between anode and cathode minus the relevant IR loss (ACI 222R 1996).

2.5. DURABILITY OF REINFORCED CONCRETE

Concrete’s limitation is that it is very weak in tension, but with the addition of steel reinforcement, combining the compressive strength of concrete and the tensile strength of steel, it becomes an entirely new construction material that is cheap and potentially durable. This durability comes from the ability of the alkaline cement paste to both promote passivation of the embedded steel and to provide a physical and chemical barrier to the ingress of aggressive ions, such as chloride. The interaction of concrete and steel reinforcement therefore produces a composite, cost effective and durable material, which is used across the world. If concrete is made with a sound, inert aggregate, deterioration will not normally occur in the absence of an external influence. Since concrete is a highly alkaline material, its resistance to other alkalis is good, but as a sequence it is very susceptible to attack by acids and other substances.
Well-constructed Portland cement structures had been found to endure for many years in seawater despite the presence of significant concentration of both chlorides and sulphates ions. On the other hand, if the concrete was of poor quality, serious damage may occur from the reactions of soluble salts with the hydrated cement compounds (Mosley et al 1996).

Concrete structures, properly designed and constructed, are long lasting and should require little maintenance. A dense, well-compacted concrete with a low water to binder ratio is all important (Mosley et al 1996). This requires appropriate planning, design and execution, particularly with regard to the properties of cover concrete (thickness, permeability, absence of aggressive substances). Consideration of durability is necessary to ensure that a structure remains serviceable throughout its lifetime. This requirement will involve aspects of design, such as concrete mix selection, adequate curing and proper cover to reinforcing bars, as well as selection of suitable materials for the exposure conditions. The detailing requirements are also important to ensure that a structure has satisfactory durability and serviceability performance under normal circumstances (Mosley et al 1996).

The durability of a concrete structure is influenced by the following factors:

(i) Exposure condition: Concrete can be exposed to a wide range of conditions such as the soil, seawater, stored chemicals or the atmosphere. The severity of exposure governs the type of concrete mix required and the minimum cover to the reinforcing steel.

(ii) Concrete quality: Cement composition significantly affects the durability performance of concrete (Rasheeduzzafar 1992). Whatever the exposure, the concrete mix should be designed and made with impervious and chemically inert aggregates (Mosley et al 1996).

(iii) Presence of cracks: It must always be remembered that reinforced concrete structures are designed to crack although there are limits, e.g. 0.3 mm, placed on the maximum allowable crack width. Such "designed" cracks are not usually a source of durability problems but the presence of "unexpected" cracking is considered as either signs of continuing movement or cyclic opening and closing.
(iv) Cover to the reinforcement: The cover to reinforcement is the shortest distance between the surface of a concrete member and the nearest surface of the reinforcing steel (Neville 1998). Adequate cover with good quality is essential to prevent corrosive agents reaching the reinforcement through cracks and pervious concrete. The thickness of cover required depends on the severity of the exposure and the quality of the concrete (Mosley et al 1996 and ACI 201-2R 1992). From the standpoint of durability, the protection of reinforcement is a function of the thickness of cover and of the quality of concrete in it (Neville 1998). It is believed that these two factors can offset each other, and some codes of practice provide tables of alternative combinations of thickness of cover together with the quality of concrete to ensure durability under given conditions. However, it is important to point out that a combination of a very large cover and a very poor concrete is entirely unsatisfactory no matter how large the cover (Neville 1998; Haque and Kawamura 1992).

2.5.1 PRINCIPLES OF REINFORCEMENT CORROSION

All good quality concretes provide protection against rusting of embedded reinforcing steel because of the alkaline environment provided by Portland cement paste (ACI 201-2R 1992). Under such high pH conditions, steel reinforcement is normally protected from corrosion by the formation of a thin surface layer of ferrous hydroxide or hydrated oxide. The formation of this surface layer is a time dependent process and its electrochemical properties change with age. Typically, for mature Portland cement paste, the pH is 12.5 and results from the presence of calcium hydroxide, liberated when the Portland cement hydrates (ACI 201-2R 1992). The presence of sodium and potassium hydroxides in the cement yield even higher values than 13.5 but these are usually consumed early during the hydration process.

The two main causes of reinforcement corrosion are known to be:

(i) Carbonation of the cover concrete, and

(ii) Presence of chloride ions around the steel.

Carbonation results from the reaction of carbon dioxide in the atmosphere with calcium hydroxide in the Portland cement paste to produce calcium carbonate resulting in reduction in the pH of the pore water to levels of 8 to 9 (ACI 201-2R
1992). In time, this will destroy the alkalinity of the surface zone concrete, and if this reaches the level of the reinforcement, it will render the steel vulnerable to corrosion in the presence of moisture and oxygen (Mosley et al. 1996). The rate of carbonation depends on the quality of the concrete, the ambient humidity and the concentration of CO₂ in the environment.

Field and laboratory studies indicate that the primary cause of deterioration of many reinforced structures in the world is attributed to the presence of chloride ions. Free chloride ions within the capillary pore water are known to increase both the electrical conductivity and solubility of the passive film that normally forms on the surface of steel in concrete. At the end of the initiation stage of the corrosion process, the passive film is destroyed or damaged and a corrosion cell is established. This signifies the onset of the corrosion growth stage (ACI 201-2R 1992).

Chloride ions can originate from the use of local “tap water” to mix the concrete, chloride contaminated aggregates, sea water spray and splash, salt bearing soil and ground water, marine salts carried by the wind, and possibly brackish curing water (Novokshchenov 1995). Chlorides can also originate in the constituents of a concrete mix, e.g. the cement, aggregate, admixtures, but are normally limited to below 0.35% total chloride ion by weight of cement by existing standards, BS 8110-1 (1985). This total chloride ion is the total of water-soluble chloride ions and the acid-soluble chloride. Moreover, the acid-soluble chloride is bounded by the cement and it can not be moved away. Whereas water-soluble chloride ions will be free and it is more easy to calculate, however, it is the more dangerous ions that attacking reinforced concrete. According to ACI 222R (1996), the presence of water-soluble chloride ions above threshold levels of 0.2 percent by mass of Portland cement, can accelerate the corrosion process.

A proportion of this chloride reacts with C₃A in the hardened cement paste, and becomes chemically bound, as calcium mono-chloro-aluminate, known as Friedel’s. The beneficial effect of higher C₃A contents in the increase of corrosion protection by retarding chloride-induced reinforcement corrosion has been known for some time (Roberts 1962; Algahtani et al. 1994). The remaining fraction of this initial chloride content “the unbound portion” is available as free chloride ions, the proportion being critically depended on the cement type. It can be seen that SRPC is much less able to
bind the initial chloride ions than OPC and, therefore, the limit on allowable total chloride ion content is reduced (Verbeck 1968; Novokshchenov 1995 and Rasheeduzzafar 1992).

The chloride content of Portland cement and PFA is typically very low; however, GGBS may have significant chloride content if quenched with salt water. These internal chlorides may be released into the free water during mixing but they become combined into the reaction products of the hydration process (ACI 201-2R 1992). As a consequence, cement blends can have significantly lower free chloride ion content than equivalent OPC concretes with the same total chloride ion content.

An important source of chloride ions is those arising from the environment. Such chlorides are by definition free chlorides ions, present in the solution, and they can move into and through the layer of cover concrete by one of three main mechanisms:

(i) Diffusion: This occurs where the capillary pores of the concrete are effectively saturated with water. As a consequence, chloride ions outside the concrete must diffuse into the saturated cover concrete. The driving force for this movement is the concentration gradient between the outside surface and the centre of the concrete.

(ii) Capillary absorption: Under alternate wetting and drying conditions, significant capillary uptake of water can occur in the outer 50 mm of a concrete surface. Where the water absorbed into the drying capillary pores contains chloride ions, they will be left deep in the cover concrete after only one wet/dry cycle. Subsequent wet/dry cycling can give rise to the rapid build-up of high chloride ion levels deep in the concrete. It has been estimated that chloride ingress by capillary absorption is more rapid than that occurs by permeability.

(iii) Flow under pressure: Concrete is a permeable material and, if in contact with water under pressure, it will allow the passage of water. If the water, which flows, is contaminated with chloride ions then chloride concentrations can build-up to high levels (Hobbs and Matthews 1998).

A major cause of chloride intrusion into concrete is ascribed to the presence of cracks, which allow infiltration at a much faster rate than by the slower diffusion processes, and establish chloride concentration cells that can initiate corrosion. Crack formation
can be reduced by the use of good quality concrete having low permeability and absorption and reduced risk of corrosion (ACI 201-2R 1992). Of course, in real structures chloride ingress occurs by a combination of these processes.

It may be noted that if there is a uniform distribution of chlorides, corrosion may be minimal. However, even if there is a uniform distribution of chlorides, significant corrosion can result because of differences in oxygen and moisture contents. It is an important observation that even in the presence of free chloride ions when concrete is kept moderately dry, corrosion of the steel can be minimised (ACI 201-2R 1992).

Additionally, it has been reported that the corrosion risk is likely to be significantly increased in circumstances where concrete is exposed to both chloride and sulphate salts (Al-Amoudi & Maslehuddin 1993). Treadaway et al. (1987) indicated that the reinforcement corrosion in the specimens admixed with sodium chloride plus sodium sulfate was higher than that for the specimens admixed with only sodium chloride (Al-Amoudi and Maslehuddin 1993; and Al-Amoudi et al 1992a, 1994; Dehwah et al 2002). This is thought to be because when both chloride and sulfate ions are present, C\textsubscript{3}A reacts preferentially with sulfate ions, as a result the proportion of free chloride ions becomes more than that when only chlorides salts are present (Dehwah et al 2002). This is believed to be the major cause of the accelerated corrosion observed in the Arabian Gulf countries (Al-Amoudi et al 1994).

2.5.2 INFLUENCE OF LOCAL ENVIRONMENT ON CORROSION PROCESS

The Nernst equation, Section 2.4.1, carries several important implications for the corrosion of metals under non-uniform conditions and indicates that a potential difference will exist between areas in electrical contact and in the presence of a common electrolyte if either a temperature gradient or difference in the material or its porosity and permeability exists. Such a difference has the potential to cause the formation of adjacent anode and cathode sites.

This has implication for steel bars in non-uniform environments, such as the use of steel reinforcement in SRPC concrete piles that are in direct electrical contact with reinforcement in OPC concrete capping beams or columns. In theory, steel
reinforcement passing through the interface between the SRPC piles and OPC crossbeams will have different potentials along their length. This could, under wet conditions, give rise to the formation of stable anodic sites and associated corrosion currents. An example of the effect that different environments have on the corrosion activity of the embedded reinforcement was reported by Chadwick (1993). In his work, Chadwick embedded three steel bars in a particular material and then applied a repair material around the central bar, as demonstrated in Figure 2.8. The performance of the interface between the parent concrete and patch material was shown to be critical (Chadwick 1993). Results reported that the presence of intense anodic regions was found at the interface on the bars extracted at the end of the exposure period. Initiation of reinforcement corrosion was much earlier for the repaired specimens than would have been expected from the results of the usage of one material (Chadwick 1993).

Figures 2.9 and 2.10 display the influence of concrete quality on the formation of anode and cathode sites in reinforced concrete samples repaired with a commercial epoxy resin. Figure 2.9 reports the potential measurements of the steel bars in Figure 2.8 when good quality concrete was used in the specimen that was repaired with the resin. In this Figure, the potential measurements of bar B was observed to be less negative than the of bars A and C, which means that bar B acted as a cathode to bars A and C.

However, when the quality of concrete was degraded, the potential of bar B became more negative than those of bars A and C (as shown in Figure 2.10), indicating that bar B became the anode to bars A and C. Therefore, the effect of local environment on the corrosion process becomes of great significance when the reinforcing steel bars are embedded in two (or more) dissimilar materials.
Figure 2.8 Specimen geometry used by Chadwick. Note bars A and C embedded in mother material while bar B surrounded by repair material, after Chadwick 1993.

Figure 2.9 Lightweight epoxy mortar repair applied to good quality OPC (0.45) concrete, after Chadwick 1993.
Figure 2.10 Lightweight epoxy mortar repair applied to moderate quality concrete OPC (0.6), after Chadwick 1993.
CHAPTER 3. EXPERIMENTAL DETAIL

3.1. GENERAL

The main purpose of this study was to examine the electrochemical behaviour of reinforcing steel embedded in different combinations of OPC, SRPC, OPC/PFA and OPC/GGBS concretes for samples tested up to an age of 5 years. The influence of different environments on the results for these mixes was investigated. The study also included a preliminary investigation up to 90 days on OPC and OPC/GGBS mortar specimens.

Tests have also been carried out on a range of un-reinforced mixes to determine density, pulse velocity and compressive strength in order to assess the general quality of the concrete. Initial surface absorption testing was carried out to assess differences in absorption between the various concrete mixes.

3.2. MATERIALS USED

3.2.1 Cementitious Materials

The ordinary Portland cement (OPC) used in this investigation was complying with (BSI 12 1996 and supplied by Blue Circle Cement Ltd. Sulphate-resisting Portland cement (SRPC) complying with BSI 4027 1996. Ground granulated blast furnace slag (GGBS), complying with BS 6699 1992, was supplied by Civil and Marine Slag Cement Ltd. Pulverised fuel ash (PFA) complying with BS 3892-1 1997 was supplied by Ash Resources Ltd.

Chemical Composition of OPC, SRPC, GGBS and PFA cements are presented in Table 3.1.
<table>
<thead>
<tr>
<th>Compound</th>
<th>OPC (Wt %)</th>
<th>SRPC (Wt %)</th>
<th>GGBS (Wt %)</th>
<th>PFA (Wt %)</th>
</tr>
</thead>
<tbody>
<tr>
<td>*SiO₂</td>
<td>20.5</td>
<td>19.8</td>
<td>36</td>
<td>48.7</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>6.8</td>
<td>5.2</td>
<td>10</td>
<td>27.8</td>
</tr>
<tr>
<td>*CaO</td>
<td>64.1</td>
<td>67.2</td>
<td>40</td>
<td>3.0</td>
</tr>
<tr>
<td>MgO</td>
<td>1.1</td>
<td>1.3</td>
<td>8</td>
<td>1.9</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>2.9</td>
<td>4.8</td>
<td>0.5</td>
<td>9.2</td>
</tr>
<tr>
<td>SO₃</td>
<td>2.4</td>
<td>1.9</td>
<td>0.2</td>
<td>0.9</td>
</tr>
<tr>
<td>Na₂O</td>
<td>0.85</td>
<td>0.98</td>
<td>0.4</td>
<td>1.3</td>
</tr>
<tr>
<td>K₂O</td>
<td></td>
<td></td>
<td>0.7</td>
<td>2.4</td>
</tr>
<tr>
<td>TiO₂</td>
<td></td>
<td></td>
<td>0.6</td>
<td>1.1</td>
</tr>
<tr>
<td>P₂O₅</td>
<td></td>
<td></td>
<td></td>
<td>0.3</td>
</tr>
<tr>
<td>Mn₂O₃</td>
<td></td>
<td></td>
<td>0.6</td>
<td>0.1</td>
</tr>
<tr>
<td>C</td>
<td>0.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S²</td>
<td></td>
<td></td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

Table 3.1. Chemical Composition of OPC, SRPC, GGBS and PFA cements, (*All to ±0.3, rest to ±0.1 or better)

3.2.2 AGGREGATES and SAND

Thames Valley flinty gravels of 10 mm particle size and uncrushed river sand were used. The water absorptions were 3.6% and 1.8% for the aggregate and sand, respectively.

3.2.3 WATER

Tap water was used in the preparation of the various mixes, whereas, distilled water was used in the preparation of the ponding solutions.

3.2.4 MILD STEEL

8mm diameter mild steel bars were supplied by Acton Bright Steel.
3.2.5 SUPERPLASTICISER

The superplasticiser used throughout this investigation was SIKAMENT 10 (S10). S10 is a vinyl copolymer based formaldehyde-free superplasticiser. Superplasticisers are water reducing agents. In the experiment it was used in the blended concrete mixes with the lower w/b to increase the workability of standard mixes without affecting w/b ratio (Aitcin et al 1994; Sika Ltd 1996).

3.3. MIX DESIGN

Four mortar mixes were used in the preliminary testing, with 5:1 or 4:1 (sand: cementitious materials ratio) and a w/b ratio of 0.9 or 0.32, Table 3.2. The cementitious materials used in those mixes were 100% OPC (OPC Mix) or a blend of 30% OPC and 70% GGBS (OPC/GGBS Mix), Table 3.2.

<table>
<thead>
<tr>
<th>Mix</th>
<th>OPC</th>
<th>Blends</th>
<th>Sand</th>
<th>w/c</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC (0.9)</td>
<td>1.00</td>
<td>-</td>
<td>5.00</td>
<td>0.9</td>
</tr>
<tr>
<td>OPC/GGBS (0.9)</td>
<td>0.30</td>
<td>0.70</td>
<td>5.00</td>
<td>0.9</td>
</tr>
<tr>
<td>OPC (0.32)</td>
<td>1.00</td>
<td>-</td>
<td>4.00</td>
<td>0.32</td>
</tr>
<tr>
<td>OPC/GGBS (0.32)</td>
<td>0.30</td>
<td>0.70</td>
<td>4.00</td>
<td>0.32</td>
</tr>
</tbody>
</table>

Table 3.2 Mix Design for Mortar Mixes (Proportions by weight)

Five concrete mixes were investigated. The mix proportions for these mixes are presented in Table 3.3. The maximum aggregate size used in these mixes was 10mm.

The concrete mix with the w/c ratio of 0.45 was considered as the reference (control) mix in this investigation, while the concrete mix with the higher w/c ratio of 0.6 and higher aggregate to cement ratio was selected to study the role of concrete quality (i.e. w/c ratio) in the corrosion performance upon exposure to chloride media, Table 3.3. The third concrete mix was exactly similar to the reference mix, however, the type of cement was replaced by SRPC to study the role of cement type (i.e. C3A content) in the durability of reinforced concrete in chloride and sulphate exposures.

In blended cements, the parent cement (OPC in the this investigation) was replaced with 30% PFA and 70% GGBS. Such replacements degrade the quality of blended cement concretes
during the early period of curing (i.e. 14 to 28 days) and, therefore, the w/b ratio of these cements was lowered to 0.4 in this investigation (as compared with 0.45 for the parent cement) to upgrade the quality of blended cement concretes. Trial mixes had been conducted on blended cement concrete mixes indicated that lowering the w/b ratio was not enough to improve their performance to the level of the reference concrete. Therefore, the sand content in these mixes was decreased and simultaneously the aggregate content was increased, as shown in Table 3.3, to upgrade the performance of these mixes. In fact, the mix design implemented in this investigation was exactly similar to that executed by Chadwick (1993), which was based on the requirements of the UK Department of Transport (DT BD27 1986).

In order to improve the workability of these blended cement mixes, S10 superplasticizer was added to these mixes at a ratio of 1% by total weight of the cementitious materials.

<table>
<thead>
<tr>
<th>Mix</th>
<th>OPC</th>
<th>Blend</th>
<th>Sand</th>
<th>Aggregate</th>
<th>w/b</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC (0.6)</td>
<td>1.00</td>
<td>-</td>
<td>4.50</td>
<td>2.80</td>
<td>0.60</td>
</tr>
<tr>
<td>OPC (0.45)</td>
<td>1.00</td>
<td>-</td>
<td>2.15</td>
<td>1.75</td>
<td>0.45</td>
</tr>
<tr>
<td>SRPC (0.45)</td>
<td>1.00</td>
<td>-</td>
<td>2.15</td>
<td>1.75</td>
<td>0.45</td>
</tr>
<tr>
<td>OPC GGBS (0.4)</td>
<td>0.30</td>
<td>0.70</td>
<td>1.50</td>
<td>2.85</td>
<td>0.40</td>
</tr>
<tr>
<td>OPC/PFA (0.4)</td>
<td>0.70</td>
<td>0.30</td>
<td>1.50</td>
<td>2.85</td>
<td>0.40</td>
</tr>
</tbody>
</table>

Table 3.3 Mix Design for Concrete Mixes (Proportions by weight)

3.4. SAMPLE PREPARATION AND CURING

Mixing was carried out in accordance with BS 1881: Part 125: 1986. After mixing, samples were cast into the moulds in accordance with BS 1881: part 108: 1983.

Steel bars were initially cleaned using abrasive cloth to remove the existing surface rust. The bars embedded in prism specimens with interface were then prepared by dipping the coated side twice within 2-3 hours in cement paste of the same composition as that to be used in the manufacture of the mix. After the drying of the cement paste coating, 2 application of a high quality marine varnish was used to seal the surface of the steel bar. The finished coating thickness was of the order of 3-5 mm. For the reinforced concrete samples, the steel bars were then carefully embedded into the fresh mix.
To protect the mixed samples from moisture losses whilst hardening, samples were left in a fog room, maintained at a constant temperature of 20°C ± 2°C and 99% relative humidity. All samples were demoulded approximately 24 hours later and left to cure in the fog room up to an age of 14 days.

The prism samples used were prepared slightly different, Figures 3.1 to 3.3. These samples are essentially rectangular prisms (295mm x 165 mm x 100mm) with a recess cast into the top with 15mm depth, Figure 3.4, to allow ponding to take place. Where half of the mix had to be cast in the first day, and the other half was cast approximately 24 hours later, Figure 3.2. These mixes were cured for 28 days.
Figure 3.2. Half cast prism specimens.

Figure 3.3. Back view of the prism specimens showing the position of the interface.
3.5. CORROSION STUDY

The experimental program for the electrochemical measurements was divided into the following two main parts:

3.5.1 PRELIMINARY INVESTIGATION

Preliminary tests were carried out on reinforcing bars embedded in cylindrical and cubic mortar samples. The aim was to compare the effect of using a blend of 30% OPC and 70% GGBS (OPC/GGBS) to that of a 100% OPC (OPC) cement on the bar potential over a period of 90 days. The influence of varying the w/c ratio was also investigated. In these tests, the specimens were partially immersed in tap water.

The cylindrical specimens (70mm diameter and 70mm deep), Figure 3.5, were cast with a high w/c (0.9), Table 3.1, in order to facilitate a high rate of steel corrosion. For each mix, two cylinder specimens were cast with one steel bar embedded centrally. 10mm cover depth was used in these specimens, Figure 3.5.
In the cubic mortar specimens, a w/c of 0.32 was used, Table 3.2. For each mix two standard 100mm cubes were cast with two steel bars embedded in each cube, Figure 3.6. The cover depth to the steel bars was 25mm.
3.5.2 MAIN INVESTIGATION

3.5.2.1 GENERAL

The main corrosion study was carried out to compare the corrosion potential of five different concrete mixes containing different cementitious materials, OPC (0.60), OPC (0.45), SRPC (0.45), OPC/GGBS (0.4) and OPC/PFA (0.4).

The influence of exposure environment on the corrosion measurements of the steel embedded in the various concrete mixes was also assessed. A solution of 1 M sodium chloride (5.8% NaCl) was used as partial immersion solution for all concrete cubes and as ponding solution for the entire prism specimens. Additionally, a solution 0.1M of sodium sulphate (1.4% Na$_2$SO$_4$) was used as partial immersion solution for the single and connected bars embedded in OPC (0.45) and SRPC (0.45). Finally, a sabkha environment was used as ponding solution for OPC (0.45) and SRPC (0.45) concrete prisms with an interface.

The sabkha environment is found in the Gulf States and the eastern region in Saudi Arabia (Al-Amoudi and Maslehuddin 1993). It consists of a solution with ranges between 0.55% and 2.1% sulphate and 15.7% chloride ions (Smith 1980). In this investigation, only one OPC (0.45) and SRPC (0.45) prism specimens was ponded in a sabkha environment consisting of 2.1% sodium sulphate and 15.7% sodium chloride ions.
PRELIMINARY INVESTIGATION

MORTAR MIX
OPC and GGBS

CYLINDERS
OPC (0.9) and OPC/GGBS (0.9)

SINGLE
CONNECTED

4 DAYS CURING

PARTILLAY IMMERSED IN TAP WATER DURING THE MEASUREMENTS

MAIN INVESTIGATION

CONCRETE MIX
OPC, SRPC, GGBS, PFA

CUBES
OPC (0.6), OPC (0.45), SRPC (0.45), OPC/GGBS (0.4), OPC/PFA (0.4)

SINGLE
CONNECTED

14 DAYS CURING

EXPOSED TO DRY/WET CYCLES, PARTILLAY IMMERSED ALL IN 1 M NaCl + TWO IN 0.1 M Na₂SO₄

PRISMS WITH INTERFACE
OPC (0.6), OPC (0.45), SRPC (0.45), OPC/GGBS (0.4), OPC/PFA (0.4)

SINGLE
CONNECTED

28 DAYS CURING

EXPOSED TO DRY/WET CYCLES, PONDED NINE IN 1 M NaCl + ONE IN SABKHA

8 DIFFERENT CONCRETE COUPLES

Table 3.4 Diagram shows the specimen type, geometry, curing regime and exposure condition.
3.5.2.2 TESTING PROGRAM

The testing program for these mixes was organized into two stages. The first stage was testing the steel bars embedded in every concrete cube with a 25mm cover, Figure 3.6. Each cube was made with the same cement type, hence there was no interface between mixes made of different cementitious materials. These steel bars were divided up on their testing arrangements into two types, single or connected, Figures 3.6 and 3.7 respectively. Single bars represent no electrical connection and were used in order to monitor the change in half-cell potential with time in a single cement environment, whereas, connected bars were used to investigate the effect of the connection on anode formation. By allowing the bars to remain connected, a current can flow and the resulting equilibrium potential of the couple can be measured.

![Figure 3.7. Connected steel bar samples](image)

The final testing stage was carried out on single steel bars embedded in concrete specimens, Figure 3.8. These specimens were designed with a direct interface, Figure 3.1 to 3.4, between one or two different types of cements. This was aimed to reflect the real design situation in the Arabian Gulf States where SRPC concrete is widely used in the foundations and OPC concrete in the above ground beams and columns. The steel bars embedded in the prism specimens were placed at depths of 35mm, 25mm or 12mm from the surface. These bars were coated at each end for 75mm leaving 100mm exposed centre, as shown in Figure 3.8.
Figure 3.8 Schematic diagram of prism ponding specimen showing location of reinforcing steel bars.
3.5.3 REINFORCEMENT CORROSION MEASUREMENT METHODS

The high level of problems caused by corrosion of steel reinforcement in concrete has led to the need for tests which can identify when and where aqueous corrosion is active without exposure of the steel or destroying the core concrete (Stephen 1996).

Two experimental techniques were employed in this investigation in order to study the corrosion behaviour of steel bars embedded in different mixes, as discussed in Sections 3.5.3.1 to Section 3.5.3.2.

3.5.3.1 HALF-CELL POTENTIAL MEASUREMENTS

3.5.3.1.1 General

Due to the electrochemical nature of corrosion of steel in concrete, electrochemical methods are well suited for the inspection and monitoring of concrete structures (Elsener et al 1994). The simplest assessment technique for rebar corrosion damage is the measurement of the corrosion potential (half cell potential) (Tullmin et al 1996). The data obtained can provide early warning of structural distress and for evaluating the effectiveness of corrosion control strategies that may have been implemented. Once rebar corrosion has proceeded to an advanced state, where its effects are apparent on the outside concrete surface, it is too late to implement effective remedial measures such as cathodic protection or chloride extraction treatments (Tullmin et al 1996).

The half-cell method utilises the fact that when steel corrodes, it forms anodic and cathodic activity at sites throughout the reinforcement. Metal is dissolved at the anodes where the oxidation of metal ions takes place, and metal oxide corrosion products are subsequently formed. Simultaneously, electrons are released and transferred through the metallic reinforcement (flow of electrical current) to a cathodic site where a reduction process occurs. The reaction typically involves the electrons, water and oxygen, which together produce hydroxyl ions within the pore solution the concrete. Associated with the flow of current is a variation in electrical potential along the length of the corroding reinforcement, which may be detected with a suitable half-cell electrode. For steel in concrete, anodic regions normally correspond to more negative potentials whilst cathodic regions correspond to less negative
potentials (Stephen 1996). The measurement of half-cell potential at the surface of the concrete is one common and standard method of estimating the degree of corrosion in reinforced concrete (Stephen 1996). Also Work carried out by the California Division of Highways, and subsequent research at the Transport & Road Research Laboratory at Crowthorne have referred to a standard 'half-cell' to determine the electrical potential of steel reinforcement (Stephen 1996; ACI 222R 1996).

ASTM C 876 (1991) describes the relation between the potential of the reinforcement and the probability of active corrosion, the basis of which is that the corrosion potential of the rebar will shift in the negative direction if the surface changes from the passive to the active state (Gowers and Millard 1999a). A measurement procedure is described in ASTM C 876 (1991). The simplified interpretation of the potential readings is presented in Table 3.5 (Tullmin et al 1996). Using Half-cell technique is possible to cover large concrete surface areas in short time frames. However, the results obtained from this technique are only qualitative, without the establishment of an actual rebar corrosion rate.

At times, very negative rebar potential readings can be misleading and may be recorded in submerged concrete, where the corrosion rate remains negligibly low due to lack of oxygen stifling the cathodic reaction rate (Tullmin et al 1996).

The values, which are obtained experimentally, are dependent upon the moisture content of the test specimen. As corroding steel reinforcement in concrete forms a half-cell, the steel may be connected to a copper/copper sulphate electrode where the potential values obtained can give an indication of the probable risk of reinforcement corrosion. By convention, potentials are considered negative when measuring the steel with respect to the copper. Present criteria indicate that the potential of the copper/copper sulphate 'half cell' as referred to a Standard hydrogen electrode is -316 mV at 22° C and the temperature correction is -0.9 mV per C (Stephen 1996). Experience in the US suggests that the following values of potential (E), with respect to a copper/copper sulphate electrode, are generally accepted as indicating the possibility of corrosion of steel reinforcement in OPC concrete, Table 3.5.
Table 3.5 Likelihood of Corrosion Damage as a Function of Corrosion Potential, after ASTM C 876 1991; ACI 222R 1996.

It should be noted that these values were typically taken for Portland cement concretes made with w/c ratios in the range 0.4-0.6 and tested at ambient temperature. UK experience suggests that, while these figures are a useful guideline, there can be significant correlation between increasing (negative) potential in the -200 to -350mV range and the occurrence of reinforcement corrosion, i.e. the -350mV value may perhaps be considered to represent the presence of significant reinforcement corrosion. However, it is probable that the onset of corrosion occurs at rather lower (more positive) potentials. This appears to be confirmed by recent work for the American Federal Highways Administration, which suggests that corrosion of steel commences at a potential in the region of -240mV. It has been found that the potential ranges typical of active corrosion and the passivity vary depending on the exposure environment (Elsener et al 1994; Gowers and Millard 1999a). For in-situ measurement of half-cell potential, it is more common to use Copper/Copper Sulphate (Cu/CuSO₄) electrode (CSE) and convert the half-cell potentials measured to values relative to the Standard Hydrogen Electrode. The Copper/Copper Sulphate half-cell has a Standard Electrode potential \( E_f = 0.32 \) Volts relative to the Standard Hydrogen Electrode. This has generally been adopted for site work in the United Kingdom (Stephen 1996). It has also been used in the US (ACI 222R 1996).

3.5.3.1.2 Experimental Technique

A schematic diagram of the 'half-cell' potential equipment used in this study is presented in Figure 3.9. The essential details are a tube full of saturated copper sulphate solution with a copper electrode (99.9 % pure) immersed in the solution. This electrode passes through a seal at one end of the cell. Sinking a sintered glass membrane (electrode) in contact with a damp sponge is used to provide the electrical contact to the concrete surface. The other end of the cell was then closed by connecting a steel bar to a high-impedance voltammeter by a flexible wire. Measurements were then taken by pressing the sponge against the surface of the
specimens and recording the meter readings. In order to ensure that sufficient electrolyte is present at the surface and to get the accuracy of the method all locations are pre-wetted (Stephen 1996; ACI 222R 1996).

![Diagram of a Copper/Copper Sulphate Half-Cell for Measuring Rebar Corrosion in the bonding specimen.]

After curing, specimens were kept for one-week in the laboratory to dry. Therefore, they were partially immersing in the exposure solutions for one week. The level of solutions surrounding the cubes was maintained at 20mm, whilst the level was maintained at 15mm on the top of prisms. The one week drying and one week wetting is considered to be one cycle. Potential measurement was carried out at the end of every wet cycle for nearly one year (25 cycles). The use of these cycles represents a (Splash Zone), which is considered to be one of the most severe corrosion accelerated techniques (Concrete Society 1987). It is worth remembering that a conducting medium was always required. This was achieved by placing the various specimens in a common electrolyte.

In the preliminary stages of the investigation, Section 3.5.1, potential measurements for single and connected steel bars embedded in OPC and OPC/GGBS mortar were measured. The first potential measurement was made after three days of curing. Samples were left partially immersed in tap water for the duration of testing. Measurements were taken every few days. The results are presented in Section 4.2.1.

The main investigation, Section 3.5.2, included the testing of the single, connected and concrete specimen with interface. However, in the case of the connected bars, the potential
differences became approximately zero and, therefore, the bars were disconnected for a period of five minutes before taking an individual bar potential measurements. This enabled the determination of the cathodic and anodic bars. After the measurements were taken, the bars were reconnected for the next cycle. The results are presented in Section 4.2.2.

3.5.3.2 CORROSION CURRENT MEASUREMENTS

3.5.3.2.1 General

Half-cell potential measurements are an indirect method of assessing potential bar corrosion, but there has been much recent interest in developing a means of performing perturbative electrochemical measurements on the steel itself to obtain a direct evaluation of the corrosion rate (Gowers and Millard 1999a). Corrosion rates have been related to electrochemical measurements based on data first reported by Stern and Geary (1957). If the potential measurements indicate that there is a high probability of active corrosion, concrete resistivity measurement can be subsequently used to estimate the rate of corrosion. This was also stated from practical experience (Figg and Marsden 1985 and Langford and Broomfield 1987).

Classifications of the severity of rebar corrosion rates are presented in Table 3.6. However, caution needs to be exercised in using data of this nature, since constant corrosion rates with time are assumed. Rebar corrosion rate data measured on a particular structure can also be correlated to other parameters such as chloride concentration profiles to determine critical chloride levels, which are useful for life prediction of a particular structure (Tullmin et al 1996). Typical polarisation resistances, corrosion rates and penetration rates for steel in concrete are shown in Table 3.6 (Gowers and Millard 1999a; Andrade and Alanso 1996). It is important to realise that a high precision and repeatability for any corrosion rate measurement of steel in concrete in the field cannot normally be expected. Each category in Table 3.6 is an order of magnitude different from the adjacent category. An electrochemical corrosion rate measurement should place the element under study into the appropriate category but small differences in corrosion rates within a particular category should not be deemed to be significant. Corrosion is a dynamic process subject to fluctuations and the interpretation of corrosion rate measurements should focus on the order of magnitude rather than the precise value obtained (Gowers and Millard 1999a).
There are five different electrochemical techniques used to measure the current rate, namely potentiostatic linear polarisation resistance (LPR), galvanostatic pulse transient analysis, AC impedance, harmonic analysis and electrochemical noise (Gowers and Millard 1999a). The simplest of these techniques is the LPR measurement. Stern and Geary (1957) showed theoretically that Ohm’s law \( V = I \cdot R \) holds approximately if the polarisation of a corrosion interface is constrained to be within +30 mV of the rest potential. If a small voltage perturbation \( \Delta E \) is applied to steel embedded in concrete, then the resultant current flow \( \Delta I \) can be used to calculate a resistance (Millard et al 1992) known as the polarisation resistance, \( R_p \), which is a measure of the resistance at the steel-concrete interface:

\[
R_p = \frac{\Delta E}{\Delta I}
\]

Where:
\( \Delta E \) = change in the potential (mV),

\( \Delta I \) = change in the current (\( \mu A/cm^2 \)), and

\( R_p \) = polarisation resistance (k Ohm cm\(^2\)).

This LPR measurement technique is known as a potentiostatic measurement and is shown schematically in Figure 3.10 (a). An alternative means of carrying out an LPR measurement is to apply a small current perturbation \( I_{app} \) and to measure the resulting change in the potential, \( \Delta E \). This method is known as a galvanostatic LPR measurement and is shown in Figure 3.10 (b).
The polarisation resistance \( (R_p) \) controls the rate of corrosion \( (I_{corr}) \), which may be calculated using the following equation (Gowers and Millard 1999a; Stern and Geary 1957; Andrade et al 1984; Moosavi et al 1994).

\[
I_{corr} = \frac{B}{R_p}
\]

Where:

- \( I_{corr} \) = rate of corrosion (\( \mu A/cm^2 \)),
- \( B \) = Tafel constant normally assumed to be 25-26 mV for active corrosion, and
- \( R_p \) = polarisation resistance (k-Ohms cm\(^2\)).

One practical difficulty with using the LPR method to assess the corrosion rate of reinforcing steel in a concrete structure is to know what is the surface area of the reinforcing steel, which is being perturbed by the measurement. Another difficulty is in deciding on a suitable time period to allow for equilibration to occur after applying the perturbation, i.e. deciding exactly when to take a measurement. These practical difficulties are both good reasons for not expecting a high precision in any corrosion rate measurement in the field (Gowers and Millard 1999a).
3.5.3.2.2 Experimental Techniques

(a) Portable Corrosion Analyser (Aston University)

The wet and dry cycles, Section 3.5.2.1, were continued on ten concrete specimens for one year. At the end of this period, current and polarisation resistance measurements were taken at Aston University concrete laboratory using a Linear Polarization Resistance (Portable Corrosion Analyser) with saturated Calomel electrode reference (SCE) (Ag/AgCl). The average of two measurements is summarized in Table 4.5.

The length of exposed bar embedded in the specimen was 10 cm, steel bar diameter (0.8 cm) with an area of 25.143 cm². The method used an automated, computer controlled, measurement system incorporating a voltage sweep of 40 mV (+20 to -20 mV) over a period of 6 minutes. The output from each measurement was as follows:

(i) E (I= 0) potential (mV),
(ii) \( R_p \): Polarization Resistance (k-Ohms cm²), and
(iii) \( I_{cor} \): rate of corrosion (µA/cm²).

The corrosion current \( I_{cor} \) could be calculated from the Stern and Geary equation.

\[
I_{corr} = \frac{B \cdot A}{R_p}
\]

Where:

\( I_{corr} = \) rate of corrosion (µA/cm²),
\( B = \) Tafel constant normally assumed to be 25-26 mV for active corrosion,
\( A = \) surface area of exposed bar (cm²), and
\( R_p = \) polarisation resistance (k-Ohms cm²).
After testing the ten concrete specimens at Aston University, Section 3.5.3.2.2 (a). The specimens were then stored in the Concrete Laboratory, University of Surrey, for further 4 years. At the end of this period, potential and current measurements were taken using a Ministat 401 Potentiostat (supplied by Thompson Electrochem Ltd.), as shown in Figure 3.11. The average of two measurements is summarized in Table 4.6.

When an electrode is displaced from its rest potential, i.e. polarized, a current is produced. The magnitude of polarization and current density is linear. The procedure for measuring the polarization current was as follows. The bar that will be measured for corrosion current was connected as the working electrode (WE). The nearest bar on the same depth was connected to other wire as auxiliary electrode (AE) that helps to complete the electric circuit. After putting the switch in the "ISOLATE" position, the rest potential of the first bar was determined and the current rest reading was recorded. Then the potentiostat was adjusted to impose a potential increase of (20mV) relative to (AE). After putting the switch in the "RUN" position, the resulting current was allowed to decay to a steady reading before being recorded. The decay period was taken to be about 10 seconds.

During the polarization period, the auxiliary electrode was disturbed from its original rest potential. Current readings were taken at the same intervals as the rest potential measurements during the measurements for each steel bar embedded in concrete specimen. Once a current reading had been obtained, the corrosion current $I_{corr}$ could be calculated from the Stern and Geary equation.

$$I_{corr} = B \times \frac{(I_r - I_i)(E_r - E_i)}{(1 \times 10000/A)} = mA/m^2$$

Where

- $B = $ Tafel constant normally assumed to be 25-26 mV for active corrosion,
- $I_r = $ Current after 10 seconds decay period (mA),
- $I_i = $ Rest current (mA),
- $E_r = $ Potentials (mV) at $I_r$, and
- $E_i = $ Rest potential (mV), and
- $A = $ Area of the exposed bar $= 25.143 \ cm^2$, $1 \times 10000/A = 398$
3.6. DESTRUCTIVE EXAMINATION

At the end of 5 years of casting, the ten specimens with an interface were broken open. The purpose of this examination was to establish the condition of the embedded steel bars and, if possible, quantify the extent of the corrosion, and to determine the effect of cement type on the location of the anodic areas for the steel bar embedded in each couple.

3.6.1 VISUAL EXAMINATION

The specimens were fractured using a small electric breaker, the bars were retrieved and their condition recorded photographically in Section 5.4.4.1.

3.6.2 CHLORIDE ION PROFILES (CONTENT)

At the end of 5 years, chloride ion profiles were obtained by drilling dust "powder"samples from all concrete specimens at the ponding surface around the steel bars with 25 mm depth. Samples were taken from each specimen using a rotary hammer drill. Two samples were taken for each couple, one sample from each concrete type. All the extracted dust samples were weighted, and then Nitrate acid solution were added to each sample and reweighed to determine the weight of the solution added. The samples were kept in for 24 hours before
analysing the Cl\textsuperscript{-} content. Ca (OH)\textsubscript{2} were added to each sample then the Cl\textsuperscript{-} content analysis was made by using Quantab Strips until the indicator bar showed that the test was complete. The concentration of chloride in the solution was obtained by reading off the scale in the capillary on the test strip (BS 1881-8 1983). An average was taken for the samples from each cement type. The results of chloride content for each concrete are presented in Section 5.4.4.2 and Table 5.6. A sample calculation of the chloride test is presented Appendix 9.1.

3.7. PROPERTIES OF THE HARDENED CONCRETE

The purpose of this section is to evaluate the performance of a range of different concrete mixes that were used in the long-term study to assess their ability to protect embedded steel against chloride induced corrosion. A series of tests were carried out on each concrete mix, Table 3.2. Three standard 100 mm cubes were cast from each mix to determine relative density, pulse velocity and compressive strength. However, three to twelve cubes were cast from each mix to determine compressive strength. Finally two cubes from each concrete were tested for initial surface absorption test. These physical property tests were carried out on the above concretes to indicate if any of the physical properties had an important influence on durability performance and corrosion protection.

The oven dried 5 mm sand and 10 mm aggregate were batched by weight and allowed to soak in the added water for 24 hours prior to mixing. Mixing was carried out for two minutes in a conventional drum mixer. The concrete mixes were compacted in two layers by using a vibrating table and were then cured at 99% RH and 20° ± 2°C prior to testing at 7, 28, 90, and 400 days. The average of these test results for the five concretes was recorded in Table 4.2 and Table 4.3.

3.7.1 COMPRESSIVE STRENGTH

Standard concrete cubes (100mm) were cured until testing. The concrete cubes were tested for each mix at different time intervals in the range 7-400 days using a Denison compression-testing machine. The cubes were placed between steel platens and tested to failure in direct compression in accordance with BS 1881 Part 116 (1983).

The hydraulic testing machine is loaded to a maximum load of 600 kN or 1200 kN. The load is applied, and the strength was found and the failure was observed. All failures were
observed to be normal. The average compressive strength results are presented in Table 4.2 and Figures 4.1 to 4.5.

3.7.2 DENSITY

The densities of the cubes were determined in accordance with BSI 1881 Part 114 (1983). At the time of testing, after removing the cube from curing room, it was placed in an electronic balance, and then the weight reading of the cube in air was taken. Afterwards, each cube was placed in a balance that gave the weight under water. Its density (kg/m$^3$) was then calculated using the following formula:

$$\text{Density} = \frac{W_{\text{air}}}{W_{\text{air}} - W_{\text{water}}} \times D_{\text{water}}$$

Where:

- $W_{\text{air}}$ Weight of concrete in air (kg)
- $W_{\text{water}}$ Weight of concrete in water (kg)
- $D_{\text{water}}$ Density of water (1000 kg/m$^3$)

The average density of 3 cubes for each mix was measured at each different interval, Table 4.3.

3.7.3 PULSE VELOCITY.

Pulse velocity was measured in accordance with BS 1881 Part 203 (1986) using the Pulsed Ultrasonic Non-Destructive Indicative Tester (Pundit). The main use of this method is in the quality control of similar concrete, (lack of compaction and a change in the water/cement ratio can be detected). There is no direct relation to the compressive strength, because of the type of coarse aggregate its content in concrete influences the relation between the strength and the pulse velocity. Other factors are age, temperature and moisture content.

The basic principle of this technique is the measurement of the transmit time of a sound wave passing from an emitter transducer through a solid material (concrete) to a receiver transducer. The pulse velocity depends on the dynamic Young’s modulus, the dynamic Poisson’s ratio, the density of the medium, the composition, degree of compaction, maturity and free water content that are inherent in concrete products and structures (BS 1881 Part 203
1986). Calibration is necessary to establish the correct zero reading for the apparatus, since the measured time is influenced by a time delay due to transmission of the electrical signal along the transducer cables. The time delay adjustment should be made while the transducers are coupled to the opposite ends of a reference bar for which the transmit time is accurately known. The two transducers must be pressed firmly against the ends of the bar and connected with it BS 1881 Part 203 (1986).

In this test, direct transmission (direct mode) was used between two opposite faces. An electrical pulse generator, a pair of transducers, an amplifier and an electromagnetic timing device were employed. To measure the pulse velocity accurately, it was necessary to place the transducers on opposite faces of the concrete element. The time \( T_1 \) taken for a pulse to travel between two opposite fine faces of the cube was measured, but not the rough surface of the cube. The cube was then turned through 90° and the time \( T_2 \) was measured. The average value of the time \( T_{av} \) was calculated for \( T_1 \) and \( T_2 \).

The pulse velocity \( (\text{km/s}) \) could be calculated knowing that the velocity \( (P_v) \) equal to distance \( (D) \) divided by time \( (T_{av}) \), whereas the distance for the cube was 10 cm. The average value of the two velocities was calculated.

Values for pulse velocity were taken prior to the compressive strength test. This test was carried out on 3 cubes for each concrete as described in Section 3.7.1. The average results were taken at different intervals and are presented in Table 4.3.

3.7.4 INITIAL ABSORPTION SURFACE TEST (ISAT)

To assess transport properties, the ISAT method was used for measuring capillary absorption of water at the actual surface of a structure. It is referred to British Standard BS 1881-5 (1970). In this test, the specimen should be pre-dried so the main drawback of site-based test is that the concrete being tested can't be properly pre-conditioned.

The initial surface absorption is the rate of flow of water into concrete per unit area after a stated interval from the start of the test and at a constant applied head and temperature. The purpose of this British Standard is to provide suitable methods of determining the initial surface absorption of concrete, both on site and in the laboratory, where most of the concrete is to be used in the samples. Thus, it is possible to provide uniformity in the methods of making and recording the tests carried out on the concrete. The ISAT provides a measure of
surface properties that is of interest in relation to performance criteria such as durability. This technique is not a method of determining the permeability of the concrete (Concrete Society 1988). Oven-dried samples were used in this study. The samples were dried in a well-ventilated oven at 50 ± 5 °C until constant weight, not more than 0.1 % weight change over any 24 h drying period, is achieved. When the sample had reached constant weight, it was placed in a suitable airtight cooling cabinet until the temperature in the cabinet falls to a temperature within 2 °C of that of the room. Each sample was remained in the cabinet until testing. The equipment used consists of a cap, reservoir, glass and capillary tubing. In this study, a metal cap was used. It provided, with a surface area of 6400mm², an area of water in contact with the surface to be tested. An inlet and an outlet tube were fixed into the cap, the former connecting to the reservoir and the latter to the capillary tube, Figure 3.12. The outlet was positioned so that it was at the highest part of the cap to allow all trapped air to escape. The cap was clamped into position using two screw clamps on either side of the cap. If any leakage was detected, the joint was made good by tightening the clamps or using more sealant material round the edges of the cap. The reservoir was filled so that a head of 200 ± 20 mm of water was applied to the surface of the concrete. The capillary tube was supported horizontally at the same level as the surface of the water in the reservoir so that the open end of the capillary tube can be raised to avoid overflow of the water between taking readings.

The tap from the reservoir was closed and the reservoir filled with water. The time of the start of the test was recorded and the tap opened to allow the water to run into the cap and out of the outlet tubing until no more air escaped. At all times, care was taken that the reservoir does not empty itself. The outlet tubing was connected to the capillary tube and any additional trapped air was flushed out by allowing the capillary to overflow and, if necessary, by sharply pinching the flexible tubing. The reservoir was replenished to maintain the specified head of water. Readings were taken after 10 min, 30 min, and 1 h from the start of the test. Just before the specified intervals, the position of the capillary tube was adjusted so that it is completely filled with water. It was then fixed in a horizontal position at the same level as the surface of the water in the reservoir. At each of the specified test intervals, the tap was closed and when water started to flow along the capillary tube, the stopwatch was started. After 5 s the number of scale units the water has moved was noted and, the period during which movement is noted was determined. When readings were taken over a 2 min or 30 s period the measurement was multiplied by 0.5 or 2 respectively, and the number of scale units the water travels in one minute was recorded. Between test intervals, the tap was left open.

66
and the level of the water in the reservoir was maintained at the specified head. The capillary tube was tilted to prevent overflow of the water. The ISAT results were recorded in mL/m²/s after the following intervals from the start of the test: 10min, 30min, and hour and are presented in Figure 4.6.

Figure 3.12 Arrangement for ISAT Measurement, after BS 1881-5 1970.
CHAPTER 4. EXPERIMENTAL RESULTS

4.1. CONCRETE PROPERTIES

The compressive strength, density, pulse velocity and initial surface absorption test (ISAT) results obtained for the five concrete mixes, mix proportions are presented in Table 3.2, when cured up to an age of 400 days at 20°C and 99% relative humidity, are summarized and presented below.

4.1.1 COMPRESSIVE STRENGTH

The average compressive strength developments for the five concrete investigated mixes are presented in Figures 4.1 to 4.5. These results are summarized in Table 4.1. The results are discussed in Section 5.1.1.

Figure 4.1. Compressive strength developments for mix OPC (0.6) concrete.
Figure 4.2. Compressive strength developments for mix OPC (0.45) concrete.

Figure 4.3. Compressive strength developments for mix SRPC (0.45) concrete.
Figure 4.4. Compressive strength developments for mix OPC/PFA (0.40) concrete.

Figure 4.5. Compressive strength developments for mix OPC/GGBS (0.40) concrete.
<table>
<thead>
<tr>
<th>Mix</th>
<th>Age (Days)</th>
<th>No of Cubes Tested</th>
<th>Strength (± standard deviation) (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC (0.6)</td>
<td>7</td>
<td>3</td>
<td>20.2±2.0</td>
</tr>
<tr>
<td>OPC (0.6)</td>
<td>28</td>
<td>6</td>
<td>27.9±0.5</td>
</tr>
<tr>
<td>OPC (0.6)</td>
<td>120</td>
<td>3</td>
<td>30.9±0.4</td>
</tr>
<tr>
<td>OPC (0.6)</td>
<td>400</td>
<td>3</td>
<td>32.6±0.9</td>
</tr>
<tr>
<td>OPC (0.45)</td>
<td>7</td>
<td>3</td>
<td>39.8±1.1</td>
</tr>
<tr>
<td>OPC (0.45)</td>
<td>28</td>
<td>12</td>
<td>48.7±2.4</td>
</tr>
<tr>
<td>OPC (0.45)</td>
<td>120</td>
<td>6</td>
<td>55.8±0.7</td>
</tr>
<tr>
<td>OPC (0.45)</td>
<td>400</td>
<td>3</td>
<td>56.0±0.9</td>
</tr>
<tr>
<td>SRPC (0.45)</td>
<td>7</td>
<td>3</td>
<td>28.6±1.8</td>
</tr>
<tr>
<td>SRPC (0.45)</td>
<td>28</td>
<td>12</td>
<td>53.5±3.4</td>
</tr>
<tr>
<td>SRPC (0.45)</td>
<td>120</td>
<td>6</td>
<td>61.7±3.0</td>
</tr>
<tr>
<td>SRPC (0.45)</td>
<td>400</td>
<td>3</td>
<td>66.0±1.1</td>
</tr>
<tr>
<td>OPC/PFA (0.4)</td>
<td>7</td>
<td>3</td>
<td>27.3±0.3</td>
</tr>
<tr>
<td>OPC/PFA (0.4)</td>
<td>28</td>
<td>9</td>
<td>42.6±2.5</td>
</tr>
<tr>
<td>OPC/PFA (0.4)</td>
<td>120</td>
<td>3</td>
<td>54.7±1.2</td>
</tr>
<tr>
<td>OPC/PFA (0.4)</td>
<td>400</td>
<td>3</td>
<td>63.0±0.9</td>
</tr>
<tr>
<td>OPC/GGBS (0.4)</td>
<td>7</td>
<td>3</td>
<td>32.2±0.6</td>
</tr>
<tr>
<td>OPC/GGBS (0.4)</td>
<td>28</td>
<td>9</td>
<td>42.0±2.1</td>
</tr>
<tr>
<td>OPC/GGBS (0.4)</td>
<td>120</td>
<td>3</td>
<td>50.0±0.8</td>
</tr>
<tr>
<td>OPC/GGBS (0.4)</td>
<td>400</td>
<td>3</td>
<td>59.0±1.3</td>
</tr>
</tbody>
</table>

Table 4.1 Average compressive strength results for concrete mixes.

4.1.2 DENSITY AND PULSE VELOCITY

The average density and pulse velocity results for the five concrete mixes are presented in Table 4.2. These results are discussed in Sections 5.1.2 and 5.1.3, respectively.
<table>
<thead>
<tr>
<th>Mix</th>
<th>Age (Days)</th>
<th>Strength (± standard deviation) (N/mm²)</th>
<th>Density (± standard deviation) (kg/m³)</th>
<th>Pulse velocity (± standard deviation) (km/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC (0.6)</td>
<td>28</td>
<td>27.9±0.5</td>
<td>2248±7</td>
<td>4.28±0.04</td>
</tr>
<tr>
<td>OPC (0.6)</td>
<td>120</td>
<td>30.9±0.4</td>
<td>2251±5</td>
<td>4.32±0.00</td>
</tr>
<tr>
<td>OPC (0.6)</td>
<td>400</td>
<td>32.6±0.9</td>
<td>2255±2</td>
<td>4.36±0.01</td>
</tr>
<tr>
<td>OPC (0.45)</td>
<td>28</td>
<td>48.7±2.4</td>
<td>2296±5</td>
<td>4.47±0.02</td>
</tr>
<tr>
<td>OPC (0.45)</td>
<td>120</td>
<td>55.8±0.7</td>
<td>2310±4</td>
<td>4.50±0.04</td>
</tr>
<tr>
<td>OPC (0.45)</td>
<td>400</td>
<td>56.0±0.9</td>
<td>2314±3</td>
<td>4.55±0.05</td>
</tr>
<tr>
<td>SRPC (0.45)</td>
<td>28</td>
<td>53.5±3.4</td>
<td>2295±6</td>
<td>4.40±0.04</td>
</tr>
<tr>
<td>SRPC (0.45)</td>
<td>120</td>
<td>61.7±3.0</td>
<td>2300±5</td>
<td>4.45±0.04</td>
</tr>
<tr>
<td>SRPC (0.45)</td>
<td>400</td>
<td>66.0±1.1</td>
<td>2307±4</td>
<td>4.50±0.06</td>
</tr>
<tr>
<td>OPC/PFA (0.4)</td>
<td>28</td>
<td>42.6±2.5</td>
<td>2345±8</td>
<td>4.53±0.05</td>
</tr>
<tr>
<td>OPC/PFA (0.4)</td>
<td>120</td>
<td>54.7±1.2</td>
<td>2352±6</td>
<td>4.60±0.03</td>
</tr>
<tr>
<td>OPC/PFA (0.4)</td>
<td>400</td>
<td>63.0±0.9</td>
<td>2355±5</td>
<td>4.63±0.01</td>
</tr>
<tr>
<td>OPC/GGBS (0.4)</td>
<td>28</td>
<td>42.0±2.1</td>
<td>2341±7</td>
<td>4.52±0.02</td>
</tr>
<tr>
<td>OPC/GGBS (0.4)</td>
<td>120</td>
<td>50.0±0.8</td>
<td>2349±4</td>
<td>4.56±0.03</td>
</tr>
<tr>
<td>OPC/GGBS (0.4)</td>
<td>400</td>
<td>59.0±1.3</td>
<td>2352±4</td>
<td>4.60±0.04</td>
</tr>
</tbody>
</table>

Table 4.2 Average compressive strength, density and pulse velocity results for the concrete mixes.

4.1.3 INITIAL SURFACE ABSORPTION TEST (ISAT)

The average 400-day ISAT results for the five concrete mixes are presented in Figure 4.6. These results are the average of two measurements. These results are discussed Section 5.1.4.
Figure 4.6. Average 400-day ISAT results (mL/m²-s) for different concrete mixes. Measurements were taken after 10, 30 and 60 minutes after starting the test.

4.2. HALF-CELL POTENTIAL MEASUREMENT RESULTS

The half-cell potentials for the preliminary and main investigation are presented below.

4.2.1 PRELIMINARY TESTING

In the preliminary testing, the potentials of steel bars embedded in cylinders and cube mortars (without interface) were measured. Theses results are presented below.

4.2.1.1 Potential measurements for single bars embedded in OPC (0.9) and OPC/GGBS (0.9) mortar cylinders.

Figure 4.7 presents the potential measurements for single steel bars embedded in OPC and OPC/GGBS mortar cylinders. Whereas the potential difference between these bars are presented in Figure 4.8, these results are summarized in Table 4.3. It should be noted that the w/b ratio for these specimens was 0.9 and the samples were cured for
only 4 days. This may explain the highly negative values of potentials of the two mixes. These results are discussed in Section 5.3.1.

Figure 4.7. Potentials measurements of single bars embedded in OPC (0.9) and OPC/GGBS (0.9) mortar cylinders.

Figure 4.8. Potential difference of single bars embedded in OPC (0.9) and OPC/GGBS (0.9) mortar cylinders.
### 4.2.1.2 Potential measurements for connected bars embedded in OPC (0.9) and OPC/GGBS (0.9) mortar cylinders

Figure 4.9 presents the potential measurements for the connected steel bars embedded in OPC and OPC/GGBS mortar cylinders, whereas the potential difference between these bars are presented in Figure 4.10. These results are summarized in Table 4.3. The potential measurements for the connected bars were taken after disconnecting the bars for a few seconds. It should be noted that w/b ratio for these specimens was 0.9 and it was cured for only 4 days. This may explain the negative values of potentials of the two mixes. These results are discussed in Section 5.3.2.

<table>
<thead>
<tr>
<th>Time (Days)</th>
<th>OPC</th>
<th>OPC/GGBS</th>
<th>Potential difference</th>
<th>OPC</th>
<th>OPC/GGBS</th>
<th>Potential difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>-231</td>
<td>-589</td>
<td>-358</td>
<td>-326</td>
<td>-457</td>
<td>-132</td>
</tr>
<tr>
<td>10</td>
<td>-215</td>
<td>-479</td>
<td>-264</td>
<td>-295</td>
<td>-395</td>
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Table 4.3 Potential measurements for steel bars embedded in OPC (0.9) and OPC/GGBS (0.9) mortar cylinders.
4.2.1.3 Potential measurements for single bars embedded in OPC (0.32) and OPC/GGBS (0.32) mortar cubes

The average potentials and the potential differences for single steel bars embedded in OPC and OPC/GGBS mortar cubes are presented in Figures 4.11 and 4.12.
respectively and are summarized in Table 4.4. These values represent the average of two bars in each cube. It should be noted that the w/b ratio for these specimens was 0.32 and the mixes were cured for only 4 days. These results are discussed in Section 5.3.3.

Figure 4.11 Average potential single steel bars embedded in OPC (0.32) and OPC/GGBS (0.32) mortar cubes.

Figure 4.12 Average of potential difference of single steel bars embedded in OPC (0.32) and OPC/GGBS (0.32) mortar cubes.
### Potential (mV) for connected steel bars

<table>
<thead>
<tr>
<th>Time (Days)</th>
<th>OPC</th>
<th>OPC/GGBS</th>
<th>Potential difference</th>
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</thead>
<tbody>
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<td>-243</td>
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</tr>
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</tr>
<tr>
<td>88</td>
<td>-107</td>
<td>-121</td>
<td>-14</td>
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</table>

### Potential (mV) for single steel bars

<table>
<thead>
<tr>
<th></th>
<th>OPC</th>
<th>OPC/GGBS</th>
<th>Potential difference</th>
</tr>
</thead>
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<tr>
<td></td>
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<td></td>
<td>-243</td>
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<td>51</td>
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<tr>
<td></td>
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<td></td>
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<td>-174</td>
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<td></td>
<td>-170</td>
<td>-235</td>
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<tr>
<td></td>
<td>-90</td>
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<td>-45</td>
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</table>

**Table 4.4** Average potential measurements and potential difference of steel bars embedded in OPC (0.32) and OPC/GGBS (0.32) mortar cubes.

### 4.2.1.4 Potential measurements connected bars embedded in OPC (0.32) and OPC/GGBS (0.32) mortar cubes

The average potentials and the potential differences for connected steel bars embedded in OPC and OPC/GGBS mortar cubes are presented in Figures 4.13 and 4.14, respectively. These values represent the average of two bars in each cube and are summarized in Table 4.4. Moreover, the potential measurements of the connected bars were taken during connecting and a few seconds after disconnecting. It should be noted that the w/b ratio for these specimens was 0.32 and it was cured for only 4 days. These results are discussed in Section 5.3.4.
Figure 4.13 Average potential of connected steel bars embedded in OPC (0.32) and OPC/GGBS (0.32) mortar cubes.

Figure 4.14 Average of potential difference connected of steel bars embedded in OPC (0.32) and OPC/GGBS (0.32) mortar cubes.
4.2.2 MAIN INVESTIGATION

In the main testing program, the potentials of steel bars embedded in the concrete cubic and prismatic specimens were measured. These concretes were OPC (0.6), OPC (0.45), SRPC (0.45), OPC/PFA (0.4) and OPC/GGBS (0.4). These results are presented below.

Potentials measurements of single and connected bars without interface are presented in Section 4.2.2.1, while, the measurements of bars with an interface are presented in Section 4.2.2.2.

4.2.2.1 STEEL BARS IN CONCRETE CUBES

4.2.2.1.1 Single bars

Figures 4.15 to 4.19 present the average potential measurements, up to 20 cycles (40 weeks), for single steel bars, embedded in the five concrete mixes, partially immersed only in 1 M NaCl solution. These values represent the average potentials of 2 bars in the OPC (0.6) mix, 6 bars in the OPC (0.45) mix and 4 bars for the remaining 3 mixes. A best-fitted curve is shown on each graph. These results are discussed in Section 5.4.1.1.

Whereas, the average potential measurements, up to 20 cycles (40 weeks), for single steel bars, embedded in OPC (0.45) and SRPC (0.45) concrete, partially immersed in 0.1 M Na2SO4 solution, were presented in Figures 4.20 to 4.21 and the results are discussed in Section 5.4.1.1.4.
Figure 4.15. Average potential of single bars embedded in OPC (0.6) concrete, partially immersed in 1 M NaCl solution. $R^2 = 0.8905$.

Figure 4.16. Average potential of single bars embedded in OPC (0.45) concrete, partially immersed in 1 M NaCl solution. $R^2 = 0.9409$
Figure 4.17. Average potential of single bars embedded in SRPC (0.45) concrete, partially immersed in 1 M NaCl solution. $R^2 = 0.9489$

Figure 4.18. Average potential of single bars embedded in OPC/PFA (0.4) concrete, partially immersed in 1 M NaCl solution. $R^2 = 0.9623$. 
Figure 4.19. Average potential of single bars embedded in OPC/GGBS (0.4) concrete, partially immersed in 1 M NaCl solution. $R^2 = 0.8086$.

Figure 4.20. Average potential of single bars embedded in OPC (0.45) concrete, partially immersed in 0.1M Na$_2$SO$_4$ solution.
Figure 4.21. **Average potential of single bars embedded in SRPC (0.45) concrete, partially immersed in 0.1M Na₂SO₄ solution.**

4.2.2.1.2 Connected bars embedded in various couples of mixes

Figures 4.22 to 4.27 present the average potential measurements, up to 20 cycles (40 weeks), for connected steel bars, embedded in four concrete mixes, partially immersed only in 1 M NaCl solution. Measurements were taken while the bars were connected and 5 minutes after disconnection. These values represent the average potentials of 2 bars in each couple. These results are discussed in Section 5.4.1.2. Whereas Figure 4.28 presents the average potential measurements, up to 20 cycles (40 weeks), for connected steel bars, embedded in OPC (0.45) and SRPC (0.45) concrete mixes, partially immersed in 0.1 M Na₂SO₄ solution, respectively. These results are discussed in Section 5.4.1.2.3.
Figure 4.22. Average potential measurements of connected bars embedded in OPC (0.45) and SPRC (0.45) couple, partially immersed in 1 M NaCl solution. Values were taken during connection and 5 min. after disconnection.

Figure 4.23. Average potential measurements of connected bars embedded in OPC/PFA (0.4) and OPC/GGBS (0.4) couple, partially immersed in 1 M NaCl solution. Values were taken during connection and 5 min. after disconnection.
Figure 4.24. Average potential measurements of connected bars embedded in OPC (0.45) and OPC/PFA (0.4) couple, partially immersed in 1 M NaCl solution. Values were taken during connection and 5 min. after disconnection.

Figure 4.25. Average potential measurements of connected bars embedded in OPC (0.45) and OPC/GGBS (0.4) couple, partially immersed in 1 M NaCl solution. Values were taken during connection and 5 min. after disconnection.
Figure 4.26. Average potential measurements of connected bars embedded in SRPC (0.45) and OPC/PFA (0.4) couple, partially immersed in 1 M NaCl solution. Values were taken during connection and 5 min. after disconnection.

Figure 4.27. Average potential measurements of connected bars embedded in SRPC (0.45) and OPC/GGBS (0.4) couple, partially immersed in 1 M NaCl solution. Values were taken during connection and 5 min. after disconnection.
Figure 4.28. Average potential measurements of connected bars embedded in OPC (0.45) and SPRC (0.45) couple, partially immersed in 0.1 M Na₂SO₄ solution. Values were taken during connection and 5 min. after disconnection.
4.2.2.2 BARS EMBEDDED IN CONCRETE SPECIMENS WITH AN INTERFACE

Figures 4.29 and 4.30 present the average potential measurements, up to 25 cycles (50 weeks), for steel bars embedded in the couples of the same mix. Figures 4.31 to 4.38 present the average potential measurements, up to 25 cycles (50 weeks), for steel bars embedded in the couples of various mixes. These values represent the average potentials of 2 bars in each couple for three investigated depths. A summary table for one year results of polarisation resistance and current density for bars, embedded in different concrete couples, is presented in Table 4.5. Also a summary table for 5-year results of potentials and current density for bars, embedded in different concrete couples, is presented in Table 4.6. These results are discussed in Section 5.4.3.

The OPC (0.45) and SRPC (0.45) concrete specimens were ponded with 1 M NaCl or Sabkha (15.7% NaCl and 2.1% Na₂SO₄) solutions, whereas, the remaining concrete couples were only ponded with 1 M NaCl solution.

![Figure 4.29](image)

Figure 4.29. Average potential measurements of bars, embedded at 3 different depths, in OPC (0.45) and OPC (0.45) couple, ponded with 1 M NaCl solution.
Figure 4.30. Average potential measurements of bars, embedded at 3 different depths, in SRPC (0.45) and SRPC (0.45) couple, ponded with 1 M NaCl solution.

Figure 4.31. Average potential measurements of bars, embedded at 3 different depths, in OPC (0.6) and OPC (0.45) couple, ponded with 1 M NaCl solution.
Figure 4.32. Average potential measurements of bars, embedded at 3 different depths, in OPC/PFA (0.4) and OPC/GGBS (0.4) couple, ponded with 1 M NaCl solution.

Figure 4.33. Average potential measurements of bars, embedded at 3 different depths, in OPC (0.45) and SRPC (0.45) couple, ponded with 1 M NaCl solution.
Figure 4.34. Average potential measurements of bars, embedded at 3 different depths, in OPC (0.45) and SRPC (0.45) couple, ponded with sabkha solution (15.7% NaCl + 2.1% Na₂SO₄).

Figure 4.35. Average potential measurements of bars, embedded at 3 different depths, in OPC (0.45) and OPC/PFA (0.4) couple, ponded with 1 M NaCl solution.
Figure 4.36. Average potential measurements of bars, embedded at 3 different depths, in OPC (0.45) and OPC/GGBS (0.4) couple, ponded with 1 M NaCl solution.

Figure 4.37. Average potential measurements of bars, embedded at 3 different depths, in SRPC (0.45) and OPC/PFA (0.4) couple, ponded with 1 M NaCl solution.
Figure 4.38. Average potential measurements of bars, embedded at 3 different depths, in SRPC (0.45)-and OPC/GGBS (0.4) couple, ponded with 1 M NaCl solution.
<table>
<thead>
<tr>
<th>Concrete couple with an interface</th>
<th>Measurements (after one year)</th>
<th>Cover Depth (mm)</th>
</tr>
</thead>
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<tr>
<td></td>
<td></td>
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<tr>
<td><strong>OPC (0.6) and OPC (0.45)</strong></td>
<td>$R_p$ (k-Ohms cm$^2$)</td>
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<td>$I_{corr}$ (mA/m$^2$)</td>
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<td><strong>OPC (0.45) and OPC (0.45)</strong></td>
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<tr>
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<tr>
<td></td>
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<td>1.4</td>
</tr>
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<td>$I_{corr}$ (mA/m$^2$)</td>
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</tr>
<tr>
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<td>$I_{corr}$ (mA/m$^2$)</td>
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Table 4.5. One year results of polarisation resistance (k-Ohms cm$^2$) and current density (mA/m$^2$) for bars embedded in OPC (0.45), OPC (0.60), SRPC (0.45), OPC/GGBS (0.4) and OPC/PFA (0.4) concrete couples, ponded with 1 M NaCl solution. (Sabkha solution (15.7 % NaCl +2.1 Na$_2$SO$_4$).)
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<th>Concrete couple with an interface</th>
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<td>Potential (mV)</td>
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<td>Potential (mV)</td>
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<td>OPC (0.45) and OPC/PFA (0.4)</td>
<td>$I_{\text{corr}}$ (mA/m²)</td>
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<tr>
<td></td>
<td>Potential (mV)</td>
<td>-515</td>
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<td>OPC (0.45) and OPC/GGBS (0.4)</td>
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<td></td>
<td>Potential (mV)</td>
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<td>SRPC (0.45) and OPC/PFA (0.4)</td>
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<td></td>
<td>Potential (mV)</td>
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<tr>
<td>SRPC (0.45) and OPC/GGBS (0.4)</td>
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<td>123</td>
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<tr>
<td></td>
<td>Potential (mV)</td>
<td>-512</td>
</tr>
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<td>OPC/PFA (0.4) and OPC/GGBS (0.4)</td>
<td>$I_{\text{corr}}$ (mA/m²)</td>
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</tr>
<tr>
<td></td>
<td>Potential (mV)</td>
<td>-570</td>
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</table>

Table 4.6. Current density (mA/m²) and potential (mV) results after 5 years for bars with an interface embedded in OPC (0.45), OPC (0.60), SRPC (0.45), OPC/GGBS (0.4) and OPC/PFA (0.4) concrete couples, ponded with 1 M NaCl solution. * (Sabkha solution (15.7 % NaCl + 2.1 Na₂SO₄)).
CHAPTER 5. DISCUSSION

5.1. PROPERTIES OF HARDENED CONCRETE

5.1.1 COMPRESSIVE STRENGTH

The average compressive strength development for each of the five concrete mixes investigated are presented in Figure 4.1 to 4.5 and summarized in Table 4.2, Section 4.1.1. The results show that, as expected, the strength of all the mixes increased with the curing period.

A relative comparison between the strength results for the various mixes as compared to that of the OPC (0.45) mix is shown in Figure 5.1. The results show that increasing the w/c ratio from 0.45 to 0.6 decreased the strength at all ages. On the other hand, the results show that the later age strength (28 days and above) of the SRPC (0.45) concrete was higher than that of the OPC (0.45). This behavior has been explained by Jackson (1984) and Algahtani et al (1994).

The strength of the OPC/GGBS (0.4) and OPC/PFA (0.4) was lower than that of the OPC (0.45) mix up to an age of 120 days. The reduction in strength in mixes containing GGBS or PFA may be related to the slower rate of hydration of these materials as compared to the OPC (Neville 1995; Concrete Society 1991). The early strength reduction of blended cements was ascribed to the lower cement content, which was replaced by PFA and GGBS. These pozzolanic materials did not contribute to the early strength development, Section 2.2.1.4. The pozzolanic reaction that was activated with time due to the release of portlandite upon cement hydration. Therefore, the 400-day results show that the OPC/PFA (0.4) and OPC/GGBS (0.4) mix reached a higher strength than the OPC (0.45) control. The higher long-term strength for the blended cement concrete compared to OPC has been reported by (Neville 1995 and Concrete Society 1991). Finally, the strength results show that the OPC/PFA (0.4) mixes achieved lower strength than the OPC/GGBS (0.4) at 7 days. Whereas, at 28 days and above, the strength of the OPC/PFA (0.4) mixes was higher than that of the OPC/GGBS (0.4). Similar trend was reported by Harrison and Spooner (1986).
These results suggest that all of the concrete mixes tested here are demonstrating characteristics that are consistent with their composition and w/b ratio.

![Graph showing relative compressive strength of various concrete mixes](image)

**Figure 5.1. Relative compressive strength of the various concrete mixes as compared to the OPC (0.45) control.**

### 5.1.2 DENSITY

The average densities for each of the five investigated concrete mixes is presented in Table 4.3, Section 4.1.2. The results show that the density of all the mixes increased with age. The results also show that increasing the w/b ratio decreased the density of the OPC concrete mixes. These observations are similar to those found in the compressive strength results.

The density results for the OPC/PFA (0.4) and OPC/GGBS (0.4) were higher than that of the OPC (0.45) mixes. This may be attributed to the slight decrease in the w/b ratio of the blended cement mixes. Finally, the density results for the SRPC (0.45) mixes, as expected, were similar to those of the OPC (0.45).

These results suggest that all of the concrete mixes tested here are demonstrating characteristics that are consistent with their composition and w/b ratio.
5.1.3 PULSE VELOCITY

The average pulse velocities for each of the five concrete mixes investigated are presented in Table 4.3, Section 4.1.2. These results follow a similar trend to that observed in the density and compressive strength results, where increasing the w/b ratio decreased the pulse velocity of the various concrete mixes.

According to pulse velocity limits reported by Neville (1995), for normal quality concrete, the quality of the mixes used in this investigation was between good (pulse velocity between 3.5 and 4.5 km/s) and excellent (pulse velocity > 4.5 km/s). In fact, all the PFA and GGBS mixes were excellent and better than plain cements.

These results suggest that all of the concrete mixes tested here are demonstrating characteristics that are consistent with their composition and w/c ratio.

5.1.4 INITIAL SURFACE ABSORPTION TEST (ISAT)

The ISAT test provides information on the absorption characteristics of the outer layer of concrete, which offers protection to reinforcement and, therefore, it is of greatest interest (Neville 1995).

The average 400-day ISAT results are presented in Figure 4.6. The results show that the absorption rate decreased with increasing the testing time, this thought to be due to the voids that been filled with water. The ISAT results follow an inverse trend to that observed in the density and pulse velocity results, since the more denser concrete will have the lower ISAT values.

In general, all the concretes examined showed a low initial surface absorption values compared to the published values, Table 5.1, (Levitt 1969).

<table>
<thead>
<tr>
<th>Absorption</th>
<th>ISAT values (mL/m²/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10 minutes</td>
</tr>
<tr>
<td>High</td>
<td>&gt; 0.50</td>
</tr>
<tr>
<td>Low</td>
<td>&lt; 0.25</td>
</tr>
</tbody>
</table>

Table 5.1 Typical ISAT values expected for well-cured, mature concrete, after Levitt 1969.
It is clear that, increasing the w/c ratio for the OPC concrete mixes increased the measured absorption rate. This presumably reflects the greater volume and continuity of capillary pores in concrete mixes with high w/b ratio. The ISAT results for the OPC/PFA (0.4) and OPC/GGBS (0.4) were somewhat lower than that of the OPC (0.45) concrete. Finally, the ISAT results for the SRPC (0.45) mixes were slightly higher than that the OPC (0.45) and blended cement mixes. A similar trend was reported by Sanjuán (2000) and Saricimen et al (1995).

These results suggest that all of the concrete mixes tested here are demonstrating characteristics that are consistent with their composition and w/b ratio.

5.2. CORROSION BEHAVIOUR OF REINFORCEMENT

According to ASTM C 876 (1991), Section 3.5.3.1 when the half-cell potential of steel bars, embedded in OPC concrete, falls below −350mV with respect to a copper/copper sulphate electrode (CSE), there is a 95% probability of corrosion occurring, Section 3.5.3.1. Therefore, the time of corrosion initiation can be measured from the first ponding cycle to the time (number of cycles) when the corrosion potential falls below this value. However, this value may vary depending on the cement type, humidity and environmental exposure (Elsener et al 1994 and Gowers and Millard 1999a).

The Nernst equation, Section 2.4.1, carries several important implications for the corrosion of metals under non-uniform conditions and indicates that a potential difference will exist between areas, at various locations along a steel bar or between electrically connected steel cages, in the presence of a common electrolyte if there is a difference in the material, the temperature gradient, the concentration of oxygen, solution, environment, ions or its porosity and permeability exists. Such a potential difference has the capacity to cause the formation of adjacent local anode and cathode sites and corrosion can be induced. Thus, if two steel bars were embedded in similar mixes that had the same aggregate content and same free/water cement ratio but differs in the cement type, a potential difference will be developed in the presence of a common electrolyte.
5.3. PRELIMINARY INVESTIGATION

The preliminary testing stage was carried out on steel bars embedded in OPC or OPC/GGBS mortar mixes partially immersed in tap water for up to 90 days after being initially cured in the curing room for 4 days (Section 4.2.1).

5.3.1 Single bars embedded in OPC (0.9) and OPC/GGBS (0.9) mortar cylinders

The potential results for the single bars embedded in OPC (0.9) and OPC/GGBS (0.9) mortar cylinders with cover depth of 10mm are presented in Figure 4.7. Whereas, the potential difference between these single bars are presented in Figure 4.8. These results are summarized in Table 4.4, Section 4.2.1.1.

Figure 4.7, shows the potential of the bars as a function of time. It can be seen that the bar in the OPC mix does indeed act as a cathode while the bar in the OPC/GGBS acts as an anode. It is interesting to note that the potential difference between the two bars, Figure 4.8, decreases with time due to the ongoing corrosion current. These initial results support the theory that bars in different cement types can corrode differently. Based on the single bar results, it seems that the bar embedded in OPC (0.9) was cathodic (potentials between -200mV and -270mV) with respect to the other bar embedded in OPC/GGBS (0.9). The result indicates that the bar embedded in OPC/GGBS (0.9) mix was thought to be corroding through out the testing period. Whereas, the bar embedded in OPC (0.9) mix was not corroding.

The poor behaviour of the bars in OPC/GGBS (0.9) is expected given the high w/b ratio and poor level of curing of these specimens. It has been documented that, concretes containing GGBS are more sensitive to poor curing than OPC concretes (Harrison and Spooner 1986). According to Al-Amoudi (1995), the high potentials of the steel bars in GGBS cement is not ascribed to actual corrosion but due to the formation H₂S on the bars giving the impression of active corrosion, Figure 4.7.

Since the cover depth for the steel bars in these specimens was 10mm, it has been reported that the surface zone of blended cement concrete was more susceptible to the poor curing. Therefore, it is not surprising that laboratory tests on blended cement
concretes that have been dried in the first few days after casting exhibited inferior performance to OPC concrete dried at the same time (Harrison and Spooner 1986).

The results also show that the first measured potential difference between the bar embedded in the OPC (0.9) and the one embedded in OPC/GGBS (0.9) was about 350 mV, this value reduced to about 250 mV after 28 days, Figure 4.8. According to Gowers and Millard (1999b), potential differences exceeding 200 mV between different locations on a surface may be considered to be indicative of corrosion activity. This provides evidence to support the initial theory that steel bars in a mixed cement environment can suffer from the formation of active anodes directly as a result of the different chemical environment. To test this idea, two bars in different cement types of OPC and OPC/GGBS, were connected and left in a common electrolyte.

5.3.2 Connected bars embedded in OPC (0.9) and OPC/GGBS (0.9) mortar cylinders without interface

Figure 4.9 presents the potential measurements for connected steel bars embedded in OPC and OPC/GGBS mortar cylinders, whereas the potential difference between these bars is presented in Figure 4.10. These results are summarized in Table 4.4.

It is clear that the connected bar embedded in OPC/GGBS (0.9) was acting as an anode while the other connected bars embedded in OPC (0.9) acted as a cathode. Finally, the potential difference between the connected bars was reduced to zero reaching equilibrium state of their potentials due to the current flow between the bars during connection.

5.3.3 Single bars embedded in OPC (0.32) and OPC/GGBS (0.32) mortar cubes

Two well-compacted mortar mixes OPC (0.32) and OPC/GGBS (0.32) with low w/b ratio of 0.32 were cast in standard cubes with two steel bars embedded in each cube with 25 mm cover. The cubes were cured for 4 days; this was followed by partial immersing in tap water during the measurement period. Referring to Table 2.3, it has been found that the time required to block continuous capillary pores with gel is 3 days of curing for w/c ratio of 0.4. The bar potentials were recorded for each mix.
The average potentials for the two bars are summarized in Table 4.4 and presented in Figure 4.11.

It is to be noted the potential difference between OPC (0.32) and OPC/GGBS (0.32), after 20 days, changes from positive to negative, Figure 4.12. It is possible that the initial 20-day behaviour is becoming dominated by the development of the pore structure of the OPC/GGBS blend. This behaviour reflects changes within the chemistry of the cement paste adjacent to the bars. After that the average potential results indicate that the bars embedded in OPC (0.32) were generally cathodic with respect to the bars in the OPC/GGBS (0.32). The relatively higher negative values for the OPC/GGBS blends was explained in Section 5.3.1. Figure 4.11 show that the average potentials for the these bars displays unstable behaviour in terms of Cathodic/Anodic sites in the initial 20-day, whereas the average potentials for the these bars displays stable behaviour in terms of C/A sites after 20 days.

At the end, the average potentials of the single bars in these cubes were <-134mV, this indicates that none of the bars were corroding. The average maximum potential difference between the single bars was -86mV, Figure 4.12. This confirms the earlier result that it is possible to establish anodic and cathodic sites on a steel bar due simply to differences in cement type.

5.3.4 Connected bars embedded in OPC (0.32) and OPC/GGBS (0.32) mortar cubes without interface.

The average potentials for two connected bars, shown in Figure 3.8, are summarized in Table 4.5 and presented in Figure 4.13. Additionally, the average potential difference results between the connected bars in both mixes are summarized in Table 4.5 and presented in Figure 4.14.

The first average potential for the connected bars embedded in OPC (0.32) and OPC/GGBS (0.32) mortar cubes was approximately -250mV, whilst the 90-day results for the average potentials of the connected bars were <-121mV, Figure 4.13. This indicates that none of the bars were corroding. Moreover, the average potential differences between the connected bars in both mixes were close to zero, this is due to
the fact that the bar potential was measured during connection, Figure 4.14. To decide which is the cathode and anode sites, the connected bars were disconnected for a period (typically few seconds in these specimens) but a period of 5 minutes was used in the main investigation.

5.4. MAIN INVESTIGATION

In the previous "preliminary" investigation, the effect of cement type on the potentials of single and connected bars embedded in cement mortar mixes partially immersed in tap water was observed. Therefore, the long term effect on the potentials of bars embedded in five different concrete cubes without interface and exposed to two different environments is evaluated. In this part, a comparison was to be made between the potentials of the connected and the single bars in relation to the cement type and the exposed environment.

At the first year, the potentials for single and connected bars embedded in various concretes were measured, Section 4.2.2.1. Also the potential of bars embedded in concrete couples with an interface was measured, Section 4.2.2.2. These potentials results were an indication for corrosion activity.

5.4.1 BARS EMBEDDED IN CONCRETE CUBES (WITHOUT INTERFACE)

5.4.1.1 SINGLE BARS

Concrete mixes were initially cured for 14 days, before potential measurements, for single bars embedded with a cover depth of 25 mm. These measurements were recorded, at the end of each wet cycle, over a period of 20 cycles (40 weeks).

The average potentials of bars embedded in five concretes, partially immersed in 1 M NaCl solution, are presented in Figures 4.15 to 4.19 (Section 4.2.2.1.1.). A comparison between these results is shown in Figure 5.2 and Table 5.2. Similarly, the average potentials of single bars embedded in OPC (0.45) and SRPC (0.45) concrete, partially immersed in 0.1 M Na₂SO₄ solution are presented in Figures 4.20 and 4.21 (Section 4.2.2.1.1). A comparison between the average potentials of single bars
embedded in OPC (0.45) and SRPC (0.45) concrete, partially immersed in 0.1 M Na₂SO₄ solutions and is presented Figure 5.4.

The average potentials of single bars embedded in OPC/GGBS (0.4) dropped sharply after 4 cycles, whereas the average potentials of the single bars embedded in OPC (0.6), OPC (0.45), SRPC (0.45) and OPC/PFA (0.4) were fluctuating for the initial 8 cycles. This was followed by a sharp drop in the average potential of the single bars, embedded in all mixes, to higher negative values. This potential drop may be attributed to the increased chloride concentration around the concrete steel bar interface.

![Figure 5.2. Average potentials (mV) at 20 cycles for single bars embedded in five concretes, partially immersed in 1 M NaCl solution.](image)

The observed downward trend is common for all corroding bars (Sanjuán 2000). At the end of measurement, 20 cycles, the average potential of single bars had more negative values, indicating that all the bars have high probability of corrosion. However, the average differences were reduced, swapping the effect of cement type.

5.4.1.1.1 Single bars embedded in OPC (0.6) and OPC (0.45) concretes in 1 M NaCl Solution.

As expected, for most of the measurements, the average potentials of the single bars embedded in OPC (0.45), Figure 4.16, were less negative than those embedded in
OPC (0.6) mixes, Figure 4.15. The time to corrosion initiation of single bar in OPC (0.6) or OPC (0.45) was 15 cycles, whereas the average potentials at the end of testing, 20 cycles, were \(-512\) mV and \(-455\) mV for OPC (0.6) or OPC (0.45), respectively. This indicates that if the bars embedded in both mixes were connected, the bar in the OPC (0.45) will act as cathode with respect to the one in OPC (0.6) mix. At the end of the experiment, all the single bars were thought to be corroding.

The higher negative potential values of bars embedded in OPC (0.6) mix as compared to those in OPC (0.45) may be due to the higher w/c ratio. This weaker cement paste will lead to less effective binder and also lead to a larger volume of capillary pores in the concrete (Neville 1995).

5.4.1.1.2 Single bars embedded in OPC (0.45) and SRPC (0.45) concrete in 1 M NaCl Solution

Figure 4.16 and Figure 4.17 show the average potential of the bars embedded in OPC (0.45) and SRPC (0.45) concretes, respectively. Moreover, a comparison between the average potential measurements for these bars is presented in Figure 5.3. These results show that the average potential difference, in the first six cycles, between single bars embedded in OPC (0.45) and SRPC (0.45) concrete was around 20 mV. The results also show, Figure 5.3, changes from cathode to anode sites were noticed in the first 7 cycles, for bars embedded in OPC (0.45) and SRPC (0.45).

The results presented in Figure 5.3 indicate that the time of corrosion initiation was around the 15 cycles for the bars embedded in these mixes. The bars embedded in SRPC (0.45) had higher negative values than those embedded in OPC (0.45). This indicates that, if these bars were connected, the bars in SRPC will acts as a stable anode. After the 16-cycle, the maximum potential difference between the bars embedded in OPC (0.45) and SRPC (0.45) was approximately 130 mV. Finally, the average potentials after 20 cycles, last measurement, for single bars in OPC (0.45) and SRPC (0.45), were \(-455\) and \(-523\) mV, respectively. This indicates that single bars embedded in both concretes had a high probability of corrosion, Figure 5.3.
Knowing that SRPC has a lower binding capacity for free chloride ions because of its low C₃A, it is a disadvantage to use SRPC for reinforced concrete in a chloride rich environment because the risk of corrosion will increase (Neville 1995; Harrison and Spooner 1986).

5.4.1.1.3 Single bars embedded in OPC/PFA (0.4) and OPC/GGBS (0.4) concrete in 1 M NaCl solution

The average potentials for bars embedded in OPC/GGBS (0.4) or OPC/PFA (0.4) are presented in Figure 5.2. The measurements taken at the first 15 cycles show that the potentials of the bars embedded in the blend mixes were more negative than those measured for bars embedded in OPC (0.45) and SRPC (0.45), Figure 5.2. However, the last 4 cycles of measurements indicate that the average potentials for bars in OPC/GGBS (0.4) or OPC/PFA (0.4) were more negative than those bars in OPC (0.45), whilst they were less negative than those bars embedded in SRPC (0.45) concrete. Additionally, the corrosion initiation time for single bars embedded in OPC/GGBS (0.4) and OPC/PFA (0.4) concretes were 7 and 11 cycles, respectively.
Both are shorter time than the 15 cycles that were measured for bars embedded in OPC or SRPC concrete, Figure 5.2.

At the first 15 cycles, the average potentials for single bars in OPC/GGBS (0.4) had more negative values than those bars in OPC/PFA (0.4) concrete, thus indicating that if the bars were connected, the bars in OPC/GGBS (0.4) act as anode for these cycles. Harrison and Spooner (1986) concluded that, concretes containing GGBS are more sensitive to poor curing than OPC/PFA or OPC concretes. Similarly, OPC/PFA concretes are more sensitive to poor curing than OPC concretes.

At the end, the average potential difference for single bars embedded in OPC/GGBS (0.4) and OPC/PFA (0.4) concrete, at the last three cycles, were reduced, Figure 5.2. This is thought to be associated with arrival of chloride ions at the surface of the bars, which initiate general corrosion of all the bars regardless of its local environment. This effectively swaps any difference in potential that arises from differences in cement type.

The results of the average potential of the single bars embedded in five concrete cubes showed the poor behaviour of the blended concrete, Section 5.3.1. Also this was attributed to the fact that the initial curing period (14 days) that was not enough for the completion of the hydration process for the blended cement concrete, due to their sensitivity for the curing. It is reported that for the blended concrete the shorter the curing period and the greater will be the degree of drying, the worse is its effect upon properties (Harrison and Spooner 1986). Moreover, the curing of blended cement concrete during its earliest stages is very important to develop the desired properties (Parrott 1991 and ACI 308 1997).

Many investigators believe that a curing period of about 28 to 90 days is required for pozzolanic cement concrete to attain properties similar to that of plain cement concrete. This is attributed to the pozzalnic reaction of these materials, which is often very slow (Cabrera 1985 and Marsh et al 1985). Similar finding by Jones (1994) was reported for OPC, OPC/PFA and OPC/GGBS.
5.4.1.1.4 Single bars embedded in OPC (0.45) and SRPC (0.45) concrete, partially immersed, in 0.1 M Na₂SO₄ solution.

Figure 4.20 and Figure 4.21 show the average potential of the bars embedded in OPC (0.45) and SRP(C (0.45) concretes, respectively. The best fitted curve for the average of both potentials was a linear equation. A comparison between the average potential of single bars embedded in both concretes is presented in Figure 5.4.

![Graph showing average potential of single steel bars embedded in OPC (0.45) and SRPC (0.45) concrete at 20 cycles, partially immersed in 0.1 M Na₂SO₄ solution.](attachment:graph.png)

**Figure 5.4.** Average potentials for single steel bars embedded in OPC (0.45) and SRPC (0.45) concrete at 20 cycles, partially immersed in 0.1 M Na₂SO₄ solution.

For the first period of 18 cycles, the average potentials of steel embedded in OPC (0.45) and SRPC (0.45) concretes, partially immersed 0.1 M Na₂SO₄, shows changing in the formation of cathode and anode sites, Figure 5.4. However, at the end, the average potentials, was -53mV and -85 mV for the single bars embedded in SRPC (0.45) and OPC (0.45) concretes, respectively. This may be because of the sulphate resistance of SRPC. The average potentials of single bars in SRPC (0.45) fluctuated gradually to less negative values. The average potentials at 20 cycles, Figure 5.4, showed that none of the single bars were corroding, thus indicating that the corrosion activity was minimal for bars in pure sulphate solutions. Similar findings have been reported by Al-Amoudi et al (1994).
5.4.1.1.5 Summary for the single bars embedded in five concrete mixes, partially immersed in 1 M NaCl solution

For the time of 20 cycles, the results showed that all the single bars, under test at the 25mm depth of cover, embedded in the five concretes, partially immersed in 1 M NaCl solution were thought to be corroding. The corrosion for the bars was induced by chloride concentration. The poor behaviour of the blended concrete was discussed in Section 5.4.1.1.3. A better performance would be expected for the blended concrete if the initial curing would increase from 14 to 28 days of curing.

As result the comparative performance of the five concretes in the form of time to corrosion initiation (cycles) and potentials (mV) has been established for single bars embedded in five concrete mixes, partially immersed in 1 M NaCl solution, Table 5.2.

<table>
<thead>
<tr>
<th></th>
<th>Single bars</th>
<th>Potential at 20 cycles (mV)</th>
<th>Time to corrosion initiation (cycles)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Best</td>
<td>OPC (0.45)</td>
<td>-455</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>OPC (0.6)</td>
<td>-512</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>SRPC (0.45)</td>
<td>-523</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>OPC/PFA (0.4)</td>
<td>-503</td>
<td>11</td>
</tr>
<tr>
<td>Worst</td>
<td>OPC/GGBS (0.4)</td>
<td>-478</td>
<td>7</td>
</tr>
</tbody>
</table>

Table 5.2. Comparative performance based on corrosion initiation time (cycles) and the average measured potentials at 20 cycles for single bars embedded in different concrete, partially immersed in 1 M NaCl solution.

Based on the results of the time to corrosion initiation and potential of single bars, Table 5.2, the formation of anode and cathode sites can be predicted for the connected bars embedded in different concrete. For example, if the bar embedded in couple of OPC (0.45) and SRPC (0.45) concrete, the potential of connected bars in SRPC (0.45) will have more negative values than those bars in OPC (0.45) and the formation of anode will be seen on the bars in SRPC (0.45). A similar prediction can be made on the couple of OPC (0.45) and OPC (0.6) concrete; being the connected bars in OPC (0.6) will act as anode with respect to the other bars in OPC (0.45). These predictions were confirmed by the results on Table 5.3.
5.4.1.2  CONNECTED BARS (WITHOUT INTERFACE)

After 14 days curing, the potentials were taken at the end of each wet cycles for a monitoring period up to 20 cycles (40 weeks), at 25 mm depth of cover. The average potentials of connected bars during connection, Figure 5.8, and after 5 min. disconnection, embedded in four concretes with six couples, were plotted, Figures 4.22 to 4.27, Section 4.2.2.1.2. A comparison between the potential and time to corrosion initiation for bars embedded in six couples are presented in Table 5.3. The arrangement of the six couples was as follows:

OPC (0.45) and SRPC (0.45) concrete

OPC (0.45) and OPC/PFA (0.4) concrete

OPC (0.45) and OPC/GGBS (0.4) concrete

SRPC (0.45) and OPC/PFA (0.4) concrete

SRPC (0.45) and OPC/GGBS (0.4) concrete

OPC/PFA (0.4) and OPC/GGBS (0.4) concrete

Whereas the average potentials of connected bars embedded in OPC (0.45) and SRPC (0.45) concrete, partially 0.1 M Na₂SO₄ solution were plotted, Figure 4.28, and Section 4.2.2.1.2.

The potential results of the connected bars presented in Section 4.2.1.2 show that the potential differences were reduced to zero during connection. Therefore, to get an indication about the anodic and cathodic sites and to determine the type of mix shifting the potential of the couple to more negative value, an attempt was made to present the average potentials of the couple bars during connection and the average potentials of the same bars after 5 minutes disconnection.

All the connected bars embedded in the four types of concretes, partially immersed in 1 M NaCl solution showed that the connection of the bars embedded in different concretes produced earlier results in the form of time to initiation of corrosion. This explains the theory that using two cement types in one structure will increase the corrosion activity. The worst behaviour was noticed in the couple of OPC and SRPC.
concrete. The average potential of connected bars in OPC (0.45) and SRPC (0.45) dropped sharply after two cycles, Figure 4.22, thus indicating that the couple bars were thought to be corroding. Whereas the average potential of bars after 5 min. disconnection indicated that the bars in SRPC were thought to be corroding.

In contrast, the average potentials of the connected bars embedded in the other five concrete couples had less negative values at the initial 6 to 8 cycles, then slightly decreased with fluctuating change of around 100mV. Afterwards, for most of the connected bars, the average potentials had taken a sharp drop to values >-350mV. This downward trend is common for all the corroded steel bars (Sanjuán 2000). At the end, the average potential difference was reduced, due to the current flow between the connected bars and the concentrated chloride around the steel bar.

5.4.1.2.1 Connected bars embedded in OPC (0.45) and SRPC (0.45) concrete

The potential difference between the bars embedded in couple of OPC and SRPC concrete, from 6 to 15 cycles, was in the range between (-115 to -166mV), indicating clearly that the bars in SRPC were acting as a stable anode and shifting the potentials of the couple to more negative values. This confirms earlier observations, that in an OPC and SRPC couple, bars in the SRPC tend to be anodic with respect to those in OPC, Section 5.4.1.1.2. It has been noted that the connection had increased the average potentials of the bars at the end. Also, it had reduced the time to corrosion initiation from 15 cycles for the single bars in SRPC, Table 5.2, to 3 cycles only, Table 5.3, for the connected bars embedded in OPC and SRPC couple. Thus, the structure life will be reduced when using both types of concretes in one structure.

After 15 cycles, the observed potential difference between the average of single bars for 5 minutes in OPC and SRPC disappears, Figure 4.22. This is thought to be associated with arrival of chloride ions at the surface of the bars, which initiate general corrosion of all the bars regardless of its local environment. This effectively swaps any difference in potential that arises from differences in cement type. At the end all the connected and single bars were thought to be corroding. The noticed lower negative values for bars embedded in OPC at the first 15 cycles, may be attributed to the beneficial effect of C3A in increasing the chloride binding in chloride solution, and increasing time to corrosion initiation.
5.4.1.2.2 Connected bars embedded in OPC or SRPC concrete with couples of OPC/PFA (0.4) and OPC/GGBS (0.4) concrete, partially immersed in 1 M NaCl solution:

Figure 4.23 represents the average potential of the connected bars embedded in OPC/PFA (0.4) and OPC/GGBS (0.4) concretes, partially immersed in 1 M NaCl solution, during connection and after 5min. disconnection, Section 4.2.2.1.2.

The time to corrosion initiation for the connected bars embedded in OPC/GGBS (0.4) and OPC/PFA (0.4), partially immersed in 1 M NaCl solution was 11 cycles. In contrast, the initiation time to corrosion of single bars in the same solution was 7 and 11 cycles for bars in OPC/GGBS (0.4) and OPC/PFA (0.4), respectively. It is noted that for most of measurements, the average potential of the connected bars embedded in OPC/PFA (0.4) had lower negative values than those bars embedded in OPC/GGBS (0.4) concrete, leading at the end to the reduced average potential difference. This confirms the earlier observation that if the bars in both blends were connected, the bars embedded in OPC/GGBS (0.4) will act as anode with respect to the other bars embedded in OPC/PFA (0.4), Section 5.4.1.1.3.

Thus, the bars in OPC/PFA (0.4) and OPC/GGBS (0.4) concrete couple performed better than OPC (0.45) and SRPC (0.45) concrete in terms of time to corrosion initiation for the couple by an order of 8 cycles.

The results of average potential for connected bars embedded in couple of OPC or SRPC concrete with blended cements showed, generally stable anode and cathode sites. The connected bars embedded in the couple of OPC and OPC/PFA concrete and the couple of OPC and OPC/GGBS concretes, show that the connected bars embedded in OPC concrete were acting as cathode with respect to the bars embedded in OPC/PFA and OPC/GGBS concretes, Figure 4.24 and Figure 4.25, respectively.

For the couple of SRPC and OPC/PFA concrete and couple of SRPC and OPC/GGBS concrete, the bars embedded in SRPC were acting as cathode with respect to the bars embedded in both cement blends, Figure 4.26 and Figure 4.27, respectively. This also confirms earlier observation for the single bars, Section 5.4.1.1.3. The higher negative values of the bars embedded in the blends concrete may be attributed to the earlier observation about the sensitivity of the blends to the reduced curing period of
14 days. Moreover, the high potentials for the bars embedded in OPC/GGBS concrete was explained in Section 5.3.1.

These results showed that the potential difference of most couples decreases with time, due to the current flow and the ingress of chloride ions to the steel concrete interface. This will swap the effect of the cement type on corrosion behaviour. However, the reduced potential difference was not noticed in bars embedded in SRPC and OPC/GGBS couple that had a stable potential difference (with 200mV max.) during the last 10 cycles. This high stable potential difference can give a warning to the increased corrosion activity.

5.4.1.2.3 Connected bars embedded in OPC (0.45) and SRPC (0.45) concrete, partially immersed in 0.1 M Na_2SO_4 solution

Figure 4.28 presents the average potential of the connected bars embedded in OPC (0.45) and SRPC (0.45) concrete, partially immersed in 0.1 M Na_2SO_4 solution, during connection and after 5min. disconnection, Section 4.2.2.1.2.

After 17 cycles, the average potential of the connected bars for bars in OPC (0.45) and SRPC (0.45) was -317mV. Whilst the average potential of the connected bars after 5 minutes disconnection were -249 and -345mV for bars in OPC (0.45) and SRPC (0.45), respectively. This indicates that the bars in SRPC act as an anode.

This was not the same behaviour at the same cycle for the single bars, Figure 5.4, since the average potential was around < -80mV for single bars embedded in both concretes. Therefore, the connection had increased the average potentials 3-4 times compared to single bars in the same concretes. At the end, the average potential for the connected bars after 5 minutes disconnection in SRPC was -167mV at 20 cycles, Figure 4.28, whilst the average potential of the single bars in SRPC (0.45) was -53mV only, Figure 5.3. The maximum average potential difference was < -180mV, whilst the average potential difference of the single bars was around 30mV only, thus indicating that even with immersing in 0.1 M Na_2SO_4 solution only, the connection increased the average potentials significantly.

The average potentials of single bars exposed in 0.1 M sodium sulphate solution rose gradually to less negative values. Similar behaviour for the connected bars was
noticed for the first few cycles but dropped rapidly for few weeks before rising again to less negative values. After partial immersing of the cubes for 20 cycles in 0.1 M sodium sulphate solution, none of the bars were thought to be corroding. The behaviour was not similar to the connected bars embedded in OPC (0.45) and SRPC (0.45) concrete, partially immersed in chloride solution, that was thought to be corroding. Sulphate ions are known to attack concrete causing either expansion and cracking or softening and reducing strength (Al-Amoudi 1998 and Swamy 2001). However, the role of sulphate ions on corrosion of reinforcing steel is very slow of their coefficients compared to those for NaCl diffusion (Al-Amoudi et al 1994). Therefore, NaSO_4 solution is less corrosive.

5.4.1.2.4 Summary for the connected bars embedded in four concrete with six couples, partially immersed in 1 M NaCl solution.

At the end of the experimental program, all the single bars, partially immersed in 1 M NaCl solution, were thought to be corroding. The corrosion was induced by chloride concentration and the connection between two bars. The connected bars embedded in four concrete with six couples, under test, produced results in the form of time to initiation of corrosion (cycles) and potentials (mV). As a result the comparative performance of six couples made with four concretes has been established for connected bars partially immersed in 1 M NaCl solution.

<table>
<thead>
<tr>
<th>Couple bars</th>
<th>Cathode /Anode</th>
<th>Time to Corrosion Initiation (cycles)</th>
<th>Potential (mV)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Best OPC (0.45) and OPC/PFA (0.4)*</td>
<td>Stable C/A</td>
<td>13</td>
<td>-493</td>
</tr>
<tr>
<td>OPC/PFA (0.4) and OPC/GGBS (0.4) *</td>
<td>Stable C/A</td>
<td>11</td>
<td>-500</td>
</tr>
<tr>
<td>OPC (0.45) and OPC/GGBS (0.4) *</td>
<td>Stable C/A</td>
<td>10</td>
<td>-439</td>
</tr>
<tr>
<td>SRPC (0.45) and OPC/PFA (0.4) *</td>
<td>Stable C/A</td>
<td>10</td>
<td>-566</td>
</tr>
<tr>
<td>SRPC (0.45) and OPC/GGBS (0.4) *</td>
<td>Stable C/A</td>
<td>9</td>
<td>-572</td>
</tr>
<tr>
<td>Worst OPC (0.45) and SRPC (0.45) *</td>
<td>Stable C/A</td>
<td>3</td>
<td>-558</td>
</tr>
</tbody>
</table>

Table 5.3. Time to corrosion initiation and average potentials after 20 cycles of connected bars (during connection) for four concrete mixes immersed partially in 1 M NaCl solution.
* The word "stable C/A" means that the determination of cathode/anode sites was based on average potentials values of bars after 5 min. disconnection, for most of measurements, indicating that the potential difference exits. However the word "unstable C/A" means that the determination of cathode/anode sites based on average potentials values of bars after 5 min. disconnection, was difficult due to the changing in the formation of cathodic and anodic sites during the measurements. The predictions of stable C/A for the different concrete couples form Table 5.2, were confirmed by the results on Table 5.3.

It can be noted that the average time to corrosion initiation was generally reduced from 7-15 cycles for the single bars, Table 5.2, to 3-11 cycles for the connected bars embedded in concrete couples, Table 5.3.

A comparative result of average potentials after 20 cycles with time to corrosion initiation (cycles) for the single and connected bars after 5 min. disconnection, embedded in OPC and SRPC concretes without interface is shown below.

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Single</th>
<th>Connected</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Potentials (mV)</td>
<td>Time to Corrosion Initiation (cycles)</td>
</tr>
<tr>
<td>OPC (0.45)</td>
<td>-455</td>
<td>15</td>
</tr>
<tr>
<td>SRPC (0.45)</td>
<td>-523</td>
<td>15</td>
</tr>
</tbody>
</table>

Table 5.4: Comparison of average potentials (after 20 cycles) and time to corrosion initiation (cycles) for single and connected bars (measurements were taken after 5 minutes of disconnection) embedded in OPC (0.45) and SRPC (0.45) concrete couples partially immersed in 1 M NaCl solution.

* For most of measurements.

From Table 5.4, it is interesting to note that the connection had reduced the time to corrosion initiation form 15 cycles (210 days) for the single bars in SRPC, to 3 cycles only (42 days), for the connected bars embedded in OPC and SRPC couple, Table 5.4. This is very important in terms of the structure life, since the connection reduces the time to initiation of corrosion by 5 times. This clearly has implication for the corrosion behaviour of steel bars in a mixed cement environment. It is very important
to avoid the connection of these two types or using them in one structure when exposed to NaCl solution.

The results of corrosion of bars embedded in the blended cement concrete would be expected to have better performance with longer initial curing period (28 days). Further, it will be expected to change the formation of the anodic sites from the bars in blended cement concrete to those in bars embedded in SRPC concrete.

Moreover, all the connected bars embedded in the six couples showed generally stable anodic and cathodic sites throughout the testing period and confirmed the earlier observations reported in Table 5.2, Section 5.4.1.1.

5.4.2 BARS EMBEDDED IN SPECIMENS WITH AN INTERFACE

The average of potential of bars embedded in different mixes with an interface, measured during the first year for 25 ponding cycles, are presented in Figures 4.29 to 4.38, Section 4.2.2.2.

Figure 5.5. Front view of concrete couple made with two concrete type and showing the recess side for bonding.
These measurements were used to assess the performance of five concretes with the following 9 couples, Figure 5.5, ponded with 1 M NaCl solution:

OPC (0.6) and OPC (0.45) concrete, Figure 5.5.

OPC (0.45) and OPC (0.45)

SRPC (0.45) and SRPC (0.45)

OPC (0.45) and SRPC (0.45)

OPC (0.45) and OPC/PFA (0.4)

OPC (0.45) and OPC/GGBS (0.4)

SRPC (0.45) and OPC/PFA (0.4)

SRPC (0.45) and OPC/GGBS (0.4)

OPC/PFA (0.4) and OPC/GGBS (0.4)

Additionally, one couple was made with OPC (0.45) and SRPC (0.45) concrete, ponded with Sabkha solution.

Moreover, the results of polarisation resistance and current measurement for bars embedded in ten concrete couples with interface, at the end of one-year, are presented in Table 4.5. For comparison purposes the average results for the one-year polarisation resistance for all bars at 25mm depth, embedded in the ten concrete specimens are presented in Figure 5.7.

Finally, two potential and current measurements were taken for two cycles at the end of 5 years after casting and presented in Table 4.6. A comparative average result for the last 5-years current measurements for all bars at 25mm depth, embedded in the ten concrete specimens are presented in Figure 5.6.
5.4.2.1 POTENTIAL MEASUREMENTS FOR 25 PONDING CYCLES

The average of one-year potential results for the ten specimens under test, at the three different depths of cover, were presented in Figures 4.29 to 4.38, whereas the average time to corrosion initiation are presented in Table 5.5.

The results of the half-cell potential measurements suggest that in seven out of the ten specimens under test, the bars at the 12mm cover depth were thought to be corroding, due to the exposure to chloride media. On the other hand, these results suggest that four out of the ten specimens and three out of ten specimens, for the bars at the 25mm cover and the bars at 35mm respectively, had a high probability of corrosion, Table 5.5. The letter (N) in Table 5.5, indicated that there no sign of corrosion of these bars within 25 cycles of potential measurements. Table 5.5 showed that in terms of the average time to corrosion initiation, the various couples can be ranked as follows:

<table>
<thead>
<tr>
<th>Couples (with an interface)</th>
<th>Figure no.</th>
<th>Average time to corrosion initiation (cycles)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>12 mm</td>
</tr>
<tr>
<td>Best OPC (0.45) and OPC (0.45)</td>
<td>Figure 4.29</td>
<td>N</td>
</tr>
<tr>
<td>OPC (0.45) and OPC/GGBS (0.4)</td>
<td>Figure 4.36</td>
<td>N</td>
</tr>
<tr>
<td>OPC/PFA (0.4) and OPC/GGBS (0.4)</td>
<td>Figure 4.32</td>
<td>N</td>
</tr>
<tr>
<td>SRPC (0.45) and OPC/PFA (0.4)</td>
<td>Figure 4.37</td>
<td>N</td>
</tr>
<tr>
<td>SRPC (0.45) and OPC/GGBS (0.4)</td>
<td>Figure 4.38</td>
<td>N</td>
</tr>
<tr>
<td>SRPC (0.45) and SRPC (0.45)</td>
<td>Figure 4.30</td>
<td>21</td>
</tr>
<tr>
<td>OPC (0.45) and OPC/PFA (0.4)</td>
<td>Figure 4.35</td>
<td>21</td>
</tr>
<tr>
<td>OPC (0.45) and SRPC (0.45)</td>
<td>Figure 4.33</td>
<td>19</td>
</tr>
<tr>
<td>OPC (0.6) and OPC (0.45)</td>
<td>Figure 4.31</td>
<td>7</td>
</tr>
<tr>
<td>Worst OPC (0.45) and SRPC (0.45) (sabkha)</td>
<td>Figure 4.34</td>
<td>11</td>
</tr>
</tbody>
</table>

Table 5.5 Average time to corrosion initiation for bars at 12, 25 and 35 mm depth, embedded in different concrete couples with an interface

119
The concrete couples in Table 5.5 were ranked between 'Best' that means no sign of corrosion for any bar at different depths and 'Worst' with high probability of corrosion for all the bars.

The average potential of the steel bars within one year, indicated that for the corroded bars showed generally, little change during the first few cycles. This constant line of potential of the steel bars, before the ingress of chloride ions, is attributed to the ability of the concrete to passivate the steel, and protect it from direct contact with the chloride ions in the ponding solution. Subsequently, a drop in the potential to more negative values occurred with time. This was attributed to the presence of chlorides ions, which had migrated through the pores of concrete (by either capillary uptake or diffusion) and acted to depassivate the steel.

The average results of the half-cell potential measurements for the bars embedded in different concrete couples, ponded with chloride solution, showed that the bars at 12mm and 25mm cover embedded in the OPC (0.6) and OPC (0.45) concrete couple had shown the poor performance since they had signs of corrosion after only 7 and 17 cycles, respectively. Finally, the average potential results showed that all the bars, at different depths, embedded in OPC (0.45) and SRPC (0.45) couple, ponded with Sabkha solution were corroded within 18 cycles and thus showed the worst performance. Thus the presence of both sulphate and chloride is necessary for the initiation and propagation of reinforcement corrosion in concrete (Al-Amoudi et al 1994; Al-Amoudi & Maslehuddin 1993 and Dehwah et al 2002).

5.4.2.2 POTENTIAL MEASUREMENTS AT THE END OF 5 YEARS

The results of half-cell potential and corrosion current measurements at the end of 5 years for the ten specimens under test at the three different depths of cover are presented in Tables 4.6.

The time to corrosion initiation of the bars at 12mm cover, as indicated by the half-cell potential, indicates that all the bars had exposed to chloride media high probability of corrosion. Haque and Kawamura (1992) had reported that one of the main causes of reinforcement corrosion is the loss of efficiency of the concrete cover,
which is known to protect the steel and keep it in a passive state. If the cover is low, the time taken for aggressive agents, such as chlorides, to reach the reinforcement will decrease and thus the time to cause onset of steel corrosion will be reduced.

However, the results of the potential measurements suggest that in eight out of the ten specimens under test, the bars at the 25mm cover depth were thought to be corroding. Finally, the potential measurements at the end of 5 years showed that in seven out of ten specimens, the bars at 35mm had a high probability of corrosion. The corrosion of the bars at 25mm and 35mm cover was thought to be induced by chloride concentration and the presence of the interface.

Based on the potential results, the bars at 25mm and 35mm cover embedded in the OPC (0.45) and OPC (0.45) couple and OPC (0.45) and OPC/GGBS (0.4) couple were thought to be not corroding. Whereas the bars at 35mm cover embedded in the OPC/PFA (0.4) and OPC/GGBS (0.4) couple were thought to be not corroding.

It is of note that the blended cement specimens cured in this series of test was cured for 28 days. This may be explain the better performance compared with the results of single bars embedded in similar concrete without interface, that were cured for 14 days only, Section 5.4.1.1.3. It has been documented that the lower permeability of blended cement concrete can result in an improved durability performance but this is crucially dependent on adequate curing to ensure that full hydration occurs (Neville 1995).

5.4.2.3 POLARISATION RESISTANCE AND CURRENT DENSITY MEASUREMENTS

The corrosion of steel reinforcing bars is a dynamic electrochemical process that requires a flow of electric current and several chemical reactions involving an exchange of ions between passive (cathodic) and active (anodic) locations on the surface of a steel reinforcing bar. The rate of corrosion is dependent on the availability of water, oxygen, and chloride ions; the ratio of steel surface area at the anode to that at the cathode; the electrical resistivity of the concrete; the temperature and changes in ambient environmental conditions (Millard et al 2001). Since the electrical resistivity of concrete is one of the factors controlling the rate of
reinforcement corrosion, it is apparent that a reduction in the electrical resistivity will lead to an increase in the rate of corrosion. In the corrosion cell circuit, ionic transport between the anode and cathode is one of the rate controlling factors (Dehwah et al 2002). The concrete resistivity regulates the ionic flow current between anodic and cathodic areas on the reinforcement surface in a chloride environment. The higher the concrete resistivity, the lower will be the currents flow between an anodic and cathodic area and therefore the lower the corrosion rate (Gowers and Millard 1999b; Baweja et al 1998). According to Millard et al (2001), the steel reinforcement in dry concrete will corrode more slowly than in moist concrete.

Results of polarisation resistance and corrosion current rate after one year, for all bars embedded in the different concrete couples are presented in Table 4.5. Additionally a comparative average polarisation resistance results between bars at 25mm depth, embedded in different concrete couples are presented in Figure 5.6.

Moreover, at the end of the monitoring life (5 years from casting), the averages of corrosion rate measurements for the bars embedded in different concrete couples are presented in Table 4.6. Additionally a comparison between averages of corrosion rate results for bars at 25mm depth, embedded in different concrete couples are presented in Figure 5.7.

Based on the data Table 3.6, the bar will be passive if its polarization resistance >250 k-Ohm cm² and current density <1 mA/m². On the other hand the bar will had very high corrosion rates if its polarization resistance was between 2.5 and 0.25 k-Ohm cm² and current density was between 100 and 1000 mA/m².

The polarization resistance and current density values for the bars, at different depths, embedded in concrete couples, showed that these measurements were more reliable than the half-cell potentials. Based on the potential results of the bars at 25mm and 35mm cover after 5 years, embedded in the OPC (0.45) and OPC/GGBS (0.4) couple were thought to be not corroding. However, the current density values for the bars at the same cover depths, were around between 14 and 38 mA/m², respectively, Table 4.6, showed that it had high corrosion rates, whereas the current density values for the bars embedded in the same concrete couple of OPC (0.45), at
the same cover depths, were around 4 and 8 mA/m², respectively, showed that it had low to moderate corrosion rates.

![Figure 5.6. Polarisation resistance values (k-Ohm cm²) for bars at 25mm embedded in concrete couples after 1 year.](image)

![Figure 5.7. Current Density (mA/m²) measured for bars at 25mm cover, embedded in concrete couples after 5 years.](image)
At the end of 5-years, the measurements showed that the lowest current density values for the bars at different depths were noticed for the bars embedded in the same concrete couple of OPC (0.45), compared with other bars embedded in different concrete couples, Table 4.6.

5.4.2.3.1 The effect of interface on polarisation and current density for bars embedded in OPC and SRPC concrete specimens, ponded with chloride solution

The average results of polarisation resistance of the bars at 25mm cover, embedded in different couples, at the end of one-year ponding with 1 M NaCl solution, Figure 5.6 and Table 4.5, showed that the $R_p$ results of bars embedded in OPC concrete or SRPC concrete, ponded with 1 M NaCl solution, was 4.6 and 2.7 times, respectively, better than $R_p$ of bars embedded in a couple of OPC and SRPC concrete, ponded with the same solution. Since $R_p$ is inversely proportional to the current density, this reflects that the bars embedded in the same concrete type will have a higher resistance to corrosion than the bars embedded in any different couples such as OPC and SRPC concrete. This was proved by the lowest current density for these bars at the end of one and 5-years measurements. It has been found that an enhanced electrical resistivity will have a strong influence in reducing corrosion of reinforcement (Rasheeduzzafar et al 1992; Rasheeduzzafar 1992; Algahtani et al 1994 and Hope et al 1987).

A similar observation was noted for bars at 35mm depth, embedded in OPC concrete specimen. This was verified by the lowest current density at the end of 5-year current measurements. Also the results of current density of the bars, at 35mm depth showed that the bar embedded in SRPC concrete had high corrosion current compared with the bar embedded in OPC concrete. Similar finings were also observed by Sanjuán (2000).

The average current density of bars at 25mm and 35mm in OPC concrete was about 6mA/m$^2$ after 5 years of casting, Table 4.6. Moreover, the results of current measurements of bars embedded in OPC concrete showed higher resistance by 2-3 times and lower current density compared with bars embedded in SRPC concrete.
This suggests that the current density decreased with increasing the C_A content in cement, which is responsible for the increased chloride binding by high C_A cements Dehwah et al (2002). This was confirmed by the increased electrical resistivity in the Ordinary Portland cements compared, to Sulfate-Resisting Portland cements.

5.4.2.3.2 The effect of interface on polarisation and current density for bars embedded in OPC and SRPC concrete specimen, ponded with chloride solution (5.9% NaCl) or sabkha solution (15.7% NaCl +2.1 Na_2SO_4)

Baweja et al (1998) suggests that the cell potential is not only a function of the electrode materials, but also of the electrolyte, because of ion concentration and polarisation effects. The R_p of bars at different depths, embedded in the OPC and SRPC couple, ponded with sabkha solution had the lowest values (25-63 k-Ohm cm^2 <250 k-Ohm cm^2), Table 4.5, compared to the other couples, ponded with chloride solution only, indicating that these bars will have the highest current density. It has been suggested by Ewins (1990) that the presence of a low concrete resistivity can be used as an indicator that the rate of corrosion is probably quite high. This was confirmed by current measurements in Tables 4.5 and 4.6. The current density at one year for bars embedded in OPC and SRPC concrete, ponded with sabkha solution was between 7 and 13mA/m^2, suggesting that the effect of interface between OPC and SRPC, ponded with sabkha solution had similar or even higher current density than those bars embedded in OPC concrete at the end of 5 years.

It been has approved that the combination of sulphate and chloride ions in sabkha environment will increase the corrosion rate compared with the same specimens ponded with pure chloride solution, since the corrosion rate is inversely proportional to the concrete resistivity R_s and R_p. Moreover, this increase in the current density for bars in the OPC and SRPC couple, ponded with Sabkha may be due to the conjoint presence of chloride and sulfate ions that reduces the polarisation resistance at the surface of the steel (Millard et al 2001 and Dehwah et al 2002). The R_p results of bars at different depths, embedded in OPC and SRPC concrete specimen, ponded with sodium chloride was higher in the order of 5.7 and 11.2 times than R_p for the bars embedded in similar specimens, ponded with sodium chloride plus sodium sulfate. Similar observation was reported by Khan (1993) and Dehwah et al (2002).
Additionally, the tendency of sulphate ions to react preferentially with the C₃A phase, have an important bearing on sub-structural components and corrosion risk is likely to be significantly increased in circumstances where concrete is exposed in soil and groundwater contaminated to both chloride and sulphate salts (Al-Amoudi & Maslehuddin 1993 and Maslehuddin et al 1997). This effect was clear in the OPC and SRPC couple, ponded in sabkha solution, due to the change in both $R_s$ and $R_p$. Thus, using reinforced SRPC concrete with low C₃A will contribute to corrosion activity problems more than those bars embedded in OPC concrete, ponded with pure chloride solution or both of chloride and sulphate solutions. Rasheeduzzafar (1992) stated that using SRPC in concrete in the Arabian Gulf States is a mistake, since the surrounding environment around the structure is not sulphates only but chlorides or combination chlorides and sulphates. This was believed to be another cause of accelerated corrosion observed in the Arabian Gulf countries (Al-Amoudi et al 1994).

5.4.2.3.3 The effect of interface on polarisation and current density for bars embedded in OPC (0.6) and OPC (0.45) concrete specimen, ponded with chloride solution

The results of hardened properties of concrete showed that OPC (0.6) concrete had the lowest strength, pulse velocity and density during the period up to 400 days. Moreover, the ISAT results at 400 days showed that OPC (0.6) concrete had the highest absorption value. Whilst hardened concrete may be attacked by a number of physical and chemical processes, the intensity of attack will usually controlled by the porosity of the concrete (Neville 1995). The average potentials, Section 5.4.1.1.1, showed that the single bars embedded in OPC (0.6) concrete was acting as anode with respect to other bars embedded in OPC (0.45) concrete.

Moreover, as comparison between $R_p$ of bars, embedded in different couples, ponded with in 1 M NaCl solution, showed that the bars in OPC (0.6) and OPC (0.45) had the lowest $R_p$ values (14 and 109 k-Ohm cm²), at 12mm and 25mm depths, respectively, Table 4.5. This indicates that these bars will have the highest current density; this was verified by the current measurements. The current density at one-year for bars embedded in OPC (0.6) and OPC (0.45) concretes was 24 and 4.3 mA/m², at 12mm and 25mm depths, respectively, Table 4.5, suggesting that the effect of interface between these couples had increased the corrosion activity. At the end of 5-year
current measurements, the bars in OPC (0.6) and OPC (0.45) concretes had higher
current density than those bars embedded in OPC concrete, being between 1414 and
3419 mA/m², at different depths, Table 4.6. Arya and Vassie (1995) and Baweja et al
(1998) suggested that the OPC (0.45) concrete with a lower water/cement ratio and
lower permeability will have lower corrosion rate, higher resistance of concrete and
lower rates of transport of oxygen and ferrous ions thereby producing restrictions to
the cathode and anode reaction kinetics, compared with high w/c ratio (0.65).

5.4.2.3.4 The effect of interface on polarisation and current density for bars
embedded in OPC or SRPC with OPC/GGBS or OPC/PFA
concrete specimens, ponded with chloride solution

The one-year results for the bars at 12mm, embedded in PFA/GGBS concrete with 28
curing, had a higher polarisation resistance, compared with other couples at the same
depth. Partial cement replacement by PFA or GGBS cement concrete caused
significant pore refinement, reduced permeability to water and chloride ions, and
increased electrical resistivity (Hussain and Rasheeduzzafar 1994). Similarly, the
results at the end of 5-year, showed that the bars embedded at 12mm depth, in the
OPC and OPC/PFA couple had the lowest average current density at this depth
<100mA/m², compared to bars embedded in other blended cement concrete couples.

It has been reported by Zhang et al (1999) and Neville (1995) that the curing of
blended cement concrete at early ages is very important to obtain satisfactory long-
term mechanical properties and durability characteristics. This was approved by the
better performance for bars embedded in blended cement concrete with 28 days of
curing compared with those having single bars and connected bars embedded in
similar cement blends with shorter curing, Section 5.4.1.1.3 and Section 5.4.1.2.3,
respectively.

Time to corrosion initiation for the single bars at 25mm, embedded in the blended
cement concrete was between 7-11 cycles, Section 5.4.1.1.3, whilst for the bars
embedded with similar blends concrete, the corrosion was not noticed within 25
cycles. Additionally, based on the potential results after 5 years, the two bars, at
25mm depth, embedded in OPC (0.45) and OPC/GGBS (0.4) concrete were thought
not to be corroding. The use of blended cement concretes is generally thought to reduce permeability, and thereby reduce chloride diffusion rates (Baweja et al 1998).

Finally, it is important to note that the bars, embedded in SRPC and OPC/GGBS, had higher current density at 5 years, compared to the other blends couples. This confirms the earlier observation that the connection of the bars between SRPC and OPC/GGBS concretes will lead to a stable potential difference of around 200mV during the measurements, Section 5.4.1.2. This higher potential difference may be considered to be indicative of corrosion activity (Gowers and Millard 1999b).

5.4.3 DESTRUCTIVE TESTING

The ten concrete specimens with an interface were broken down after 5 year of casting. This was carried out in order to visually examine the condition of the embedded steel bars and measure the chloride content present in these samples.

5.4.3.1 VISUAL EXAMINATION

The purpose of this examination was to establish the effect of cement type on each side of the interface on the extent of corrosion and location of the anodic areas for the steel bars embedded in each couple. The bars were retrieved and their condition recorded photographically in Figures 5.8 to 5.13. Finally, a comparison was made between the visual extent of corrosion observed and the current and polarisation resistance measured for the bars, Section 5.4.3.

Figure 5.8 presents a sample of the bars that were embedded in an OPC/OPC couple with a cover depth of 25mm. The figure shows a uniform corrosion developing at both sides of the interface. Additionally, the average current and polarisation resistant measurement recorded for these bars was 4 mA/m² and 1400 kOhms cm², respectively.
Figure 5.8. Steel bars embedded in an OPC/OPC couple with a cover depth of 25mm.

Figure 5.9 presents a sample of the bars that were embedded in an SRPC/SRPC couple with a cover depth of 25mm. Similar to the OPC/OPC couple, the figure shows a uniform corrosion developing at both sides of the interface. Finally, the average current and polarisation resistant measurement recorded for these bars was 10 mA/m² and 850 kOhms cm², respectively.

Figure 5.9. Steel bars embedded in an SRPC/SRPC couple with a cover depth of 25mm.

Figure 5.10 presents a sample of the bars that were embedded in an OPC (0.45) / SRPC (0.45) couple with a cover depth of 25mm. In contrast to the observation made in the OPC/OPC and SRPC/SRPC couples, the figure clearly shows that the corrosion developed only on the SRPC side of the interface, whilst the bars embedded in OPC shows very little or no corrosion. Additionally, the average current and polarisation resistant measurement recorded for these bars were 90 mA/m² and 330 kOhms cm², respectively.

Figure 5.10. Steel bars embedded in an OPC/SRPC couple with a cover depth of 25mm.
The visual examination for the bars embedded in OPC / OPC, SRPC / SRPC and OPC / SRPC couples, confirms the observation that was obtained by comparing the current measurement for these bars, which was an order of magnitude higher when embedded in an OPC / SRPC couple as compared to that when it was embedded in an OPC / OPC or SRPC / SRPC couples. Additionally, the results show a decrease in the polarisation resistance values for the bars embedded in couple of different concretes. These results clearly show that the formation of the anodic and cathodic sites can be found on the bars passing between two different concrete types with an interface. These observations may explain the relative high current and low polarisation resistance measurements for these mixes when compared with those embedded in single concrete couples. Finally, the observation made in this section, confirms the earlier findings for the single and connected bars embedded in a couple of OPC (0.45) and SRPC (0.45) concrete, Section 5.4.1.1.2 and 5.4.1.2.1, respectively.

Figure 5.11 presents a sample of the bars with a cover depth of 25mm that were embedded in the OPC (0.45) / SRPC (0.45) couple, ponded with sabkha solution.

![Image of a steel bar embedded in a OPC/SRPC couple with a cover depth of 25mm, ponded with Sabkha solution.](image)

**Figure 5.11. Steel bar embedded in a OPC/SRPC couple with a cover depth of 25mm, ponded with Sabkha solution.**

This figure again shows that the corrosion developed only on the SRPC (0.45) side, whereas, little corrosion took place on the OPC (0.45) side. This confirms the findings noted above. However, the immersion in the Sabkha environment raises the measured corrosion current to 270mA/m² and decreases the polarisation resistance to 30KOhm cm². Despite this increased severity the anodic site formed within the SRPC, concrete remains stable and appears to protect the section of the bar within OPC.

Figure 5.12 presents a sample of the bars that were embedded in an OPC (0.6) / OPC (0.45) couple with a cover depth of 25mm. The figure shows that the corrosion developed on the OPC (0.6) side, whereas little corrosion took place on the OPC.
(0.45) side. This again confirms the ability of a bar in a mixed environment to preferentially form anodic and cathodic regions. This formation of anodic and cathodic sites within the bar was different than the uniform corrosion in the bar passing through one concrete type only such as OPC (0.45), Figure 5.8. This was confirmed by the higher current of 1420 mA/m² and the lower resistance of 105KOhm cm² for the bar passing through OPC (0.45) and OPC (0.6).

![Steel bar embedded in a OPC (0.6)/OPC (0.45) couple with a cover depth of 25mm](image)

Figure 5.12. Steel bar embedded in a OPC (0.6)/OPC (0.45) couple with a cover depth of 25mm

Finally, Figure 5.13 presents a sample of the bars that were embedded in a GGBS/PFA couple with a cover depth of 25mm. The figure shows that even when using two different cement blends, corrosion was developed on the GGBS side, with relatively little corrosion took place on the PFA side. Thus confirm the fact that even when using two cement blends together, will form cathodic and anodic sites, this was confirmed earlier in Section 5.4.1.1.3 and Section 5.4.1.2.2.

![Steel bars embedded in a GGBS/PFA couple with a cover depth of 25mm](image)

Figure 5.13. Steel bars embedded in a GGBS/PFA couple with a cover depth of 25mm.

5.4.3.2 CHLORIDE ION PROFILES

The concentration of chloride in the solution was obtained by reading off the scale in the capillary on the test strip. An average was taken for the samples from each
cement type, Section 3.6.2. A sample calculation of the chloride test is presented in Appendix, Table 9.1. The results are presented in Table 5.6.

<table>
<thead>
<tr>
<th>Concrete sample</th>
<th>Cl(^{-}) concentration (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC (0.6)</td>
<td>2.46</td>
</tr>
<tr>
<td>SRPC (0.45) Sabkha</td>
<td>0.94</td>
</tr>
<tr>
<td>OPC (0.45) Sabkha</td>
<td>0.63</td>
</tr>
<tr>
<td>SRPC (0.45)</td>
<td>0.5</td>
</tr>
<tr>
<td>OPC (0.45)</td>
<td>0.2</td>
</tr>
<tr>
<td>OPC/PFA (0.4)</td>
<td>0.14</td>
</tr>
<tr>
<td>OPC/GGBS (0.4)</td>
<td>0.16</td>
</tr>
</tbody>
</table>

Table 5.6. Chloride ion content for the five concretes for all couples at 25mm depth around the steel bars, after (Age 5 years).

The results showed that the lowest chloride ion content was for OPC/PFA and OPC/GGBS cement, that was expected due to the higher chloride binding of the blended cements, Section 2.2. However, the chloride ion content of OPC cement was lower than SRPC cement because of the beneficial effect of the higher C\(_3\)A in OPC. Moreover, the results showed that the highest chloride ion content was found in OPC(0.6) cement, due to the larger capillary pores in this cement that make easy route for the chloride ions. A porous concrete, in which the voids are interconnected, allows the penetration of aggressive agents, thereby increasing the available area for reactions to take place and therefore leading to intensive attack (Neville 1995). The results showed that OPC(0.45) and SRPC (0.45) cement that were ponded with chloride solution only had lower chloride ion content compared to both cements ponded with chloride and sodium solution. Similar findings has been reported by Al-Amoudi & Maslehuddin (1993) that showed a substantial decrease in the chloride binding capacity of cements immersed in sodium chloride and sodium sulphate solutions, thereby releasing the chloride ions into the pore solution. The free chlorides so liberated cause depassivation of steel (Maslehuddin et al 1997).
5.4.4 SUMMARY

The ten concrete couples made with OPC (0.6), OPC (0.45), SRPC (0.45), with PFA or GGBS can be ranked in terms of visual examination and current measurements, for the embedded steel bars as follows:

Best: OPC (0.45) and OPC (0.45) (uniform corrosion along the bar)
SRPC (0.45) and SRPC (0.45) (uniform corrosion along the bar)
OPC (0.45) and OPC/PFA (0.4) (little corrosion on the bars in OPC/PFA (0.4)
OPC (0.45) and OPC/GGBS (0.4)(little corrosion on the bars in OPC/GGBS (0.4)
SRPC (0.45) and OPC/PFA (0.4) (little corrosion on the bars in SRPC (0.45)
OPC/PFA (0.4) and OPC/GGBS (0.4) (corrosion on the bars in OPC/GGBS (0.4)
SRPC (0.45) and OPC/GGBS (0.4) (corrosion on the bars in SRPC (0.45).
OPC (0.45) and SRPC (0.45) (corrosion on the bars in SRPC (0.45)
OPC (0.45) and SRPC (0.45) (sabkha) (corrosion on the bars in SRPC (0.45)

Worst: OPC (0.6) and OPC (0.45) (corrosion on the bars in OPC (0.6)

Based on the results of polarisation resistance, current measurements and visual examinations on the magnitude of the formation of anodic and cathodic sites of different couples with interface, it can be related to the following factors:

- The type and quality of cement used, the mix design and properties of concrete
- Degree of curing
- Thickness of concrete cover to reinforcement
- Details of the construction (i.e. interface)
- Electrochemical (Exposure) conditions of the reinforcement in contact with solution.
It would appear that the ability of the interface to resist the electrochemical corrosion process is not necessarily limited to the performance of the adjoining concrete materials as single material but how they perform as a couple. This was clear in the couple of PFA and GGBS concretes that led to the formation of cathode and anode sites.

The final result in this study concludes that using two concrete types in the same structure will lead to the preferential formation of anode and cathode sites. Thus, even when using two types of blended cement concretes in one structure will cause the formation of cathode and anode sites, but the corrosion rate was less than those of bars embedded in the couple of OPC (0.45) and SRPC (0.45) concrete. This confirms the earlier observations for the comparison between the connected bars embedded in a different concrete couples, Section 5.4.1.2.4 and Table 5.3, that the formation of anodic and cathodic sites for two concrete in a couple can be predicted based on the magnitude of the corrosion activity for a bar in single concrete.

Finally, it has generally been noted that the ideal couple is one which matches the properties of the other concrete as closely as possible. In this study, the couple made with OPC concrete only had the lowest current values at the end of 5 years. It can be also expected that using single blend such as PFA or GGBS concrete in one structure will lead to better performance than OPC concrete due to their lower porosity and permeability and higher chloride ions bindings.

The performance of the interface between the two different concretes with an interface was shown to be critical in most couples, but with different magnitude, due to the changes in the material properties. The differing behaviour of cement types may also have implications for steel, which, either through design or repair is embedded in different mixes. Thus, it is important to avoid using two different types of concrete in one structure exposed to marine environment.
CHAPTER 6. CONCLUSIONS

The corrosion behaviour of steel bars embedded in concrete mixes containing more than one type of cement was investigated. The main findings and conclusions are presented below.

6.1. PROPERTIES OF CONCRETE

- The strength development of the various concrete mixes that were used in this study exhibited the expected general trends. The short-term strength gain of the OPC mixes was greater than of the equivalent concrete made with either PFA or GGBS. The long-term strength (measured at 400 days) show that both the PFA and GGBS blends continue to gain strength with time reaching compressive strength better than the equivalent OPC mix.

- The density and pulse velocity of the concrete mixes followed the general trends expected. The density and pulse velocity of all the mixes was found to increase marginally with increasing the curing time and decreasing w/c ratio.

- Based on the measured pulse velocity, all the mixes used in this study could be considered of good to excellent quality (Neville 1995).

- ISAT values for the hardened concrete at 400 days showed that all of the concrete mixes had low absorption characteristics (Levitt 1969). In terms of ranking, the OPC (0.6) concrete was found to have the highest ISAT value, the measured ISAT values decreased in the order of SRPC (0.45), OPC (0.45), while the PFA and GGBS blends having the lowest values. This is consistent with the expectation that long-term cure of the blends leads to greater pore filling due to continued long-term hydration.

- Moreover, it can be concluded that the quality of the OPC (0.6) concrete was generally inferior to the OPC (0.45) concrete, while the blendes of OPC/PFA (0.4) and OPC/GGBS (0.4) showed the best overall quality.
6.2. BEHAVIOUR OF CONNECTED BARS EMBEDDED IN DISSIMILAR CONCRETE MATERIALS

- Steel bars that were electrically connected but which were embedded in concretes manufactured from different cement types in the absence of a direct interface have been found to become anodic or cathodic with respect to each other. The formation of either anode or cathode is controlled by the environment (cement type) around the individual bars.

- Based on the measured potential of steel bars embedded in concrete with a single cement type, and without a direct interface, it is possible to rank the different environments (Table 5.2) that generally can be used to predict the likelihood of a bar becoming anodic or cathodic when connected to another bar in a concrete with a different cement type.

- Long-term study of bars that were electrically connected, and were embedded in concretes manufactured with different cement types and without a direct interface exhibited reduced times to onset of corrosion when subjected to chloride ion ingress. In the worst case, the connection of the a bar in OPC (0.45) concrete and one in SRPC (0.45) concrete caused the bar in the SRPC (0.45) mix to start corroding after only 3 cycles (6 weeks) compared to 15 cycles (30 weeks) for a single unconnected bar in the same SRPC (0.45) concrete.

- It was observed that in the short-term (1-day to 3-months), bars in OPC (0.32) and OPC/GGBS (0.32) mortars can swap from being anodic to cathodic with respect to one another. This may reflect the changes in pore structure and chemistry associated with the ongoing hydration of the various cement types employed.

- In particular, the hydration process of both the OPC/PFA and OPC/GGBS blends are known (from strength development) to continue for a considerable period. In comparison, both the OPC (0.45) and SRPC (0.45) mixes achieved most of their properties within 2 to 3 months, although they still continue to react slowly if they remain wet.
• For most of the connected couples tested, it was observed that as the time of immersion in NaCl solution increased, the potential difference between the bars decreased as both bars become actively corroding in the presence of chloride ions. Thus, the observed anodic/cathodic behaviour of the connected bars, which become clearly established after several months, is eventually swapped by the impact of the chloride ions, which cause both bars to undergo active corrosion.

6.3. SPECIMENS WITH A DIRECT INTERFACE

• Measurements of the half-cell potential, polarisation resistance and corrosion current, all confirm that a single bar passing the interface between two different concretes manufactured with different cement types can suffer the formation of anode and cathode regions that depend on the local environment. For example, a steel bar passing from an OPC (0.45) concrete to a SRPC (0.45) concrete will preferentially form an active anode within the SRPC concrete. This has implications for concrete structures manufactured using SRPC foundations and OPC superstructure, as often used in the Arabian Gulf States. Similarly, a bar passing from OPC (0.6) to OPC (0.45) will preferentially become anodic within the concrete with the higher w/c ratio.

• Within a given couple arrangement, it is possible to predict which region will become anodic with respect to the other based on the data for the individual bars in a concrete with a single cement type.

• It is an important observation that specimens manufactured with the same concrete on each side of the interface did not suffer from the preferential creation of anode and cathode sites and therefore exhibited generally much greater resistance to the initiation of corrosion when subject to chloride ions ingress. Indeed even if corrosion had been initiated in a bar passing between OPC (0.45) and OPC (0.45) or SRPC (0.45) and SRPC (0.45), the observed corrosion current was much less than a bar passing between OPC and SRPC subjected to the same chloride environment.
• After 5 years of testing, the bars broken out from specimens containing an interface between concrete manufactured with different cement types exhibited very clear evidence that one side of the bar had become anodic relative to the other, Figure 5.10.

• Taken together, the potential, polarisation resistance, corrosion current, and visual evidence all confirmed a consistent behaviour of bars that pass an interface between two concrete types. It has been found that steel bars can preferentially form anode sites in certain materials. In general, the worst behaviour was observed when a bar passed from OPC (0.6) to OPC (0.45), followed by OPC (0.45) and SRPC (0.45) concrete.

• The behaviour of bars passing across an interface was consistent generally with the behaviour of the electrically-connected bars embedded in concrete with different cement types but with no direct physical interface. Thus, the initiation observed in behaviour of different couples was not dependent on the presence of a physical interface. The only requirement is a direct electrical connection between the anode and cathode regions and a continuous ion migration path.
CHAPTER 7. RECOMMENDATIONS FOR FUTURE WORK

Upon completing this research program, it was decided that the following areas would benefit from further research:

7.1. ENVIRONMENTAL CONDITIONS

- It is recommended to investigate the influence of the Arabian Gulf climate on the various properties of concrete structures containing more than one type of cement with interface.

- It was reported that Sabkha environment, which is present in the Arabian Gulf region, might contain magnesium sulphate in addition to sodium sulphate and sodium chloride. Therefore, it is important to examine the influence of a solution of all these materials together on the properties of concrete structures containing more than one type of cement with the presence of interface.

7.2. CORROSION TESTING

- The application of accelerated testing conditions, such as longer dry cycles, should be investigated and compared with those used in this study.

- A more detailed study should be conducted on the polarisation resistance and corrosion current measurements of bars embedded in concrete mixes, which contain more than one type of cement with the presence of interface, when placed with a varying cover depth and measured at different time intervals.

7.3. DURABILITY

- Investigate the influence of varying the blend replacement level on the properties and durability of concrete mixes that contain more than one type of cement with the presence of interface.

- The chloride content measurements, especially around the interface area, should be assessed at different time intervals and different depths. These
measurements should be compared and related to the potential and corrosion measurement.

• The influence of using more than one cement type with an interface on the porosity and permeability of concrete structure should be investigated further especially at early ages.
CHAPTER 8. REFERENCES


• BS 1881-203 (1986). Recommendations for measurement of velocity of ultrasonic pulses in concrete.


• DT BD27 (1986). Department of Transport, Materials for the repair of concrete highway structures.


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• Stephen K (1996). Reinforcement electro potentials half-cell potential measurements, Technotrade, UK.


• Verbeck G J (1968). Field and Laboratory Studies of the Sulphate Resistance of Concrete, Performance of Concrete-Resistance of Concrete to Sulphate and Other Environmental Conditions, Thorvaldson Symposium, University of Toronto Press, Toronto, pp. 113-124.

CHAPTER 9. APPENDICES

The sample for the calculation of chloride concentration (content) % is shown below.

<table>
<thead>
<tr>
<th>Concrete sample</th>
<th>Reading</th>
<th>Sample</th>
<th>ACID ppm(mg/L)</th>
<th>Cl' concentration (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC (0.6)</td>
<td>9.8</td>
<td>34.4</td>
<td>81.7</td>
<td>OFF (1234)</td>
</tr>
<tr>
<td>SRPC (0.45) sabkha</td>
<td>9.2</td>
<td>38.3</td>
<td>71.5</td>
<td>OFF (1020)</td>
</tr>
<tr>
<td>OPC (0.45) sabkha</td>
<td>9</td>
<td>30.98</td>
<td>41.5</td>
<td>OFF (963)</td>
</tr>
<tr>
<td>SRPC (0.45)</td>
<td>7.45</td>
<td>40.7</td>
<td>74.4</td>
<td>566</td>
</tr>
<tr>
<td>OPC (0.45)</td>
<td>4.6</td>
<td>40</td>
<td>82.5</td>
<td>200</td>
</tr>
<tr>
<td>PFA</td>
<td>3.4</td>
<td>37.1</td>
<td>84.3</td>
<td>119</td>
</tr>
<tr>
<td>GGBS</td>
<td>3.2</td>
<td>24.1</td>
<td>68.7</td>
<td>108</td>
</tr>
</tbody>
</table>

Table 9.1. Chloride ion content for the five concretes for all couples at 25mm depth around the steel bars, after (Age 5 years).

The density of SRPC concrete mix cement = 2350 Kg/m³

SRPC (0.45)

The reading 7.45 is equal to 566 ppm(mg/L)Cl on the scale

\[ W_{\text{acid}} = 74.4 = 75\text{ml} \]

\[(566/75) \times 1000 = 7,547 \text{ (42.45mg Cl)}\]

The total proportions of SRPC mix for cement, sand and agg. was 4.9

\[ W_{\text{sample}} = 40.7 \text{ gm} \]

\[ 40.7/4.9 = 8.3 \text{ gm of cement from SRPC} \]

The content of Chloride ( %) = \[(42.45/8.3) \times 100 = 0.5\%\]