CONCRETING AND HIGH PERFORMANCE CONCRETE IN HOT WEATHER

Proceedings of the

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Edited by:

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Khaldoun N. Rahal

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Kuwait Chapter

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Preface

Since its establishment in 1997, American Concrete Institute – Kuwait Chapter (ACI-KC) took on the responsibility to improve local concrete practices and spread awareness of the best techniques suitable to the local environment and quality of workforce. It promoted and encouraged technical forums that serve as a platform to exchange experiences and find sound engineering solutions to specific regional problems. ACI-KC maintained its duty to spread knowledge to improve concrete practice and serve the community.

During the past few years, ACI-KC conducted many technical activities. It organized three major exhibitions accompanied by a series of technical seminars, conducted many technical training courses, organized technical seminars, published technical documents, and participated in local and regional technical activities in addition to many other technical and social activities. The ACI Executive Vice President, Mr. James G. Toscus, stated that ACI-KC is the only international chapter that has published technical documents based on local experience (Guide to Damage Assessment and Repair of Concrete Structures, Guide to Proper Concreting Practices Part-I Mixing and Curing, Guide to Implementation of Concrete Repair, Guide to Proper Concreting Practices Part II Placement, Compaction and Finishing). Mr. William R. Folley, Senior Managing Director of ACI International has noted that ACI-KC is one of the most active and well-organized international chapters.

To continue this track record, the ACI-KC Board of Directors decided to organize its first international conference from September 29 to October 1, 2003. The theme of the conference was selected based on the most relevant topic to this region, *Concreting and High Performance Concrete in Hot Weather*. The topics covered by the conference include: new trends in construction materials, durability and deterioration, specifications and codes, repairs and retrofitting, quality assurance and control, and advanced construction technology. A total of 38 abstracts were received from 10 countries and twenty-five technical papers were finally accepted for this conference and are included in the proceedings.

ACI-KC hopes that this document adds to the technical knowledge in the theme of this conference. We look forward to the readers' constructive comments and feedback to improve our future activities.

Dr. Naji M. Al-Mutairi, Chairperson
ACI-KC First International Conference and Fourth Exhibition

Acknowledgement

The success of this conference is attributable to the organizing and scientific committees and the support of ACI-KC Board of Direction. To insure a successful event, members of the committees contributed enthusiastically to the various aspects of the organization of the conference activities for over fourteen months. Without their active participation, determination and dedication, it would not be possible to organize the conference.

ACI-KC highly appreciates the encouragement and help of the Kuwait Society of Engineers president Eng. Adel Al-Kharafee, past president Dr. Hasan Al-Sanad, General Manager Eng. Talal Al-Kahtanee, Eng. Tareq Al-Saqabee, Eng. Abdullah Al-Daajanee, and many more colleagues at the society.

The organizing committee expresses its deep appreciation to the keynote speakers, the chairpersons of the scientific sessions as well as the distinguished speakers and participants for the effort and contributions they made during the conference activities. Particular thanks are extended to Tin Cho Mohammed for his devotion to the conference administrative activities and for Tareq Abduljaleel, Melita Alvares, and Sheba Mathew for their help.

Finally, the organizing committee wishes to extend special thanks to the sponsors of this conference, Kuwait Society of Engineers, Kuwait Foundation for the Advancement of Sciences, and ACI-International. The support of the co-sponsors, Gulf Consult and Kuwait Portland Cement Company is also gratefully acknowledged.

Proceedings of the ACI-KC First International Conference & Fourth Exhibition. September 29-October 1, 2003, Kuwait — ***Concreting and High Performance Concrete in Hot Weather.***

ELECTROCHEMICAL SOLUTIONS FOR DURABLE CONCRETE REPAIRS

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ABSTRACT:

The understanding of the electrochemical nature of corrosion has led to the realization that durable repair of reinforced concrete entails more than simply replacing defective concrete and corroded reinforcing steel. Concrete Repair technology has developed in recent years and now employs the use of shrinkage controlled mortars; high fluidity micro-concrete; as well as nondestructive methods such as Electrochemical Chloride Extraction, Realkalization and more recently, internal galvanic anode technology. The availability of impressed current as well as galvanic techniques provides a ‘third generation’ approach to arresting the widespread problem of concrete deterioration.

The paper will elaborate on the importance of addressing the electrochemical activity relating to rebar corrosion, as well as discussing the various techniques available to address the root causes rather than just the symptoms; and hence provide long-term, durable concrete repair solutions. The application of such techniques to a range of situations ranging from small patch repairs to major rehabilitation of marine structures is examined.

Keywords:
Reinforcement corrosion, incipient anode effect, electrochemical techniques, chloride extraction, realkalisation, cathodic prevention, galvanic protection

1. INTRODUCTION

The corrosion of reinforcing steel in concrete is a problem which has been highlighted since the 1960's. As the Gulf region has rapidly developed over the past decades, the use of reinforced concrete has emerged as the most widely used construction material due to its versatility. However, it is not uncommon to see extensive corrosion of reinforcing steel in diverse structures including industrial & residential structures; ports and power & desalination plants; and which is caused by factors such as inadequate specifications, poor workmanship and/or contamination by deleterious elements, chlorides being foremost amongst these in the Gulf region.

The highly alkaline nature of concrete leads to the formation of a passive oxide layer on the surface of the reinforcing steel that reduces corrosion to negligible levels. However, the permeability of concrete can allow the ingress of chemical agents which lead to a breakdown in the protective passive layer and subsequent corrosion of reinforcing steel [ref.1]. The two most commonly encountered processes leading to rebar corrosion are (a) carbonation in which a pH reduction in the concrete pore solution is induced by the action of carbon dioxide and (b) chloride attack [ref. 2]. These processes lead to breakdown of the passive oxide layer and subsequent formation of expansive corrosion products, which can lead to cracking and spalling of the concrete surface.

Chloride attack on reinforcing steel can be as a result of chloride salts which have diffused in from the external surface, for example, wind-blown sea salt, or from cast-in chloride salts. Above a certain chloride level threshold [ref.3], steel corrosion is initiated. According to Tutti [ref.4], initiation is followed by a propagation period, which eventually leads to a cracking/spalling stage.

Many methods of concrete repair have been developed over the past few decades, however, it is now understood that in order to overcome the highly damaging effects of corrosion, the only durable course of action is to address the electrochemical nature of the problem. A variety of solutions to the problem of reinforcement corrosion exist which can be used to treat structures in the initiation/propagation phases or following failure [ref. 5]. In recent years, improved understanding of the corrosion process has led to the development of electrochemical techniques, which attempt to address the actual cause of the problem rather than just the symptoms.

Cathodic protection, a technique developed in the 19th century [ref. 6], is used extensively for galvanically protecting ships hulls and more recently offshore structures. In the 1970's, impressed current cathodic protection (ICCP) was first applied to a reinforced concrete structure [ref.7] and has since been applied to a number of structures suffering from chloride induced corrosion problems. In the Middle East alone, it is now estimated that over 500,000m^2 of reinforced concrete has been cathodically protected.

Another option is to use galvanic anodes to supply the required protection current to the steel, a technique that has been used preferentially in applications where the absence of an external power source is deemed advantageous. This paper describes a number of electrochemical treatments for protecting reinforced concrete structures, with examples of actual installations and supporting data obtained.

1.1 Concrete Repair Options

Understanding the cause and extent of reinforcement corrosion is the key to selecting an appropriate repair solution. A number of corrosion control techniques are now available to the building owner or engineer to prolong the life of the structure and minimize future maintenance. The solutions have been developed to meet the wide range of cost/lifetime requirements of building owner, and include Realkalization, Chloride Extraction, Impressed Current Cathodic Protection (ICCP), and Galvanic Protection. Some of these techniques may be used proactively to prevent corrosion damage from occurring; whilst others may be used to complement traditional structural concrete repairs in order to ensure durability.

2. TRADITIONAL PATCH REPAIR

The most common method of addressing spalling concrete on structures is reinstatement with a formulated repair mortar. This involves removal of loose concrete and breakout to and beyond steel, removal of steel corrosion products through mechanical cleaning, priming, and application of the repair mortar. More recent technical advances have led to the development of mortars with low shrinkage characteristics to improve patch longevity.

Such an approach is effective in aesthetically upgrading the structure in the short term but does not address the problem of on going corrosion due to existing carbonation or chloride contamination. This is attributable to the fact that this repair technique does not guarantee removal of chloride bearing concrete, which may remain in areas adjacent to the repair. There is a possibility that the differential in chloride concentration between patch and parent concrete can lead to the formation of an electrochemical corrosion cell. This can ultimately lead to failure at the periphery of a patch repair – a phenomenon commonly referred to as the 'incipient anode effect' [ref. 8] or 'corrosion ring'.

A potential solution is to break out <u>all</u> contaminated concrete. This is effective but expensive, dusty, noisy and disruptive and may necessitate temporary propping of the structure due to mass concrete removal.

In order to overcome this problem, a zinc-based galvanic sacrificial anode was developed [ref. 9]. This development has been described in more detail elsewhere [ref. 10], but has proved to be a practical and cost effective method of alleviating the issue of 'incipient anode' formation by restoring an electrochemical balance.

3. GALVANIC PROTECTION TECHNIQUES (SELF POWERED)

Galvanic protection using sacrificial anodes has been known for more than 150 years, but it is only recently that sacrificial anodes have been used for protecting reinforced concrete. This operates on the principle of a more reactive metal than steel (usually zinc), corroding preferentially to the steel as a result of a difference in potential between the two metals; and the resultant electromotive potential gives rise to a corrosion preventive current.

3. 1 Internal Discrete Galvanic Anodes

As discussed earlier (Section 2), 'incipient anodes' caused as a result of such patch repairs, can be neutralized using embedded sacrificial anodes tied to the steel at the edge of the repair One system (Galvashield XP™), comprising a zinc/active mortar composite, is attached to the reinforcing steel at the edge of the required patch repair to prevent the setting up of 'incipient anodes'. No external rectifier is required as the galvanic anodes are effectively self powered. The zinc anodes slowly corrode sacrificially over their life time and display intelligent behavior by adjusting current output to reflect environmental conditions. Optional monitoring can provide the building owner or specifier with current output and polarization data if required. Table 1 shows typical current density results obtained from such data.

Table 1.- Typical Current Density outputs from monitored internal discrete galvanic anodes (ref. 19)

reading no.	**temp (deg C)**	**current output A1 (mA/m^2 steel)**	**current output A2 (mA/m^2 steel)**	**current output A3 (mA/m^2 steel)**	**Avg individual current output A3 (mA/m^2 steel)**
1	17	4.3	4.3	11.8	1.3
2	17	4	4	6.6	0.73
3	10	5.6	5.3	7.2	0.8
4	23	8.6	8.3	15.5	1.72
5	11	7	6	10	1
6	24	6.3	6.6	8.2	0.9
7	23	11	8.3	13.1	1.45
8	24	9	9	10	1.11
9	26	5.3	5.6	8.3	0.92
10	22	6.3	6.3	6.2	0.68
11	14	3.3	3.3	4.2	0.46
12	22	5	4.6	8	0.9
13	27	6	5.3	8.5	0.95
14	26	7.33	7.66	6.99	0.78
15	27	9.66	10.66	6.66	0.74
16	30	9.99	10.99	16.66	1.85
17	27	6.33	8.66	10.33	1.15
18	44	14.33	14.99	15.66	1.74
19	37	10.66	7.99	15.33	1.7
20	37	12.33	8.66	15.83	1.76
21	38	13.33	13.99	15.49	1.72
22	39	13.99	14.99	13.99	1.55
23		no reading	Monitoring unit connection problem suspected – then rectified		
24		no reading			
25	38	13.66	4.33	8.63	0.96
26	38	11.66	4.1	10.8	1.2
27	36	10.66	3.66	16.8	1.86
28	37	12.01	5.1	19.25	2.14
29	32	11.33	4.33	18.6	2.06
30	32	8.33	3.33	17.93	1.99
31	33	6.33	3.33	17.93	1.99
32	32	6	1.66	16.93	1.88
33	33	10.33	3.33	13.78	1.53
34	40	11.33	2.99	11.95	1.33
35	32	5.66	1	5.15	0.57
36	35	6.99	1.66	6.97	0.77
37	32	4.33	0.66	3.82	0.42
38	30	4.99	0.66	3.15	0.35
39	28	3.99	0.66	11.78	1.3
40	29	3.66	0.66	9.79	1.09

The 24 hour depolarizations taken vary from 30 to 100mV. This is what can be expected to achieve the level of protection required based on the background potentials of the areas outside of the zone of influence of the anodes.

A logical step is to extend the galvanic corrosion prevention to areas outside these repair patches. To this end, an enhanced sacrificial anode has been designed in a form which facilitates discrete installation into pre-drilled holes in reinforced concrete. This comprises cylindrical, mortar encased zinc anodes wired together in sequence and inserted into the pre-drilled holes in a grid configuration. When connected to the steel reinforcement the grid impresses a galvanic current through the steel reinforcement, suppressing anodic reactions and controlling corrosion. Such a system (Galvashield CC™) has been installed on a number of structures, suffering from chloride contamination, and trial data from a typical example is discussed below.

A19 Seaton Bridge trial

The current output data shows that all the anodes are providing good current output and the estimates of current density on the steel surface indicate that, according to the values in BSEN12696, should be capable of preventing pitting corrosion initiation and thus countering corrosion activity.

The magnitude of polarisation is such that it represents a significant reduction of the corrosion current and therefore corrosion rate.The trial has shown that this protection can be achieved on structures that are contaminated with chloride and at risk of corrosion.

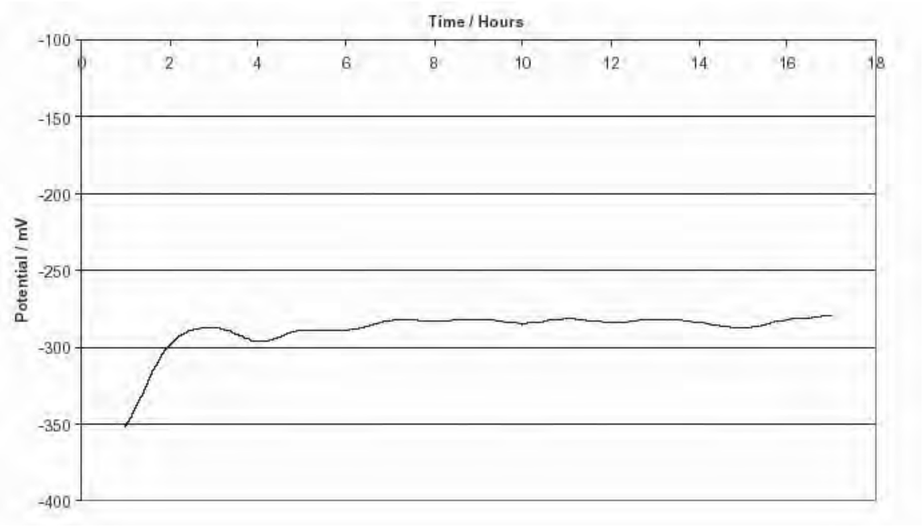

Figure 1 -Typical depolarization curve for reference electrode

The life of the anode system is directly related to the current output that is achieved and the results of the trial to date indicate that a typical anode life of 15 years can be achieved. [ref. 11]

This form of discrete galvanic protection system is a lower cost option than realkalization, desalination and ICCP. It can provide long-term, maintenance-free protection and is particularly useful for those seeking an easy-to-install system with no ongoing monitoring costs. It is suitable for all types of structures including pre-stressed and post tensioned structures where ICCP, realkalization and desalination are often precluded due to potential hydrogen evolution and the associated risk of steel embrittlement.

3.2. Marine Galvanic Jacket Techniques

Studies conducted by the Florida Department of Transportation (FDOT) have shown that zinc anodes can provide long term cathodic protection in the tidal zone of steel reinforced concrete structures. Pile Jacket systems with an inbuilt zinc mesh anode can represent an attractive alternative to conventional impressed current CP based on their low installation and maintenance costs, simplicity and by their reliable, self-regulating performance [ref. 12].

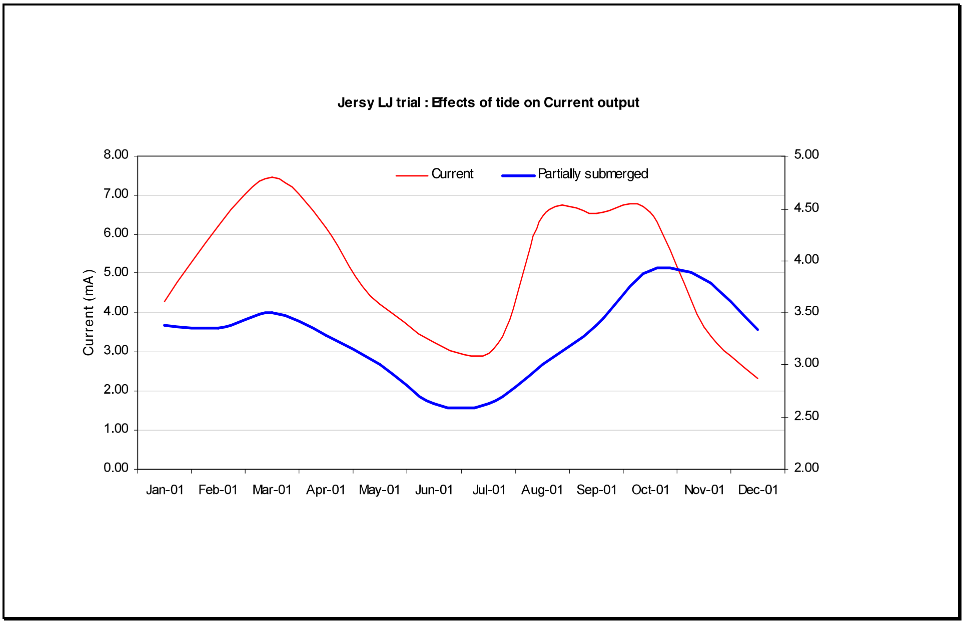

Figure 2 – Self – regulating current output vs. tide levels

The Zinc mesh pile jacket system amply satisfies the 100 mV polarization criterion, however due to the concrete remaining moist within the jacket cavity and the restriction of oxygen and chloride replenishment by the inert fiberglass form, the polarization decay criterion is not always satisfied at 4 hours, and the 24 hour value may be taken instead. The system would literally take months to fully depolarize if the wire connection became severed or disconnected (ref. 13).

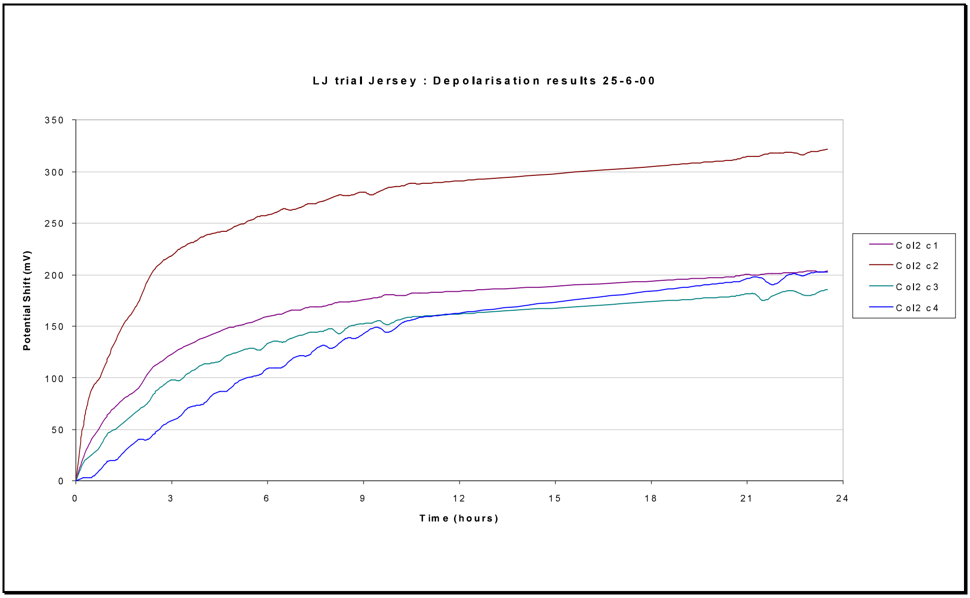

Figure 3 - Typical Depolarization vs. time curves for Galvanic Jackets

4. CHLORIDE EXTRACTION & REALKALISATION

In general, these nondestructive techniques are particularly appropriate in situations where condition surveys identify a significant risk of future corrosion; but in areas where the structure has not yet been affected by substantial delamination & spalling. Timely application of these techniques can thus avoid the need for costly & potentially disruptive patch repairs.

4.1 Electrochemical Chloride Extraction (ECE)

Sometimes also referred to as 'desalination', electrochemical chloride extraction [ref.14] is based on a process of reducing chloride ions from around the reinforcing steel in contaminated concrete down to a level below the corrosion threshold. This works on an induced current principle, whereby an externally mounted metallic anode (in this case a temporary one), is embedded into an electrolyte reservoir and a D.C current (typically in the range of 1 A/m^2 of concrete) is applied to the reinforcing steel which becomes a cathode. The (negatively) charged chloride ions are repelled away from the (negatively) charged reinforcing steel cathode and migrate towards the (positively) charged anodic mesh. Simultaneously, the electrolytic production of hydroxyl ions at the steel surface results also in the displacement of chloride ions and subsequent re-passivation of the steel with an effective buffer zone.

On delivery of the required no. of Amp/hours, the current is switched off, and the external anode, with its electrolyte reservoir is removed and discarded.

4.2. Electrochemical Realkalization (ERA)

Developed in the late 1980's to combat the growing carbonation problem in Scandanavia, ERA works on the basis of electro-osmosis and can be used to realkalize carbonated concrete by creating the existing reinforcement as the negative electrode, or cathode [ref.15]. As with ECE, a temporary external metallic anode mesh is installed and embedded in a disposable electrolytic mass containing an alkaline solution. Ionic migration occurs on applying a voltage between the two electrodes under the influence of current and electro-osmosis.

Over time (typically 4 to 8 days), the concrete is saturated to beyond the cover zone with the alkaline solution reinstating the pH level of the concrete to an initial level of 12, thereafter equalizing to around 10.8. When the entire cover concrete is impregnated, as evidenced by phenolpthalein indicator test, the current is switched off, and the external anode, with its electrolytic mass, is removed and discarded. A protective coating can then be applied to the surface of the structure to prevent further carbonation or chloride ingress.

5. IMPRESSED CURRENT CATHODIC PROTECTION (ICCP) (EXTERNALLY POWERED)

This is now the oldest of the electrochemical techniques and has been recommended for use on reinforced concrete structures since the 1970's. This technique involves the application of a low, direct current from a permanent anode to the reinforcing steel. Sufficient current is used (ranging from 0.25 to 25 mA/m^2 of steel) to prevent anodic reactions at the steel reinforcement and to maintain the reinforcing steel in a cathodic state thereby protecting it from further corrosion. One of the features of the system is the permanent requirement for a D.C power source (Rectifier unit) and a permanent anode system. In addition to the cost of initial installation, ICCP requires investment in periodic monitoring to ensure that the system is operating effectively.

Anodes for use in ICCP applications can take a variety of forms dependent on the type of structure to be protected and the practicalities of installation; and can utilize either discrete or external anode systems.

5.1 External Iccp Anodes

External systems can comprise either titanium mesh anodes overlaid by a thin layer of low resistivity mortar; titanium ribbon placed into slots in the concrete; or conductive paint systems.

MMO coated titanium wire mesh is commonly used to protect large flat areas of reinforced concrete. Installation of the mesh on the structure is followed by the application of a sprayed cementitious overlay, as described more fully elsewhere [ref.5], however, this technique may not be appropriate for every situation, where deadweight or appearance restrictions apply.

MMO coated titanium ribbon is applied into chases cut into the concrete surface at an approximate spacing of 300mm. The ~2cm width ribbon can be applied in either dimension and is then grouted using an appropriate material. The technique is generally applied to structures having a reasonable depth of cover, as this avoids potential electrical shorts.

Conductive paints/mortars can be applied to the surfaces of reinforced concrete structures and will behave as anodes provided continuity can be maintained between the concrete surface and the coating. These materials can prove cost effective on structures having relatively low steel densities, and where environmental conditions facilitate application.

5.2 Discrete Iccp Anodes

The facility to bury the discrete anodes within the concrete structure allows protection of deeply buried steel or multiple layers which may not be protected by a surface mounted system. In addition the applied system adds no weight loading to the structure, and can easily be applied to difficult geometry. Several units are linked together to the power supply, the density of units can be varied dependent on required steel current density

One type of a typical discrete system comprises cylindrical, mixed metal oxide (MMO) anodes connected by wire and grouted into drilled holes adjacent to the reinforcing steel.

An alternative discrete anode system (Ebonex) comprises a cylindrical material (~7-28mm diameter x 75-600mm length) formed from a patented electrically conductive ceramic tube. This material is highly corrosion resistant and as such is capable of withstanding high applied current densities required to protect adjacent reinforcing steel.

The Ebonex discrete anodes are installed into holes drilled into the concrete structure between reinforcing bars, an acid tolerant grout being used to fill the annulus.. Each discrete anode is equipped with a vent tube which allows discharge of gases formed at the anode surface during operation.

The operation of an anode in CP systems applied to reinforced concrete will result in the effective consumption of water and generation of oxygen and H+ ions, at the anode surface. It is important that the applied anode/mortar system is stable under these conditions, as acid etching can result in void formation around the anode, as has been observed previously in the case of mesh anodes. This will lead to steadily increasing circuit resistance and the formation of discontinuities, with ultimate failure of the system.

Trials have demonstrated that the high alkalinity of the Ebofix grout assists in neutralizing the acid generation, and the gas venting system allows dissipation of gaseous oxygen. The Ebonex anode/Ebofix mortar composite remained intact in full contact with the surrounding concrete matrix, even at exceedingly high current levels of ~900mA/sqm. In contrast an unvented titanium tubular anode demonstrated severe damage to the surrounding mortar, with substantial loss of cement binder [ref.16].

These results confirmed the hypothesis that both a formulated mortar of high alkali reserve and a gas venting mechanism should form an essential part of a discrete anode design, and with these items in place, long term performance of the Ebonex discrete anode system should be assured.

Site evaluation data of Ebonex discrete anodes in UAE.
Ebonex anodes were installed in a grid pattern, at locations between the two layers of reinforcing steel, with anode separations of 600mm and 1m. These units were powered up using a standard rectifier/control unit and half-cell potential measurements were taken at

various times using a copper/copper sulphate reference electrode, to determine the extent of protection offered by the CP system. Current output was 19mA/m^2 of steel.

Table 2 – 24 hour Depolarisation data from UAE Ebonex trial.

Ebonex Anode reference	Distance from Reference Electrode to Ebonex Anode (cm)	24 hour Depolarisation (mV)
A1	40	172
	50	163
	60	69
	70	89
A2	20	195
	50	119
	60	80
	65	79
A3	25	193
	45	102
	60	136
A4	10	158
	45	123
	50	111

The data in Table 2 clearly demonstrate that a significant level of polarization has been achieved, and in fact the criterion of 100mV depolarization detailed in NACE 0290-90 is achieved at a distance of over 50cm from the discrete anodes, suggesting cathodic protection has been conferred. It is also clear that the influence of the discrete anode unit on steel polarization decreases with distance as would be expected form the increasing contribution from the concrete resistivity.

6. CONCLUSIONS

Concrete repair can be an expensive business. It is labour intensive, access costs are often high, and there may be a consequential loss of income due to temporary closure of the structure.

By embracing the technologies described above, it is possible to offer the building owner or specifier a full range of repair and protection options to ensure that their concrete repair project addresses not only visible defects but the hidden corrosion that, if left unchecked, may incur even more costly and disruptive repair works in the future. These technologies are now gaining significant track record around the world, with meaningful data providing a high degree of confidence with a number of clients and specifiers.

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AC IMPEDANCE TECHNIQUE FOR CONCRETE DURABILITY AND CORROSION TESTING

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ABSTRACT

Corrosion of steel in concrete is one of the major problems with respect to the durability of reinforced concrete structures. Predictions concerning comparative performance of commercial quality mix design of ordinary Portland cement (NoCl-cast), chloride containing cement mortar (Cl-cast), calcium nitrite in mortar cement (CN-cast) and silica fume (SF) in mortar cement have been investigated.

The main objective of the investigation is to present comparative data on electrochemical impedance spectroscopy (EIS) for steel reinforcement in mortar cement with two different admixtures during the hydration process of cement (i.e. 28 days).

The Impedance behaviors of the small specimens are studied using a special corrosion cell designed to accommodate small test specimens of mortar cement. Thereafter, to qualitatively characterize the EIS spectrum for each system in terms of Nyquist and Bode plots, as well as in terms of the general equivalent circuit elements.

Results of examples from laboratory and site mixes are presented to illustrate the viability of EIS to be used as a diagnostic tool for characterizing the protective quality and behavior of the proposed mortar concrete mix design.

In this study EIS has been found to be rapid and accurate technique for measuring durability and more specifically corrosion rate of reinforced steel in concrete. Therefore, EIS has a great potential to be used very effectively to distinguish between acceptable and un-acceptable concrete mix design for field application practice.

Keywords: Electrochemical Impedance Spectroscopy (EIS), calcium nitrite, silica fume, and ordinary Portland cement.

INTRODUCTION

Electrochemical impedance spectroscopy (EIS) has been applied successfully to the study of corrosion systems for more than 20 years (Mansfeld 1982). The reason EIS has found increasing applications in corrosion research is because of the possibility of obtaining information about the chemical mechanism involved. An important advantage of EIS over other laboratory techniques is the possibility of using very small amplitude signals without disturbing the properties being measured. Another major quality of EIS is the possibility of working in low or variable conductivity environments. Thus EIS has been applied successfully to study the corrosion of Fe and carbon steels in neutral waters or deoxygenated formic acid (Zeller 1986).

The following points have been addressed on correlating the AC impedance measurement to the protection afforded by commercial quality admixture systems:

(i) To determine whether AC impedance test a reliable form of accelerated test for the smaller specimens, and how predictive can it be with the silica fume and calcium nitrite systems of commercial quality during dehydration process. (ii) Whether it is only useful as quality control or condition monitoring. The out put results from EIS behavior was examined and compared with respect to the lollipop specimens from previous experiments.

EXPERIMENTAL METHOD

The mortar mixture proportions of the samples prepared for this experiment were designed to yield a moderate range of transport properties and are based upon concrete mixture proportions from previous experiments. The cement was ASTM Type 1; the chemical composition of the mix proportion is given in Table 1. The pozzolanic mineral admixture was silica fume in slurry form. A chemical inhibitor in the form of Calcium Nitrite was used, as indicated in Table 1. The mixture proportion of the samples in this experiment for water cement ratio was a 75 % (by mass) aqueous solution. The chloride percentage by weight of the mixing water was 1.75 %.

Table 1. Laboratory-Prepared Concrete Mix Proportions

Specimen Type	Specimen Designation	Concrete Specifications
Plain Concrete	NoCl-cast	Type 1 ordinary Portland cement W/C 0.75
Plain Concrete with Cl	Cl-cast	Type 1 ordinary Portland W/C 0.75
Calcium Nitrite	CN-cast	Type 1 ordinary Portland W/C 0.75
Silica Fume	SF-cast	Type 1 ordinary Portland W/C 0.75

The mortar mixtures were prepared according to the procedures in ASTM C 109. The samples were cast in 100mm X 150mm molds made of Perspex materials, covered, and stored in a 100% relative humidity chamber. At 24 hours of age, the specimens were removed from the molds and stored in a humidity chamber until they were tested periodically at intervals of 28 days. Although no temperature controls were used, the laboratory temperature could be characterized by the interval 20± 2 ° C.

Each specimen was prepared for testing according to the specifications of the admixture proportion recommended by the suppliers of the materials. The present test cell and mold types were designed in order to have a small specimen of mortar cement concrete (10cmX15cm). The main advantage of this design is to study the response of the EIS spectrum on smaller sample and correlate it with the standard ASTM test specimen. The specimen consisted of polished steel rebars approximately 6.9 mm in diameter as working electrode and two identical graphite electrode approximately 8 mm in diameter as auxiliary electrodes.

The purpose of using two embedded graphite electrodes was to verify the validity of the conductivity measurement for EIS technique and to reduce overestimated scattering in the signal noise. Conductivity, an intrinsic property, is independent of specimen geometry and size. A reliable technique for determining specimen conductivity should obtain equivalent results from replicate specimens with different lengths.

AC impedance measurement and data acquisition was controlled using Solartron Model 1287 Potentiostat/Galvanostat interfaced with Solartron Model 1260 Frequency Response Analyzer (FRA) to provide sweep frequency measurement. Both instruments were interfaced with computer for data logging, storage, and analysis. The applied potential amplitude was in the range of (10 - 100 mV) in the nominal frequency range of 100 kHz to 10 MHz. The AC impedance test data were obtained at the open circuit potential.

The impedance tests were performed using Solartron instruments with two and three electrode configuration:

(a) Auxiliary with two embedded graphite electrodes (as counter electrode).
(b) Saturated silver/silver chloride electrode as a reference electrode.
(c) Test specimen reinforced steel bar in mortar cement (10 cm X 15 cm)
(i.e. as working electrode).

The AC impedance test cell consisted of two plastic sheets made of Perspex. The plastic sheets were rested on the top and bottom of each sample, thus holding the sample, a wet filter paper saturated with 0.5M NaCl solution was rested on top of the specimen for potential measurement with reference electrode, whilst EIS measurement were made. Three identical test cells were designed and constructed for this study and the mortar steel specimen selected for test was mounted horizontally.

RESULTS AND DISCUSSIONS

Electrochemical impedance spectroscopy (EIS) have been utilized in this study to characterize four different compositions of duplicated cement concrete samples. Predictions concerning comparative performance of commercial quality mix design of ordinary Portland cement (NoCl-cast), chloride containing cement mortar (Cl-cast), calcium nitrite in mortar cement (CN-cast) and silica fume in mortar cement have been investigated. The evaluation has been attempted first, to examine the comparative ranking of EIS spectra obtained for each mix system when combined or superposed together in one impedance plot with respect to the corresponding corrosivity of each protection system. The second reason is to assess whether the early predictions made by ASTM corrosivity test and Electrochemical Impedance Spectroscopy (EIS) for selected materials of Lollipop samples are still valid.

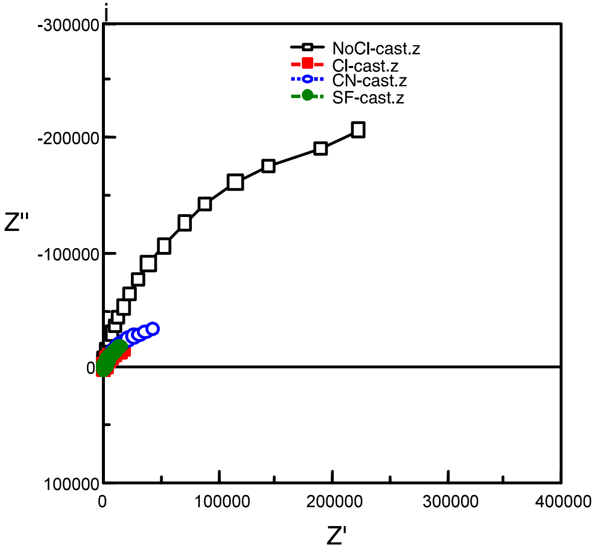
The impedance results of Fig. 1 (a, b) to Fig. 4 (a, b) indicated impedance spectrum in the form of Nyquist and Bode plots for the plain concrete (NoCl-cast), chloride concrete (Cl-cast), calcium nitrite mortar cement (CN-cast), and silica fume (SF-cast).

Fig. 1, represent EIS measurement results in the form of Nyquist and Bode plots for the mortar cement specimens taken immediately after casting while Fig. 2, 3 and 4 represent the same test obtained after 3, 14, and 28 days respectively during dehydration process in the humidity curing chamber.

Qualitative analysis of the Nyquist impedance spectrums have indicated that the corrosion performance of the different concrete mixtures are in agreement with the ASTM test results of corrosion rate and potential values obtained in previous experiments. The impedance results have shown that the lab prepared ordinary Portland concrete structure during casting exhibited

the highest impedance and capacitance values among the rest of the specimens. This is shown clearly in Fig. 1 a, b and Fig, 2 a, b during casting and after 3 days exposure, respectively.
In other word the presence of a Warburg diffusion response indicated that the slow diffusion processes of the water species through porosity in the material and formation of passive film

(a)



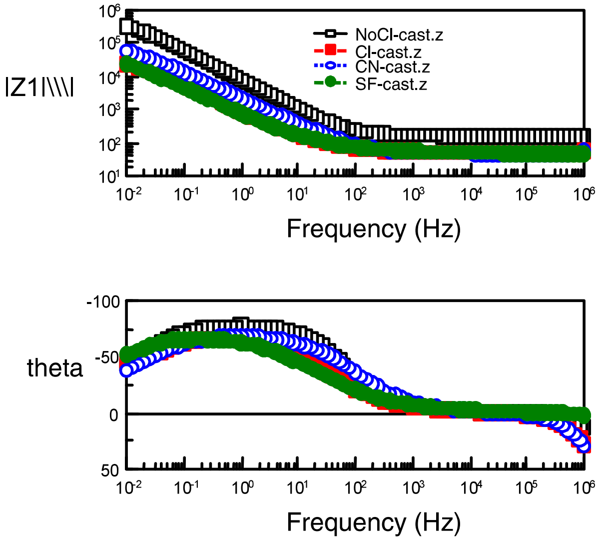


Figure 1. EIS Nyquist plot (a) and Bode plot (b) for the mortar specimens immediately after casting in the mold.

due to mortar alkalinity was the rate controlling factor for the mechanism of concrete corrosion protection at this stage

The natural process of alkaline passive formation and build up for this particular sample affects this process. Similarly water saturated concrete tends to show resistivities on the order of a few Kohm-cm, while oven-dried concrete can approach resistivity values typical of electrical insulator as it is reflected on the EIS spectrum taken before exposure for other concrete samples.

As it is expected from the protective nature of the silica fume mortar specimen (SF-cast), the dehydration process of SF concrete slowed down the diffusivity of the chloride species to reach the steel substrate. This is very much related to the mechanism of coalescence and interaction of the micro silica cement element in the pores structure of the concrete whilst in the dehydration process.

In contrast to the ordinary mortar concrete and silica fume mortar, the inhibited concrete (CN-cast) and chloride mix mortar cement (Cl-cast) concrete samples showed the lowest impedance and capacitive behavior. This is due to the high rate of permeability and adsorption of the chloride corrosive species to the steel substrate surface.

In the case of inhibited concrete sample, the attempt of studying such inhibitor with concrete is to show the viability of the EIS technique to be used as a tool for condition monitoring (Fig. 3 and 4). The EIS spectrum after 14 and 28 days is attributed to the improper application of inhibitor concentration and or due to under curing which may cause the corrosion process to become even worse than ordinary concrete cement without inhibitors (OPC).

The hypothesis behind the unexpected behavior of the inhibited concrete sample was more or less related to the improper film coverage and adsorption of the nitrite species to the passive oxide or hydroxide layer of steel surface with subsequent formation of unprotected micro-corrosion cells. If chloride ions arrive first as contaminant to the surface of the steel, the surface tends to become active when the molar ratio of Cl^- to Nitrite and hydroxyle ions in the pore solution reaches a critical value that exceeded 0.9. Microscopically, the concrete mix acts as a non-homogeneous electrolyte, which varies greatly with the overall moisture content, in addition to the rate and degree of diffusivity of corrosive species and inhibited ions such as oxygen, chloride and nitrite. This finding is in agreement with EIS spectrum previously obtained for the Lollipop ASTM samples.

In the case of the as-received specimen with no chloride addition an almost purely capacitive and diffusion response was observed during experimental testing. This effect has been reflected as straight spectra line in the Nyquist and Bode plot shown in the same figures (1 to 4) and even after 28 days of curing. There is little apparent change during this time, suggesting that the system is relatively stable and that there is no more water uptake and the material is acting as some sort a die-electric component.

In the case of the chloride contaminated concrete it is expected that Nyquist spectrum will reach its semi-circular shape and will become smaller and smaller with time. Similarly, the same EIS response would be expected from calcium nitrite specimen. Both specimens have shown a greater tendency toward reaching its charge transfer resistance and polarization resistance at a faster rate once exposed to the environment.

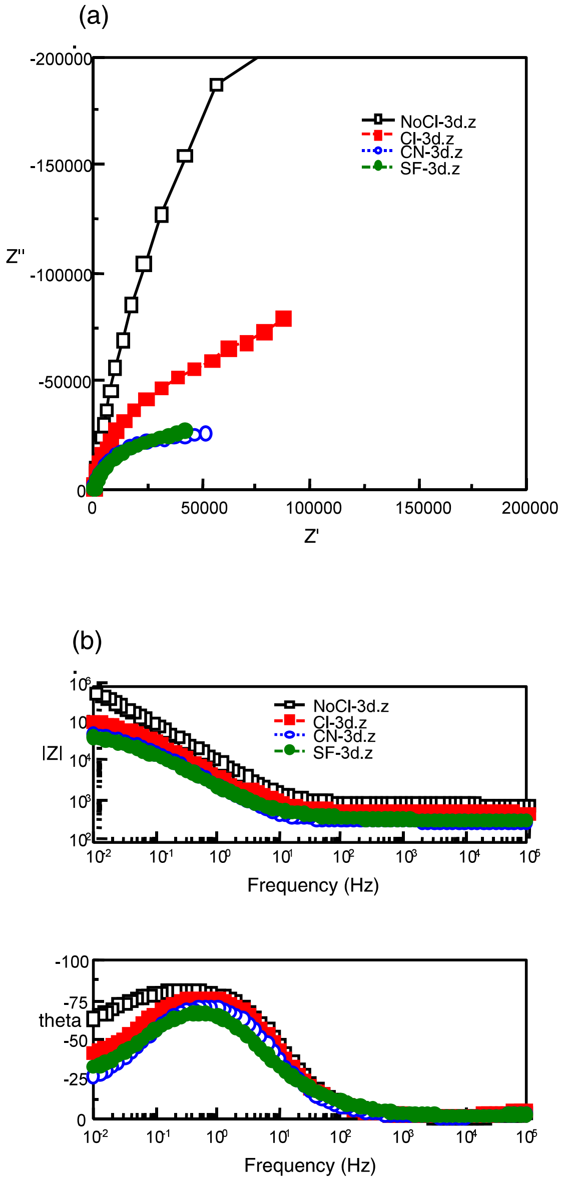


Fig. 2. EIS Nyquist plot (a) and Bode plot (b) for the mortar specimens immediately after 3 days of casting in the mold.

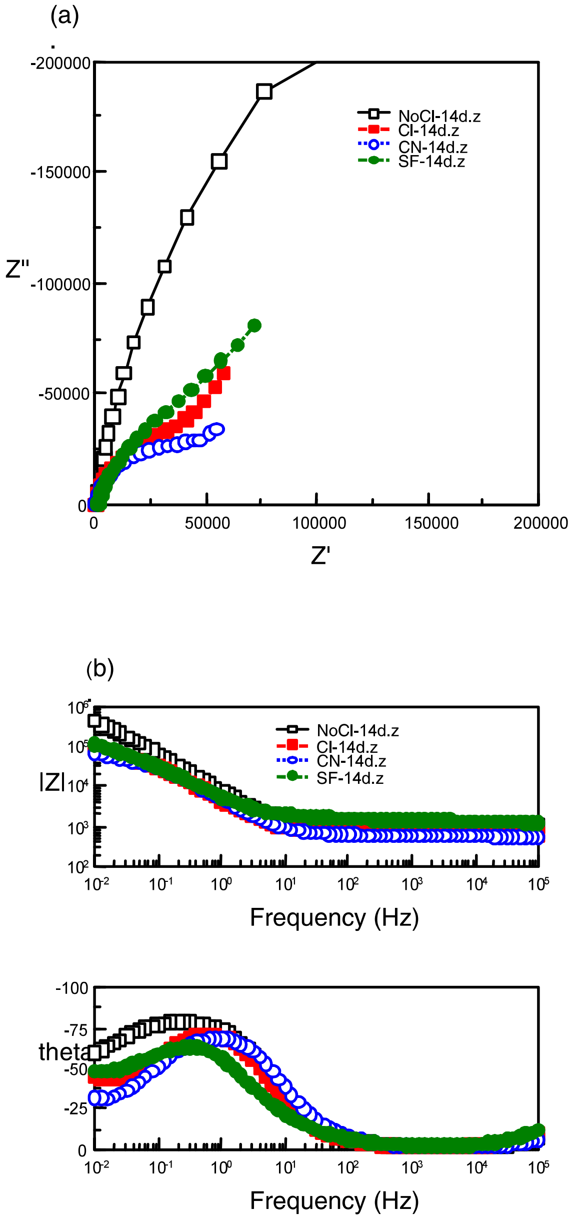


Fig. 3. EIS Nyquist plot (a) and Bode plot (b) for the mortar specimens after 14 days of casting in the mold.

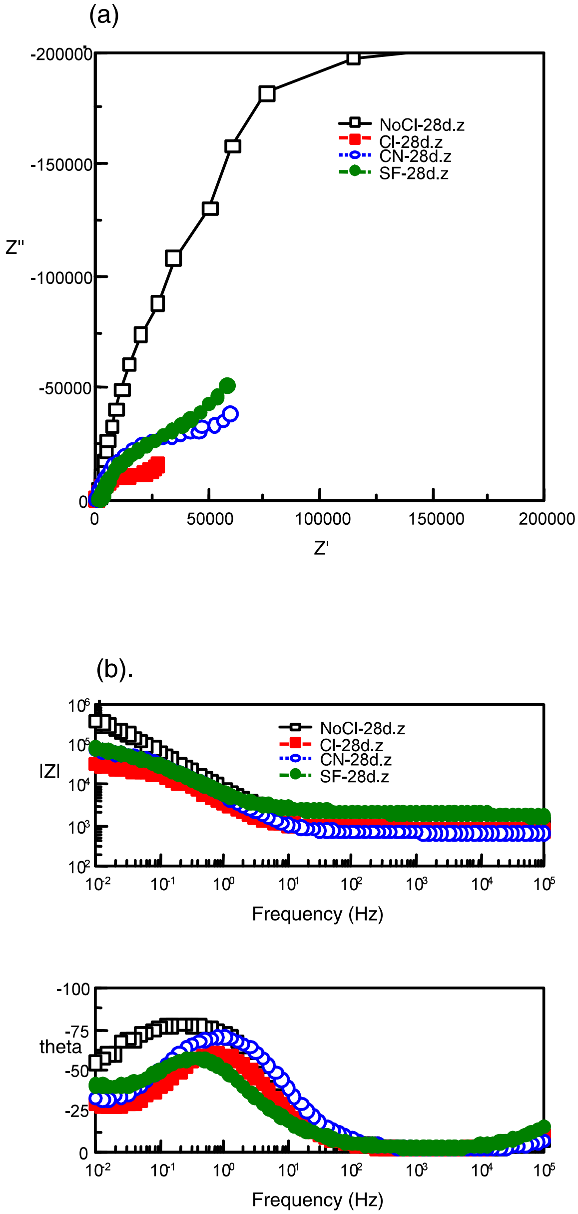
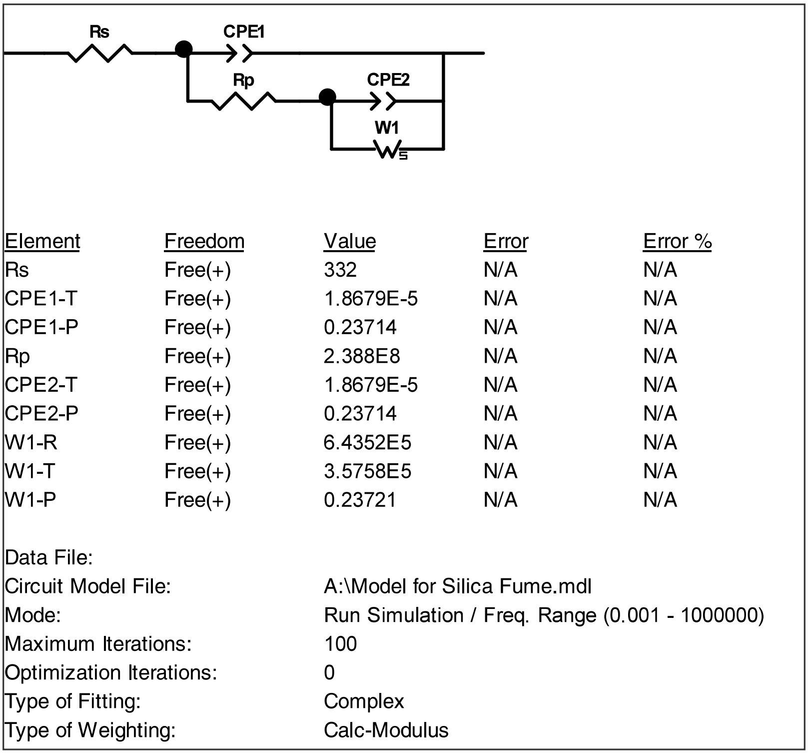


Fig. 4. EIS Nyquist plot (a) and Bode plot (b) for the mortar specimens after 28 days of casting in the mold.

EIS SPECTRUM ANALYSIS

The equivalent parallel R-C circuit in figure 5 explains and represents quite well the frequency impedance response of the concrete mix design systems used in this study.



Element	Freedom	Value	Error	Error %
Rs	Free(+)	332	N/A	N/A
CPE1-T	Free(+)	1.8679E-5	N/A	N/A
CPE1-P	Free(+)	0.23714	N/A	N/A
Rp	Free(+)	2.388E8	N/A	N/A
CPE2-T	Free(+)	1.8679E-5	N/A	N/A
CPE2-P	Free(+)	0.23714	N/A	N/A
W1-R	Free(+)	6.4352E5	N/A	N/A
W1-T	Free(+)	3.5758E5	N/A	N/A
W1-P	Free(+)	0.23721	N/A	N/A

Data File:	
Circuit Model File:	A:\Model for Silica Fume.mdl
Mode:	Run Simulation / Freq. Range (0.001 - 1000000)
Maximum Iterations:	100
Optimization Iterations:	0
Type of Fitting:	Complex
Type of Weighting:	Calc-Modulus

Fig. 5. Example of equivalent circuit model with curve fitting analysis for capacitance and impedance elements for Silica Fume metal/solution interface under-study.

The real meaning of C (or CPE1) and Rp for the complex system metal/concrete/electrolyte in this study has now been very well established. In the author's view, at the beginning of the test C_c could be related to the capacitance of the concrete mortar cover film, acting as a dielectric material. Whereas (R_p) the ohmic electrical resistance of the mortar film or (pore resistance), acting as a barrier to the permeation of substances across it. With time, new electrochemical processes would take place and the equivalent circuit would consequently change, introducing now the double layer capacitance (C_{dl} or CPE2) prevalent at the substrate/cement interface and charge-transfer resistance (R_{ct}) that combined together with the Warburg diffusion impedance (Z_W or W1s). Thus, the high frequency semicircle is considered to represent information on the concrete or mortar layer (C_c, Rs, and R_{po}), whilst the semicircle at lower frequencies is thought to indicate the information on the metal substrate corrosion tendencies in conjunction with (C_{dl} & R_{ct}).

In this study, to explain the overall variation of R_{po} data with time for silica fume mix design, two phenomena are assumed to occur simultaneously but with variable rates depending on the exposure or hydration curing time:

(1) The increase in pore density, and (2) the partial sealing of the pores by cement hydration process or iron corrosion products and/or other types of protective micron size species that become more prominent over long immersion times. (Hepburn 1986) reported similar observations, which described the depression of the low frequency semi-circle (shown in the previous figures) to the presence of two distinct types of corrosion sites: sites that are initiating and sites that are becoming blocked by corrosion products and or protective elements. The value of C_c is generally considered to provide information on the degree of water penetration or diffusion through the concrete and in principle its value is expected to increase with time. This behavior can be qualitatively analyzed as functions of time on the Nyquist plot that form series of semi-circles with reduced/increased diameters. In this work there is little variation of C_c with testing time for the smaller plain concrete samples (NoCl) and silica fume (SF) may indicate a high rate of film stability and dehydration combined with oxygen diffusion through the concrete mortar system. The same effect of EIS behavior was observed for the ASTM specimens in previous experiments (Abdul Salam 2002). The relatively constant value of C_c during exposure time for sample of concrete with Cl and calcium nitrite in Figs. (2 and 4). Suggests that the values of capacitance of concrete cover film of the small pathways or pores are much less than the parallel capacitance of the remainder of the film which do not contribute significantly to the overall capacitance, even though the pathway controls the performance of the concrete composition. Another assumption may be attributed to the weakness of self-inhibition nature of the calcium nitrite and the high aggressive nature of the destructive chloride species in localized density on the steel substrate and particularly during the initiation of pores.

CONCLUSIONS

1. This work has shown the feasibility of using EIS as a monitoring tool for detecting corrosion processes on steel reinforcement in concrete with mix design technology.
2. EIS is considered to be a viable and accelerated diagnostic tool for the corrosion evaluation of the steel reinforcement in concrete mix design system for both smaller specimens and larger ASTM Lollipop specimens.
3. The prediction made by AC impedance, in this study with respect to material performance and environmental effect, is in agreement with the ASTM tests for concrete durability in terms of corrosion rates, corrosion potential and galvanic corrosion for previous exposures in laboratory and Kuwait environment.
4. For all concrete with NaCl addition, the reaction sequence would follow initially one of the diffusion controlled (passivation reactions) and changed to charge transfer control (polarization reactions). For concrete without the NaCl addition, the corrosion process is characterized by capacitive and diffusion controlled reaction sequence.

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Proceedings of the ACI-KC First International Conference & Fourth Exhibition. September 29-October 1, 2003, Kuwait — ***Concreting and High Performance Concrete in Hot Weather.***

CORROSION PROTECTION SYSTEMS FOR IMPROVING CONCRETE PERFORMANCE IN ARID REGIONS

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ABSTRACT

Improving concrete performance and minimizing corrosion-induced deterioration of reinforced concrete structures are certainly required by the Building Codes and Specifications for arid regions such as the Arabian Gulf. Therefore using corrosion protection systems such as supplementary cementing "pozzolanic" materials, epoxy-coated rebars, and corrosion inhibiting admixtures, are becoming increasingly important for enhancing the quality of concrete and prolonging the life of reinforced concrete structures. Nevertheless, it is also important to examine these systems to evaluate their performance and to assess their impact on durability of the structures under the severe weather conditions of this region. At Kuwait Institute for Scientific Research, an experimental laboratory and field test program was implemented to identify and evaluate performance of some of the most suitable corrosion protection systems under the prevailing weather conditions in Kuwait. The study concluded that supplementary cementing materials contributed to improvement of concrete performance and its resistance to chloride diffusivity. Also, it concluded that epoxy-coated rebars are vurnable to corrosion pitting in saturated chloride environment, especially when the epoxy coating is damaged as received. As for the calcium nitrite corrosion inhibiting admixture, the study concluded that they are most effective when applied by the correct dose to compete with all available chloride ions, and when in combination with microsilica.

INTRODUCTION

It is becoming increasingly obvious that aggressive environment and severe conditions pose great threat to reinforced concrete structures in Arabian Gulf region. Whenever concrete deterioration is investigated in this region, it is rarely goes without mentioning corrosion–induced deterioration inflicted on structures integrity, and the premature damages and degradation taking place that trigger unplanned repair works which often ineffective, inadequate and repetitive. The region provides an extremely aggressive environment, which is characterized by high ambient temperature and humidity conditions, and severe ground and ambient salinity with high levels of chlorides and sulfates in the soil and groundwater. Structures exposed to the marine environment, groundwater conditions and industrial pollution have suffered the most. Concrete corrosion is becoming inescapable phenomenon in this region even to structures of high performance concrete, which are relatively new (10-15 years of construction age). Recent survey study that took place in 1997 on premature deteriorated structures (1), concluded that repair and rehabilitation cost are often expensive, not highly effective in arresting the corrosion processes, and would have to be repeated within few years. The study conducted in 1992 (2), on impact of metallic corrosion on Kuwait economy, showed that cost of direct and indirect corrosion was 5.2% of the gross national product (GDP), out of which 17.4% was an avoidable cost that could have been saved by application of corrosion control technologies. The economical impact was estimated to be KD. 266 millions paid as unrecoverable cost of unavoidable corrosion. The study concluded that cost of corrosion in the construction industry was 0.5% of the GDP. That had demonstrated clearly the economical consequences of the corrosion-induced deterioration of reinforced concrete structures.

Corrosion protection systems reduce the risk of corrosion in reinforced concrete structures in many parts of the world. The use of these systems though might increase the cost of construction; the increase is marginal compared with the cost for repair of the structure as a result of premature deterioration. It is estimated that when these protection systems are used, the increase in construction cost is typically only 10 % of the cost for repair of the premature corrosion-induced deterioration that occurs in the absence of the protection systems (3). As for the corrosion protection systems, such as supplementary cementing materials, corrosion inhibiting admixtures, and epoxy coated rebars, though they have been used for building constructions in very limited projects in the region, they have not been subscribed or standardized for in the national building code of practices in the region. This can be attributed to:

1. No serious attempts or collective well have been made to develop and update the national construction guidelines and code of practices to match the development in the concrete technology that suite the environmental conditions in the region.
2. Adopting foreign construction code of practices without paying attention to the specific nature of the construction materials and the environmental and service conditions prevailing in the region.
3. Adopting concrete construction practices that not necessary guarantee high performance and durability of the reinforced concrete structures under the aggressive, hot environmental conditions.
4. Lack of awareness and unfamiliarity with the use of these systems as well as the lack of comprehensive database on the long-term effectiveness of the systems under the environmental and service conditions prevailing in the region.

In 1997 Kuwait Institute for Scientific (KISR), supported by Kuwait Foundation for Advancement of Science (KFAS), and the major companies of material suppliers, ready mix concrete, and construction and consulting firms, have seized the opportunity to conduct this urgent study which aimed at identifying corrosion protection systems that are most suitable for application in reinforced concrete structures in Kuwait and to demonstrate enhancement achieved in concrete durability when these systems are used. Also among the main objectives are to establish performance data on the selected corrosion protection systems under typical local service conditions and to determine their economic benefits.

The corrosion protection systems that involved in the study are:

1. Epoxy coated reinforcing bar. Steel bars are protected with a coating of powdered epoxy that is fusion-bonded to the steel. The coating physically blocks chloride ions. Cracking and chipping of the coated bars may occur during transportation, storage and field handling, particularly where unskilled labors are used, as is prevalent in the Arabian Gulf region. Damages of such nature could be very detrimental to the integrity of these bars and the protection against corrosion (4).
2. Silica-fume (Microsilica) and Ground Granulated Blastfurnace Slag (ggbs). These are effective pozzolanic materials that significantly reduce concrete permeability and, thereby, reduce chloride ion ingress. The decreased permeability substantially increases resistance to chloride penetration and reduces the rate of steel corrosion and carbonation. Microsilica and ggbs concretes typically have low chloride diffusivity. On the other hand, though ggbs concrete has an early delay rate of hydration, it has lower heat of hydration and progressive compressive strength after 28 days, which superceded that of ordinary concrete and continue to rise after 56 days of curing(5). As for microsilica the concern includes the reduction in the pH value of the concrete which makes carbonation more likely and could cause bound chlorides to be liberated from the hydration products and reduction in the resistance of concrete to salt weathering which is caused by crystallization of salts in the concrete pores (6)
3. Calcium Nitrite Corrosion-Inhibiting Admixtures. It enhances the stability of the passivating layer on the surface of the reinforcing steel. In this concrete system chloride and nitrite ions compete for ferrous ions on the steel bar. If the chloride ion concentration is greater, the corrosion process will start. If, on the other hand, the nitrite ion concentration is greater, a passive layer will form to close off the iron surface. The effectiveness of the calcium nitrite admixture, therefore, is dependent on an accurate prediction of the chloride loading of the structure over its expected design life and, hence, on the selection of an appropriate dosage of the admixture (7).

TESTING PROGRAM

The testing program was designed to include standard and popular laboratory testing methods, accelerated and normal testing methods. The program also included field assessment studies, where specimens are prepared and placed in an exposure site of multiple conditions. The corrosion test program covered three different evaluations:

1. Strength development properties.
2. Corrosion activities-related tests.
3. Chloride ingress characteristics.

For specimen preparations local building materials were used such as Type I & Type V cements, sand, aggregates, and ordinary steel reinforcing bars. Chemical admixtures were provided by local suppliers, whereas local promenant construction companies provided the research program with the required amounts of microsilica (from Elkem Microsilica), ggbs (from Falcon Cement), epoxy coated rebars (from Protech Dubai), and calcium nitrite (from Al-Gurg Fosroc Dubia). Table 1 presents test program summary designed for the study.

Table 1. Test Program Summary

Test	Specimen Size	Measurement
ASTM G-109	Beams: 279 x 152 x 114 mm	Macro-cell current Half-cell potential
Corrosion Rate (Lollipop)	Prism: 380 x 200 x 76 mm	Macro-cell current Half-cell potential
Time-to-Corrosion	Blocks : 300 x 300 x 200 mm	Corrosion rate Macro-cell current Half-cell potential
ASTM C-1202 (AASHTO T 277-86)	Cylinders: 100 x 200 mm	Total charge (conductivity)
AASHTO T 259-80	Slab: 300 x 300 x 75 mm	Chloride profile
Chloride Diffusivity	Cylinders: 100 x 50 mm	Chloride content
Outdoor Exposure	Beams: 120 x 120 x 350 mm With 2 bars With 1 bar Cylinders: 100 x 200 mm	Visual examination Half-cell potential Steel mass lost Chloride content

Strength Development Properties

This is presented in terms of compressive strength. Strength gain development is monitored and recorded at certain curing intervals. Any strength development with time can be related to change in permeability and chloride ingress characteristics. As hydration takes place and the curing process progresses, capillary pores are filled with hydration products. The capillary porosity of the paste depends on both w/c ratio of the mix and the degree of hydration. The type of cement influences the degree of hydration achieved at a given age. At a high water/cement (w/c), ratio the volume of the formed cement gel is not sufficient to fill the capillary pores, which are mainly responsible for permeability of hardened cement paste and its vulnerability to chloride ingress.

Corrosion Activities-related Tests

Time-to-Corrosion Initiation (Modified ASTM G-109). This is a stringent testing method that evaluates the effectiveness of concrete in protecting embedded steel bars from corrosion when salts are applied externally (8). The test requires 48 weekly test cycles to complete. Measurement proceeds after 96 h of salt-water ponding, followed by vacuum removal of the salt water and immediate freshwater rinse and vacuum removal again. This is followed by 72 h of air-drying. The weekly measurements involve readings of half-cell potential, corrosion rate, and concrete resistivity, which are recorded with respect to a copper-copper sulfate reference electrode (CSE) of –350 mV.

Corrosion Rate Test (Lollipop Test). This is a popular laboratory test method, which demonstrates the effectiveness of corrosion protection systems exposed to the marine environment. The test takes its name from the shape of the test specimen. The test simulates the wicking of chlorides by concrete in seawater. The test studies the microcell corrosion current that occurs in a localized area on the steel rebar. Corrosion rate measurements are recorded every month, and involve half-cell potential and microcell corrosion current. This test takes at least one year for the initial results to appear. This test provides excellent chloride ingress profile indication, which qualifies the permeability characteristics of the different corrosion protection systems.

Chloride Ingress Characteristics Tests

The rate at which ions, particularly chloride ions, can diffuse through concrete is important with regard to possible corrosion of steel reinforcement. Ions diffuse through concrete due to differences in ion concentration, which are often independent of hydraulic pressure gradient. Ion diffusivity is generally determined by measuring the time for the concentration at a given point to reach a particular value. There are at least three test methods to determine ionic diffusion rates:

1. The first involves taking incremental samples at different depths by drilling and measuring the chloride content at each increment.
2. The second involves measurement of ionic diffusion by concentration difference between two sides of a specimen after ponding with a chloride solution at one side for certain period of time.
3. In the third method, ionic diffusion is measured by the change in electrical properties that results from changes in concentration.

Chloride Diffusivity. This is a popular long-term duration laboratory test method. This method is designed to assess the chloride ingress characteristics of cylindrical specimens; epoxy-coated on all surfaces, then cut at one end to expose the concrete The exposed concrete surface provides access for one-dimensional diffusion of chloride ions of 3% NaCl solution. Chloride concentration profiles are developed by periodical assessment of chloride concentration at certain depths (0-10 mm, 10-25 mm, 25-30 mm, and 30-50mm). For certain concentrations, at each depth, the time will be recorded and compared.

Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration ASTM C-1202-91). This is a standard test method designed to determine the electrical conductivity of concrete to provide a rapid indication of its resistance to the penetration of chloride ions (9). The test consists of monitoring the amount of electrical current passed through 100 mm diameter by 50 mm long cores when one end of the core is immersed in sodium chloride and a potential difference of 60 V dc is maintained across the specimen for 6 h. The total charge passed in coulombs is related to chloride permeability.

Resistance of Concrete to Chloride Ion Penetration (AASHTO T 259-80). This is a standard test method that evaluates the effect of variations in concrete properties on resistance to chloride ion penetration (10). The test results are correlated with findings of ASTM C1202 to confirm ionic diffusion by chloride ion concentration. Chloride diffusivity is measured by assessment of chloride concentration at different depths, after 90 days ponding with 3% NaCl solution.

RESULTS AND DISCUSSION

In this paper results and performance of each corrosion protection system will be compared to performance of ordinary concrete and the ability to arrest the corrosion process and control the corrosion activities within each system.

Performance of GGBS Concrete

Table 2 presents the concrete mix proportions of the ordinary concrete and the ggbs concrete as 50 % of cement weight was replaced by ggbs. The water- to- cement ratio was selected to be relatively low to represent high performance concrete.

Table 2. Concrete Mix Proportions

Ingredients	Ordinary Concrete	GGBS Concrete
Water-to-cement Ratio	0.36	0.36
Cement type	I	I
Cement (kg/m^3)	472	236
GGBS (kg/m^3)	-	236
Sand (kg/m^3)	550	550
* Aggregate 20 mm (kg/m^3)	770	770
* Aggregate 10 mm (kg/m^3)	380	380
Water (l/ m^3)	170	170
** Chemical Admixture (Caplast NE/EDS) (l/m^3)	5.5-8.5	5.5-8.5

* SSD = Saturated surface Dry , ** Super plasticizer water reducer

Results shown in Table 3 indicates the improvments achieved in the compessive strength at 28 and 56 days in spite of the delay in maturity in the third and seventh day of the curing period. Improvements occurred as a result of 50 % replacement of cement weight by ggbs have affected positively the concrete resistivity to chloride ions penetration and the corrosion activities on the protected steel reinforcing bars. In Fig. 1, and according to ASTM C-876-91 (11) for corrosion risk assessment of half-cell potential of steel bars in concrete, the half cell potential of the steel bar protected by the ggbs concrete was less than –200 mV-CSE after 48 cycles of testing according to modified ASTM G-109, which indicates that steel bar condition in the region described as 90 % probability no corrosion activity occuring on the surface of the steel bar. This is incomparable to results of steel bars protected by ordinary concrete, when its corrosion potential recorded –215 mV after 35 cycles of testing, which describe the steel bar condition as in the region of uncertain corrosion risk area. After 40 cycles of testing and at 280 day the onset of corrosion activities occurs at half-potential of –240 mV-CSE.

Table 3. Improvements in Mechanical and Physical Properties of Ordinary Concrete as 50% of Cement Weight Replaced by GGBS

Properties / Concrete Type	Compressive Strength (kg/cm^2)				ASTM C-1202		AASHTO T-259 Percentage of chloride Absorption After 90 days	
	3 days	7 days	28 days	56 days	28 days	56 days	13 mm Depth	25 mm Depth
Ordinary Concrete Type I	443	490	579	600	2590	1713	0.120	0.052
GGBS Concrete	356	440	632	780	1220	636	0.034	0.021

Table 4 shows results of the corrosion activities for both concretes represented as time-to-corrosion initiation, magnitude of corrosion activities, corrosion current density, concrete electrical resistivity, and corrosion penetration rate. Corrosion rate determined by linear polarization and other electrochemical techniques, expressed in terms of corrosion current density (I_{corr}) can be converted into penetration rates by the following expressions based on Faraday's law (12):

$$\text{Corrosion penetration rate} = K\frac{ai}{nD} \qquad (1)$$

where a = atomic weight of metal (55.8); i = current density,μa/cm^2; n = number of electrons lost, valence charge (2); D = density of steel, g/cm^3 (7.86); K = constant depending on the penetration rate desired with K = 0.129, mpy (mils penetration per year), K = 3.27 , mm/yr, and K = 0.00327 , m/yr

Table 4. Improvement in Concrete Permeability and Its Effect on Corrosion Activities on Steel bars

Properties / Concrete Type	Time–to Corrosion (Modified ASTM G-109) (days)	Magnitude of Corrosion Activities (μA. day)	Corrosion Current Density (μA.cm^2)	Corrosion Penetration Rate (μm/year)	Concrete Electrical Resistively (kΩ. cm)
Ordinary concrete Type I	280	2081	0.166	1.927	70
GGBS Concrete	Corrosion Potential less than–200 mv	749	0.110	1.2	100

As results showed above there are significant improvements in concrete properties when 50% of cement replaced by ggbs, particularly with respect to chloride ions penetration and corrosion activities on steel bars, which qualify ggbs concrete for use in marine and off shore styructures.

Performance of Silica Fume Concrete in Comparison to Ordinary Concrete and Concrete with Calcium Nitrite

Results showed significant improvements in concrete properties with respect to chloride ions penetration, by 10% replacement of cement weight with microsilica, which had a direct effect on delaying the corrosion activities progress on steel reinforcement bars. According to AASHTO T-259 Table 5 presents results of chloride ions obsorption of concrete cover after 90 days of concrete surface ponding with 3% sodium chloride solution. Results showed substantial decrease in chloride ions obsorption that ranged from 60 – 80% depending on the variation in the water-to-cement ratio of the different concretes. The above results are in agreement with results of ATSM C-1202 presented in Fig. 2 that indicate concrete resistivity to chloride ions penetration expressed as total charge passed through concrete in six hours. According to FHWA/RD-81-119 report (13) for chloride ion penetration as presented in Table 6,

Table 5. Rate of Chloride Absorption at Different Depths of Concrete Surface

Concrete Code	Average Absorbed (%)		Maximum Absorbed (%)	
	1.6-13mm	13mm-25mm	1.6-13mm	13mm-25mm
* OU—50	0.161	0.118	0.186	0.177
**OUSF 50	0.089	0.049	0.111	0.111
*** OU –45	0.086	0.041	0.098	0.049
**** OUSF 45	0.069	0.006	0.096	0.009

* Ordinary Concrete: w/c =0.5, ** silica fume concrete: w/c = 0.5

*** Ordinary concrete: w/c = 0.45, **** silica fume concrete: w/c = 0.45

Table 6. Chloride Permeability Based on Charge Passed

Charge Passed (Coulombs)*	Chloride Ion Penetrability	Typical of
> 4,000	High	High w/c (>0.6). Ordinary concrete
2,000 – 4,000	Moderate	Moderate w/c (0.4 – 0.5). Ordinary concrete
1,000 – 2,000	Low	Low w/c (>0.4). Ordinary concrete.
100 – 1,000	Very Low	Latex modified concrete Internally sealed concrete
< 100	Negligible	Polymer impregnated concrete. Polymer concrete.

*Reference 13

The concrete classification has changed from high chloride ions penetration (>4000 coulombs) to low chloride ions penetration (2000 coulombs) when 10% of cement weight replaced by microsilica. As for the effectiveness of silica fume concrete in protecting the steel reinforcement, Fig. 3 clearly shows that it proceeded the capability of ordinary concrete and the concrete with calcium nitrite. According to modify ASTM G-109 test and the interpretation given by ASTM C-876 for corrosion risk assessment, results of the corrosion potential of the steel bars protected by the different concretes, indicated that after 48 cycles of drying and wetting with 15% sodium chloride solution, silica fume concrete continued to provide protection to the reinforcement while those protected by calcium nitrite concrete exhibited uncertain corrosion activities on their surfaces.

Performance of Epoxy Coated Rebars

Results indicated in Figs. 4 and 5 show clearly that the effectiveness of the epoxy coated rebars is reliant on the quality of the concrete cover and the condition of the epoxy coating on the steel bar at the time of use. Fig. 4 shows the effect of steel bar coating condition on the corrosion potential of the steel bar. It is clearly evidant that the epoxy coating on the steel bar (OU-50C) has improverd corrosion activities resistance compared to uncoated bars (OU-50U). Also it shows the deterioration in the corrosion potential when the epoxy coating is damaged (OU-50D), it reveals the increased corrosion activities on the steel bar. Fig. 5 indicated the effectiveness of the silica fume concrete (OUSF50D) in delaying the penetration of the chloride ions and its attack on the damaged epoxy coating. It also shows the healing effect of the calcium nitrite when creating a passive layer on the damaged epoxy coated rebars to hinder chloride ions attack and hence corrosion activities from taking place. The discussion above has drawn the attention to the effectiveness of the epoxy coated rebars, nevertheless Fig. 6 is a cause for concern and second thought when epoxy coated rebars are used. It is quite often that the monitoring and the electrochemical measuring devices used for investigating the corrosion activities, will fail to detect any damages on the epoxy coated rebars and hence overlook any change in the corrosion potential readings unless the steel bars are physically removed and visually examined. That was the reason behind the sudden failure and the deterioration which took place on 1986 in the state of Floride in USA, when application of epoxy coated rebars became very popular and expansive without precautionary measures or previous knowledge of such danger.

SUMMARY AND CONCLUSIONS

As a summary to all above, its concluded that the supplementary cementing materials such as ggbs concrete and silica fume concrete when replace certain percentage of the cement weight in the concrete mixure, they have significant effect in enhancing concrete resistivity to chloride ions penetration and hence decrease the corrosion activities on steel bars protected by such concretes. As for the epoxy coated rebars results showed concerns related to the condition of the epoxy coating layer on the bar and to their application in high chloride laden environments. On the other hand it is proven that corrosion inhibiting admixures such as calcium nitrite are more effective when used with other protection systems such as ggbs and microsilica. Accordingly conclusions can be made as following:

1. Significant imrovements in concrete performance when suplementary cementing materials are used, especially with respect to resistivity to chloride ions petration and corrosion activities.
2. Corrosion protections systems satisfy durability requirements of reinforced conctere structures in marine environment.

3. Effectiveness of the epoxy coated rebars is reliant on the quality of the concrete cover and the condition of the epoxy coating on the steel bar at the time of use.
4. Effectivness of corrosion inhibiting admixtures such calcium nitrite is highly reliant on their concentration with respect to chloride ion concentration in the same environment. Any unaccounted for increase in the level of chlorides over the nitrite ions would lead to unexpected form of corrosion on the steel bars. That why such corrosion protection system would work better in multiprotection environment.
5. It is of great importance to conduct field research studies under the actual prevailing environmental to be able to conclude more realistic results that would make practical applications of such materials and systems more effective. Studies of that nature usually requires long time that spans to 5-10 years.

ACKNOWLEDGEMENT

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THE OPTIMAL FILLER CONTENT GIVING THE BEST PERFORMANCES OF CONCRETE

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ABSTRACT:

Concrete producers are prompted to promote the utilization of the limestone based aggregates due to the limitation on extraction of alluvial materials in Constantine and the increased transportation costs. These fine aggregates constitute the principal deposits of Constantine.

The utilization of limestone-based aggregates for filler content has shown some discrepancies internationally. As the majority of our production units use excess of fine aggregate in their production, we attempted to undertake some tests on the influence of the filler content on concrete properties.

The basic parameters studied are: the variation of the fine content (0 to 24% with a step 6%), the provenance of the sands (two sites) and the granular distribution (continuous and discontinue).

Varying these three parameters, we have measured and observed various qualities of concrete consistency, mechanical strength, shrinkage and loss in weight.

In conclusion, significant results demonstrated that we could attain the best performances with 18% of fine content. For shrinkage we registered an acceptable behavior of almost 18% from the tests.

Keywords:
Aggregates, Fines, Concrete, Granular Classes, Sand, Limestone, Shrinkage

INTRODUCTION

Until recently, the aggregates supply has never been a source of major problem in most regions of Algeria. Since then several factors have developed and environmental and other factors can be held responsible for the lack of natural resources.

This state is due to the decrease of current deposits, especially problems linked either to the environment or exploitation of resources.

Moreover, the global changes in the cost of construction, transport and material have led to new initiatives to utilize available national materials. Chalky rocks have proven useful in Algeria, notably in regions where problems of aggregates supply are the most crucial. Similarly, Constantine, Mila, Oum El Bouaghi[1], can favourably respond to the problem of aggregate supply.

However, enterprises prefer the use of alluvial aggregates from hard rocks, showing reluctance to use crushed aggregates which seem to create several problems.

These problems are mainly linked to characteristics depending on type of the aggregates. There are important contents in fine elements (more than 10%), which imply that it is necessary to add more water than for rolled aggregates based concrete to obtain the same workability of concrete. This is detrimental to concrete qualities because the concrete placement is more delicate and its mechanical performances are notably reduced.

Several researches have been conducted in Spain, France, Morocco, Algeria and other countries on the use of chalky sand containing fillers in mortar and concrete confection[2]. The results obtained from these studies show that the tolerated limit of fillers contents vary from one country to another, as shown in Table 1.

The most important problem is to know the effects of these very fine products on the fresh and hardened concrete properties[3].

The objective of this research is to present the possibility of chalky crushed aggregates used in hydraulic concrete confection and also to determine the acceptable filler content limits in sands according to the petrographic nature of the original rocks[4].

In order to prove the influence of the filler presence on free concrete deformations, we have undertaken dimensional variation measurements on test tubes containing different filler rates and exposed to the free air by measuring the shrinkage and the loss in weight.

Table 1. Limits of filler contents authorized in different countr*ies.*

R.F.A.	Canada	Italy	U.S.A.	France	U.K
< à 4%	< 3 - 5 %	< 3 - 5%	< 7%	< 10 - 12%	< 16%

EXPERIMENTAL PROGRAM

Twenty concrete mixes containing different filler rates have been reviewed, based on the mineralogical nature of the aggregates as well as the granular skeleton of the concrete and their influence on rhiological properties of fresh concrete (slump, occluded air and density), and their mechanical properties (compressive and tensile strength) measured at the age of 7, 28 and 90 days. Shrinkage and weight loss measurements were undertaken since the moulds were removed until the age of 90 days.

The sand was fully washed and then reconstituted in large proportion of filler rates (0%, 6%, 12%, 18% and 24%). Two granular skeletons have been retained (Continuous and Discontinuous:8/15 grade suppressed). The cement content was kept constant (350 Kg / m3). Initially, the quantity of water was fixed to 182 l / m3 and then adjusted gradually in order to maintain a constant workability (8 to10cm), measured on ABRAMS cone. See Table 2.

Table 2. Identification of the twenty concrete studied

N°	Notation	Designation
1	AC0	Continues granular concrete; S1 Sand with 0 % de fillers
2	AC6	Continues granular concrete; S1 Sand with 6 % de fillers
3	AC12	Continues granular concrete; S1 Sand with 12 % de fillers
4	AC18	Continues granular concrete; S1 Sand with 18 % de fillers
5	AC24	Continues granular concrete; S1 Sand with 24 % de fillers
6	AD0	Discontinues granular concrete; S1 Sand with 0 % de fillers
7	AD6	Discontinues granular concrete; S1 Sand with 6 % de fillers
8	AD12	Discontinues granular concrete; S1 Sand with 12% de fillers
9	AD18	Discontinues granular concrete; S1 Sand with 18% de fillers
10	AD24	Discontinues granular concrete; S1 Sand with 24% de fillers
11	KC0	Continues granular concrete; S2 Sand with 0 % de fillers
12	KC6	Continues granular concrete; S2 Sand with 6 % de fillers
13	KC12	Continues granular concrete; S2 Sand with 12 % de fillers
14	KC18	Continues granular concrete; S2 Sand with 18 % de fillers
15	KC24	Continues granular concrete; S2 Sand with 24 % de fillers
16	KD0	Discontinues granular concrete; S2 Sand with 0 % de fillers
17	KD6	Discontinues granular concrete; S2 Sand with 6 % de fillers
18	KD12	Discontinues granular concrete; S2 Sand with12 % de fillers
19	KD18	Discontinues granular concrete; S2 Sand with18 % de fillers
20	KD24	Discontinues granular concrete;S2 Sand with 24 % de fillers

Characterization of Constituents

The Cement: The cement used for the concrete confection is the CPJ 45 accordingly to the Algerian norm NA 442. It comes from the cement factory ERCE (EL Hamma - Constantine). The chemical and mineralogical composition of the clinker is shown in Table 3.

Table 3. Portland clinker characteristics (%)

<table>
<tr><td colspan="10"><i>Mineralogical composition</i></td></tr>
<tr><td colspan="3">C_3S</td><td colspan="2">C_2S</td><td colspan="3">C_3A</td><td colspan="2">C_4AF</td></tr>
<tr><td colspan="3">56.60</td><td colspan="2">22.98</td><td colspan="3">9.78</td><td colspan="2">8.25</td></tr>
<tr><td colspan="10"><i>Chemical Composition</i></td></tr>
<tr><td>SiO_2</td><td>Al_2O_3</td><td>Fe_2O_3</td><td>CaO</td><td>MgO</td><td>SO_3</td><td>Na_2O</td><td>K_2O</td><td>other.</td><td>L.O.I</td></tr>
<tr><td>27.83</td><td>6.21</td><td>3.12</td><td>57.22</td><td>0.94</td><td>2.02</td><td>/</td><td>/</td><td>2.28</td><td>2.41</td></tr>
</table>

The Granulates: The specimen used comes from two different sites; the first one, "S1" which is found at 25 Km South/East of Constantine in the region of El Khroub, exploiting the deposit of Djebel Oum Setta. This site is composed of great extent of Cenomanienne rock of clear beige colour, safe, with 98% of very clean tender limestone. The second site, "S2" is situated at 20 Km South/West of Constantine in the region of Ain Smara, exploiting the deposit of Djebel Felten extending to Eocene rock white to greyish colour. The rock, fissured and altered with clays, composed of 85% of limestone and 8% of silica[5].

Gravels are industrial chalky crushed products of 3/8, 8/15 and 15/25. Physical, morphological and mechanical properties are found in Table 4.

Table 4. Aggregates properties

	S1	S2	Gravel		
Aggregates Grading	0/3	0/3	3/8	8/15	15/25
Physical Properties					
Density (Kg/m^3)					
▪ Absolute	2.73	2.65	2.57	2.58	2.60
▪ Apparent	1.44	1.46	1.36	1.35	1.34
Superficial tidiness (P)	-	-	1.50	1.28	0.56
Equivalent of sand	78.67	65	-	-	-
$CaCO_3$ (%)	86	98	85	83	84
Morphological Properties					
Flattening Coefficient	-	-	18	13	17
Mechanical Proprieties					
Los Angeles (LA)	-	-	20	23	22
Micro Deval (MDE)	-	-	16	17	-

The Fillers: We designate fillers with crushed elements that have not been subjected to any processing and having a granular grade of 0/d with d equal to 80 microns[6].

The fillers come from the chalky crushed sand dust. These fillers conform perfectly to the norm (NFP 18-501), their coefficient activity is in the order of 0.5. The chemical composition and the physical characteristics of the fillers are grouped in Table 5.

The filler content has an important role on the physical sand characteristics.

Table 5. Chemical composition of fillers

SIO_2	AL_2O_3	Fe_2O_3	Ca CO_3	MgO	SO_3	LOI	PH
0.06	0.09	0.02	99	0.01	0.01	43.8	9

Influence of Filler Rate on Physical Sand Properties

Apparent Density: We notice that the apparent density increases slightly for fillers with rates between 0 and 6%, then increases continually from 6 to 18% and it stabilizes over 18%, see Figure 1. The peak compactness is obtained between 18% and 24%. These two sands follow almost the same appearance. A light difference between these two sands is observed and it is due to this fact that the sand S2 always contained a small quantity of fillers after fully washing.

Absolute Density: We observe that the absolute density increases rapidly for fillers with rate ranging from 0 to 12% and it increases very feebly between 12% and 18%. Then it stabilizes, as shown on Figure 2.

Fineness Modulus: The fineness modulus (Fig.3) varies almost linearly with fillers rate and varies considerably between sand S1 and S2. The S1 sand is coarse and has FM equal to 3.2, adding fillers to this sand decreases FM to 75%. The FM of sand S2 varies from 2.6 to 2.3 considered proper sand according to specifications.

Piston Equivalent Of Sand: observation of Figure 4, leads us to imply that a correlation between the fillers percentage and the diminution of the equivalents of sand exists. We notice that the value of the equivalents remains superior to 65 in the interval 0 to 18% of fillers rate. For these two sands and according to the French standard the last value is the limit for the piston equivalent of sand. Over 18% of fillers rate, the value of sand equivalent decrease.

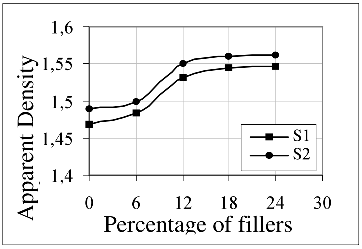

Fig.1 : Apparent Density versus fillers rate

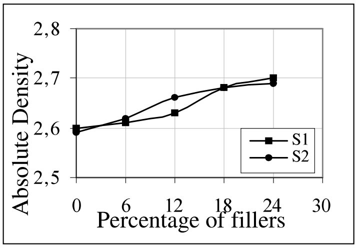

Fig.2 : Absolute Density versus fillers rate

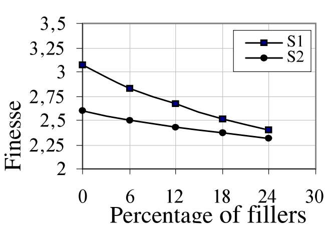

Fig.3 Finesse Modulus versus fillers rate

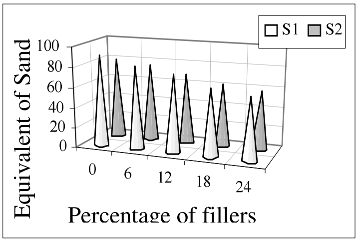

Fig.4 Piston equivalent of sand versus fillers rate

Concrete Mixture and Confection of Specimens

The method used for the concrete design is that of Dreux and Gorise[7]. Proportions of the different compositions are represented in Tables 7 and 8. Specimens have been made (mixing, placement in the moulds, tightening) according to Algerian standard[8], then conserved in water at 20°C after demoulding at the age of 24 h, until the time of the testing. The specimens destined for shrinkage and the weight loss measurements were conserved in the ambient air (60% HR, T=20°C).

RESULTS AND DISCUSSIONS:

Influence of Filler Rate on Fresh Concrete Properties

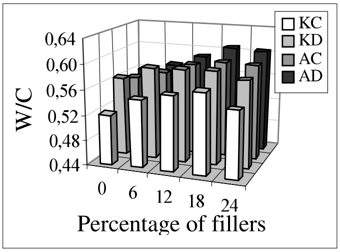

Fig.5 W/C versus fillers rate

W/C: The fine elements play an important influence on the W/C. In reality, more the filler rate increases, more water is needed to keep a constant slump (see Figure 5). For a filler rate approximately equal to 18%, the demand for water becomes less important. This statement may be explained by the fact that fillers having a very important specific surface, form a colloidal microstructure that retains water until the value of approximately 18%. This observation remains valid for these two granular (continuous and discontinuous) skeletons, not taking into account the mineralogical nature of the sands.

At approximately 18% of filler rate, fillers behave as a fluidizer. According to R. Bertrandy[7] for a plastic consistency the optimal filler rate required seems to be between 8 and 12% of filler contents in the sand. In addition to this, it is necessary to consider a new parameter by allowing the fillers colloidal activity to intervene. As seen in these results:

- For a high coefficient of activity, the best workability is reached[6] with a filler rate of 8%.
- The wastage water becomes considerable for a higher coefficient of activity.

In our case (according to technical form) the coefficient of activity is equal to 0.5, which explains that we have gone until 18% to reach the optimum.

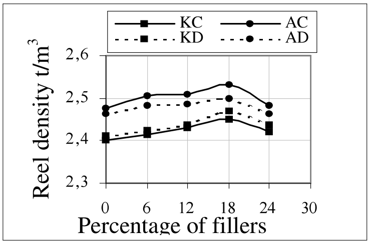

Fig. 6 Concrete reel density versus fillers rate

Real Density: Measures of real density in function of the fillers rate (Figure 6) show that these characteristics increase rapidly for filler rate rising from 0% to 12%. The optimal compactness is obtained between 12% and 18% of fillers where we have noticed the maximal value. Over this interval, a decrease is observed for all the curves which imply that limestone fillers have sometimes a positive and sometimes a negative influence on the density depending on their content. The different curves present a flatten bell with an optimum situated between 15% and 18% of fillers. The evolution of concrete densities based on aggregates issued from site S1 is better than those of site S2. The influence of fillers is much more steady on discontinuous aggregate concrete. This confirms the porosity effect in discontinuous granular skeleton.

Occluded Air: Contrary to the variation of the real density and as shown in figure 7, the percentage of the occluded air decreases with respect to filler rate. We notice that the optimum is reached between 12 and 18% for the different curves. This verification is elsewhere confirmed by the preceding measurements of the density where we notice that in this interval, concrete structure is denser. We notice three zones:

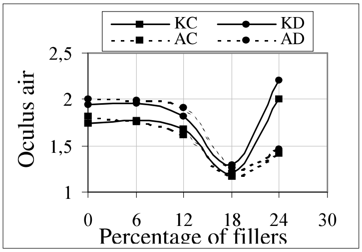

Fig.7 Occluded air versus filler rate

- From 0% to 12% of fillers, the curves are almost constant.
- From 12% to 17% of fillers, we observe an abrupt decrease.
- Over 17% of fillers, the curves increase for the site S2 in a rapid manner but for the site S1 they are slightly flat. In addition, we notice over the 18% stage there is a resumption of the occluded air. This resumption is apparently provoked by the interaction activity of cement-fillers-granulates, this statement need to be confirmed.

The Influence Of Fillers On Hardened Concrete Properties

Normalized mechanical tests have been conducted at the stages of 7, 28 and 90 days, to follow the progressive evolution of the compressive strength on cylinders 16x32cm and that of the bending traction prisms 7x7x28 cm.

Compressive Strength: The compressive strength evolution in function of the age (Figure 8) presents the same appearance for all filler rates. Globally, at 28 days the concrete compressive strength for 6%, 12% and 18% of fillers rate is superior to that of the concrete without fillers. On the other hand, for the content having 24% of fillers rate, the obtained resistance is always weaker for the two granular skeletons. Between 7 and 28 days, the increase of resistance in function of the age is appreciably linear. Over 28 days the filler effect tends to stabilise gradually until 90 days. This allows us to say, that at longer term, the contribution to the resistance becomes minimal.

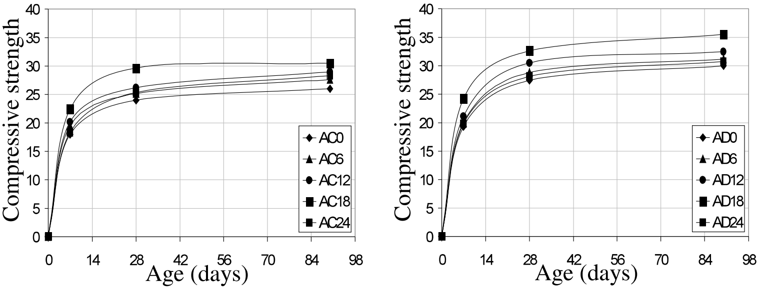

Fig.8-A Compressive strength in function of the fillers rate and age (Site S1)

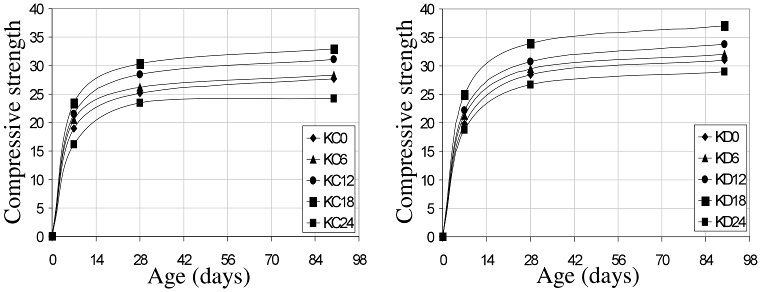

Fig.8-B Compressive strength in function of the fillers rate and age (Site S2)

The limestone fillers play, therefore, a beneficial role on the resistance until the content of 18% “the optimum value". The best compressive strength is obtained with the discontinuous granular skeleton of the site S1.

Tensile Strength: The observation of the presented values on figures 9 prompts us to deduce the same conclusion for the compressive strength. The best performance is obtained for the continuous granular skeleton of the site S2 with a filler rate of 18%, while the weakest acceptable performance is that of the concrete having 24% fillers rate.

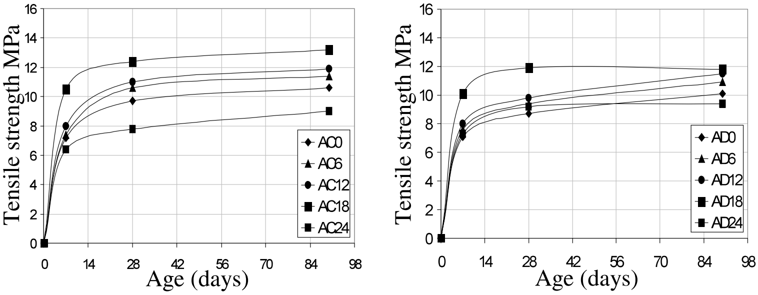

Fig.9-A Tensile strength versus age and fillers rate (Site S1)

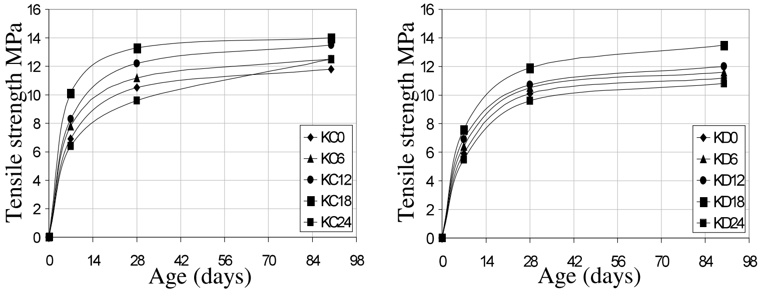

Fig.9-B Tensile strength versus age and fillers rate (Site S2)

Shrinkage And Weight Loss: We know that the shrinkage evaluation function of time, evaporation of water taken in concrete and by desiccation, by over 28 days evolution begins to stabilise. Globally, the totality representative curves obtained in our tests (figure 10), have an identical behaviour. Whereas, we notice that the withdrawal grows proportionally with the addition of the fillers. In the interval (0 to 6%) of fillers, the deformations due to the shrinkage grow in the same manner for the different curves during the first days. Over 5 days composition containing high filler percentage begins to plagiarize slowly from the weak content compositions. We have recorded that the shrinkage grows to an order of 50% from 1 day to 7 days, and 20% between 7 days to 28 days. We notice that sands with high content of fillers favour the shrinkage which is proportional to the weight loss (figure 11).

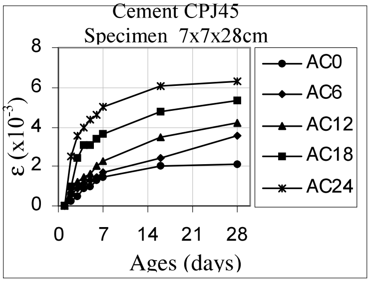

Fig.10-A Shrinkage versus the age and fillers rate (Site 1)

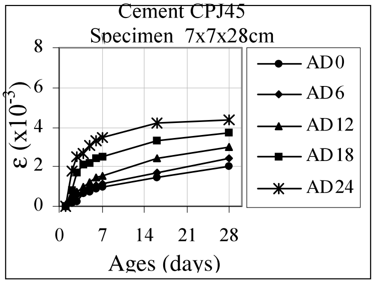

Fig.10-B Shrinkage versus the age and fillers rate (Site 1)

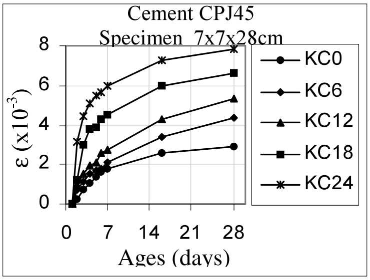

Fig.10-C Shrinkage versus the age and fillers rate (Site 2)

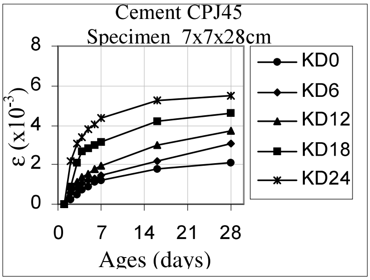

Fig.10-D Shrinkage versus the age and fillers rate (Site 2)

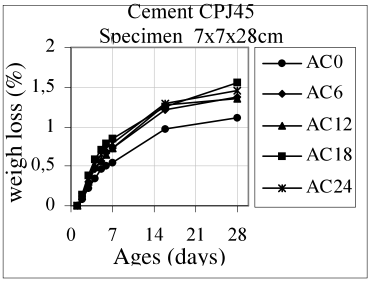

Fig.11-A Weight loss versus the age and fillers rate (Site 1)

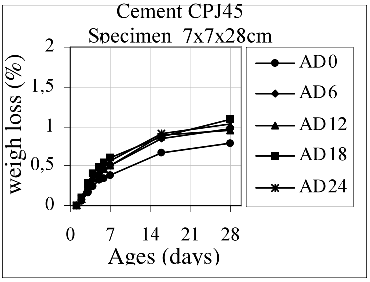

Fig.11-B Weight loss versus the age and fillers rate (Site 1)

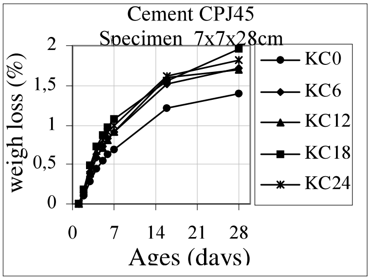

Fig.11-C Weight loss versus the age and fillers rate (Site 2)

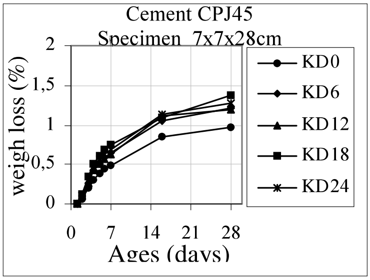

Fig.11-D Weight loss versus the age and fillers rate (Site 2)

CONCLUSION

Exploitable resources of limestone aggregates in Algeria are important. Intrinsic characteristics conform to specifications needed for hydraulic concrete. In all careers of Algeria, the fillers rate is too high (going until 30%) and dusting processing are inexistent; fully washing of sand may eliminate the fillers included in sands, which give poor resistance and reduces the concrete workability.

Mechanisms of limestone filler actions in the concrete are based on physical effects. Thus the filler particles grading of average dimensions, weaker than those of sand, contribute to identify the microstructure by reducing the matrix pore.

Fillers confer to the fresh concrete a power of water retention that allows it to resist the bleeding and provide a cohesion that maintains homogeneity (absence of segregation).

Fine grains slip between larger ones to increase the compactness and participate in the optimisation of the curve grading.

The fillers incorporation during crushing is beneficial for the concrete. The optimal content that allows obtaining the highest mechanical resistance, undertaken on a constant workability concrete, is in the order of 18% of fillers, whatever is the aggregate source and the granular skeleton. The demand for water decreases over 18% of filler rate and fillers begin to behave as a water reducing adjutant. The cement-fillers-aggregates interaction provokes a resumption of the occluded air when the filler rate is over 17%. Straight gains are linked to the improvement of the compactness obtained by the addition of fillers, and, on the other hand the straight decrease is due mainly to the increase of the W/C, the fillers effect tends to stabilise. The **optimal fillers content** that allows the obtaining of highest resistance is close to 18%.

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DETERIORATION OF PLAIN AND BLENDED CEMENTS IN MARINE EXPOSURE

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ABSTRACT

This study was conducted to investigate the sulfate resistance of two plain cements (Type I and Type V) and three blended cements made with fly ash, silica fume and blast furnace slag in marine environments. In addition to assessing the strength developments, the performance of these cements was evaluated by exposing the specimens to a "fresh" sea water for a period of two years and measuring the reduction in compressive strength and expansion of the mortar specimens; visual inspection, weight change of the concrete specimens; and mineralogical analysis using x-ray diffraction technique of cement paste specimens. The results of these tests indicated that deterioration due to sulfate attack was somewhat hindered on all plain and blended cements despite the high sulfate concentration in the sea water medium which is classified as "aggressive".

Keywords: Plain cements, blended cements, sulfate attack, sea water, marine exposures, performance.

1. INTRODUCTION

Deterioration of concrete in marine environments has been the subject of major concern and research since the invention of cement as a binding material [Al-Amoudi, 2002]. Due to the extensive spread of seas all over the world, as compared with the continental land available for human beings, marine structures and offshore oil facilities will grow and will continue to be made of concrete [Mehta, 1980; 1991] because concrete is not only the most economical structural material for construction of such structures, but also the most durable. The performance of concrete structures in sea water is of great importance because the durability of these structures is the most complicated system to investigate; where concrete deterioration is caused by a maze of interwoven mechanisms (i.e. chemical, physical and mechanical factors) [Al-Amoudi, 1992].

Research on durability of reinforced concrete in sea water is mostly concerned with two deteriorating actions; chloride-induced corrosion of reinforcing steel and sulfate attack on concrete [see the list of references]. The need for continued research on these aspects stems from the fact that there have been considerable changes in the physico-chemical characteristics of Portland cement in the past century, particularly in terms of C_3A and C_3S phases. Further, the addition of supplementary cementing materials, such as fly ash, silica fume and blast furnace slag, in Portland cement has significantly increased, particularly in the aggressive exposures of the Arabian Gulf. Moreover, the concentration of salts in marine exposures, particularly the sulfates and chlorides, may give the impression of a constant composition all over the world. However, Table 1 displays significant variation in the concentration of various salts, with the sea water in the Arabian Gulf being the most concentrated open sea [Al-Amoudi, 2002]. As part of a comprehensive research program initiated at KFUPM in the early 1990s to study the durability of plain and blended cements in high sulfate-chloride media [Al-Amoudi, 1992], this paper summarizes the investigation conducted to assess the sulfate attack on plain and blended cements exposed to marine environments.

Table 1: Concentration of Major Ions in Sea Water in Some Areas of the World [Al-Amoudi, 2002]

Major constituents	Concentration (mg/l) in								
	Black Sea [4]	Marmara Sea [4]	Mediterranean Sea [4]	North Sea [5]	Atlantic Ocean [5]	Baltic Sea [5]	Arabian Gulf [6]	BRE** Exposure	Red Sea
Sodium (Na^+)	4,900	8,100	12,400	12,200	11,100	2,190	20,700	9,740	11,350
Potassium (K^+)	230	340	500	500	400	70	730	400	1,350
Calcium (Ca^{++})	236	328	371	430	480	50	760	400	531
Magnesium (Mg^{++})	640	1,035	1,500	1,110	1,210	260	2,300	1,200	1,867
Chloride (Cl^-)	9,500	14,390	21,270	16,550	20,000	3,960	36,900	18,200	22,660
Sulfate (SO_4^{--})	1,362	2,034	2,596	2,220	2,180	580	5,120	2,600	3,050
Bicarbonate (HCO_3^-)	217	182	158	–*	–*	–*	128	–*	152
Total dissolved solids (TDS)	17,085	26,409	38,795	33,060	35,370	7,110	66,650	32,540	40,960
pH	7.4	7.9	8.0	–*	–*	–*	8.3	–*	6.3
Concentration ratio***	3.90	2.52	1.72	2.02	1.88	9.37	1.00	2.05	1.63

* Not reported
**Building Research Establishment, England
***Concentration of total dissolved solids compared to the Arabian Gulf sea water

2. EXPERIMENTAL PROGRAM

ASTM C 150 Type I and Type V Portland cements with a C_3A content of 8.5% and 3.5%, respectively, were used in preparing plain cement paste, mortar and concrete specimens. ASTM C 618 Class F fly ash (FA), silica fume (SF) and blast-furnace slag (BFS) were used at 20, 10 and 60% replacements by weight of Type I cement, respectively, to prepare FA, SF and BFS blended cements. An effective water to binder (w/b) ratio of 0.50 was kept invariant in all the paste, mortar and concrete specimens. In the mortar specimens, the sand to binder ratio was maintained at 2.75, while a binder content of 350 kg/m^3 and a coarse-to-fine aggregate ratio of 2.0 by weight were kept constant in all the concrete mixtures. The aggregates were 19-mm maximum size crushed limestone and dune sand from eastern Saudi Arabia.

After casting, all the specimens were covered with wet burlap in the moulds for 24 hours and, thereafter, cured in potable water for a further period of 14 days. They were then air-dried in the laboratory (23 ± 2°C) for one day before being placed in the sea water. The sea water was obtained from the Arabian Gulf beside KFUPM beach (see Table 1 for the concentration of various ions in the sea water) and was changed every month.

Cement paste specimens were used to study the effect of sulfate ions in sea water on the hydration products using X-ray diffraction (XRD) technique after 24 months of exposure to the marine environment.

Mortar specimens were used to determine the strength development in water and strength reduction and expansion due to exposure to sea water. The strength tests were conducted on 25-mm cube specimens according to ASTM C 39, while the expansion measurements were conducted on two 25×25×285 mm prismatic specimens as per ASTM C 441. Concrete cylinders, 76 mm in diameter and 152 mm high, were used to study the effect of sea water on the weight loss of concrete. See Al-Amoudi [2002] for further details on these tests.

3. RESULTS

The data on compressive strength development in all the plain and blended cement mortar specimens placed in water are depicted in Figures 1 and 2, respectively. All the cements exhibited somewhat similar strength after 14 days of curing, except the blast furnace slag (BFS) cement. After 180 days of curing, the maximum compressive strength was observed in the silica fume and fly ash cements. The strength development in BFS cement specimens, however, was the lowest at all curing periods due to the high quantity of BFS (60%) in this cement.

The reduction in strength in plain and blended cement mortar specimens exposed to sea water is plotted in Figures 3 and 4, respectively. The strength reduction was less than 20% in all types of cements after one year of exposure. After two years of exposure, the reduction in strength was in the range of 22 to 26% in all the cements except in the BFS cement which was 13%. The lowest reduction in strength in BFS cement may be attributed to their initially low strength, as was stated earlier.

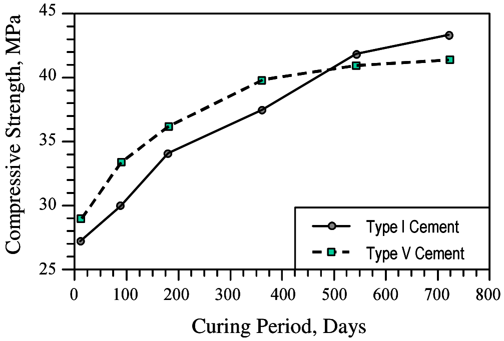

Figure 1: Strength Development of Plain Cements.

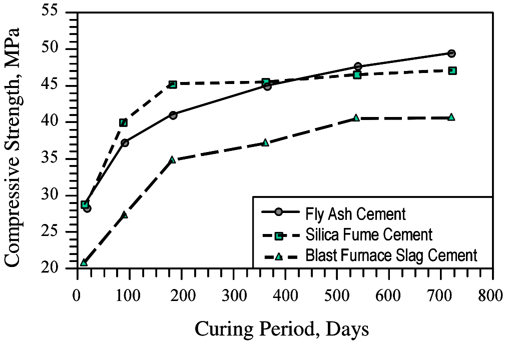

Figure 2: Strength Development of Blended Cements.

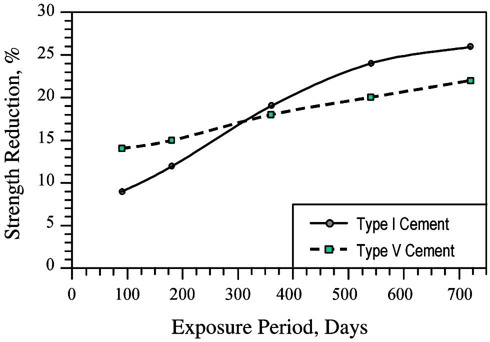

Figure 3: Reduction in Compressive Strength of Plain Cements.

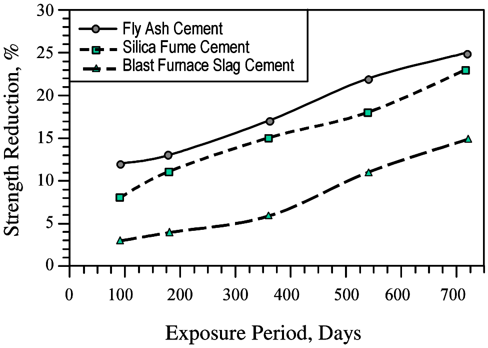

Figure 4: Reduction in Compressive Strength of Blended Cements.

The expansion data are presented in Figures 5 and 6 for plain and blended cement mortar specimens, respectively. After 540 days of exposure to the marine environment, the expansion in plain cements was more than that in all the blended cements. The better performance of blended cements could be ascribed to the dilution of the reactive cement phases (i.e., mainly C_3A and C_3S) in the parent cement due to its replacement by the pozzolanic materials [Al-Amoudi, 2002]. The maximum expansion of 0.08% was noted in

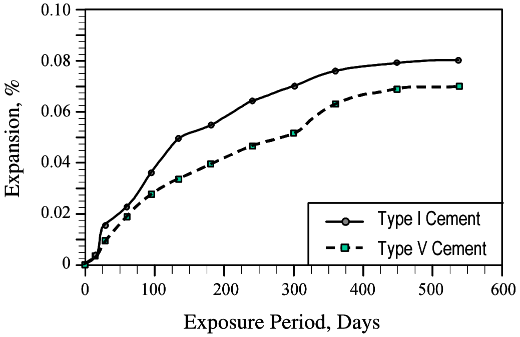

Figure 5: Expansion of Mortar Specimens Made with Plain Cements.

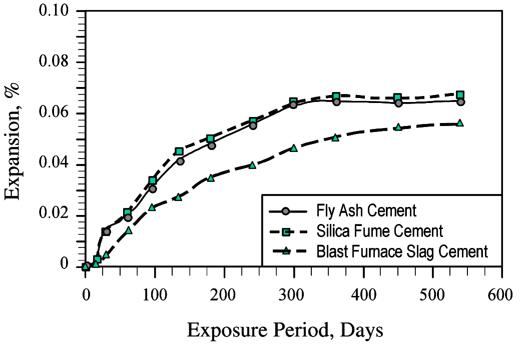

Figure 6: Expansion of Mortar Specimens Made with Blended Cements.

Type I cement after 18 months, followed by Type V cement, which had an expansion of 0.070%. The marginal improvement of Type V cement might be ascribed to its relatively lower C_3A. Among blended cements, BFS cement exhibited distinctly the lowest expansion of 0.056%. After 540 days, the expansion was about 0.067% in both FA and SF cements.

The data on weight change for the concrete specimens made with plain and blended cements are plotted in Figures 7 and 8, respectively. Surprisingly, an increase, rather than a decrease, in the weight was noted in all the concrete specimens, even after 24 months of exposure to the sea water. Visual inspection of these specimens did not reveal any type of "distinct" deterioration below or above the sea water level, as was observed in other aggressive exposures [Al-Amoudi, 1998; Al-Amoudi et al., 1994]. Only salt deposition was noted on the specimens in a way exactly similar to what has been observed on the specimens placed in sabkha (i.e., high sulfate-chloride) brines [Al-Amoudi, 1995]. The salt precipitation increased in proportion to the increase in weight. In fact, salt precipitation was noted even on the portions of the specimens above the sea water level though to a lesser extent.

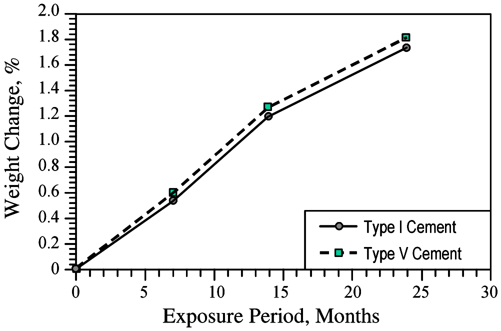

Figure 7: Changes in Weight of Concrete Specimens Made with Plain Cements.

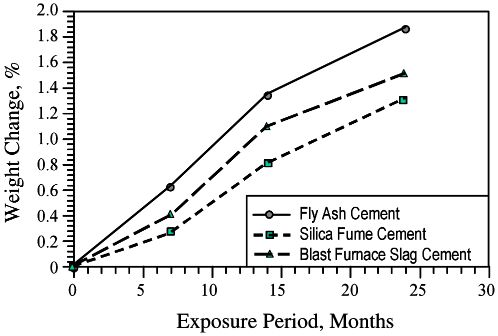

Figure 8: Changes in Weight of Concrete Specimens Made with Blended Cements.

The X-ray diffractograms (XRDs) for plain and blended cement pastes exposed to the sea water for a period of 24 months are presented elsewhere [Al-Amoudi, 2002]. The XRD data indicate that the portlandite peaks noted in the specimens cured in water were either totally eliminated or significantly reduced when the specimens were exposed to the marine environment. Most of the peaks noted in the pastes exposed to sea water were very small; either the remnants of portlandite or gypsum or the calcium chloro-aluminate, known as Freidel's salt, or the magnesium hydroxide [Al-Amoudi, 1992]. These peaks were small and cannot be easily identified, as was noted previously in the case of the specimens exposed to a high sulfate-chloride environment [Al-Amoudi et al., 1994]. However, the formation of calcite, probably due to carbonation of cement, was noted in both the specimens cured in water and exposed to sea water.

4. DISCUSSION OF RESULTS

The data reported in this paper indicates that the maximum strength reduction noted in this investigation was about 25% in Type I and FA cements after two years of exposure to the marine media. Such a reduction can be considered as "small" for the following reasons: (i) the duration of exposure to the marine environment was relatively long (24 months); (ii) the specimen size was designed to be small (25 mm cubes) so as to accelerate the deterioration phenomena by the sulfate ions in the sea water; and (iii) the sea water had a sulfate (SO_4^{2-}) concentration of 5,120 ppm [Al-Amoudi, 2002], which is considered as severe, according to the Canadian Standard CAN3-A23.1-M77 [Mindness and Young, 1981] and ACI 318 [American Con., 1995]. The strength reduction in all the mortars was less than 30% that is generally considered as the threshold value for failure due to sulfate attack on small specimens similar to those utilized in this investigation [Al-Amoudi and Maslehuddin, 1996].

The expansion data also supports the results of strength reduction whereby the expansion of both plain and blended cements was vividly low. The expansion of all cements during the 18-month exposure to the marine environment did not exceed the 0.1% that is generally specified as the failure criterion for expansion [Al-Amoudi, 2002].

The relatively mild attack of sea water on all the cements used in this investigation is further supported by the weight change data generated by exposing the concrete specimens to sea water. As stated earlier, an increase in the weight of these specimens was noted thereby confirming the absence of noticeable concrete deterioration after two years of maritime exposure. The increase in weight was ascribed to salt penetration and precipitation that was visually noted on all the concrete specimens, both below and above the sea water level, as was discussed previously. Similar observation was noted when fifteen different concrete mixtures were exposed to a high chloride-sulfate "sabkha" solution [Al-Amoudi, 1995].

Literature review indicates that there are two forms of concrete deterioration that are ascribed to sulfate attack [Al-Amoudi, 1998]. The first mode of deterioration is akin to eating away of the hydrated cement paste and its progressive reduction to a cohesionless granular mass leaving the aggregate exposed and leading to loss of strength and reduction in weight. This mode is attributed mainly to the formation of gypsum and the non-cementitious magnesium silicate hydrate, and is known as the **acidic type** of sulfate attack. The second mode of deterioration, which is normally characterized by expansion and cracking, takes place when the reactive hydrated aluminate phases, present in sufficient quantities, are attacked by sulfate ions, thereby forming tricalciumsulfo-aluminate hydrate, also called ettringite or Candlot's salt. This **expansive type** of sulfate attack is ascribable to the formation of a colloidal form of ettringite in the presence of high concentrations of $Ca(OH)_2$ in the pore solution.

Visual inspection of the specimens exposed to sea water did not reveal any of the above two forms of deterioration during the two year exposure period. Previous studies in the 1950s and 1970s had reported excessive expansion and extensive deterioration in concrete structures exposed to sea water [Mehta and Haynes, 1975; Kalousek and Benton, 1970; Figg, 1979]. The reason for the increased proneness of old cements to expansion and spalling by sea water is probably ascribable to the high C_3A content of "old" Type I cements that were manufactured before 1950s. When the C_3A content is more than 11%, the susceptibility of concrete to expansion and cracking is significantly increased [Gjorv. 1971]. On the contrary, the present-day Type I cements often have a maximum C_3A content of about 9%. Accordingly, the expansion of Type I portland cement when exposed to sulfate media is much mitigated. The expansion characteristics are further reduced by the conjoint presence of chlorides with the sulfate salts in marine environments [Figg, 1979; Harrison, 1990]. Even if ettringite or gypsum is formed, the expansive stresses will be much reduced due to the concomitant presence of chloride ions [Lea, 1970; Al-Amoudi et al., 1994].

Considering the long-term (i.e., more than 20 years) performance of concrete, some researchers [Mehta and Haynes, 1975] reported that all Portland cements, including Type V cements with C_3A of up to 3%, will be significantly affected by sea water. The reason may be attributed to the first "acidic" type of sulfate attack, which is ascribable to the prevalence of portlandite produced by the hydration of C_3S and C_2S phases. In fact, the portlandite content is proportional to the C_3S to C_2S ratio of the cement and this ratio is responsible for the early strength development of the cement. Since the manufacturers of modern cements do care a lot about the high early strength only, they deliberately make the C_3S/C_2S ratio extremely high thereby increasing the portlandite content. Therefore, these cements will readily deteriorate after long exposure periods. The principal form of deterioration will be the gypsum "acidic" type, whereby the portlandite produced by the hydration of calcium silicate phases will react with the sulfate ions in sea water to produce gypsum according to the following reaction:

$$Ca(OH)_2 + SO_4^{2-} \xrightarrow{2H_2O} CaSO_4{\cdot}2H_2O + 2OH^-$$

To mitigate the inferior role of portlandite, it is recommended to use pozzolanic (i.e., mineral) admixtures to consume this portlandite as shown in the following "pozzolanic" reaction:

$$3Ca(OH)_2 + 2SiO_2 \longrightarrow 3CaO{\cdot}2SiO_2{\cdot}3H_2O$$

In addition to consuming the portlandite, the pozzolanic reaction produces secondary C-S-H, which reduces the permeability of concrete thereby improving the sulfate resistance of blended cements. Therefore, the long-term performance of these cements, particularly those incorporating silica fume, is expected to be much better than plain portland cements exposed to marine environments, especially when the concrete mixture is to be prepared at a low water to binder ratio, similar to what is being currently used in offshore structures [Mehta, 1991].

5. CONCLUSIONS

Cement paste, mortar and concrete specimens prepared using Type I and Type V cements and Type I cement blended with fly ash (20%), silica fume (10%) and blast furnace slag (60%) were exposed to sea water for a period of 720 days. The performance of these cements was evaluated through visual inspection, measuring the reduction in compressive strength, expansion and weight change. Based on the results developed in this investigation, the following conclusions can be drawn:

The reduction in compressive strength and expansion data indicated better performance by blended cements, particularly the BFS and SF cements. The performance of Type V cement was marginally better than that of Type I cement.

An increase, rather than a decrease, in weight was noted in all the concrete specimens. The lowest increase in weight of about 1.50% was noted in SF and BFS after two years of exposure. The weight change was 1.85% in plain and FA cements.

No signs of deterioration were observed on plain and blended cements, even though the sulfate concentration in the sea water was high. The weight loss, expansion and reduction in compressive strength were less than the threshold values reported in the literature to denote failure due to deterioration by sulfate attack, even after two years of exposure.

Considering the long-term durability performance of concrete in marine and offshore structures, SF or BFS cement can be used at a low water to binder ratio. To further enhance the resistance against deterioration by sulfate ions and/or salt crystallization, additional protective measures, such as the application of a water-resistant epoxy-based coating, may be considered.

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ACHIEVING DURABLE CONCRETE SURFACES IN HOT CLIMATES BY THE USE OF ZEMDRAIN® CONTROLLED PERMEABILITY FORMWORK (CPF) LINERS

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ABSTRACT

In hot aggressive environments improvements to concrete durability are still necessary in order to reduce annual multi-million dollar bills for concrete refurbishment and replacement. Research has resulted in new standards being developed which have involved the development of new exposure classifications with associated strength requirements, maximum w/c ratio and minimum cement content recommendations. These requirements can all be classed as "product" related. Ideally they should lead to a higher quality, more durable concrete, which will have the required design life for the prevailing exposure conditions.

Unfortunately, perfection is never achieved on site and every "process" problem that will arise, inevitably leads to significant reductions in the concrete's resistance to all aggressive elements. Hot climates exacerbate these problems. Thus, no matter what cement replacement or admixture is added to the mix, the outer 20mm of the surface will always be the worst concrete in the structure This paper reviews the problems arising from process and reviews how the use of CPF liners like Zemdrain® can offer a solution.

Keywords

Controlled permeability formwork, durability, curing, blemishes, cover, product, process, carbonation, chloride ingress, micro-organisms.

INTRODUCTION

The quality of the outer 20mm of the concrete cover zone is not determined by the cement content and w/c ratio of the mix, but is largely determined by:

- the use of impermeable formwork face contact materials which trap excess air and water at the formwork/concrete interface
- the use of release agents whose residues contaminate the concrete surface
- the quality of compaction
- the quality of curing
- the overall quality of workmanship

The above process problems are exacerbated in hot and aggressive environments due to:

- arid regions with bright sunshine and little cloud cover
- long spells of hot weather
- peak temperatures above 50^0C
- varying humidity levels and dry blistering winds with significant speed fluctuations

Reductions in surface quality due to the above can be quite significant and can have a dramatic influence on the theoretical design life of any structure. Detailed analysis of the problems arising from process demonstrates conclusively that the use of CPF improves the quality of the outer 20mm of the surface helping to overcome most of these problems.

EFFECTS OF PROCESS ON FINISHED CONCRETE SURFACES

The effects of a harsh environment combined with process problems can significantly effect the properties of the concrete surface up to a depth of 20mm. The use of oiled impermeable formwork face contact materials, such as plywood, steel and plastic, results in water gain, cement reduction and trapped air bubbles in this critical zone.

1. Surface Blemishes

Blemishes that can normally occur on a concrete surface, as a result of process problems have been well documented, (Monks 1981). Blemishes attributed to the excess air and water normally trapped at the concrete/formwork interface include: -

- blowholes and pinholes
- scouring, crazing and plastic cracking
- dusting, retardation and oil discolouration

Each of the above reduces surface quality and durability by making the concrete more susceptible to early age weathering and to the ingress of deleterious agents.

2. Contaminated Surfaces

Biological growth thrives on a porous surface with blowholes and contaminated with release agent residues. This is particularly the case in hot environments with cycles of wetting and drying. Independent research, (Franke 1993) has shown that residues of oil/release agents used with impermeable faced formwork can penetrate up to 5mm into such a concrete surface. These chemicals are not subject to UV degradation once they are away from the surface inside the blowholes or pinholes. The residues of release agent not only provide nourishment for bacteria and fungi, but may also contaminate the water supply and affect the performance of applied coatings and surface penetrants, (Price 2000).

3. Surface Cement Content and Porosity

In the outer 20mm of the surface, the type of formwork face contact material used determines the porosity, w/c ratio and cement contents. A UK government sponsored research project; (Dhir 1999) involved laboratory plus in-situ testing on twelve construction sites. It was found that that surface cement content was typically reduced by up to 40 kg/m^3 below design values whilst porosity and w/c ratio were increased by up to 25%. This was irrespective of cement types (PC, GGBS, PFA, silica fume), the mix strength (25 to 60 N/mm^2) and the type of admixture (water reducers, superplasticisers, pore blockers, air entrainment) used and formwork orientation. The combination of reduced cement content and minimal surface blemishes leads to a surface with an increased porosity and reduced resistance to chloride ingress, expansive slats, chemical attack, abrasion and carbonation.

4. The Importance of Curing

The curing affected zone is judged to vary between 20 and 50mm depending upon the type and duration of curing employed. Adequate curing is, arguably, the most critical of all construction operations as regards concrete durability, unfortunately, as detailed in many learned journals it is rarely carried out effectively, (Price 1998). All the benefits of correct specification of materials and high standards of mix design and construction practice can be lost if curing is not properly carried out. The hot, dry and windy environment in the region necessitates the very highest standard of curing that can be obtained.

It is well known that good curing practice improves the performance of the concrete surface by reducing porosity and increasing resistance to carbonation, chloride ingress and abrasion. Curing is known to be particularly beneficial for concretes with cement replacements, including PFA, GGBS and Microsilica and in hot climates, (Walker 2002). The porous surface produced by conventional oiled formwork facilitates an immediate significant moisture loss upon formwork removal. The inevitable time delay between formwork removal and installing a curing regime result in less effective curing.

5. The Importance of Cover

The quality and depth of cover is always critical, but perhaps more so in aggressive environments. Surveys of in-situ structures in various areas across the world have revealed that it is almost impossible to consistently achieve the design cover, with shortfalls of 10 to 15mm below minimum common, (Slater 1999). This reduction in cover combined with the poor quality outer 20mm arising from process problems, results in premature reinforcement corrosion, even for well designed and placed mixes.

6. Summary

The majority of engineers make the erroneous assumption that the covercrete produced by conventional impermeable faced formwork will always be of the same durability and quality as the heartcrete. As the above has shown this is not the case, the worst concrete in any structure will always be at the surface, precisely the area where the best concrete is needed.