DURABILITY OF MICROSILICA CONCRETE, SUBJECTED TO ARABIAN GULF EXPOSURE

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الخلاصة :

إن تدهور المنشآت الخرسانية يعتبر مشكلة خطيرة جداً بمنطقة الخليج العربي. ولما كان معظم هذا التدهور بسبب نفاذ الكلوريدات خلال الخرسانة وما ينتج عنه من صدأ حديد التسليح؛ فقد انتشر في السنوات الخمس الأخيرة استخدام المايكروسيليكا في الخرسانة المسلحة أثناء عمل المهندسين لإطالة عمر الخرسانة التي تكون معرضة لأعمال الطقس القاسية بالمنطقة. ويُستخدم حالياً المايكروسيليكا بإضافته بنسبة بين ٥٪ إلى ١٠٪ وذلك حسب درجة تعرض الخرسانة للأحوال القاسية. والبحث يجيب عن: هل كمية المايكروسيليكا هذه تكفي للحصول على العمر الزمني الملائم للمنشآت الخرسانية ؟ وكم هي كمية المايكروسيليكا التي نحتاجها للحصول على خرسانة ذات عمر أطول تحت ظروف التعرض للاحوال المختلفة ؟ ونقترح في هذه الورقة، نماذج لتقدير كيفية نفاذ أيونات الكلوريد الى داخل خرسانة الاسمنت البورتلاندي والخرسانة المزودة بالمايكروسيليكا عند تعرضها لاحوال مختلفة. وتظهر النتائج أن نسبة قليلة من المايكروسيليكا تكفي طالما تم المحافظة على غطاء خرساني مناسب لحديد التسليح وكذلك تحديد مقدار تعرض الخرسانة للكلوريدات بصورة صحيحة.

ABSTRACT

Premature deterioration of reinforced concrete structures is a serious problem in the Arabian Gulf. Much of this deterioration is caused by chloride ingress through the concrete, which results in the corrosion of the reinforcing steel. Over the last 5 years, the practice of specifying microsilica in concrete has widened, as engineers work to improve the durability of concrete, which is subjected to the severe exposure conditions in the region. Current practice is to use microsilica additions ranging from 5% to 10% depending on the degree of exposure. Are these amounts of microsilica sufficient to maintain a realistic life span for concrete structures? How much microsilica is really needed to provide durable concrete with an appropriate life under the varying exposure conditions? In this paper, models are proposed to estimate the chloride ion ingress into Portland cement concrete and microsilica enriched concrete under different exposure conditions. The results show that small amounts of microsilica are sufficient as long as appropriate concrete covers to reinforcement are maintained and chloride exposure to the concrete is correctly assessed.

DURABILITY OF MICROSILICA CONCRETE IN THE ARABIAN GULF ENVIRONMENT

INTRODUCTION

Deterioration of reinforced concrete structures is a problem of serious proportions in the Arabian Gulf, with the most common form of deterioration being caused by chloride ingress through concrete, which results in the corrosion of the reinforcing steel [1]. When the steel rusts, it expands by up to 4 times its original volume, creating tensile stresses in the concrete, which lead to cracking and spalling of the concrete cover zone. Chloride ingress can then rapidly accelerate and eventually deterioration of the structure results. In 1992, Rasheeduzzafar [2] has reported the beneficial effect of using the pozzolans, Pulverized Fuel Ash, PFA, Ground Granulated Blast Furnace Slag, GGBFS, and Microsilica, MS, with Type 1 ordinary Portland cement, OPC, over the more commonly used sulfate resisting cement, SRC, to achieve longer life in concrete structures subject to the extreme exposure conditions. (see Figure 1).

In response to this research, engineers began specifying microsilica in structures subject to extreme chloride exposure conditions. The most immediate concern of the Client's was the high cost of the pozzolans specified. Microsilica sells for eight times the cost of cement in Saudi Arabia. Microsilica costs SR1500 to 1750 per ton, Portland Cement costs SR 200 to SR 220 per ton. When supplied from ready mixed concrete plants the quoted costs of concretes containing pozzolanic materials are summarized in Table 1.

Blast Furnace Slag with a high replacement level of 70% makes it a costly option. Microsilica, with lower replacement levels of 5 to 10%, is currently the most popular choice for durable concrete in the Gulf Region. Researchers have also shown that OPC mixes modified with microsilica are also resistant to sulfate attack [3]. The use of microsilica in the Eastern Province in recent years is shown in Table 2.

In all cases, the engineers have specified that the concrete should be tested using the ASTM C1202 Rapid Chloride Permeability Test, and have a low chloride ion permeability as classified by AASHTO T 277, with a test result of less than1000 Coulombs. The coulomb results achieved are shown in Figure 13, below.

Microsilica is a pozzolan like PFA and GGBFS and Table 3 shows typical properties of these materials.

The primary effect of microsilica on fresh concrete is to increase the cohesiveness. This increases the water demand and, as a result, plasticizing and superplasticizing admixtures are required to maintain the low w/c ratio and workability. Secondly, the filling of the major voids in the fresh concrete results in virtually no bleed water. Microsilica will increase the strength of the hardened concrete and reduce the permeability by densifying the matrix and thus providing a more durable concrete

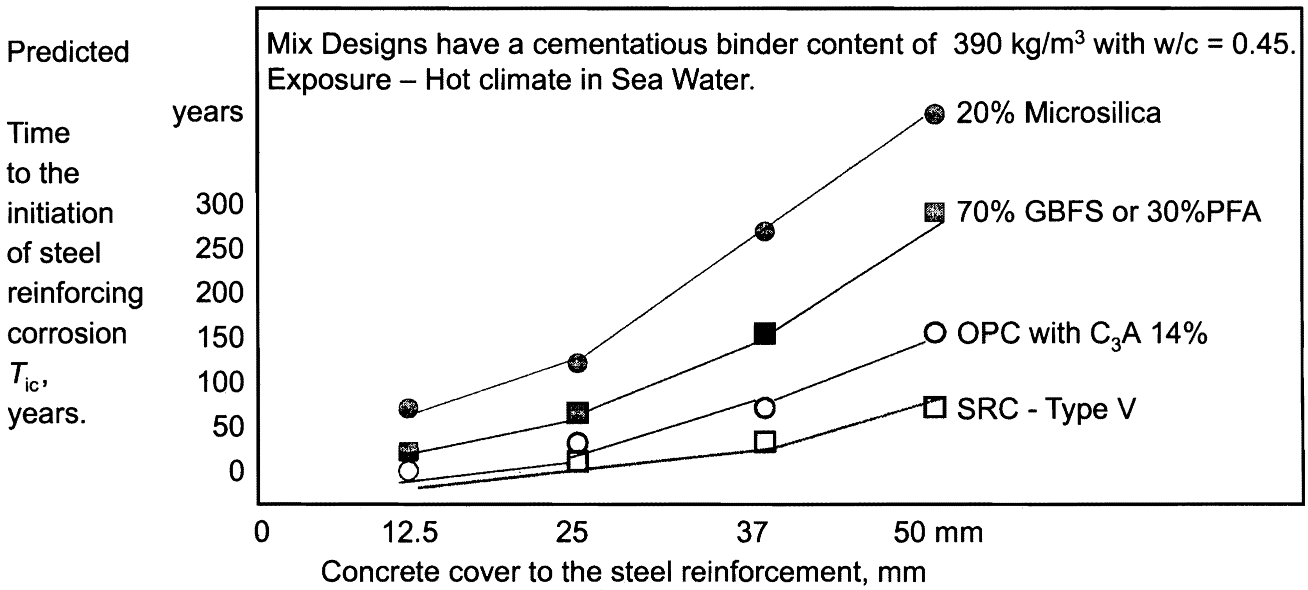

Figure 1. Predicted Time to the Initiation of Reinforcement Corrosion, at Various Covers for Concrete Made with SRC, OPC, and OPC Modified with PFA, GGBFS, and Microsilica Under Severe Chloride Exposure [2].

with a longer life. The refined pore structure reduces the passage of harmful ions like chlorides [4]. However, experience in hot weather has shown that high doses of around 15% may be susceptible to plastic shrinkage cracks forming during the initial set of the concrete. To prevent this cracking, care is required to ensure the recommendations given in Table 2.1.5 of ACI 305, the code of practice for hot weather concreting, are strictly adhered to. In particular cooling the concrete with chilled water and ice, and not pouring concrete on hot or windy days when humidity levels are low [5].

Table 1. Cost of Adding Pozzolans to Concrete in Saudi Arabia.

Mix Design	Mix Cost SR/m^3	Time to start of Corrosion[(a)] Years	Cost/Year of Design Life SR/Year
OPC + 20% MS	Not recommended as prone to cracking.		
OPC + 10% MS	250	380	0.66
OPC + 5% MS	220	240	0.92
OPC + 70% G B F Slag	620[(b)]	250	2.48
OPC + 30% PFA	335[(b)]	250	1.34
OPC with C_3A = 8%	180	100	1.8
SRC, with C_3A = 2%	185	60	3.1

All mixes have total cementations content 390 kg/m^3. Cover to reinforcement = 50mm.
[(a)]Rasheeduzzafar [2]. [(b)]Costs as quoted by a leading RMC supplier in August 1994. Other costs 1998.

Table 2. Microsilica Replacement Levels Used in Eastern Province 1993–7.

Year	Project	Microsilica %	Cover mm
1993	SCECO specification 71-SMSS-5 Rev2	10–15%	–
1993	SCECO power line piles to Half Moon Bay	11%	–
1993	Aramco, Executive Beach Club, Half Moon Bay	8%	75
1993	Beach Villa Foundations on Half Moon Bay	7%	–
1994	Aziziyah Desalination Plant	7–10%	–
1995	Aziziyah Beach Club	10%	75
1995	SCECO specification 71-SMSS-5 Rev3	8%	–
1995	Royal Commission of Jubail & Yanbu	8.2%	–
1997	Alkhobar Building Basement	5–7%	75
1997	Al Khaleeg Village on Half Moon Bay	8%	–
1998	Sunset Beach Development. Aziziyah	7%	–
1998	Gazlan Power Station. Specification	10–15%	85

Cement + microsilica = 390kg/m^3 unless noted otherwise.
% microsilica = microsilica as a percentage of the total cement + microsilica weight.

SCOPE OF THIS RESEARCH

As the development of codes of practice are lagging behind the development of new products and technology within the construction industry, this paper is intended to contribute to the body of knowledge on microsilica enriched concrete, and answer the following questions: are specification engineers justified in using microsilica levels ranging from 5 to 10% to obtain durable concrete?; what are the expected years to the initiation of reinforcement corrosion for these mixes under

Table 3. Physical and Chemical Properties of Microsilica Compared with Other Pozzolans.

Physical Data	Cement	PFA	GGBFS	Microsilica
Surface Area m^2/kg	350–500	300–600	300–500	15 000–20 000
Bulk density kg/m^3	1300–1400	1000	1000–1200	200–650
Specific gravity	3.15	2.3	2.9	2.2
Chemical Data	**Cement**	**PFA**	**GGBFS**	**Microsilica**
SiO_2	%	20	50	38 85–98
Fe_2O_3	%	3.5	10.4	0.3 0.2–2.5
Al_2O_3	%	5	28	11 0.2–2
CaO	%	65	5	40 0.2
MgO	%	0.1	2	7.5 0.2
$Na_2O + K_2O$	%	0.8	3.2	1.2 2.0

various exposure conditions?; and how much cover is required to the reinforcement? This research report will answer these questions, firstly through a literature search of current codes, specifications, and research on modeling chloride profiles, and secondly by proposing new models for calculating chloride diffusion coefficients for plain concrete and microsilica enriched concrete. Thirdly, by predicting the time to the initiation of corrosion of steel reinforcement in a range of concrete mixes under varying exposures. Finally, recommendations for designing durable concrete will be made.

LITERATURE REVIEW ON CURRENT DESIGN GUIDELINES FOR DURABLE CONCRETE MIX DESIGN

Bamforth [6] has clearly shown in (Table 4) the inadequacy of European codes in preventing corrosion of reinforcement in concrete subject to extreme chloride exposure.

Table 4. Inadequacy of European Codes, When Used in the Gulf Region [34].

Code	Exposure Condition	Cement kg/m^3	Cover mm	D_{eff} m^2/s	Time to onset of corrosion - Years
BS8110 Structures	Seawater spray	400	50	2.57e-12	5.4
BS5400 Bridges	Seawater spray	360	40	3.93e-12	5.5
BS6349 Marine Str.	Seawater spray	400	50–75	2.57e-12	5 to 12

The CIRIA Guide

The CIRIA Guide to concrete construction in the Arabian Gulf Region [7], provides guidelines for categorizing environmental exposure conditions; then a range of specification limits are given for selecting the appropriate mix design and the recommendations are summarized in Table 5. However the recommendations are inadequate as shown by Bamforth [6]. *(Note that this guide is currently being updated.)*

The existing international codes are inadequate in providing realistic life spans for major structures in the extreme exposure of the Arabian Gulf region. Firstly, the chloride diffusion coefficients for ordinary Portland cement mixes are not low enough to provide an adequate service life. Secondly, increased cover is required to the reinforcement. Finally, concrete containing either pulverized fly ash, ground granulated blast furnace slag or micro silica is required to provide an adequate service life.

Local Specifications for Concrete Mix Designs

Comprehensive specifications used by local organizations such as SCECO East [8], ARAMCO [9], and The Royal Commission for Jubail and Yanbu [10], provide guidance to engineers working in Saudi Arabia. A summary of the requirements, for durable mix designs, are given in Table 6.

Table 5. Exposure Conditions and Concrete Mix Design from CIRIA SP31 [7].

Exposure Condition	Cement Min. kg/m^3	Max W/C Ratio	Min Cover mm	Additional Requirement
A. Superstructure not exposed to Cl	300–320	0.52–0.5	30	None
B. Superstructure exposed to Cl	320	0.5	40	None
C. Foundations not exposed to Cl	320–350	0.5–0.45	40–50	None
D. Foundations exposed to Cl	300–420	0.5–0.42	40–50	Tanking membrane
E. Marine structures	370–400	0.45–0.42	75–100	

Table 6. Local Saudi Arabian Specifications for Durable Concrete Design.

SCECO [8]	Environmental classifications RMC Spec'n 71-SMSS - 5, Rev 3	Design recommendations Cement kg	MS kg or %	W/C Ratio	Coulomb Test
Severe	Exposed to sea water, or raw water wash down, exposed to Cl. concentration >0.1%	359	31 or 8%	0.4	<1000
Moderate	In contact with soil, with chloride <0.1% Exposed to fresh water, occasional raw water	350	nil	0.4	
ARAMCO [9]					
Severe	Exposed to sea water or near the coast, *Specified on a case by case basis*	370	30 or 7.5%	0.4	<1000
	Structural concrete to 09-SAMSS-097	350–370	nil	0.4	
Royal Commission of Jubail & Yanbu [10]					
1995	Royal Commission of Jubail & Yanbu	315–405	28–36 or 8.2%	0.4	<1000

Traditionally, engineering codes of practice give minimum cement contents and minimum covers, and maximum water to cement ratios to ensure durability of concrete. These guidelines are largely based on research into the permeability of concrete [11]. However, today it is more appropriate to specify minimum amounts of pozzolanic additives to achieve the longevity needed in structures exposed to severe environments. The local specifications summarized in Table 6, provide better advice for engineers in the region, than the European codes of practice in Table 4 .

The Rapid Chloride Permeability Test

This test (ASTM C1202) is frequently specified to test concrete containing microsilica in Arabian Gulf. The results are reported in Coulombs. The Concrete's chloride permeability is then assessed according to the AASHTO T277 classification shown in Table 7.

Table 7. AASHTO T277, Chloride Permeability Classification for Concrete.

Coulombs	Chloride Permeability	Typical of
4000	High	W/C > 0.6
2000 – 4000	Moderate	0.4 < W/C < 0.6
1000 – 2000	Low	W/C < 0.4
100 – 1000	Very Low	Microsilica Concrete
< 100	Negligible	Polymer Impregnated
Concrete		

The chloride ion permeability ratings have been confirmed by 90 day ponding tests.

A relationship between the Coulomb values and the effective Chloride Diffusion Coefficients, for concrete, would be useful. It would allow design engineers to calculate chloride profiles at the mix design stage from the results of 28 day Coulomb tests, and then the predicted design life could be calculated. However, at present no such relationship has been proven and hence the ASTM C1202 Coulomb test is only an appropriate tool for quality control purposes. It should be used at appropriate intervals to check that the correct amount of microsilica has been added to the concrete. Research is needed to verify the relationship between Coulomb values and chloride diffusion coefficients for concrete mixes with Coulomb values between 100 and 2500.

Local specifications provide good advice in recommending the use of pozzolans to produce durable concrete. But how long will concrete mixes, containing between 5% and 10% microsilica, last under severe exposure? And how much cover is required to provide adequate protection, to the steel reinforcement, from the severe exposure experienced in the Arabian Gulf region? To answer these questions we require models, that predict the rate at which chloride ions diffuse through the concrete, for varying concrete mix designs. The models could then be used, to calculate the time to the initiation of steel reinforcement corrosion, and hence an estimate of the design life could be calculated for varying mix designs.

Literature Review on Chloride Diffusion into Concrete

The main transport processes leading to the deterioration of concrete in the Arabian Gulf, are water absorption from capillary forces and wick action, followed by water evaporation and the resulting salt crystallization on the surface and in the pores of the drying zone. Once transported to the surface, the chloride then diffuses through the concrete and eventually the reinforcement corrodes. This process occurs in all structures in contact with salty sea or ground water or damp salty soil. To model this chloride diffusion process, engineer's use Fick's Second law which enables the chloride profile in the concrete to be calculated [12–14].

Fick's Second Law

Figure 2 shows a typical chloride profile calculated using Fick's diffusion laws for modeling the diffusion of chloride ions into concrete over a prolonged period of exposure.

This results in a solution that can be used to calculate C_{xt}, the concentration of diffusing atoms at a distance x inside a material after time T. The solution is referred to as the Fick's chloride profile equation.

$$C_{xt} = C_s - (C_s - C_o) \times erf\,(x/(2\sqrt{(D_{\text{eff}} \times T)})) \tag{1}$$

Where;

C_{xt} = The concentration of chloride at a distance, x, inside the concrete after time T, as % weight of concrete

C_o = Original chloride concentration in the concrete from all the mix materials, as a % of the concrete weight

C_s = Surface concentration of chloride, as a % of concrete weight

x = distance from the surface in cm

T = Time period for chloride buildup in the concrete in years

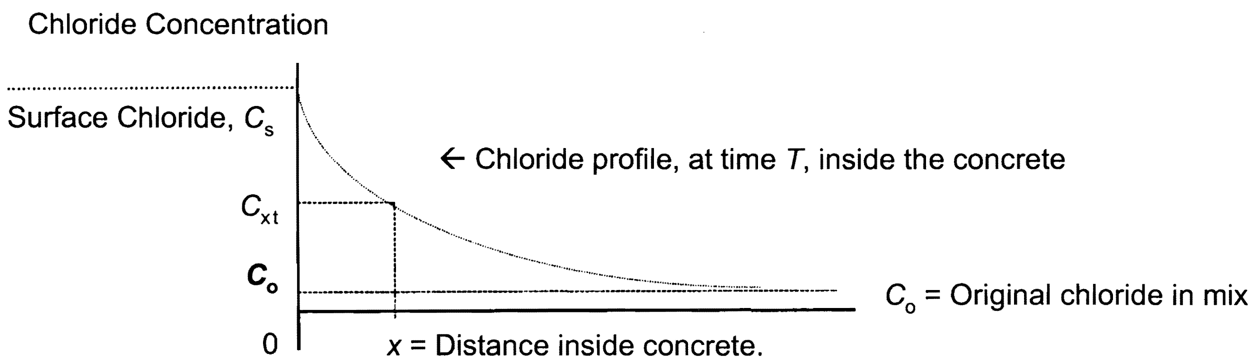

Figure 2. Relation Between Chloride Concentration and Distance from Concrete Surface. The chloride profile calculated using Fick's Second Law [15].

erf = the error function from standard tables [15]

D_{eff} = the effective diffusion coefficient over the design life of the structure = $D_{ei} \times K_{ti} \times K_{te}$, in cm^2/year. (2)

D_{ei} = the initial diffusion coeff at 28 days, in cm^2/year.

K_{ti} = time related effect on the diffusion coefficient

K_{te} = temperature related effect on the diffusion coefficient.

Original Chloride Concentration C_o, in the Concrete from all the Mix Materials

Local and international specifications, firstly limit the chloride content of the mix ingredients, then, secondly, that of the total mix, as shown in Table 8. Practically, we should aim to have the lowest chloride level possible with locally available materials, particularly for concrete exposed to sea water. That is, a maximum $C_o < 0.10\%$ by weight of cement for all concrete. In some parts of the Gulf region this may not be achievable. If so, then a higher grade of concrete should be specified, with more cement and microsilica, to compensate for a higher C_o in the initial concrete mix. Alternatively corrosion inhibiting admixtures may need to be used.

In the Gulf, it is also important to wash the aggregates, as the fines often contain high concentrations of deleterious substances like sulfates and chlorides.

Test measurements taken on concrete used in the Eastern Province of Saudi Arabia, presented in Table 15, show that with adequate quality control procedures, the original chloride level in the concrete, C_o, can be kept below 0.1% by weight of cement or 0.017% by weight of concrete. Hence, C_o = 0.017% by weight of concrete will be used by the author to model chloride profiles and assess the time to the initiation of corrosion of reinforcing steel.

Table 8. Local and International Code Limits to the Total Chlorides in the Original Concrete Mix, C_o.

Maximum Acid Soluble Chlorides in a Concrete When Mixed, C_o as a % by Weight of Cement.

Local limits	SCECO	ARAMCO	R.Comm.J&Y
Type of concrete	71-SMSS-5 Rev3	09-SAMSS-097	SAB 03347
Reinforced concrete in a moist environment and exposed to chloride	<0.10%.	<0.25%	<0.13%

International limits	American ACI 201	British BS 8110	British BS 5400	FIP Hot Weather	CIRIA SP31
RC exposed to chloride in service	<0.10				Gulf
Reinforced in SRC		<0.20	0.06	0.15	0.15
Reinforced in OPC			0.35	0.15	0.30
Reinforced in OPC & PFA or BFS		<0.40			

Environmental Chloride and the Concrete Surface Chloride Concentration, C_s

The concrete surface chloride ion concentrations C_s, for different mix designs, which undergo varying exposure conditions, are required to predict the chloride ingress using Fick's law [16–20, 22]. The Surface chloride values plotted in Figure 3, show typical values measured in hot marine environments.

The C_s values are greatest in areas subject to wetting and drying. When salty water evaporates, any chloride in the water will remain in the surface zone in a crystalline form. In marine structures, the C_s value will be at a maximum just above the sea water level in the splash zone. In bridges and buildings, the C_s value will be at a maximum just above ground level where water is drawn through the concrete and then evaporates from the surface.

Effect of Temperature on C_s

In the Arabian Gulf, the effect of the high temperatures on chloride penetration in concrete needs to be considered. Al-Khaja *et al.* [21] have shown that C_s more than doubles, from 0.39 to 0.93, with a temperature increase from 20°C to

45°C. The temperatures in the region vary between low extremes of 3°C in winter to high extremes a 48°C in summer, with an annual mean temperature of 26.5°C in Bahrain and Al Khobar. Hence, we could expect an increase in C_s, from 0.39 at 20°C in the lab, to C_s of 0.5 at 26.5°C, that is by 22% for mean site conditions, to a doubling of C_s in the hot summer months [21].

Effect of Mix Design on C_s in the Gulf Environment

Sharafi A. *et al.* [22] have studied chloride ingress into concrete in the UAE. They measured the surface chloride level in the concrete after 1 year and their results are summarized in Table 9. The results indicate that C_s varies according to the concrete mix design. When compared to the OPC control mix, the lab trials show that C_s was higher in the microsilica concrete, however, in the tidal zone C_s was much lower for microsilica concrete.

These results are not conclusive and further research is required to clarify appropriate C_s values for microsilica enriched concrete. Bamforth [23], has shown that C_s eventually reaches a maximum level after a year and then stays relatively constant during the life of the structure, depending on the type of concrete used.

Table 9. Surface Chloride, C_s, Varies with Mix Design [22].

Surface chloride	C_s, as a % of concrete weight, measured in Dubai trials			
	Lab Trials	Site Exposure Trials		
	In 3% NaCl	Above ground	Below ground	Tidal zone
Mix with 10% microsilica	0.30	0.031	0.03	0.25
Control mix 370 kg/m³ OPC	0.19	0.002	0.02	0.53
Environmental Chloride concentrations measured at the exposure site.				
Environment %NaCl	3	0	0.229	3.70

Effect of Exposure on C_s

Bamforth [23] has proposed that engineers use the C_s values shown in Table 10 for chloride modeling under varying exposures.

The values proposed by Bamforth[23] for blended cements containing PFA, GGBFS or microsilica appear high. The $C_s = 0.9\%$ for extreme environments corresponds with values shown in Figure 3 that were measured in GBFS concrete on the Bahrain Causeway, by Bijem *et al.* [17, 19]. Similar high values have been measured at the Qatar wharves for SRC concrete by Akili [16]. However, for microsilica concrete, research done in Dubai [22], shows C_s values as low as 0.25, for

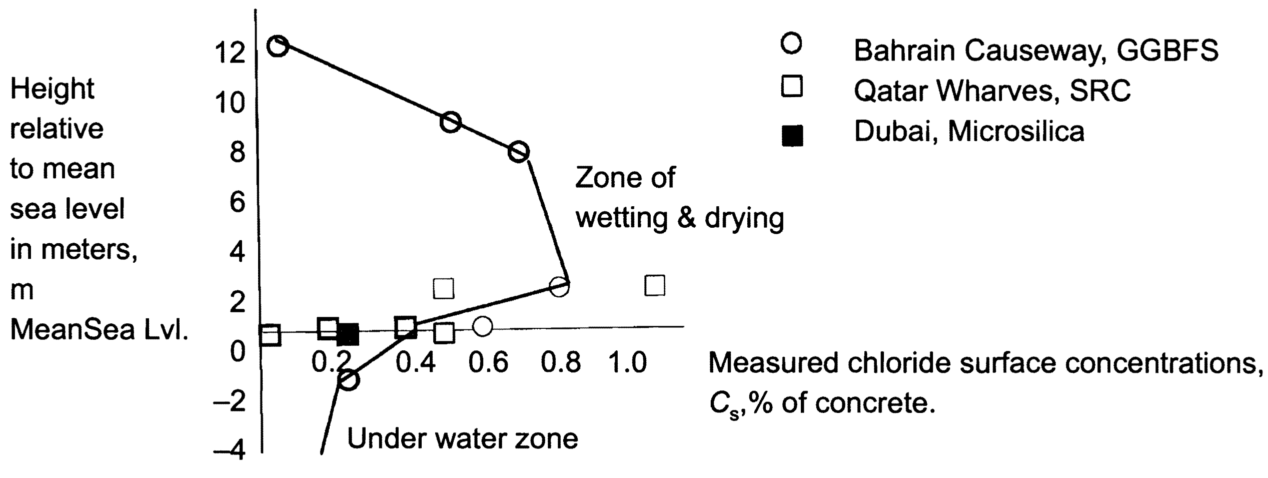

Figure 3. Relation Between the Location of Exposure and the Measured Surface Chloride Level, C_s, of the Concrete in Hot Marine Climates – Extreme Exposure [16, 17, 19, 20 & 22].

mixes having low w/c ratios and exposed to sea water. One would expect C_s values for low w/c ratio microsilica concrete to be lower than OPC at the same w/c ratio as the pore structure is much finer, as demonstrated by Hussain [24].

The range of surface chloride levels, C_s, expected under varying exposures, and the actual values used by the author for modeling chloride ingress, are shown in Table 11.

Note that C_s values may also vary with the amounts of pozzolans used but more research is required before differentiating between C_s and mix type. Bamforth [23] has proposed that C_s for blended cements are 20% higher than for Portland cements. But Sharafi *et al.* [22], have shown that microsilica concrete exposed to seawater in Dubai has 50% lower C_s than Portland cement concrete under the same exposure. Once the maximum surface chloride level has been established, and the initial chloride level measured, then the chloride diffusion coefficient is required to estimate the chloride profile over the life of the concrete.

Table 10. Proposed C_s values, % by wt. Concrete, Suggested for Design Purposes [23].

Exposure	C_s for Portland cement	C_s for cements containing pozzolans PFA, GGBFS or Microsilica
Extreme	>0.75%	>0.9%
Severe	0.5% to 0.75%	0.6% to 0.9%
Moderate	0.25% to 0.5%	0.3% to 0.6%
Low	<0.25%	<0.3%

Table 11. Environmental and Surface Chloride Levels, C_s, Under Varying Exposure Conditions.

Exposure	Typical Condition	Cl concentration % Conc.	C_s range % Conc. [23]	**C_s modeled** % Conc.
Extreme	Sea water in hot climate	3.5% in Gulf	0.6–0.9	**0.9**
Severe	Sea water in temperate climate	2.5% in Europe	0.3–0.6	**0.6**
Moderate	In contact with ground water or raw water wash down	0.1–0.5%	0.15–0.3	**0.3**
Low	Foundation & Superstr. to 1st Floor in hot climates within 3m of water table	<0.1%	0–0.15	**0.15**
Nil	Superstructures above 1st Floor	Nil	Nil	

The Effective Chloride Diffusion Coefficients, D_{eff}

The effective chloride diffusion coefficient defines the rate at which chloride ions migrate from the surface of the concrete to the reinforcement. The rate varies according to the mix design, the concrete age, and the exposure temperature.

$$D_{eff} = D_{ei} \times K_{ti} \times K_{te}\,, \qquad (2)$$

Where;

D_{ei} = the initial chloride ion diffusion coefficient, at 28 days at laboratory temperatures.

K_{ti} = a time related coefficient dependant on mix design.

K_{te} = a temperate related coefficient dependant on exposure temperature.

Chloride Diffusion and Mix Design

Research by Malikakkal at KFUPM [14], has related the initial chloride diffusion coefficient in concrete to the water/cement ratio and cement content, as shown in Equation (3).

Malikakkal's Diffusion equation for SRC concrete in Dhahran:

$$D_{ei} = 0.315 \times (82.7 - 426 \times w/c + 568 \times (w/c)^2 + 4.26 \times C^{-6})\ \text{cm}^2/\text{year at } 20°\text{C}. \qquad (3)$$

Where, w/c = water to cement ratio, C = (cement content/350) for concrete made with SRC.

Hussain [24] has shown that for OPC mixes, $D_{eiopc} = 0.736 \times D_{eisrc}$. (4)

Hence, for standard ready mixed concrete mixes, the chloride ion diffusion coefficients under laboratory conditions, D_{ei}, have been calculated using Equations (3) and (4) and are shown in Table 12.

Table 12. Initial Chloride Diffusion Coefficients, D_{ei}, for Saudi Arabian SRC and OPC Concrete [14, 24].

Mix psi	Cement kg/m^3	W/C Ratio	D_{eisrc} cm^2/year	D_{eiopc} cm^2/year
5000	400	0.40	1.65	1.44
4500	360	0.45	3.08	2.68
4000	340	0.50	5.34	4.65
3500	295	0.53	9.0	7.83

Researchers [2, 24, 25] have shown that type 1 cement, blended with pozzolans, will give much lower diffusion coefficients and hence far better resistance to chloride ion penetration, than plain Portland cements. As this research was done on cement pastes one can assume that the results are lower than those that could be obtained on concrete containing these constituent materials and aggregates. They provide the lowest limit that one might expect of these materials, and clearly show the benefit of pozzolans, and in particular microsilica mixes, in reducing pore radii and chloride diffusion coefficients.

Time Related Reduction in Diffusion Coefficients K_{ti}

Maage [25] has shown that the chloride diffusion coefficients, for concrete containing microsilica, reduce with increasing the microsilica dosage and with age, as demonstrated in Figure 4. Note that, for concrete the chloride diffusion coefficients reduce with age, but, the reduction is greater when pozzolans are added.

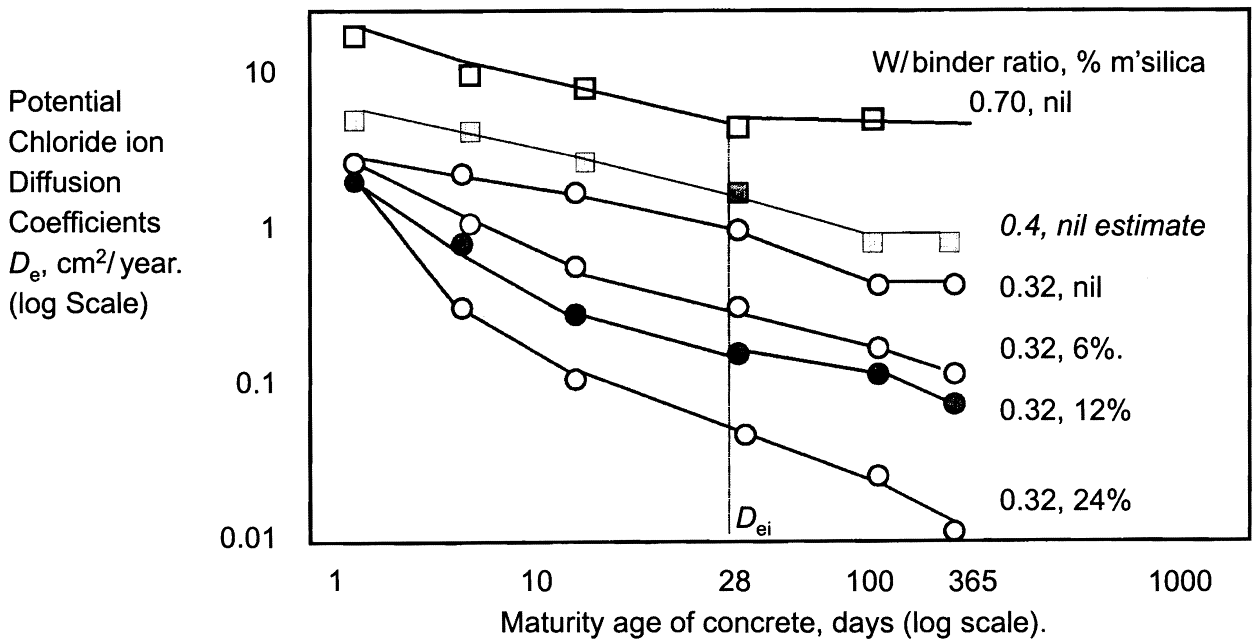

Figure 4. Relation Between Chloride Ion Diffusion Coefficient's and Age of Concrete for Various w/c *and Microsilica Doses [25]. Define* D_{ei} *as the chloride ion diffusion coefficient at a concrete age of 28 days.*

Maage has used low water/cement ratios, hence, his results will represent a line near the lowest achievable for microsilica concrete. The addition of microsilica will reduce the diffusion coefficients by an order of magnitude, which, will extend the time to the onset of corrosion. Figure 5 shows the reduction of diffusion coefficients during a typical 100 year design life for a concrete structure, particularly when blended cements are used. Hence, in our model for calculating the time to the onset

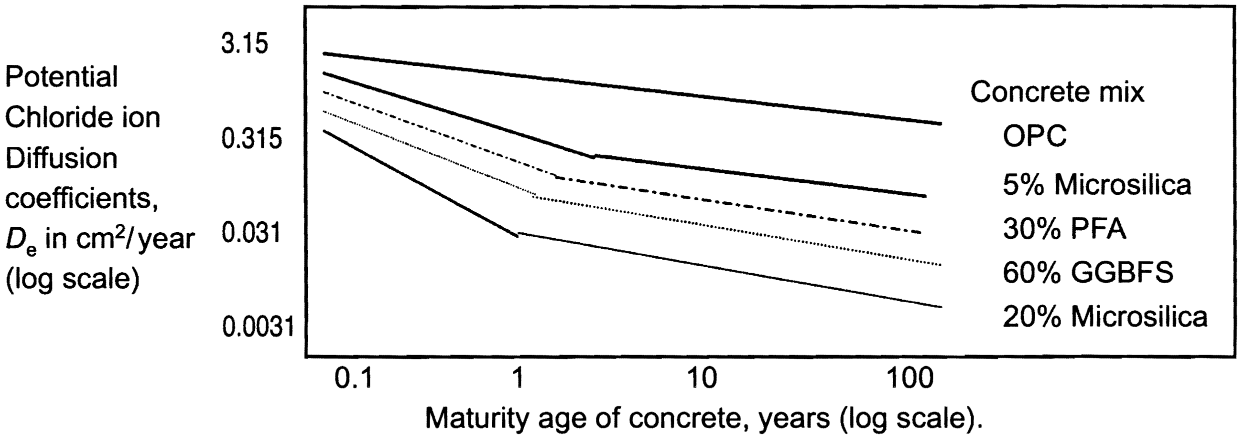

Figure 5. Time Related Reduction in the Chloride-Ion Diffusion Coefficients for Concrete Containing Pozzolans Over the Design Life of a Concrete Structure [12, 25].

of corrosion of the reinforcing steel, which may be up to 100 years, we must take account of firstly, the effect of the dose of pozzolans and secondly, the time related drop in diffusion coefficients over the design life of the structure [12, 25].

Theoretical Models for Calculating Chloride Diffusion into Concrete

For Saudi Arabian concrete's, containing local materials, models 1 to 5 are proposed to calculate the chloride ion diffusion coefficients for concrete containing OPC, SRC, and microsilica, and the Coulomb test results for microsilica concrete. These modeled diffusion coefficients will then be used to calculate the relationship between mix design, cover, and life expectancy of a concrete structure to ensure an appropriate durable mix is chosen at the design stage, using Figures 9 to 12, (below).

Model 1. Chloride Diffusion Coefficients of Concrete Made with Sulfate Resisting Cements

This model is proposed to help engineers calculate the effective chloride-ion diffusion coefficients, D_{eff}, over the design life of a structure, from the concrete mix design for a structure built with concrete containing SRC.

$$D_{effsrc} = D_{eisrc} \times K_{ti} \times K_{te} \, , \tag{2}$$

Where;

D_{eisrc} = The initial chloride ion diffusion coefficient, at 28 days at laboratory temperatures, for Saudi Arabian concrete's containing SRC, from Malikakkal's diffusion equation [14] (Equation (3));

$$D_{eisrc} = 0.315 \times (82.7 - 426 \times (w/c) + 568 \times (w/c)^2 + 4.26 \times C^{-6}) \text{ cm}^2\text{/year},$$

K_{ti} = the time related coefficient = $1/(1 + 2 \times \log(T+1))$, for T years of design life.

K_{te} = 1 for cold & temperate climates and K_{te} = 2 for hot & tropical climates like the Arabian Gulf.

There is a lack of technical research data to support doubling the diffusion coefficient, however, this will add a degree of conservatism to the predictions for hot climates until more research has been done. Thus, for modeling chloride diffusion, the effective diffusion coefficient becomes;

$$D_{effsrc} = 2 \times (1/(1 + 2 \times \log(T + 1))) \times 0.315 \times (82.7 - 426 \times (w/c) + 568 \times (w/c)^2 + 4.26 \times C^{-6}) \text{ cm}^2\text{/year}. \tag{5}$$

For structural concrete made with SRC for a 100 year design life in the hot Arabian Gulf region.

$$D_{effsrc} = 0.126 \times (82.7 - 426 \times (w/c) + 568 \times (w/c)^2 + 4.26 \times C^{-6}) \text{ cm}^2\text{/year}. \tag{6}$$

Model 2. Chloride Diffusion Coefficients of Concrete Made with Ordinary Portland Cement

This model is proposed to help engineers calculate the effective chloride-ion diffusion coefficients over the design life of a structure, from the concrete mix design. The chloride ion diffusion coefficients, for concrete containing OPC, are estimated using Malikakkal's diffusion Equation (3) [14], but with a correction to lower the diffusion coefficient by thirteen percent, as a result of the research by Hussain [24].

$$D_{\text{effopc}} = 0.110 \times (82.7 - 426 \times (w/c) + 568 \times (w/c)^2 + 4.26 \times C^{-6}) \text{ cm}^2/\text{year}. \tag{7}$$

Model 3. Proposed Chloride Diffusion Coefficients of Concrete Made with OPC and Microsilica

This model is proposed to help engineers calculate the effective chloride-ion diffusion coefficients, D_{effms}, over the design life of a structure, from the concrete mix design for a structure containing microsilica.

$$D_{\text{effms}} = D_{\text{eims}} \times K_{\text{tims}} \times K_{\text{te}} , \tag{8}$$

Where K_{tims} is a time related coefficient and K_{te} a temperature related coefficient;

$$D_{\text{eims}} = 1.5 - 1.25 \times \log(ms + 1) + ms^2/1500, \text{ cm}^2/\text{year} \tag{9}$$

ms = percentage replacement of microsilica = $100 \times Ms/(C + Ms)$, Ms is microsilica content in kg/m^3 of concrete and C is the OPC content in kg/m^3 of concrete. This equation is based on the diffusion coefficients published by researchers [12, 13, 20, 23, 25] at an age of 28 days. The following boundary conditions must be given to Equation (9). It is only proposed for microsilica replacement levels between 5% and 15%. Concrete mixes must have $W/(C + Ms) < 0.4$ and be made of OPC cement. The concrete mixes should have between 370 kg and 400 kg of cement and microsilica per cubic meter. Super plasticizing and retarding admixtures should be used to maintain the low w/c ratio.

For predicting chloride profiles over lifetimes of up to 100 years it is necessary to allow for reducing the diffusion coefficients with age. It has been shown by Maage [25] and Bamforth [12], that the average diffusion coefficient will significantly reduce with time, particularly for higher microsilica replacement levels. To simulate this effect, the age related coefficient for microsilica K_{tims}, is calculated as follows.

$$K_{\text{tims}} = 1/(1 + 3 \times \log(T + 1) + 0.1 \times ms \times \log(T + 1)). \tag{10}$$

T = age in years for predicting chloride level, ms = % replacement of microsilica up to 15%.

K_{te}, the temperature related coefficient, $K_{\text{te}} = 1$ for cold climates and $K_{\text{te}} = 2$ for hot climates,

$$\text{Using } D_{\text{effms}} = D_{\text{eims}} \times K_{\text{tims}} \times K_{\text{te}}, \text{ then} \tag{8}$$

$$D_{\text{effms}} = 1/(1 + 3 \times \log(T + 1) + 0.1 \times ms \times \log(T + 1)) \times (1.5 - 1.25 \times \log(ms + 1) + ms^2/1500) \times K_{\text{te}} \text{ cm}^2/\text{year}. \tag{11}$$

Now for 100 years in hot climates.

$$D_{\text{effms}} = 1/(7.01 + 0.2 \times ms) \times (1.5 - 1.25 \times \log(ms + 1) + ms^2/1500) \times K_{\text{te}} \text{ cm}^2/\text{year}. \tag{12}$$

Typical values of D_{effms} are plotted in Figure 6.

Model 4. Proposed Coulomb Prediction Equation for Microsilica Enriched Concrete, C_{ms}

Based on published results from microsilica suppliers, Binnex [26], Elkem [4], it is possible to model expected Coulomb values for microsilica enriched concrete. Equation (13) is proposed to estimate the ASTM C1202-91 Coulomb value of microsilica enriched concrete, C_{ms}, having $W/(C + Ms) < 0.4$ and with a total OPC cement and microsilica content of 370 to 400 kg/m^3 with microsilica contents between 5% and 15%.

$$C_{ms} = 2250 - 1650 \times \log(ms + 1) + 0.7 \times ms^2, \text{ Coulombs}. \tag{13}$$

Where, $ms = 100 \times Ms/(C + Ms)\%$, Ms = microsilica content in kg/m^3 and C = OPC content in kg/m^3.

The predicted Coulomb values for microsilica enriched concrete, calculated using Equation (13), have been plotted against microsilica manufacturers literature in Figure 7. Equation (13) represents a line on the upper limit and 95% of measured values should fall below this line.

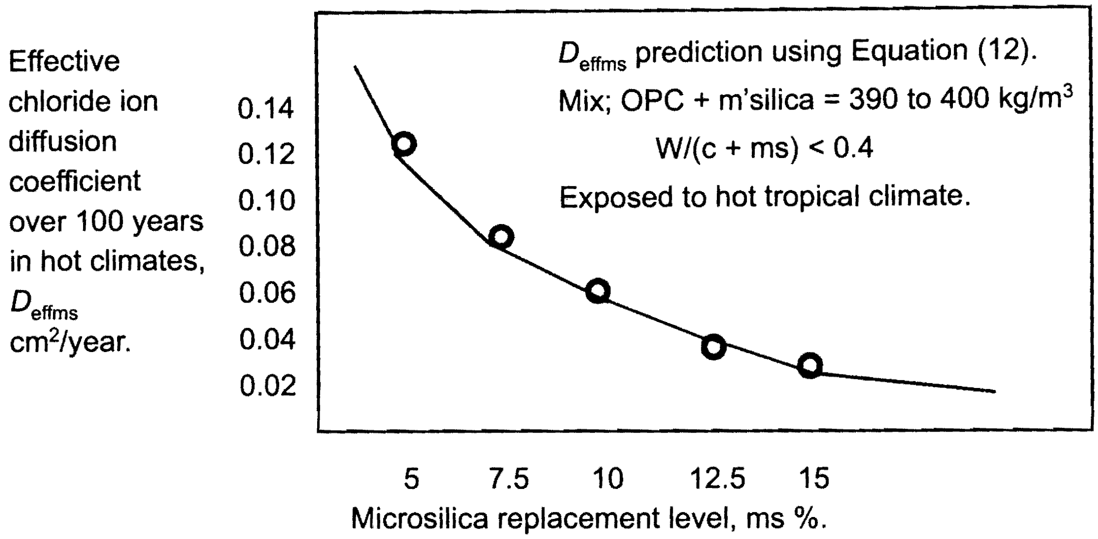

Figure 6. Proposed Relation Between Effective Chloride Ion Diffusion Coefficients Over 100 Years, and Microsilica Replacement Levels, for Mixes Exposed to Hot Marine Environments, Calculated Using Equation (12).

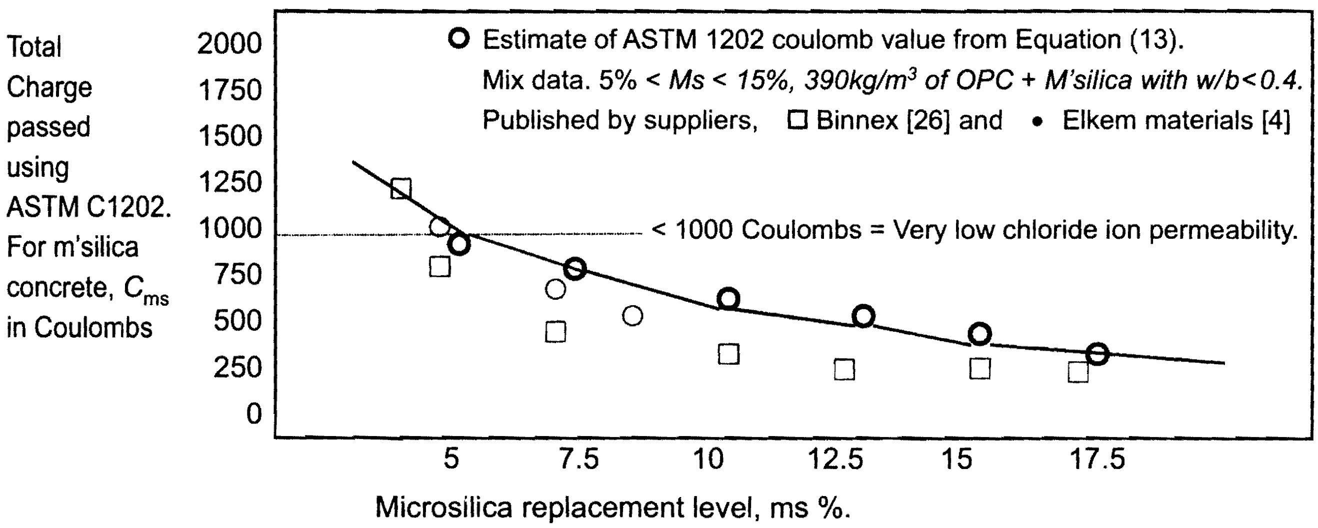

Figure 7. Proposed Relation Between Rapid Chloride Permeability in Coulombs and Percentage Microsilica, Calculated Using Equation (6).

Model 5. Proposed Coulomb–Diffusion Coefficient Relationship for Microsilica Enriched Concrete

The following equations are proposed to help engineers estimate the chloride ion diffusion coefficients for microsilica concrete, using the ASTM C1202 Coulomb test results, C_{ms}, The effective diffusion coefficient;

$$D_{\text{effms}} = D_{\text{eims}} \times K_{\text{tims}} \times K_{\text{te}} . \tag{14}$$

Where the estimated initial Chloride Ion diffusion coefficient at 28 days.

$$D_{\text{eims}} = C_{ms}/1600 + 2.1/\sqrt{(C_{ms})}.\ \text{cm}^2/\text{year}, \qquad \text{where } C_{ms} = \text{Coulomb value at 28 days.} \tag{15}$$

The ageing coefficient:

$$K_{\text{tims}} = 1/(1 + 3 \times \log(T+1) + 0.1 \times ms \times \log(T+1)),\ \text{at age of } T \text{ years.} \tag{10}$$

K_{te}, the temperature related coefficient is 1 for cold climates and 2 for hot climates like the Arabian Gulf .

Hence, for mixes containing microsilica replacements, *ms*, from 5% to 15%, over a life time of 100 years in hot climates.

$$D_{\text{effms}} = 2 \times (C_{ms}/1600 + 2.1/\sqrt{C_{ms}}) \times (1/(7.01 + 0.2 \times ms)). \tag{16}$$

Using Equations (13) and (16), we can estimate Coulomb values and diffusion coefficients given the microsilica replacement levels, the results are plotted on Figure 8.

The diffusion coefficients calculated in Table 13, and shown in **bold** type, will be used to predict the time to the initiation of corrosion of reinforcing steel and the results plotted in Figures 9 – 12.

Table 13. Estimated Chloride Ion Diffusion Coefficients and Coulomb Values, for Plain and Microsilica Enriched, Concrete at an Age of T = 100 yrs in the Arabian Gulf Climate from Proposed Equations (6), (7), (12), (13), & (16).

Concrete mix design. Cement, w/b, microsilica				Equation 6	Equation 7	Equation 12	Equation 13	Equation 16
kg/m^3	ratio	Kg	%	D_{effsrc} cm^2/year	De_{ffopc} cm^2/year	De_{ffms} cm^2/year	C_{ms} Coulombs	D_{effms} cm^2/year
295	0.53	0	0	3.59	3.13			
340	0.5	0	0	2.13	1.86			
360	0.45	0	0	**1.23**	**1.07**			
400	0.4	0	0	**0.66**	**0.57**			
370	0.4	20	5			**0.136**	984	0.137
361	0.4	29	7.5			**0.088**	756	0.093
351	0.4	39	10			**0.059**	602	0.064
342	0.4	48	12.5			**0.040**	494	0.045
332	0.4	58	15			**0.029**	421	0.032

*Note that in the Arabian gulf climate great care needs to be taken using more than 15% microsilica replacement, especially during the hot summer months, and it is not recommended.

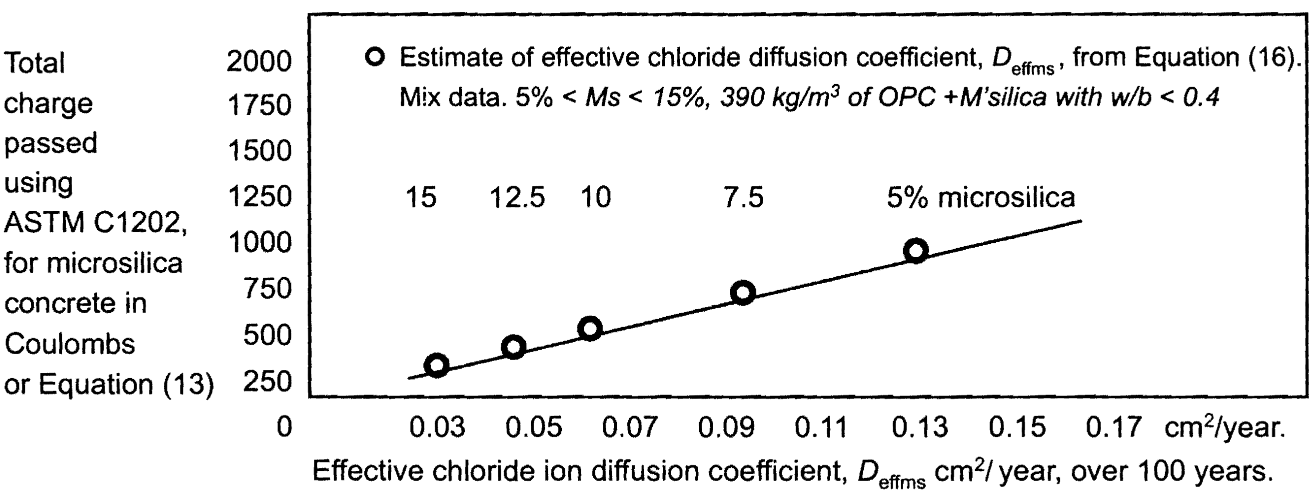

Figure 8. Proposed Relation Between the Rapid Chloride Permeability at 28 Days and the Chloride Ion Diffusion Coefficients, at 100 Years, in Hot Climates, for Microsilica Enriched OPC Concrete, from Equation (16).

Model Prediction of the Time to the Initiation of Reinforcement Corrosion, T_{ic} in Years, for Various Mix Designs Under Different Exposure Conditions

The life expectancy of structures can be estimated by using Equation (1).

$$C_{xt} = C_s - (C_s - C_o) \times erf\,(x/(2 \times \sqrt{(D_{\text{eff}} \times T)}). \tag{1}$$

By calculating the time taken for the concrete chloride level, C_{xt}, to reach the threshold chloride level that results in steel corrosion, C_{ic} at the position of the reinforcement, x cm from the surface of the concrete, we can calculate the time to the initiation of corrosion T_{ic} and obtain a measure of the design life that can be expected for the concrete structure. By substituting C_{ic} for C_{xt} and T_{ic} for T in Equation (1) the resulting equation is;

$$(C_s - C_{ic})/(C_s - C_o) = erf\,(x/(2 \times \sqrt{(D_{eff} \times T_{ic})}), \tag{17}$$

Where T_{ic} is the years to the initiation of reinforcing steel corrosion at cover of x cm.

Note that standard tables must be consulted for values of the error function, *erf* [15].

Modeling Constants C_{ic}, C_o, C_s, and D_{eff} Used in Equation 17 to Prepare the Design Figures 9 to 12

Threshold Chloride Levels, C_{ic}

Threshold chloride levels, C_{ic}, that result in depassivation of the protective oxide layer and the onset of corrosion of the steel reinforcement, vary depending on the type of cement used as shown in Table 14.

Table 14 shows the better performance of the Type 1 cements and in particular those with high C_3A contents over Type 5, SRC, in preventing corrosion of reinforcing steel. But high C_3A cements are not readily available and are not generally used by industry. Bamforth [12, 23] advises that for PFA and GGBFS enriched concrete, a reduction in threshold corrosion levels by 20% should represent a reasonably conservative assumption. He also suggests that for microsilica the threshold corrosion levels may be even lower. However, Talib *et al.* [27] have shown that the time to initiation of corrosion for microsilica concrete's is increased. Until more research is been done to clarify this, we will assume that microsilica concrete has similar threshold corrosion levels to OPC concretes of similar w/c ratios.

For modeling chloride profiles over the design life of structures, Bamforth [6], advises that even though the reinforcement begins to corrode at low threshold levels around 0.4% of chloride by weight of cement, the initiation of rapid corrosion and cracking may not occur until the chloride levels reach 1% of chloride by weight of cement.

Threshold Chloride Levels

The Threshold Chloride Levels that Result in the Initiation of Corrosion C_{ic}, for different cementatious materials, used for modeling are shown in Table 15.

Note that for microsilica concrete, researchers are divided, Bamforth [19] suggests lower values should be used for microsilica, while Talib *et al.* [27] have shown higher values. Therefore, we assume C_{ic} for microsilica will be the same as OPC for modeling until future research clarifies this.

Table 14. Threshold Chloride Levels, C_{ic}, that Result in the Initiation of Corrosion of Steel in Mortar [24].

Cement Sample	C_3A content % by wt. of cement	Threshold acid soluble chloride level % by wt. of cement	% by wt. of concrete
1. SRC	2.43	0.35	0.06
2. OPC	7.59	0.62	0.10
3. OPC	14.00*	1.00	0.16

*14% C3 A cement not readily available.

Table 15. Threshold Chloride Levels, C_{ic} for Different Cementatious Materials Used for Modeling.

Cementatious material	C_{ic} % by weight of cement	C_{ic} % by weight of concrete
SRC	0.35	0.058
OPC	0.60	0.100
M'Silica	0.60	0.100

Original Chloride in the Concrete Mix, C_o

C_o = 0.017% by weight of concrete has been used to calculate the chloride profile.

Surface Chlorides, C_s

$C_s = 0.9$ for extreme exposure

$C_s = 0.6$ for severe exposure

$C_s = 0.3$ for moderate exposure

$C_s = 0.15$ for low exposure.

Refer to Table 16 for details of typical exposure conditions.

Chloride Ion Diffusion

The Effective Chloride Ion Diffusion Coefficients, D_{eff}, for 100 years in hot climates, shown in Table 13, have been calculated using Equations (6), (7), and (12) for varying mix designs, and used in the Model predictions;

$$D_{\mathrm{effsrc}} = 0.126 \times (82.7 - 426 \times (w/c) + 568 \times (w/c)^2 + 4.26 \times C^{-6})\ \mathrm{cm^2/year}. \quad (6)$$

$$D_{\mathrm{effopc}} = 0.110 \times (82.7 - 426 \times (w/c) + 568 \times (w/c)^2 + 4.26 \times C^{-6})\ \mathrm{cm^2/year}. \quad (7)$$

$$D_{\mathrm{effms}} = 2/(7.01 + 0.2 \times ms) \times (1.5\text{-}1.25 \times \log(\mathrm{ms}+1) + ms^2/1500)\ \mathrm{cm^2/year}. \quad (12)$$

Using these equations and constants it is now possible to calculate the time to the initiation of corrosion of the reinforcing steel for varying mix designs and concrete covers. The results are plotted in Figures 9–12.

PROPOSED FIGURES FOR ASSESSING THE MIX DESIGN AND COVER REQUIRED, TO PROVIDE DURABLE CONCRETE OVER A SPECIFIED DESIGN LIFE, IN SAUDI ARABIA

The following results, in terms of years to the initiation of corrosion, were obtained using the above mentioned models for extreme, severe, moderate, and low chloride exposures, for various mix designs at various covers.

Concrete Mixes and Cover Requirements for Extreme Chloride Exposure of Sea Water in Hot Climates

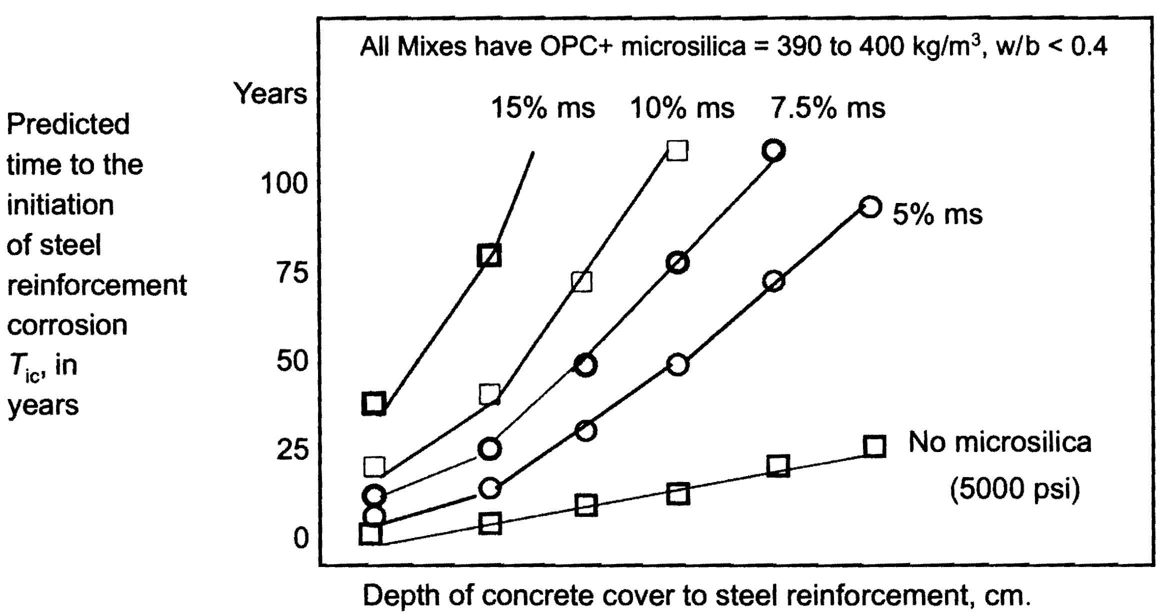

Figure 9. Predicted Time to the Initiation of Corrosion of Steel Reinforcement, at Various Covers in Concrete Under Extreme Chloride Exposure in Hot Climates, Using Equations (6), (7), (12), and (17).

Concrete Mixes and Cover Requirements for Moderate Chloride Exposure

Any structure in contact with raw water with TDS from 4 000 to 5 000 ppm in hot climates is subject to moderate exposure.

Concrete Mixes and Cover Requirements for Severe Chloride Exposure of Sea Water in Cool Climates

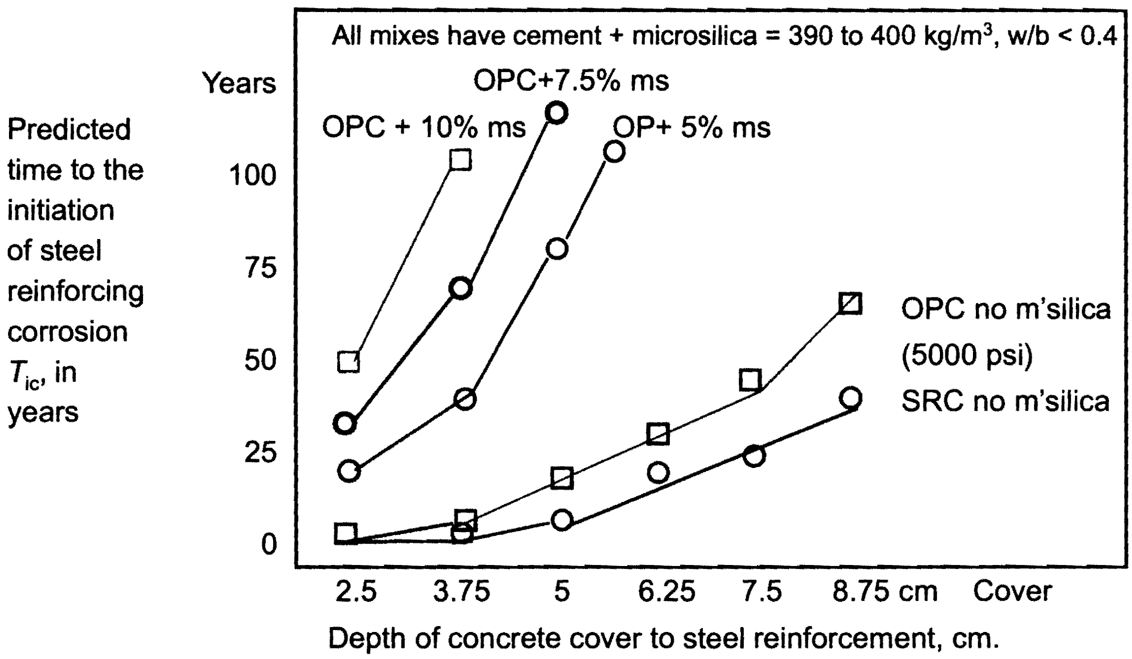

Figure 10. Predicted Time to the Initiation of Corrosion of Steel Reinforcement, at Various Covers in Concrete Subject to Severe Chloride Exposure in Temperate and Cool Climates, Using Equations (6), (7), (12), and (17).

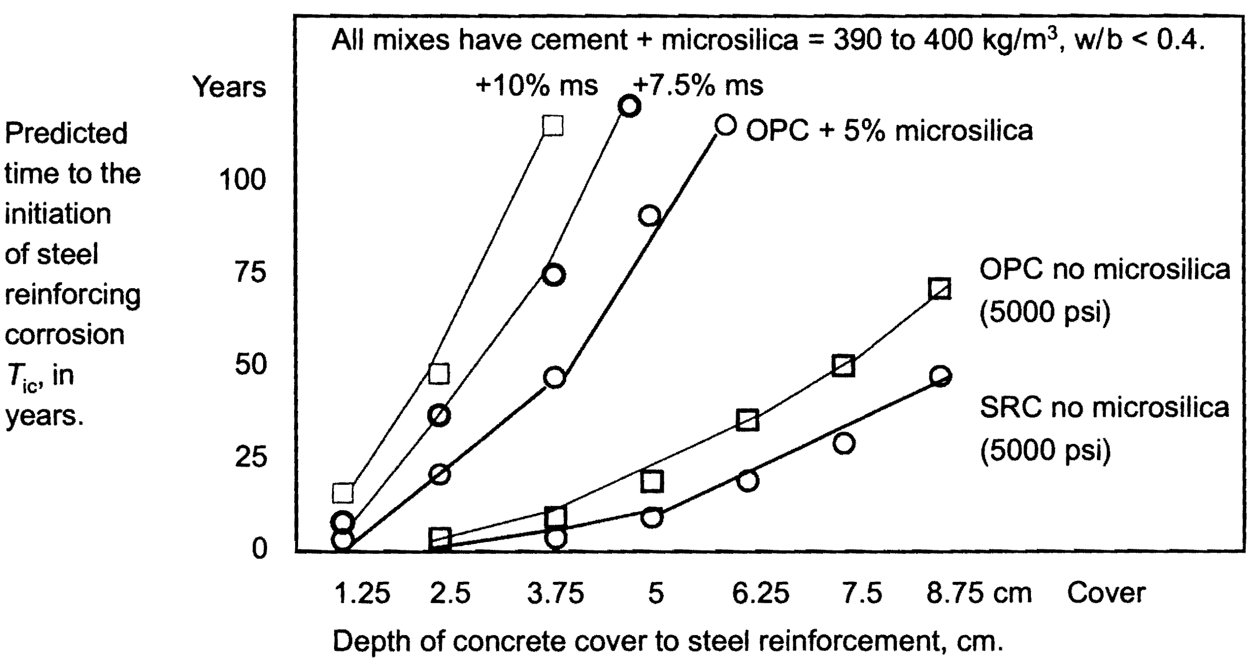

Figure 11. Predicted Time to the Initiation of Corrosion of Steel Reinforcement, at Various Covers in Concrete, Under Moderate Chloride Exposure in Hot Climates, Using Equations (6), (7), (12), and (17).

Concrete Mixes and Cover Requirements for Low Exposure

Low exposure refers to any structures in the ground within 3 m of the water table, up to 3 m above the ground level in hot climates.

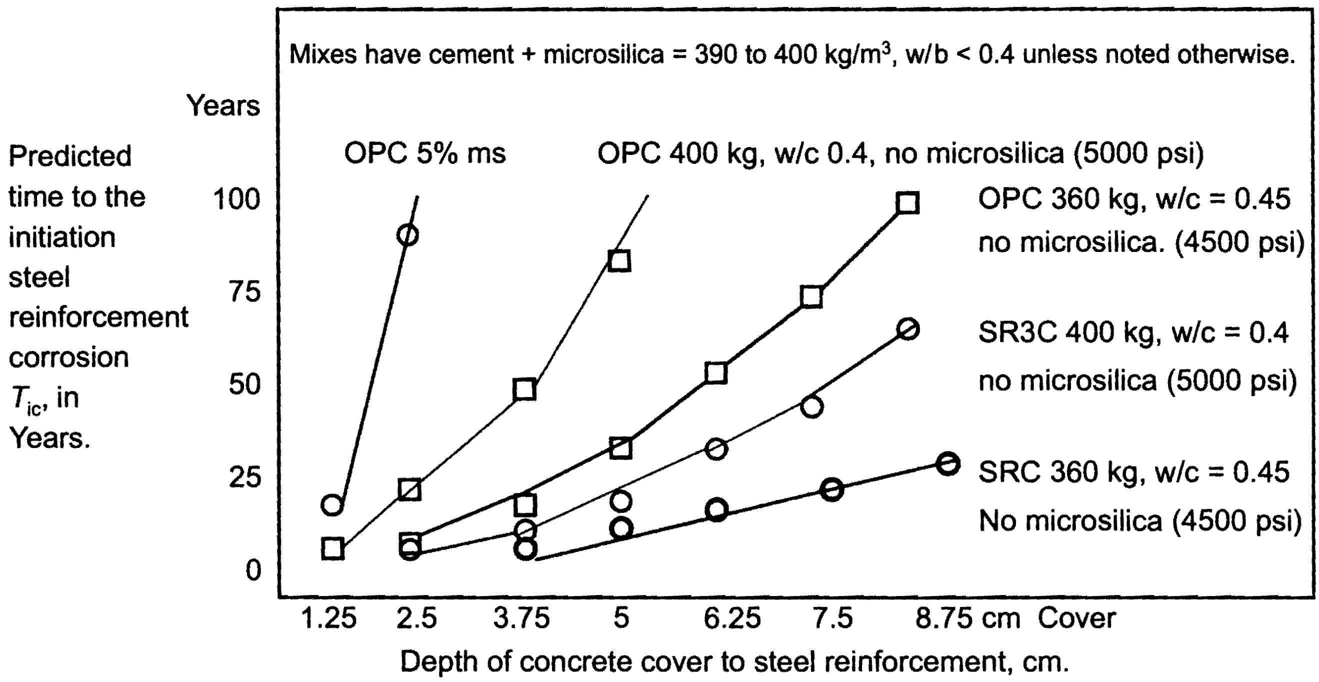

Figure 12. Predicted Time to the Initiation of Corrosion of Steel Reinforcement, at Various Covers in Concrete Under Low Chloride Exposure in Hot Climates, Using Equations (6), (7), (12), and (17).

EXPERIMENTAL PROGRAM

The experimental program involved measuring chloride levels in; the environment where structures are to be built; in the concrete constituent materials and in the hardened concrete. The testing was carried out, by Independent testing laboratories, in Al Khobar and Dammam. The following test methods were used; ASTM D-512 to measure chlorides in water; BS812 to measure chlorides in aggregates; and BS1881, Part 124, 1988 to measure chlorides in hardened concrete. The chloride contents of the groundwater and soil were measured using ASTM D-512 for testing the ground water and BS812, Part 117 for testing the soils. The Rapid Chloride Permeability or Coulomb test, ASTM C1202-91, was used to measure the Coulomb values of hardened concrete. A selection of mix designs, tested by the author, are shown in Table 16.

TEST RESULTS AND DISCUSSION

Concrete Ingredients

The chloride levels measured in the concrete ingredients are shown in Table 17. Quality conscious ready mix concrete suppliers, in Saudi Arabia, have no difficulty in maintaining ARAMCO and SCECO material requirements. That is, keeping Cl < 0.1% by weight of cement or 0.016% by weight of concrete.

Provided dry wind blown dune sand is used, it will usually have chloride levels within specified limits. However if the sand is taken from below the groundwater table level, then the chloride levels may become excessive and fail to meet the specified limits. Coarse aggregates, especially the smaller nominal grading sizes of less than 12 mm must be washed to remove dust and chemicals like chlorides and sulfates.

Table 16. Microsilica Mix Designs Tested, by the Author, to Ascertain the Benefit of Microsilica on Concrete.

Mix	Cement kg/m³	Microsilica kg/m³	Aggregate kg/m³	Sand kg/m³	Water litre/m³	MS % 100ms/(c+ms)	Water/Binder W/(c+ms)
1	350opc	Nil, 25 & 35	1313 AH	505	140	0.0, 6.7 & 9.1	0.38
2	400src&opc	Nil	1115 AH	620	160	0.0	0.40
3	370opc	25 & 33	1070 AH	600	153	6.3 & 8.2	0.38
4	370opc	15, 25 & 30	1110 RR	600	154	3.9, 6.3 & 7.5	0.40
5	370opc	27 & 30	1110 RR	630	158	6.8 & 7.5	0.40
6	370opc	40 & 45	1150 RR	620	164	9.8 & 10.8	0.40
7	370opc	20, 30 & 40	1070 AH	745	155	5.1, 7.5 & 9.8	0.40
8	370opc	20, 30 & 40	1180 RR	605	151	5.1, 7.5 & 9.8	0.39
9	420opc	Nil & 30	1173 RR	786	138	0.0 & 6.7	0.33
10	363opc	20, 27 & 39	1150 RR	683	160	5.1, 6.9 & 10	0.41

Water reducing, retarding and super plasticising admixtures complying with BS 5075 and ASTM C494 were used to keep w/c ratios <0.4 in all the above trials.

AH = Abu Hadriyah, RR = Riyadh Road coarse aggregate sources.

Environmental Chloride Levels

Soils and water were tested at several sites in Eastern Province and the results obtained are shown in Table 18.

The results of the soil tests show that a large variation of chloride levels occurs in a relatively small geographical area. Hence, it is important to measure chloride levels in soils, prior to placing concrete in the ground, especially in sabkha salt flats where high levels of chlorides and sulfates are expected.

Rapid Chloride Permeability Test Results for Different Microsilica Mix Designs

Over the past 5 years, using the mix designs shown in Table 16, the results shown in Figure 13, have been obtained in the Dammam–Al Khobar area of Saudi Arabia. The predictions from Equation (13) show good agreement with the upper limit of the measured coulomb values.

Unfortunately, chloride profiles for microsilica concretes are unavailable at this time to verify insitu diffusion coefficients.

Table 17. Typical Concrete Material Acid Soluble Chloride Levels.

Material	proportion % mix by wt.	Max Specified Chloride % concrete	Measured Chloride Cl as % concrete
Coarse Aggregate	48	0.011%	0.01 to 0.02 % Washed with clean water
Sand	27	0.05%	0.017% Unwashed but dry wind blown sand
Cement	16		
Microsilica	1		0.06
Water	6	0.03%	0.009%
Air	2	Nil	
Concrete	100	0.016% concrete or 0.10% by cement	0.008 to 0.02%

Table 18. Environmental Chloride Levels Measured in Eastern Province of Saudi Arabia.

Location	Chloride concentration	Exposure
Gulf sea water at Aziziyah	2.90% (TDS = 48,021)	Extreme
Sabkha around half moon Bay	1.2 –1.8%	Severe
Aziziyah soils report on coast	0.30%	Moderate
Aziziyah soils 100m from coast	0.14%	Moderate
Well water, Khobar	0.15% (TDS = 3779ppm)	Moderate
Dammam 2nd Ind. City.	0.066%	Low
Dammam warehouse	0.057%	Low
Khobar Golden Belt	0.026%	Low
Khobar City, Pr. Hamoud Street	0.010%	Low

Design Life Considerations

What is a reasonable design life and time for corrosion initiation? That depends on the clients' intended use of the structure and the consequences of early deterioration in terms of loss of life, environmental damage, and the cost of replacing the structure. Once corrosion starts, it is difficult and sometimes impossible to stop. The corrosion will only accelerate once spalling and cracking occur, leading to the eventual failure of the structure. The proposed design guidelines given in Figures 9–12 use the time to the initiation of rebar corrosion to give a conservative design life. The actual structural failure will take many more years to occur.

The design life of a structure could be defined as the utilization period specified by the owner or consultant, with respect to structural safety, serviceability, and durability. That is the period for which the structure is to be used for its intended purpose with anticipated minimum maintenance but without major repair being necessary. Recommendations, for appropriate design lives of structures, can be made by subdividing the structures according to their use and importance, as shown in Table 19.

Sharp [28] gives further guidance on Usage and Importance.

Usage

Usage can be categorized as follows:

General Infrastructure is classified as general works used by the general public, including public buildings.

Buildings are all structures intended for residential and commercial use.

Industrial Infrastructure is classified as works in the service of a particular industrial installation or associated with the use of transitory natural deposits of resources like mines or oil extraction facilities.

Importance

Importance or security levels given in Table 19 can be categorized as follows:

Level 1. Works of local interest with small risk of loss of human life or environmental damage in the case of failure. Including work in minor ports, small craft harbors, pavements, small towns, commercial installations, and suburban residential buildings.

Table 19. Design Life Recommendations for Structures in Years [28].

Use and Importance	Level 1	Level 2	Level 3
General Infrastructure [28]	25	50	100
Buildings	20	40	75
Industrial Infrastructure [28]	15	25	50

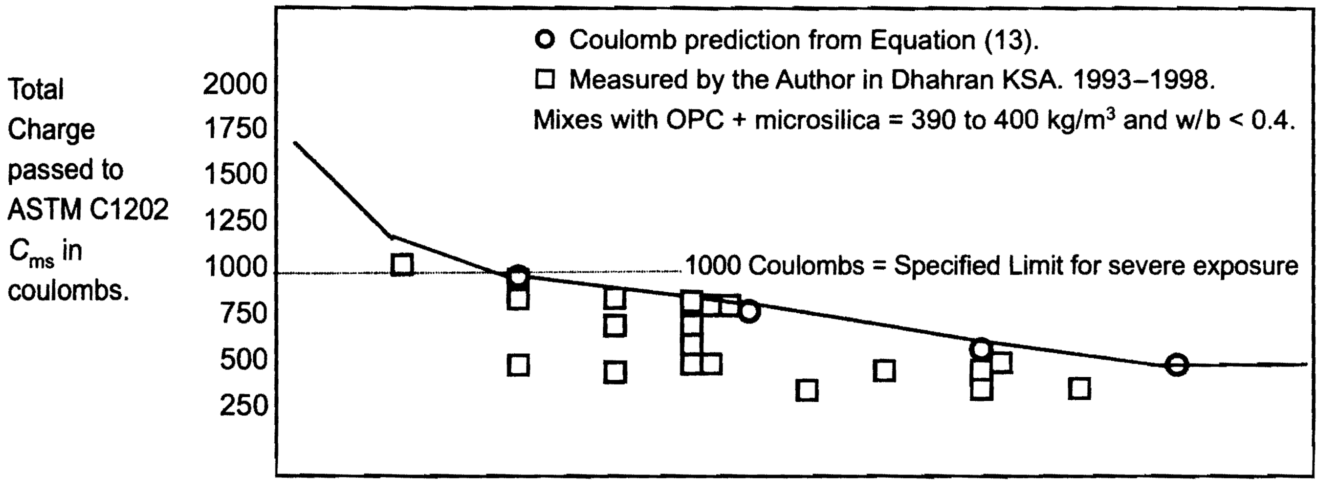

Figure 13. Comparison Between Rapid Chloride Ion Permeability Test Results in Coulombs, Predicted Using Equation 13, and the Actual Measured Coulomb Values, at Various Microsilica Replacement Levels.

Level 2. Works of general interest with moderate risk of loss of human life or environmental damage in case of failure. Including work in major ports, out-falls of large cities and medium rise buildings in cities, road bridges, and retaining walls on interstate highways.

Level 3. Works and installations for protection against inundations or international interest, with an elevated risk of human loss or environmental damage in the case of failure. Including defense of urban or industrial centers, desalination plants, power stations, dams, hospitals, monumental, and high rise buildings, and international highway bridges and tunnels.

However, in the Gulf where good concrete materials are difficult to find it is virtually impossible to guarantee a structure for more than 50 years, as very few concrete structures are that old. We may conclude that the 50 to 100 year design lives of Level 3 structures will only be achieved by using pozzolanic materials in the foundations and parts of those structures that are subject to chloride exposure.

RECOMMENDATIONS FOR DESIGNING DURABLE CONCRETE

To ensure that concrete remains in an acceptable condition, over the anticipated working life of a structure, the following factors should be considered during the design, construction, and maintenance stages of a project.

Design Stage

An adequate design life should be specified in consultation with the client.

Specify an adequate strength based on loading data, then limit deflection and thermal cracking.

Do environmental testing and select an appropriate exposure classification as shown in Table 18.

Choose an appropriate mix from Figures 9–12 depending on exposure and specify adequate cover to the reinforcement.

All concrete constituent materials and the hardened concrete should be tested for chlorides.

Specify an adequate testing programme during the construction phase.

The concrete mix design must allow for the difficulties in placing of concrete in hot climates. To prevent honeycombing high slumps must be used. Typically, 100 to 125 mm slump can be obtained by using superplasticizing and retarding admixtures complying to ASTM C494.

Construction Stage

Monitor concrete supply with Coulomb tests at the same frequency as strength tests.

Conduct strict site inspections.prior to casting to maintain cover requirements.

Use impermeable cover blocks to hold the reinforcing steel, made with similar cementatious blend to the structural concrete or use moulded plastic spacers to ensure that the rebar cage does not move during concreting.

Check cover with a cover meter after concrete placing.

Control concrete temperatures by using ice to keep the temperature below that recommended in ACI 305, the guide to placing concrete in hot weather [5], to prevent cracking during initial setting on the fresh concrete.

Finish the concrete surface paying particular attention to closing any plastic shrinkage or settlement cracks by hand floating the surface as the initial set occurs.

Curing is also very important and must be maintained for at least 7 days by ponding with sweet water on horizontal surfaces [11]. Curing compounds should be used on other surfaces, due to the difficulty of keeping them wet, in air at 45•C in the hot summer months.

Construction joints should be wire brushed to expose the aggregates to ensure a good interlock to the next pour.

Maintenance Stage

Protect concrete from unnecessary salty or raw water wash down.

Record any deterioration during annual inspections and repair deteriorated concrete as soon as it becomes visible, and apply surface coatings to limit future deterioration.

Take cores every 5 years to monitor chloride profiles and the status of reinforcement corrosion, if any.

CONCLUSION

While the above research is quite comprehensive, I must caution readers with regard to using Figures 9–12 for design purposes. Firstly, they are undoubtedly more reliable than existing codes of practice but the results of insitu testing of chloride profiles in existing structures after 25 to 75 years of extreme exposure is required to confirm the validity of the models proposed.

Secondly, there are areas in the Eastern Province where high concentrations of both sulfates and chlorides exist simultaneously. Further research into the use of microsilica in concrete exposed to combined sulfate and chloride attack is required. We do know that the past widespread use of sulfate resisting cement, has proved inadequate in resisting chloride attack under combined exposure conditions, and the practise of specifying microsilica enriched OPC concrete is increasing in the Eastern Province of Saudi Arabia.

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