

An Investigation of corrosion of steel reinforcement in concrete in the Eastern Province of Saudi Arabia

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Civil Engineering

Abstract

Field and laboratory investigations have been carried out to study the corrosion of steel reinforcement in concrete in the Eastern Province of Saudi Arabia. Mechanisms of steel corrosion and concrete spalling have been discussed in the light of recent research. More than sixty cores obtained from ten field structures have been analyzed using various chemical, destructive and non-destructive techniques to study the effect of chlorides, concrete quality, cover to reinforcement and the local concrete making materials on the corrosion of reinforcement and the associated spalling of concrete. Supportive controlled laboratory investigations on concrete slab samples have also been carried out to develop data on the effect of the aforesaid parameters on the corrosion of rebars. Data from condition surveys of structures and inspection surveys of construction sites and quarries have been used to evaluate the construction practices prevailing in the Eastern Province.

Recommendations have been made regarding the threshold values of chlorides, minimum cover, evaluation of local materials, type of cement, mix design and construction practices for obtaining durable concrete construction in the region.

An Investigation of Corrosion of Steel Reinforcement in Concrete in the Eastern Province of Saudi Arabia

by

Ahmad Saad Al-Gahtani

A Thesis Presented to the

FACULTY OF THE COLLEGE OF GRADUATE STUDIES

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DHAHRAN, SAUDI ARABIA

In Partial Fulfillment of the
Requirements for the Degree of

MASTER OF SCIENCE

In

CIVIL ENGINEERING

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AN INVESTIGATION OF CORROSION OF STEEL
REINFORCEMENT IN CONCRETE IN THE
EASTERN PROVINCE OF SAUDI ARABIA

by

AHMAD SAAD AL-GAHTANI

THESIS

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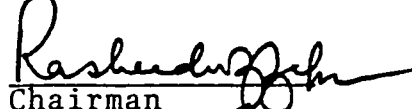
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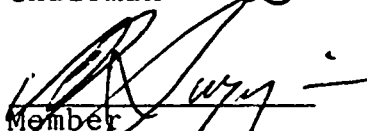


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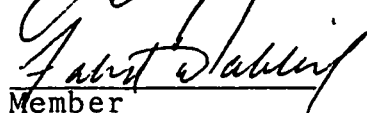
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ABSTRACT

Field and laboratory investigations have been carried out to study the corrosion of steel reinforcement in concrete in the Eastern Province of Saudi Arabia. Mechanisms of steel corrosion and concrete spalling have been discussed in the light of recent research. More than sixty cores obtained from ten field structures have been analyzed using various chemical, destructive and non-destructive techniques to study the effect of chlorides, concrete quality, cover to reinforcement and the local concrete making materials on the corrosion of reinforcement and the associated spalling of concrete. Supportive controlled laboratory investigations on concrete slab samples have also been carried out to develop data on the effect of the aforesaid parameters on the corrosion of rebars. Data from condition surveys of structures and inspection surveys of construction sites and quarries has been used to evaluate the prevailing construction practices in the Eastern Province.

Recommendations have been made regarding the threshold values of chlorides, minimum cover, evaluation of local materials, type of cement, mix design and construction practices for obtaining durable concrete construction in the region.

Chapter 1

INTRODUCTION

1.1 CONCRETE DETERIORATION PROBLEM

The boom in the construction activity in the Gulf States for the last two decades has brought in its wake the concrete frame and the concrete block as the most popular form of construction in this region. An unprecedented demand for concrete buildings and structures for a variety of functions has built up immense pressures on the local construction industry. However, the local construction industry is beset by an inadequate infrastructure, shortages of suitable materials, equipment, skilled manpower and inadequate specifications and construction practices. The outcome of the resulting paradox is seen in structures which are showing an alarming degree of deterioration within a short span of 10 to 15 years. The deterioration is accentuated by the environmental conditions which are characterized by high temperature-humidity regimes combined with severe ground and ambient salinity.

The root of the concrete deterioration problem lies in the cumulative interaction of (i) defective construction practices, (ii) presence of chloride and sulphate salts in the ground, the water, the atmosphere and the aggregates, (iii) adverse climatic environment, (iv) geomorphic conditions resulting in reactive and marginal aggregate material, and (v) general lack of appreciation of the seriousness of the problem.

Along the coastal flats, specially along the eastern seaboard of the Arabian peninsula, where much of the development is founded, these factors combine to produce particularly difficult conditions. The ground water table is relatively high in this area and the evaporation is intense. The capillary rise of moisture and frequent flooding followed by evaporation leaves a heavy crust of salt in the upper few feet of soil profiles. This leaves the ground, the water, the atmosphere and the aggregates heavily contaminated with chlorides and sulphate salts. The concrete construction in coastal habitations is also continually exposed to frequent and persistent winds charged with sea water and sea salts which permeate concrete to cause steel corrosion and concrete spalling. High ambient temperatures accelerate both the chemical attack and physical deterioration.

The main causal factors can be identified as : rebar corrosion, shrinkage and early thermal movements, sulphate attack, salt weathering, and alkali-silica and alkali-carbonate reactions. The major types of deterioration caused by these factors are : reflective cracking over reinforcement shortly after concrete placement due to shrinkage and early thermal movements (Plate 1.1), expansive cracking of concrete with loss of strength and cohesion due to sulphate attack and salt weathering (Plate 1.2), loss of steel metal and severe associated concrete spalling due to rebar corrosion (Plate 1.3) and map cracking and exudations due to alkali-silica reactions (Plate 1.4). Minor forms of deterioration such as popouts (Plate 1.5), pitting (Plate 1.6), hair checking (Plate 1.7), crazing (Plate 1.8) and corrosion stains have all been observed (Plate 1.9).

Condition surveys were carried out at UPM on 42 concrete framed structures located in Alkhobar, Dhahran and Dammam habitations of the Gulf coast and the results of these surveys have been published¹. The results show an alarming conditions of structures constructed 15-20 years hence. 38 of these 42 structures were constructed during the years 1960-64 and remaining during and after 1974. Although the general condition of each structure was

recorded as comprehensively as possible, an overwhelming majority of the detailed deterioration recordings were made only on concrete exposed to the ambient environment. This enabled to hold at least two variables - age and in-service exposure - reasonably constant in order to study the effect of other variables on concrete deterioration. The detailed methodology of the surveys is given elsewhere. Figure 1.1 classifies on a six point scale the general condition rating of 168 study areas from these 42 structures. The figure shows that 48% of the observed points group in the classifications 5 and 4 which correspond to a very unsatisfactory condition range of the rating and about 19% in classification 3 which is also far from satisfactory. Only 26% manifest slight or no deterioration. Illustrations of classifications 5 and 4 are shown in Plate 1.10 and Plate 1.11 respectively.

The commonly occurring distress conditions observed during the condition surveys were : reinforcement corrosion, concrete spalling, cracking and surface mortar deterioration.

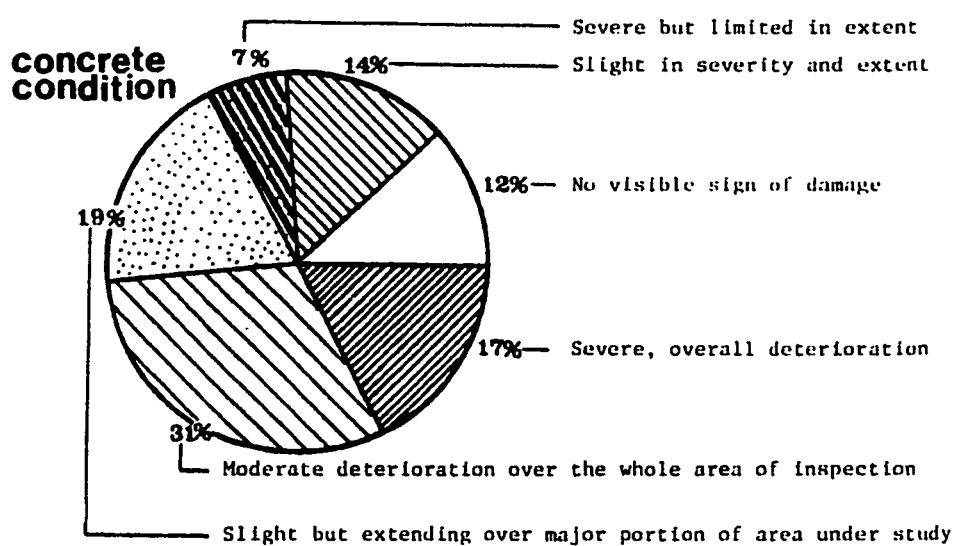


Fig. 1.1 : Classification of Condition of 168 Observations

1.2 REBAR CORROSION NUMBER ONE DURABILITY PROBLEM IN THE EASTERN PROVINCE OF SAUDI ARABIA

Table 1.1 summarizes the results of the aforesaid condition surveys related to the prevalence format of the observed deterioration type on 168 observations. It shows that the main causes of concrete deterioration are attributable in decreasing order of importance to : corrosion of reinforcement, sulphate attack and salt weathering, early age cracking due to thermal gradients and cracking due to aggregate-cement reactivity.

As a substantial portion of the author's current research dealt with field studies, he had to look for structures of about 22-27 years of age for concrete sampling and investigations from the Alkhobar-Dhahran-Dammam complex of habitations. In this process observations and deterioration distribution were recorded on more than twenty concrete building structures. The author's findings in relation to the distribution of deterioration confirms the results of the above mentioned condition surveys by indicating that corrosion of reinforcement associated with concrete cracking and spalling by far outweighs any other of the several forms of deterioration observed. The author believes that Alkhobar-Dhahran-Dammam complex as a small

TABLE 1.1
Distribution of Deterioration

Total Observations : 168

Type of Deterioration	Possible Cause	Incidence		Remarks
		Numbers	Percent	
(i) Steel Corrosion	Chloride attack accentuated by poor quality concrete	76	45.24	
(ii) Spalling of concrete	Steel corrosion	68	40	All observation points are common to (i)
(iii) Surface mortar deterioration	Salt weathering, sulphate attack, wear	72	43	16 observations common to (i) and (ii)
(iv) Pattern Cracking	Aggregate-cement reaction	4	2.4	

geographical unit is typically characteristic of the prevailing environmental and other affecting parameters along the eastern seaboard of the Arabian peninsula. And if these condition studies on about 65 concrete structures in this region are representative of the deterioration pattern and distribution, then it is indicated that corrosion of reinforcement is the number one concrete durability and in-service performance problem in the Eastern Province and possibly also in the other Gulf States located on the Eastern coast sharing common features of climate, ground conditions and associated construction problems.

1.3 CORROSION AS A DOMINANT DURABILITY PROBLEM ELSEWHERE

Generally speaking, coastal structures everywhere in the world will be subjected to noticeable rebar corrosion due to their exposure to marine atmosphere characterised by wind, spray and rain charged with sea salts. A large number of feedback reports^{2,3,4} confirm this. Characteristic case histories of such deterioration are reported by Halstead and Woodworth² for reinforced concrete structures located near the sea on the South Coast of Natal, South Africa. The structures were exposed to the maritime atmosphere and

to the hot, humid, sub-tropical climate experienced in this area. Halstead et al. report in the several cases the complete conversion of steel to the end products of corrosion due to the presence of sea salts resulting in a complete disruption of many structures. Hiroshi Seki³ reports the corrosion deterioration of concrete coastal structures in Japan. Test data from field samples showed salt content in concrete upto 0.5% causing the corrosion of the rebars. Griffin⁴ has reported the deterioration of reinforced concrete structures in the South Pacific by wind bearing seawater and high atmospheric temperatures that prevail in the tropical marine environment. Investigations carried out at the U.S. Naval Civil Engineering Laboratory, Port Hueme, California indicate reinforcement corrosion due to salts the major cause of concrete deterioration. The findings show that the critical salt content is inducted by wind bearing sea water that enters the concrete.

It is relevant to refer here to the results of a survey carried out by the British Building Research Station on the durability of concrete in the U.K. through a comprehensive questionnaire sent to a randomly selected sample of members of the Concrete Society⁵. Results of five

comparatively more relevant questions are listed in Table 1.2. Although the magnitude of the problem is modest in that only 1.4% of the total value of the concrete used in structures would be accounted for by repairs and replacements, 72% of replies indicated that there was a problem of concrete durability in the United Kingdom. The results of the survey indicate that corrosion of reinforcement is the number one offender. The largest group considered the retention of design characteristics for the required time to be a prerequisite of durable concrete and that the relevant period to retain the characteristics was 50 years. This criterion of concrete durability is in line with that proposed by Nurse⁶ as : "the length of time for which the material will continue to serve the purpose for which it was designed while exposed in a specified environment".

The largest group considers the *poor compaction* and *inadequate cover* to reinforcement the principal contributory factors to poor concrete durability.

Figg^{7,8} reporting on the basis of British experience states: "The rusting of reinforcement in concrete is the single most important cause of poor durability of structural concrete. The costs of corrosion in the

TABLE 1.2

Results of British Building Research Station Questionnaire
on the Durability of Concrete in the U.K.

<i>What do you understand by durable concrete ?</i>	<i>%</i>	<i>What should define concrete durability</i>	<i>%</i>
1. High strength material	6	1. No adverse change in appearance	23
2. Long life	17	2. No reduction below designed strength for service life of structure	
3. Retain its appearance	20	3. Unchanged in strength and appearance for 10 years	2
4. Unaffected by chemicals		50 years	32
5. Retains its design characteristic for the required time	31	100 years	12
		4. Others: please state	8
<i>In your experience, is there a problem with concrete durability in the UK?</i>	<i>%</i>	<i>What do you consider are the principal causes of poor concrete durability in the UK</i>	<i>%</i>
Yes	72	1. Sulphate attack	8
No	28	2. Corrosion of reinforcement	24
<i>What do you consider as principal contributory factors to poor concrete durability?</i>	<i>%</i>	3. Aggregate instability	6
1. Incorrect cement	2	4. Cement-aggregate reaction	5
2. Incorrect aggregate	8	5. Shrinkage	19
3. Unsuitable additive	5	6. Frost	13
4. Poor mix design	26	7. Leaching	4
5. Poor compaction	30	8. Acid attack	5
6. Inadequate cover to reinforcement	30	9. Organic growths	4
		Impact	7
		Overloading	2
		Abrasion	4

construction industry are of the order of £700 million per year, and of this a significant proportion is due to corrosion of steel reinforcement⁹.

The corrosion of reinforcement and the subsequent spalling of concrete as a major concrete performance problem in the United States was dramatically illustrated in recent years when it achieved national prominence as the "Bridge Deck Problem"^{10,11,12,13} with its consequent heavy impact on the maintenance and construction budgets of the state highway agencies. The genesis of the problem is as follows : With the placement of fresh concrete subsidence, shrinkage and thermal stresses cause small tension cracks to occur over the reinforcing bars. These provide paths of ingress for water and deicing salts (sodium and calcium chlorides) to the reinforcement (Figure 1.2). Subsequent corrosion of the reinforcing bars generates forces of sufficient magnitude to cause the spalling or delamination of the concrete (Plate 1.12).

The bridge deck problem has been precipitated in incidence and severity during the last decade following the adoption by various highway agencies of the "bare pavement" policy, stressing safe, all weather driving conditions. This resulted in a phenomenal increase in the application

of chloride deicing chemicals to the bridge decks in the United States (Figure 1.3), from less than half a million tons in 1947 to 12 million tons in 1976. In 1973 it was reported¹⁴ that FHA estimated the annual cost for bridge deck repairs in the United States at \$ 70 million. By 1975, that figure had increased to \$ 200 million per year. It is indicated that these figures are deceptively low as they account for repairs to only part of the total damage. A yet more disturbing feature is the early age at which deterioration begins to appear. A Pennsylvania State University survey¹⁵ shows that nearly one fourth of the 249 bridges surveyed showed reinforcement corrosion and concrete spalling within four years of their construction and 71% of the decks exhibited transverse cracks which are forerunners to the impending concrete spalling and delamination.

Another area in which corrosion of steel poses a serious threat is prestressed concrete construction. Corrosion of prestressed reinforcement is considerably more serious than that of normal reinforcement because of much smaller size of the wire used and also because of the more significant role prestressed steel plays in the stability of structure. Szilard¹⁶ has reported the findings

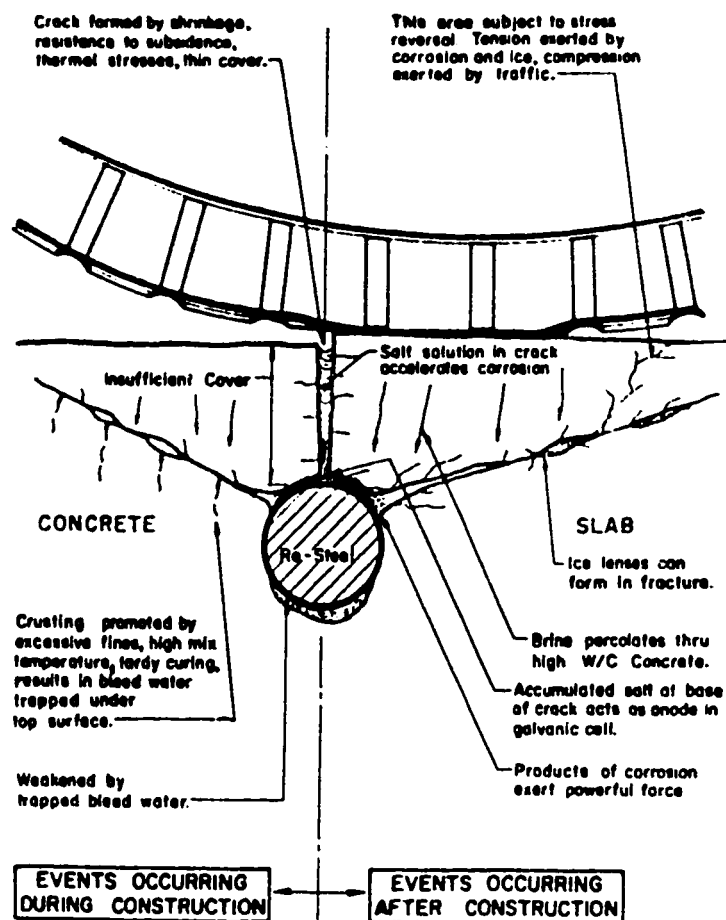


Fig. 1.2 : Corrosion of Bridge Deck Caused by Subsidence Cracks and Deicing Salts

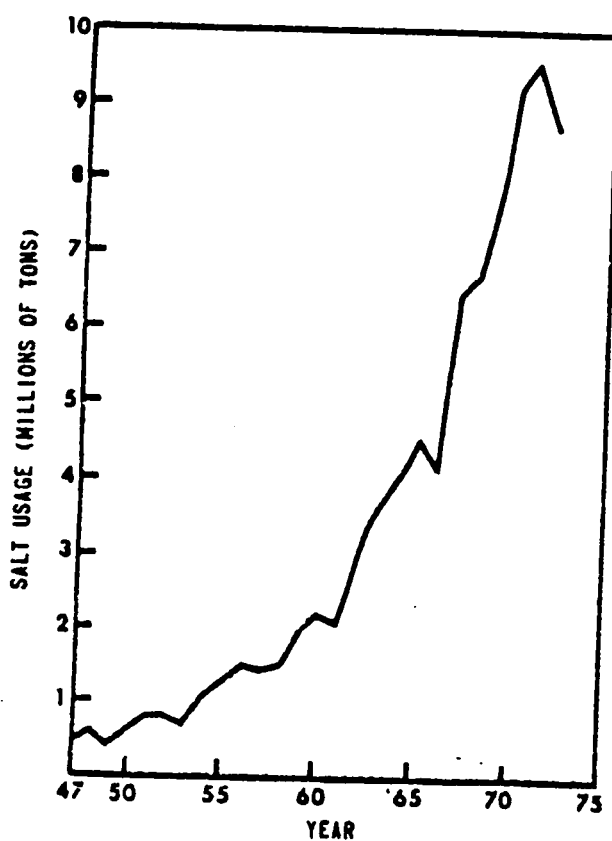


Fig. 1.3 : Increased Use of De-icing Salts Causing Bridge Deck Problem

of a survey on the durability of prestressed concrete structures in the United States, Canada, and Pacific and Far Eastern countries. Monford and Verbeek¹⁷ have reviewed the reported failures of prestressed concrete structures due to corrosion of prestressed steel. Both the reports indicate on the whole a satisfactory in-service performance of prestressed concrete. However, these reports do cite several failures which after investigations were attributed to the corrosion of the prestressing cables. The most serious case¹⁸ of corrosion damage of the tendons in prestressed concrete bridges was reported by the State of Washington. The damage occurred in a floating bridge over seawater. The failure of two prestressed concrete tanks¹⁹ is reported in the USA. Hill²⁰ has reported the corrosion in prestressing cables around a tank in England and Bouvy²¹ has cited the corrosion failure of the wires in a circular tank in Algeria. Corrosion failures²² in the wires of a dome of the sprinkling filter unit at the Regina, Saskatchewan have been attributed to the calcium chloride used in the shotcrete mortar. Another notable failure by corrosion of prestressed steel on 36-in diameter concrete pipes at Regina, Saskatchewan, has been thoroughly studied and reported²³. The failure included a number of complicated

factors the dominant being the use of calcium chloride.

A close examination of the case histories reported in the foregoing discussion reveals one common denominator : the aggressive salt environment characterized by the presence of chloride ions most of the time. Another common feature of the reported field deteriorations of this type is some kind of weakness in concrete which acted as a forerunner and feeder for rebar corrosion by facilitating the ingress of salt-laden moisture.

1.4 PARAMETERS AFFECTING REBAR CORROSION IN CONCRETE

Before discussing the parameters affecting corrosion of reinforcement in concrete it is necessary to briefly recount the mechanism of corrosion. This topic forms the subject matter of chapter 2 where it is discussed in greater detail. At this stage only the salient points are recalled.

In the environment of the fresh concrete, amongst the hydration products of Portland cement, are present the alkaline calcium and other hydroxides having $\text{pH} > 12$; these alkaline hydroxides form a thin insoluble passivating or protective film of gamma ferric oxide (Fe_2O_3) on the steel bars. Corrosion of reinforcement will not take place as

long as this passivating environment is intact. However, it is established that chloride ion is a specific and unique destroyer of this protective film. If sufficient chloride ions are available at or near the steel surface the protective environment provided by the concrete alkalinity breaks and the first condition for corrosion becomes obtainable. The second prerequisite for the occurrence of corrosion is the presence of an electrical potential which is usually provided by either differential aeration or differential ion concentration or differential surface properties. The presence of an electrical potential results in setting up an electrochemical cell of the type shown in Figure 1.4. The basic chemical reactions in the corrosion cell are shown in the flow diagram.

Thus from the above discussion and as shown in the chemical reaction diagram in the normal concrete situation where pH is greater than 11.5, three things are essential for corrosion to commence and proceed at the steel-paste interface :

- * Chloride
- * Oxygen
- * Water

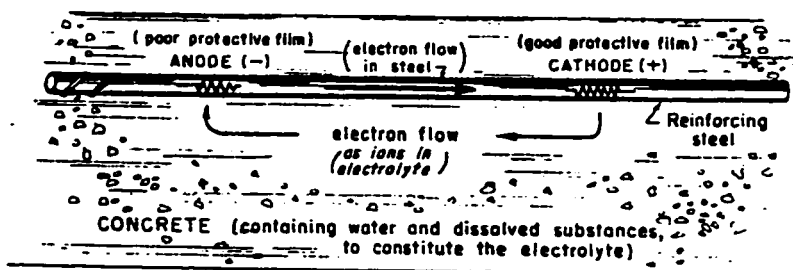
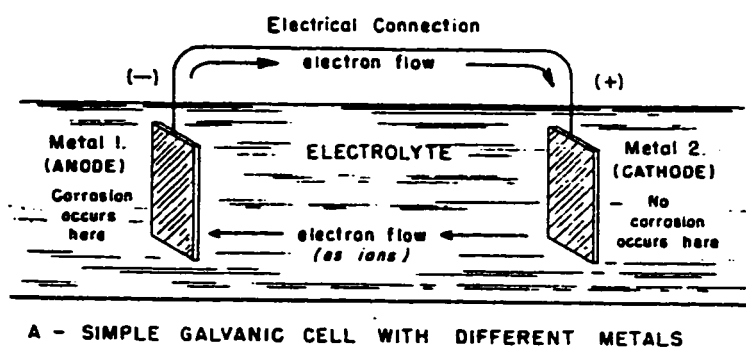


Fig. 1.4 : B - SIMPLE GALVANIC CELL IN CONCRETE

—Galvanic corrosion cells.

For no corrosion to occur, obviously, these should be kept out of concrete.

The foregoing discussion entails that the most significant active parameters affecting rebar corrosion are :

- (i) Presence of *chloride ion* in the concrete as a destroyer of the protective film on steel.
- (ii) Presence of *moisture* in concrete which acts as a carrier of salt and oxygen and provides the hydroxyl ion for rust formation. With its dissolved salt, moisture also provides an electrolyte of low electrical resistivity, thus permitting corrosion currents to flow readily through concrete.
- (iii) *Oxygen* permeation into concrete to provide an essential component of the chemical reaction resulting in the formation of rust.
- (iv) *Concrete mix variables* which affect the alkalinity (pH), the quantity of uptake and transmission of water, oxygen and chlorides. These factors are established to directly and significantly affect the magnitude and speed of corrosion rate of the embedded steel. The concrete mixture variables that affect the corrosion process are : *w/c ratio, aggregates, cement*

and *admixtures*.

- (v) *Construction variables* which influence the conditions that facilitate the movement of water, chlorides and oxygen to the location of the embedded steel. Some of these construction variables like *cover to reinforcing steel*, *consolidation* and *curing* of concrete are known to have profound effect on the corrosion process.

1.5 THE UNIQUE GULF SITUATION AS PROMOTER OF STEEL CORROSION

Almost all the forelisted parameters are active in an adversely intensive and cumulative fashion in the Gulf States. It seems that the single most damaging factor affecting steel corrosion in this region is the abundant presence of the chloride salts. In the Arabian peninsula sulphates and chlorides occur at several horizons in the geological formations. Numerous salt domes in the Gulf, around its shores and in the Oman desert constitute "built-in" sources of salt contamination especially when they are located in the zones of groundwater circulations.²⁴ Calcium sulphate ("gypcrust" or "gypcrete") and sodium chloride ("salcrust") are among the most widespread "duricrusts" which characterize most of the regional outcrop sequences.

A yet another salt source are the numerous Sabkhas which constitute natural evaporating pans saturated with brines generating chloride, sulphate and carbonate minerals in their surface crusts. These salts are wind blown and have been found to heavily contaminate dune sands upto distances of 40 kms.

As a result of these geomorphic conditions chloride and sulphate salts pollute the ground, water, the atmosphere and the aggregates. Top layers of some of the limestone outcrops, which are a major source of aggregate in the Gulf areas, are rich in salts and hence the aggregate product usually has a high chloride content. The commonly used mix water usually has 0.14% of chloride salts.

Table 1.3 compares²⁵ the chloride content in the atmospheric air in Dhahran region (July-August) with air in Long Beach, California, along the Atlantic coast. It is seen that the chloride pollution in the ambient environment in Dhahran is 490 times that in the air in a marine atmosphere in USA.

Table 1.4 compares²⁶ the chloride contents of various sea waters with that of the Arabian Gulf at Dhahran Beach. It shows that the chloride content of the later is about 1.6 to 2 times as high as the sea waters from Mediterranean and the Atlantic.

TABLE 1.3

Chloride & Sulphate Content in Atmospheric Air
of Gulf Coast (Dhahran) and Atlantic Coast
(Long Beach, Morehead City)
(July-August)

	Dhahran	Long Beach	Morehead City
CL ⁻ μmg/m ³ of air	63.7	0.13	0.13
SO ₄ μmg/m ³ of air	33.8	0.32	0.54

TABLE 1.4

Chloride Content of Sea Water Compared with
Arabian Gulf Water (Dhahran Beach)

Mediterranean	Atlantic	Mean Sea Water	Arabian Gulf (Dhahran Water Beach)
21,380	17,840	19,800	33,660

Concrete construction on the Gulf coast is, therefore, continually exposed to ground and atmosphere charged with salts, and aided by capillary action and high humidity conditions the salt contaminated ground water and the salt-laden airborne moisture find an easy ingress in the exposed concrete matrix. This supply would be in addition to an already substantive base salt content already present in the concrete through the mix water and the salt contaminated aggregates.

The ingress of chlorides, moisture and oxygen into the concrete matrix is facilitated by a rare combination of favourable circumstances. Firstly, the concrete cracking due to plastic shrinkage early thermal movements and subsidence stresses. This cracking which is random as well as reflective of the reinforcing mesh pattern appears within hours of the concrete pouring. Secondly, subsequent cracking due to drying shrinkage and due to the effect of sharp temperature gradients on surfaces exposed to intense heat on the outer face and an air-conditioned cool environment on the inside surface. Thirdly, the round the year changes in the day and night temperatures cause continuous thermal expansion and contraction which may lead to the cracking of concrete. Some of the dolomitic limestones used in the

Gulf have a much lower coefficient of expansion than the surrounding hardened cement paste resulting in cracking or the accentuation of an already existing cracking²⁷. Salt weathering occurs when saline ground water, salt-laden moisture or even salt dust penetrates concrete pores or cracks. Subsequent to the evaporation of water these salt crystallise in the crevices and cause expansion and cracking. Fourthly, sulphate attack causes expansive cracking and mortar deterioration. Fifthly, the alkali-silica reactions in cases where silica-reactive aggregates are used cause pattern or map cracking. And finally, poor workmanship and lack of adherence to specified and good construction practices and quality control results in a poor quality and porous concrete facilitating the ingress of chlorides, moisture and oxygen.

The early cracking or lack of denseness of concrete matrix, per se, may not impair the function or serviceability of the structure. Such conditions are, however, a significant forerunner or harbinger of more serious conditions of distress subsequently. These early cracks or the high porosity act as feeders by facilitating the ingress of salt-laden water and moisture which attacks concrete and reinforcement in the Gulf environment causing

corrosion and consequent disintegration of concrete.

The extreme Gulf climatic conditions²⁸ are particularly helpful to the chemical reactions involved in the corrosion and other disintegration processes. The data summarized in Figure 1.5 illustrate the climatic variations typical of the Arabian subcontinent and bordering areas. Mean annual temperatures are generally 8°C above the value of 18°C usually taken to define a hot desert with summer air temperatures in the area frequently reaching $45^{\circ} - 50^{\circ}\text{C}$. The annual rainfall is almost always less than 127 mm isohyet often used to surround the zone of greatest aridity. From May to October the rainfall is virtually zero but the evaporation is intense in this period of very high temperatures often combined with high wind speeds. The combined effects of frequent drying winds, high temperatures and low precipitation result in excessive evaporation of Gulf waters (124 cms/year) causing high salinities especially in coastal areas (Figure 1.6). The Gulf salinities²⁹ are quite high compared to 38.74% and 32.62% for Mediterranean and Atlantic oceans respectively.

Daily variations in humidity and temperature can be greater. For example, relative humidity near the coast often ranges from 40% to 100% over 24 hour and temperatures

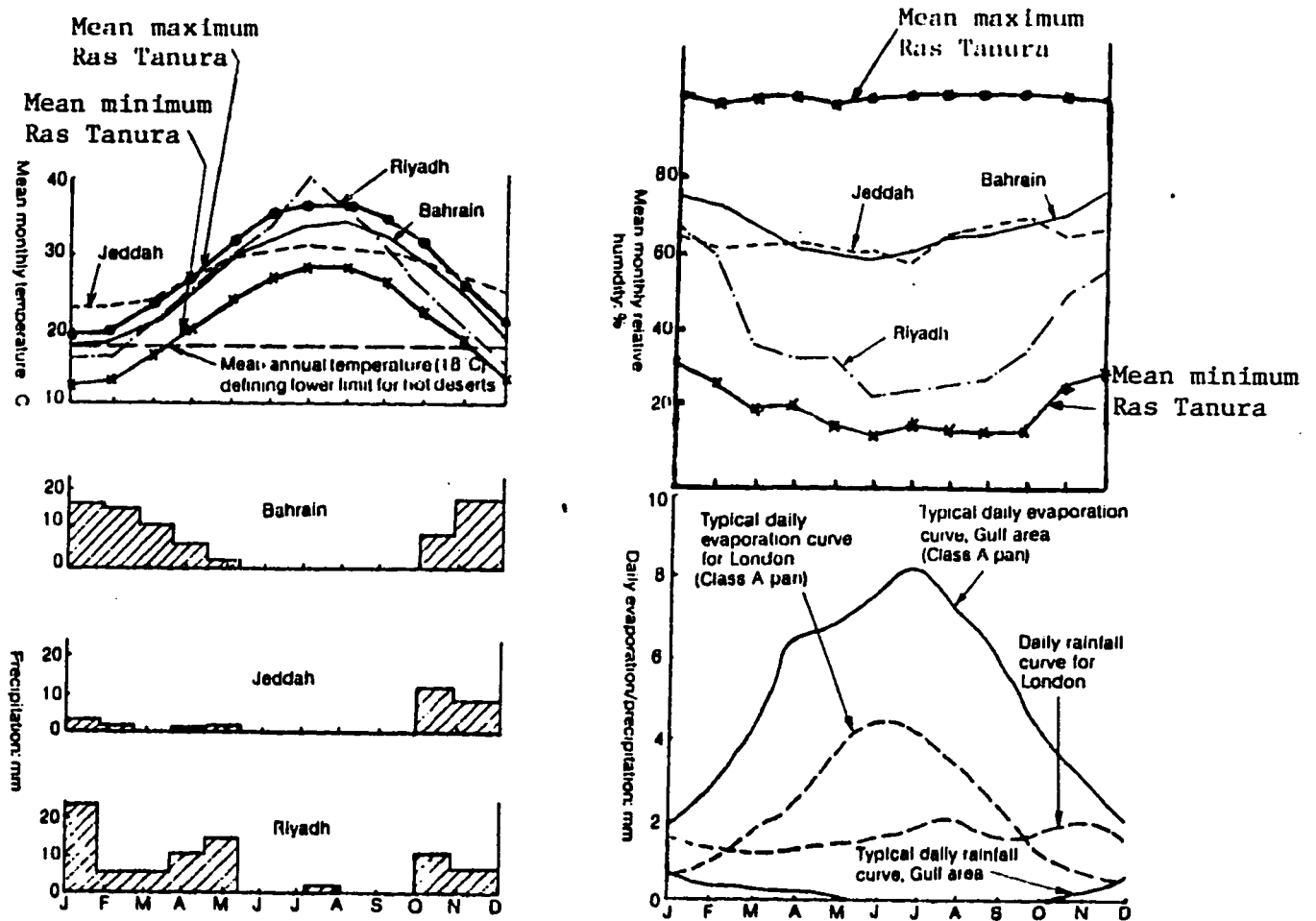


Fig. 1.5 : Representative climatic data for Arabian subcontinent

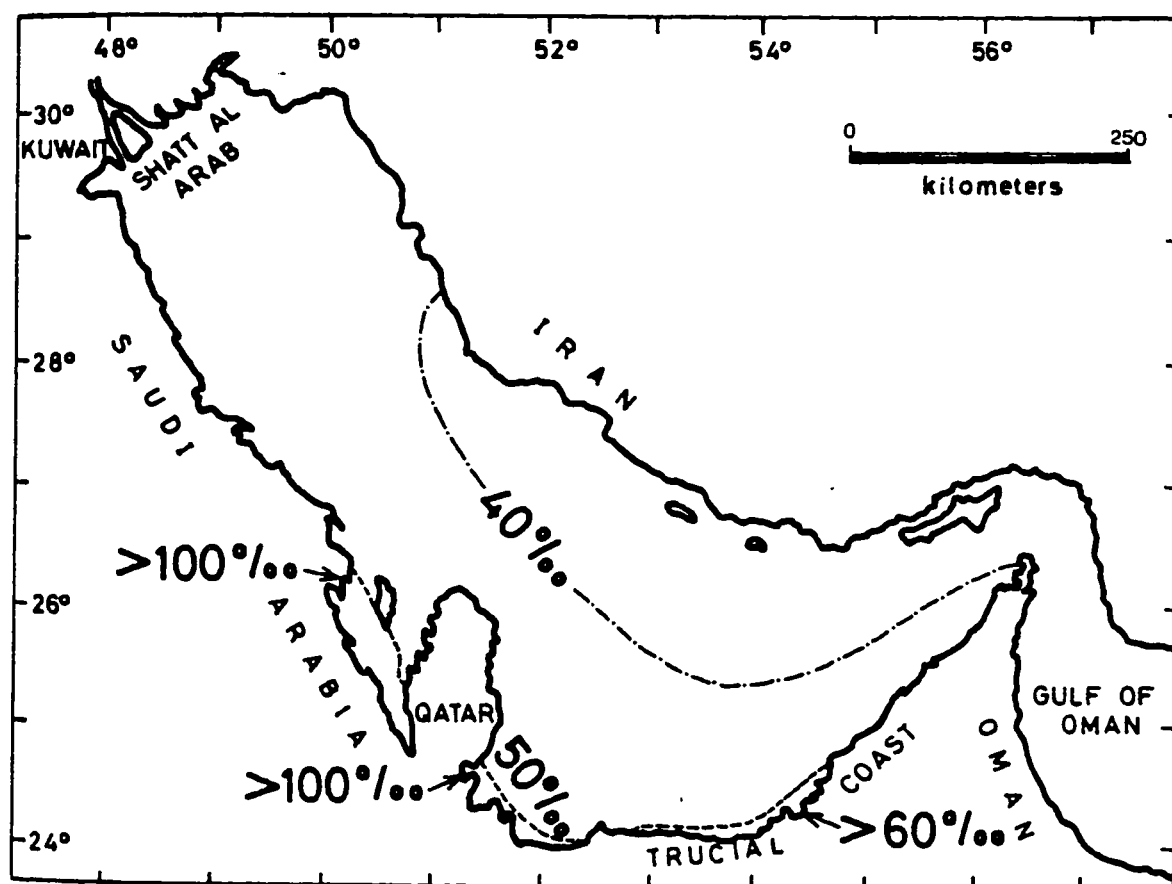


Fig. 1.6: Map showing major salinity trends within the Persian Gulf.
(Based on Emery, 1956; Brettschneider, 1970; and Shell Research data)

may vary as much as 20°C during one day in summer. Dew often forms overnight at any time of the year. Thus both moisture and high temperature are available to accelerate chemical reactions and as a rule of thumb, the speed increases by 70% for every 10° rise in temperature. An investigation³⁰ carried out on the effect of temperature and humidity on corrosion process shows that when circumstances are such that corrosion can occur its rate is increased by higher temperature and higher humidity. The rate of corrosion appears to be sharply increased by increase of temperature in the range of 20°C to 40°C , especially at high humidity.

Knofel³¹ citing German experience observes that corrosion is increased by moisture, high relative air humidity and impurities. The average relative air humidity in some areas of West Germany being higher in winter than in summer, Knofel reports that iron is found to rust five times as fast in winter than it does in summer. Uhlig³² cites data showing that it is not necessary to have 100% relative humidity to produce condensation on metal surfaces. Hygroscopic impurities in the atmosphere cause condensation at much lower values. He reports that the critical relative humidity above which corrosion of steel will proceed on an accelerated rate is between 50% and 70%. As an example,

the corrosion behaviour of iron in a 0.01% SO_2 atmosphere at various relative humidities is shown in Figure 1.7. The figure shows a dramatic increase in corrosion rate beginning at 60% relative humidity.

The combined effects³³ of temperature and humidity on the corrosion of crevice coupons mounted on vehicles during a winter season (November-April) in Winnipeg are demonstrated in Figure 1.8. The corrosion rates of those crevice coupons that were mounted on vehicles and were periodically housed in heated garages were approximately triple the corrosion rates of those mounted on vehicles left in the open or in unheated garages.

Another distinctly unique feature of the concrete disruption in the Gulf region is the complex interactive relationship between the various causal factors. The way in which concrete is attacked is rarely simple. One form of attack tends to feed on the weakness created by others until concrete eventually disintegrates completely. The total deterioration is by no means a simple arithmetic summation of the deteriorations caused by each of the operative causal factors individually. Disintegration due to one factor initiates and sometimes accelerates another form of attack and the total deterioration is cumulative

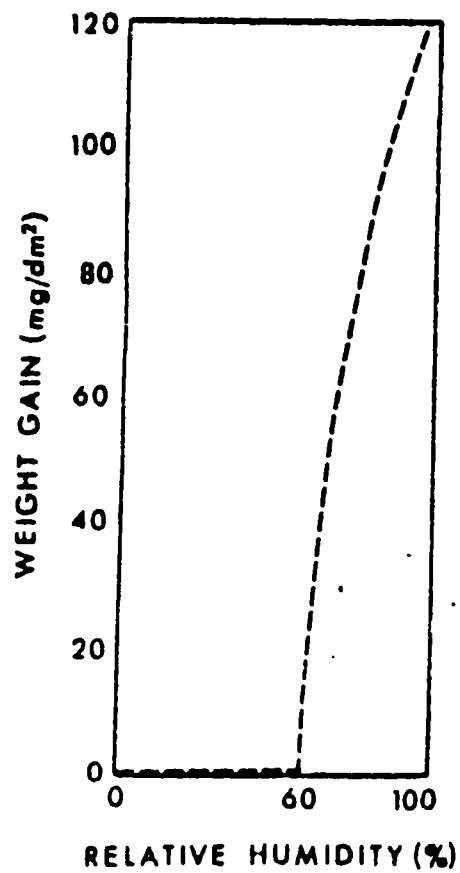


Fig.1.7 : Corrosion of iron in air containing 0.01% SO_2 , showing critical relative humidity

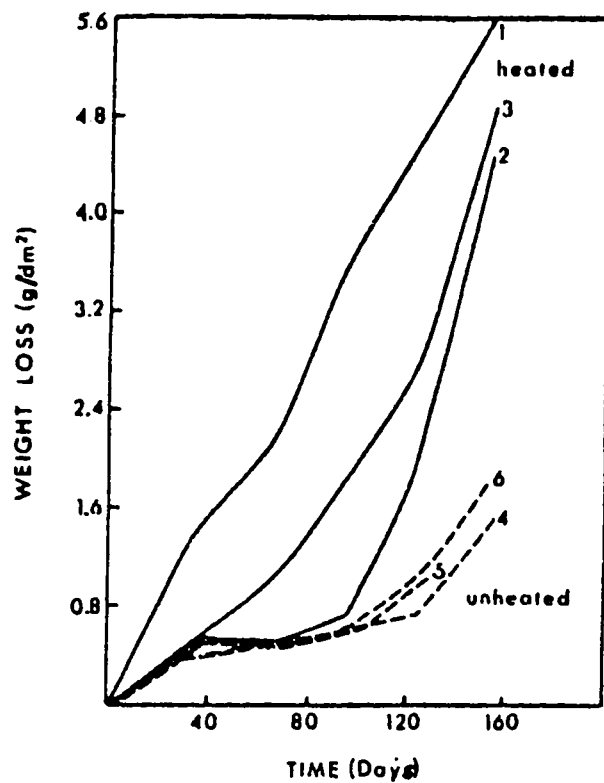


Fig.1.8 : Comparative corrosion rates of crevice specimens attached to vehicles stored in heated garages vs unheated garages

in a complex way. Thus cracking due to plastic shrinkage or sulphate attack may immensely accelerate the hitherto slow corrosion process by opening channels of easy flow of chloride, moisture and oxygen to the steel location in a reinforced concrete member.

1.6 OBJECTIVES OF THE PRESENT STUDY

The in-service wide gap between the present knowledge based on laboratory tests and performance of structures in practice about factors adversely affecting concrete durability is universally acknowledged. Although factors affecting concrete durability have been adequately identified, and with respect to some of these factors such as steel corrosion, sulphate attack, alkali-aggregate reaction and early-age cracking considerable lab data exists to set up useful guidelines. However, lab tests have often proved difficult to interpret in terms of field performance and fail to provide a pragmatic and reliable estimate of concrete deterioration in the field under interactive factors. On these grounds it is widely recognized that to be objective and meaningful the formulation of design and practice recommendations for *durability* (in conjunction with apparent strength) have to be based on the feedback and

experience gained from the performance and behaviour of actual structures in the field. Field performance of concrete in the Gulf area provides a unique opportunity to evaluate parameters bearing directly on the durability characteristics of concrete construction in an aggressive service environment. It is felt that an organised collection of field data supported wherever necessary by lab investigations on influential parameters causing deterioration of concrete in the Eastern Province of Saudi Arabia will prove to be particularly useful in planning practical attempts to deal with concrete deterioration problem in this part of the Kingdom and in other areas sharing common factors of climate, ground condition and associated construction problems.

Keeping this broad objective in view field and laboratory investigations have been carried out to collect data on the corrosion and spalling of concrete which was identified as the premier deterioration causal factor in the Gulf region. Specifically speaking, the following parameters have been investigated :

Firstly, the effect of chloride ion on the extent of corrosion.

Secondly, the effect of concrete quality on the

extent of corrosion.

Thirdly, the effect of cover to reinforcement on the extent of corrosion.

Fourthly, a study of the materials and construction practices on the corrosion of reinforcement, and

Fifthly, a review of the remedial measures.

Throughout this investigation, field studies were emphasized for the reasons outlined in the beginning of this section.

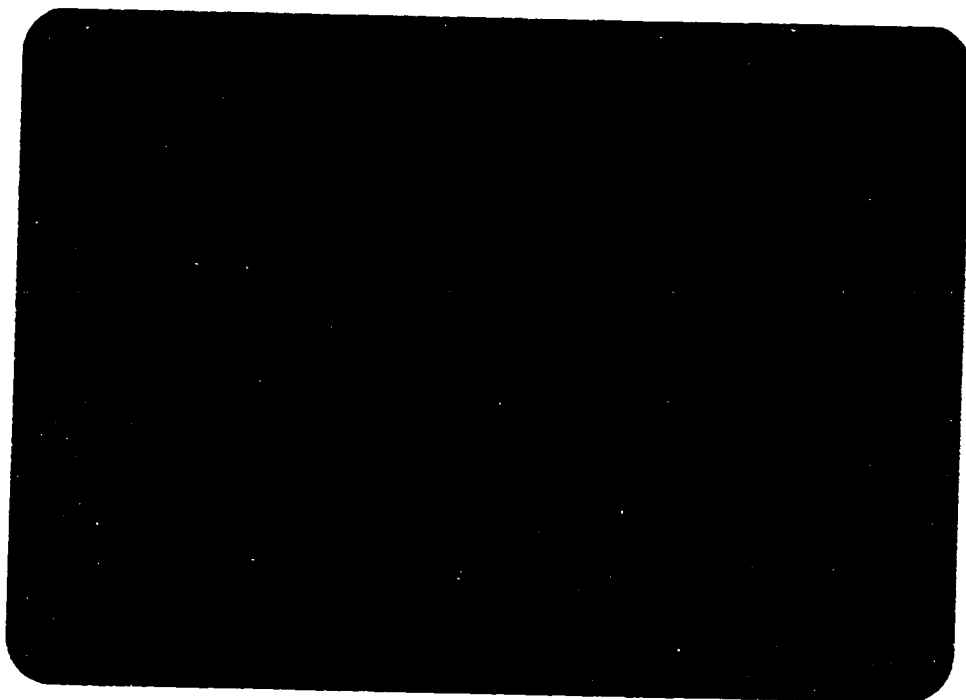


Plate 1.1 : Cracks due to shrinkage and early thermal movements



Plate 1.2 : Concrete deterioration due to sulphate attack and salt weathering

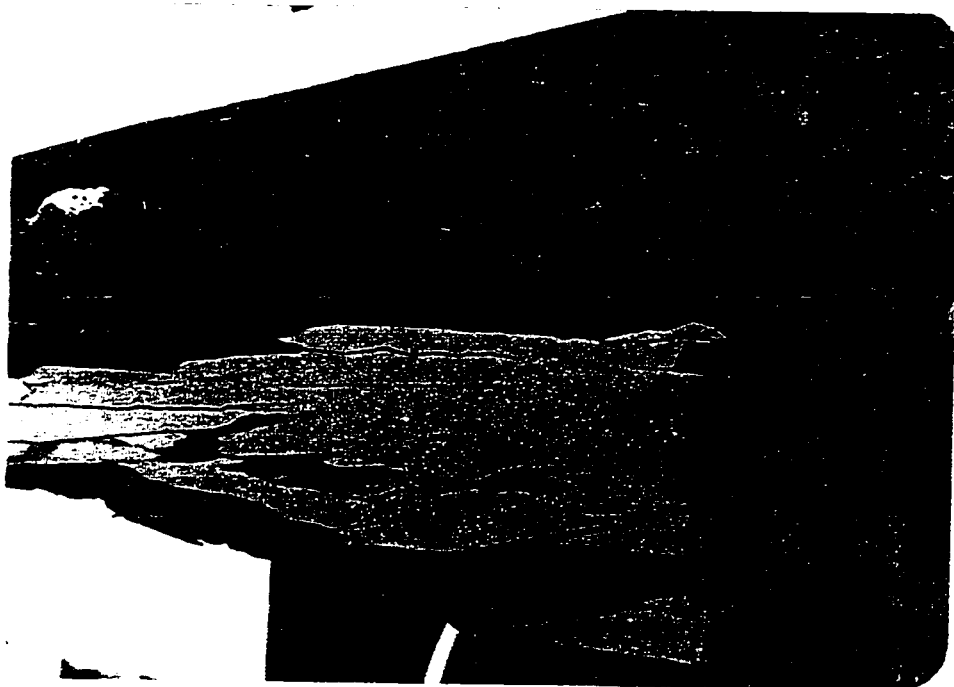


Plate 1.3 : Concrete spalling due to rebar corrosion

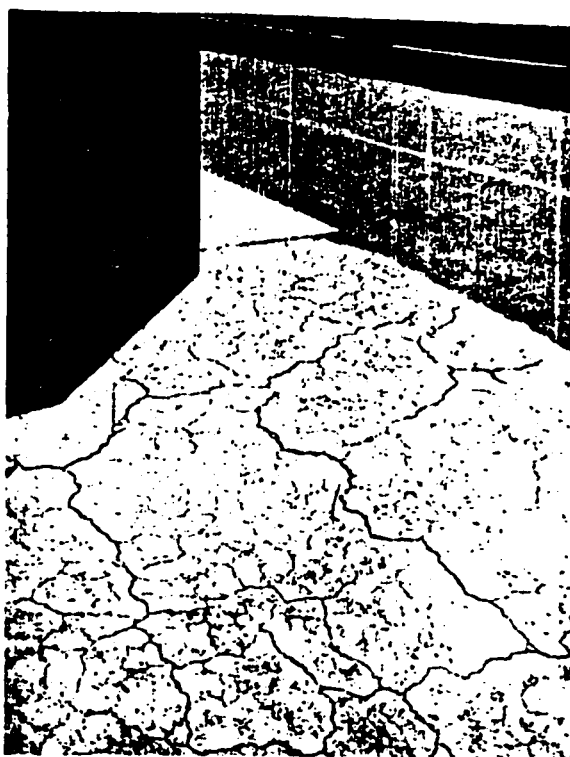


Plate 1.4 : Concrete cracking due to alkali-silica reaction

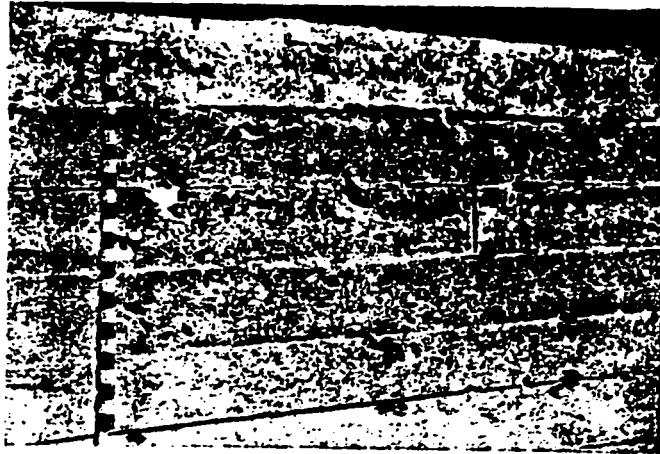


Plate 1.5 : Concrete deterioration due to popouts



Plate 1.6 : Corrosion pitting on rebars

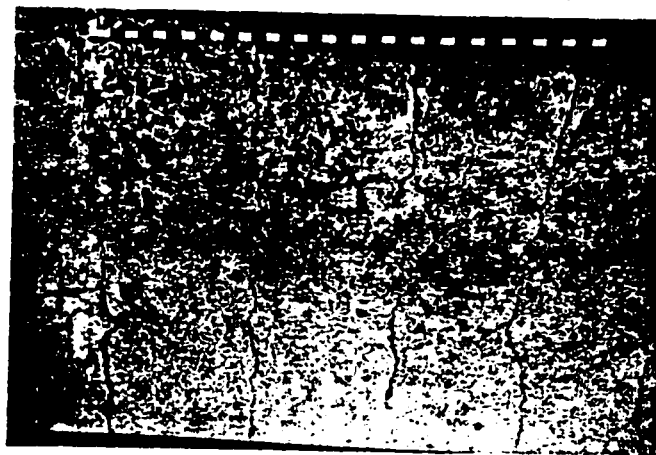


Plate 1.7 : Hair cracking

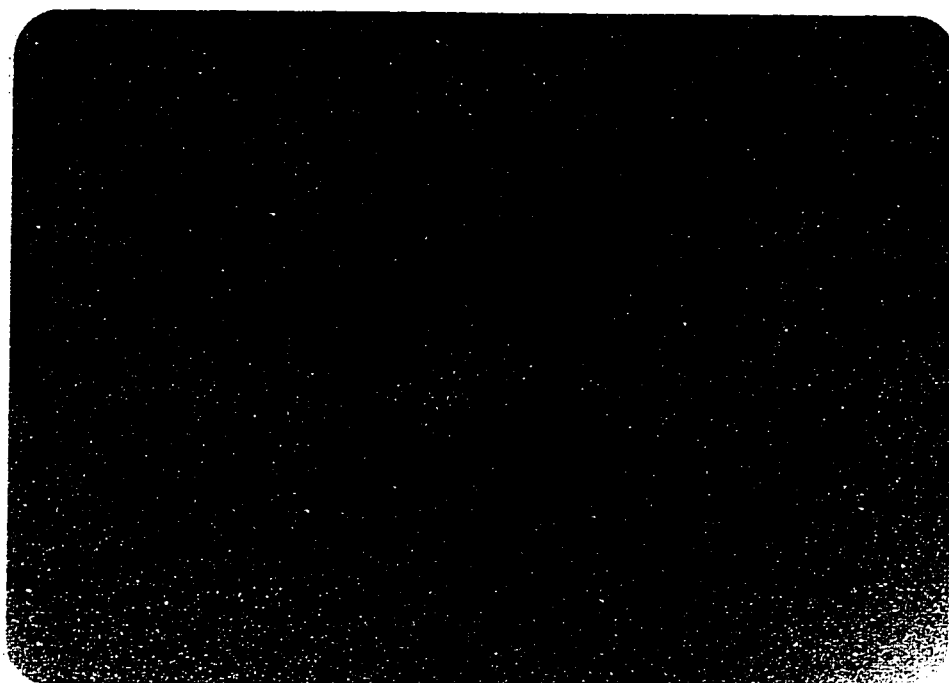


Plate 1.8 : Crazing



· Plate 1.9 : Corrosion stains on concrete surface

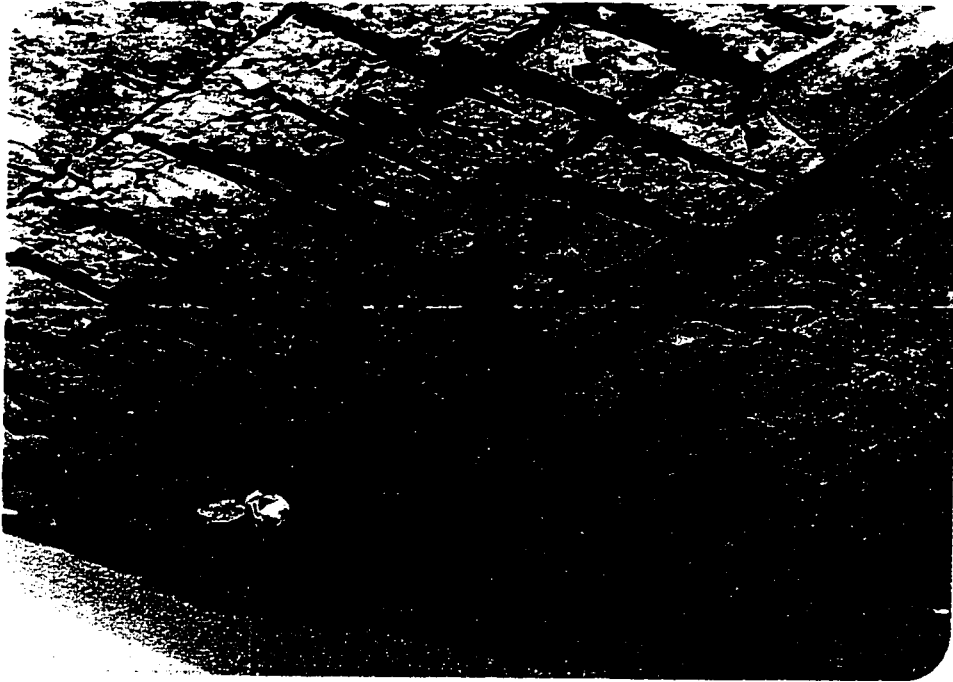


Plate 1.10 : Illustration of severe, overall deterioration
(Classification 5)



Plate 1.11 : Illustration of moderate deterioration over the
whole area of inspection (Classification 4)

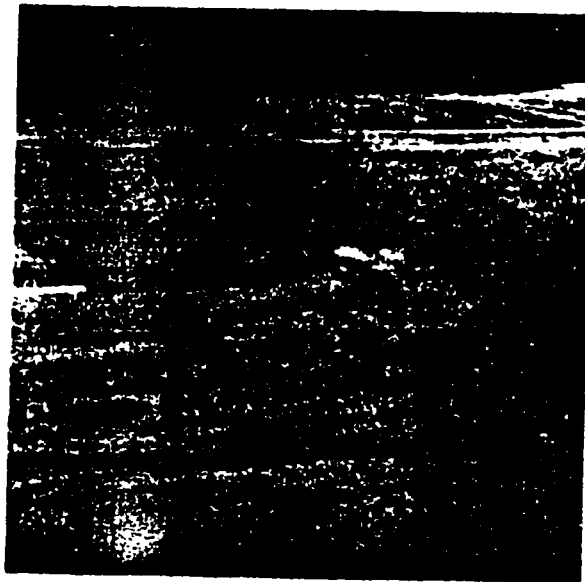


Plate 1.12a : Local crackover reinforcements



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Plate 1.12b : Loss of a bridge deck surface due to spalling

chapter **2****MECHANISM OF STEEL CORROSION
AND CONCRETE SPALLING****2.1 INTRODUCTION**

Professor F. Tamas of the Hungarian Academy of Sciences has described concrete as "the most noble material on earth"³⁴. In respect of corrosion of reinforcement also the foregoing statement is quite true. High quality concrete, properly placed, compacted and cured provides an optimum alkaline environment at the steel concrete interface that will normally prevent corrosion of embedded steel. However, reports from the field abound with evidence of serious problems due to rebar corrosion in concrete structures both in terms of the frequency of occurrence and the magnitude of the resulting damage. Reinforced concrete highway structures (specially bridge decks) and structures exposed to salt-laden environment usually characterized by exposure to sea water or marine atmosphere are typically severely prone to this problem. The common denominator is the presence of chlorides which results from the application of deicing salts in the former instance and from the environment in the later.

The Eastern Province of the Kingdom groups with the second category. Only the attack is visible in a far intensified form due to the abnormally high salinity of the Arabian Gulf and due to the presence of salts in the duricrust formations of the region due to the peculiar regional geomorphic conditions.

This chapter describes the mechanism of corrosion of steel embedded in concrete with special reference to the prevailing conditions in the Gulf States in general and the Eastern Province of the Kingdom in particular. The other salient features which define the scope of the material in this chapter are : firstly, the mechanism is discussed from the point of view of an engineer rather than a corrosion scientist; secondly, the core material describing the mechanism is sifted from a large number of technical papers and research reports, sometimes presenting conflicting information, so as to obtain a degree of consistency from the point of view of available evidence and data; and thirdly, the discussion is relevant only to corrosion of steel in the characteristic concrete environment and other forms of corrosion such as by direct oxidation or by acid attack which are not pertinent to the concrete situation have been overlooked.

2.2 ELECTROCHEMICAL CORROSION MECHANISM RELEVANT TO STEEL EMBEDDED IN CONCRETE

The current state of knowledge on corrosion has evolved rapidly over the past 30 years and there is unanimity that corrosion of steel in concrete takes place through an electrochemical process, i.e., a process involving chemical reaction as well as a flow of electricity. Further, the flow of electricity mentioned is mostly an internally generated process through electric cells, called galvanic cells.

2.2.1 Galvanic Cells

The aforesaid galvanic cells are generated in various ways but to be practically operative they all essentially require :

Firstly, two dissimilar electrodes electrically connected by a conductor.

Secondly, presence of an electrolyte whose ions can conduct electricity, and

Thirdly, presence of an electrical potential between the two electrodes.

A simple galvanic cell incorporating the above prerequisites is shown in chapter 1 (Figure 1.4). Figure 2.1 shows a simulated model of a galvanic cell in concrete with

the aforesaid essential features marked on the diagram itself and elucidated below :

- * *Two zones* A and C on the rebar act as *electrodes* (anode and cathode respectively). These electrodes are connected by the bar length AC acting as a *conductor*.
- * *Water and moisture* with dissolved salts within the pores of concrete matrix act as an *electrolyte*.
- * A *differential electrical potential* between cathode and anode may arise from almost any conceivable heterogeneity of the system, that is, any asymmetry of the metal, or the environment, or both. Specifically, an electrical potential may be caused due to any of the following reasons :
 - (i) *Differential aeration* : Differences in concentration of oxygen over the surface of the steel.
 - (ii) *Differential ion concentration* : Differences in concentration of metal ions, dissolved salts in the vicinity of steel.
 - (iii) *Differential concrete environment* : Differences in the alkalinity (due to alkali-aggregate reaction, preferential leaching, carbonation), cracking and microcracking, and temperature within the concrete;

bleeding, segregation or poor consolidation may also result in variations in the internal concrete environment.

(iv) *Differential surface properties of steel* : Variation in the surface characteristics of the steel such as mill scales or break in coatings.

2.2.2 Chemical Reactions

Corrosion is aptly defined as the destruction of a metal due to interaction with its environment. Table 2.1 is the electromotive force series. All metals listed above hydrogen are electronegative and have a tendency, in proportion to their upward position in the series, to oxidize to a metal ion in an aqueous solution. These metals, such as steel or iron, are manufactured by reduction from ores as oxides found in nature. The process of conversion from oxides to base metal is energy-intensive and their resultant metal is therefore metastable due to its high energy state and tends to revert to a lower-energy, combination state³⁵. This process, when it takes place in an uncontrolled manner as a result of reaction with the environment, gives rise to corrosion and in the case of iron is known as rusting. Metals which behave in this manner are listed electronegative with reference to a standard hydrogen electrode in the electromotive

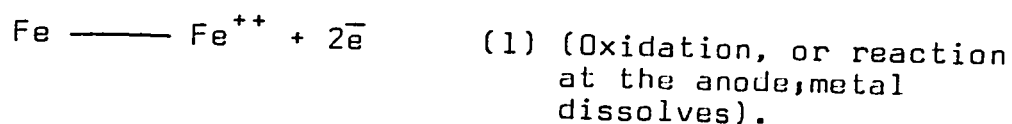
TABLE 2.1
Electromotive-Force Series (77°F)

Element	Electrode Reaction	Standard Electrode Potential, v ^a
(Active end)		
Sodium	$\text{Na} \rightarrow \text{Na}^+ + e$	-2.712
Magnesium	$\text{Mg} \rightarrow \text{Mg}^{++} + 2e$	-2.34
Beryllium	$\text{Be} \rightarrow \text{Be}^{++} + 2e$	-1.70
Aluminum	$\text{Al} \rightarrow \text{Al}^{+++} + 3e$	-1.67
Manganese	$\text{Mn} \rightarrow \text{Mn}^{++} + 2e$	-1.05
Zinc	$\text{Zn} \rightarrow \text{Zn}^{++} + 2e$	-0.762
Chromium	$\text{Cr} \rightarrow \text{Cr}^{+++} + 3e$	-0.71
Iron	$\text{Fe} \rightarrow \text{Fe}^{+++} + 3e$	-0.44
Cadmium	$\text{Cd} \rightarrow \text{Cd}^{++} + 2e$	-0.402
Cobalt	$\text{Co} \rightarrow \text{Co}^{++} + 2e$	-0.277
Nickel	$\text{Ni} \rightarrow \text{Ni}^{++} + 2e$	-0.250
Tin	$\text{Sn} \rightarrow \text{Sn}^{++} + 2e$	-0.136
Lead	$\text{Pb} \rightarrow \text{Pb}^{++} + 2e$	-0.126
Hydrogen	$\text{H} \rightarrow 2\text{H}^+ + 2e$	0.000 (reference)
Copper	$\text{Cu} \rightarrow \text{Cu}^{++} + 2e$	+0.345
Copper	$\text{Cu} \rightarrow \text{Cu}^+ + e$	+0.522
Silver	$\text{Ag} \rightarrow \text{Ag}^+ + e$	+0.800
Platinum	$\text{Pt} \rightarrow \text{Pt}^{++} + 2e$	+1.2
Gold	$\text{Au} \rightarrow \text{Au}^{+++} + 3e$	+1.42
(Noble end)		

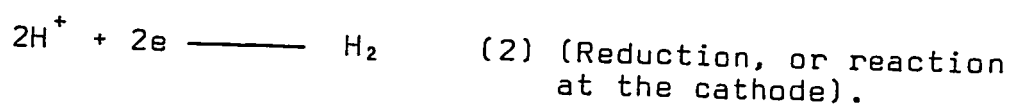
^aThere are two schools of thought on the plus versus the minus sign on these voltages; the main thing to keep in mind is that the more active (the more minus in this table), the more the metal tends to oxidize (lose electrons or corrode).

force series because on immersion in an electrolyte they have a tendency to release ions into solution leaving a residual negative charge on the metal.

The ionization of the metal at the anode is often referred to as the primary stage of the corrosion reaction and may be represented by the following equation^{36,37}



The anodic region of the metal now has an excess of electrons as indicated by equation (1). To maintain equilibrium of electric charges an equivalent quantity of hydrogen is plated out at adjacent surfaces of the metal:



This results in a thin invisible film of hydrogen around the cathode (Figure 2.2) known as cathodic polarization, thus providing an insulating blanket that reduces current flow and practically stops corrosion. Consequently, any subsequent reaction will cease unless the hydrogen film is removed in some manner.

The destruction of the hydrogen film may occur in one of the two ways : (a) oxygen depolarization at the cathode,

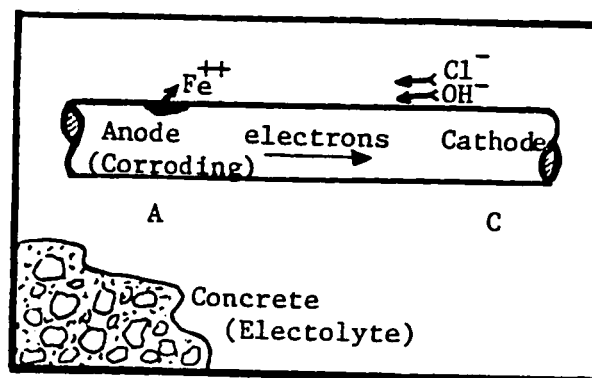


Fig. 2.1 : Schematic diagram of the corrosion cell

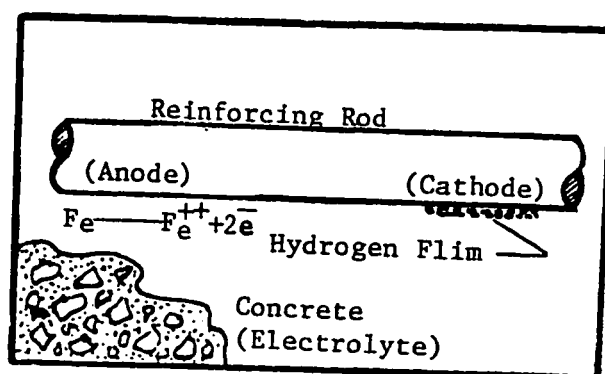
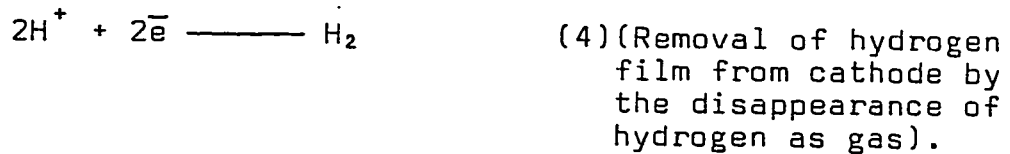
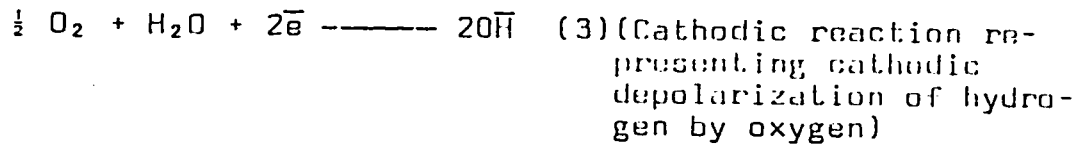


Fig. 2.2 : Primary reaction - formation of hydrogen film at the cathode which inhibits subsequent region

and (b) hydrogen evolved as a gas as shown in Figure 2.3.



Concrete matrix is usually permeated with moisture or water containing dissolved oxygen. The cathodic depolarization by oxygen is therefore, very widespread and is taken for granted. On the other hand, reaction (4) is generally not characteristic of the corrosion of steel in concrete. Hence it is customary to omit equation (2) from the reaction process and to represent the cathodic reaction directly by equation (3). This is indicated accordingly in the flow diagram of the chemical reactions in the corrosion cell shown in Figure 2.4.

The reactions at the cathode are termed as secondary reactions and permit the primary reactions at the anode to proceed with the accumulation in the solution of ferrous ions, Fe^{++} , which in the presence of water and oxygen at the cathode are oxidised and precipitated as rust in accordance with the following reaction :

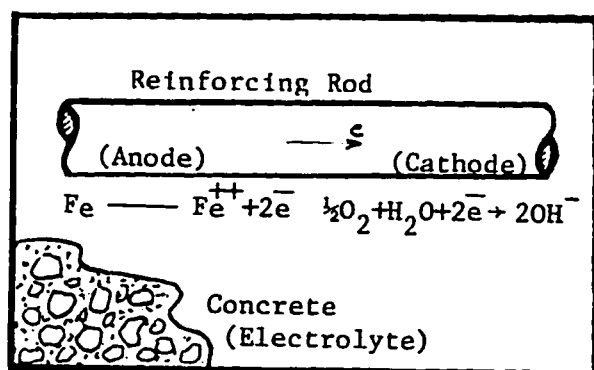


Fig. 2.3 : Secondary reaction - break down of the hydrogen film thus allowing the corrosion process to continue

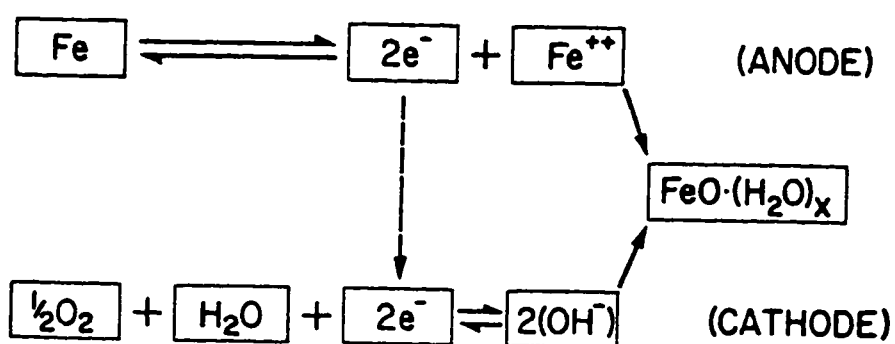
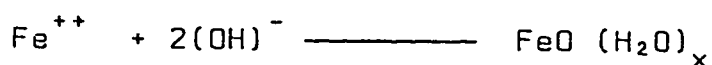


Fig. 2.4: Flow diagram showing the basic chemical reactions in the corrosion cell



2.2.3 Types of Corrosion Cell in the Gulf Environment

In the high chloride environment of the Gulf two types of corrosion cells can be envisaged:

Type A

Steel (Anode)	Permeable Concrete	Less Permeable Concrete	Steel (Cathode)
	Low pH High cl	High pH Low cl	
	High O ₂	Low O ₂	

Type B

Steel (Anode)	High Water Saturation of Concrete and High Porewater Content of Soluble Salts	Low Water Saturation of Concrete and Low Porewater Content of Soluble Salts	Steel (Cathode)
	High salt	Low salt	
	Low oxygen	High oxygen	

High salt, low oxygen and low pH regions form anode and low salt, high oxygen and high pH regions form cathode.

It can be seen that type A is basically a *differential salt and pH cell* which is supposed to overshadow the *secondary*.

opposing oxidation potential. This cell is based on the *permeability conditions* of concrete. Variations in permeability conditions allow a differential permeation of salt and oxygen, the ingress being higher where concrete is less dense and more porous and vice versa. Such a cell is supported by the reported research of Lewis and Copenhagen⁶².

Type B cell recognizes the importance of the oxidation potential and envisages its influence as additive to the differential salt cell. The physical viability of the cell is explained on the basis of two assertions : firstly, high water saturation in some locations of concrete lowers air and oxygen permeability and diffusibility into water filled concrete matrix, and secondly, higher soluble salt content in pore water will lower the dissolved oxygen concentration. Thus region of *high salt/low oxygen* and *high water saturation/low oxygen* will become anode and the region of *low salt/high oxygen* and *low water saturation/high oxygen* will act as a cathode. This type of cell is supported by Finley⁶³ and by the informative work of Rhaschl and Marsh⁶⁴. Evans⁶⁵ and Bhatt et al.⁶⁶ have also documented the influence of oxygen.

The author feels that differential oxidation potential plays a very important role in promoting corrosion. There

is evidence that sometimes corrosion has progressed to damaging limits purely on the strength of the differential oxidation cell and in the total absence of chlorides or other salts. Shermer⁶⁷ has reported typical corrosion of reinforcing steel in a lumber drying kiln where the concrete was exposed to warm moist air. The concrete was of fair quality, testing between 3000 and 4000 psi but there was little cover over the steel and in order to facilitate casting the small beams a high w/c ratio was used producing permeable concrete.

The author, during the chloride-metal loss analysis of his field samples, has also noticed that in several cases the loss of metal was totally out of proportion to the low chloride content. This observation supports the significant role of the differential oxidation potential as a source of corrosion.

2.2.4 Corrosion Products

The initial corrosion product is a ferrous hydroxide usually found directly on the steel surface. From this first stage product a whole series of more or less hydrated ironoxides will be formed depending on the particular corrosion conditions, especially the availability of oxygen and water^{37,38,39}. However, a very common second state is

hydrated ferric hydroxide formed a little away from the surface where it is in contact with more oxygen as shown in Figure 2.5:

The structure and composition of the rust varies considerably with the conditions prevailing during its formation and the structure plays an important role in the subsequent corrosion process^{37,39}. If the rust layer is hard, dry, and fairly adherent to the metal surface, then it may retard corrosion by forming a protective coating. On the other hand, if the layer is spongy and readily detachable it will absorb oxygen and moisture from the surrounding media and consequently will add to further activity.

The products of corrosion occupy a space 2.2 to 15 times that of the original metal and may develop a mechanical pressure within concrete⁴⁰ as high as 4700 psi which is far in excess of the tensile strength of concrete.

2.2.5 Corrosion Rate

Speller³⁷ has separated the factors which govern the tendency of the corrosion reaction to proceed from those which influence the rate of corrosion. The magnitude of the electrochemical potential mainly determines the tendency of the reaction to proceed. As for rate, if a process, like the electrochemical corrosion, comprises two or more

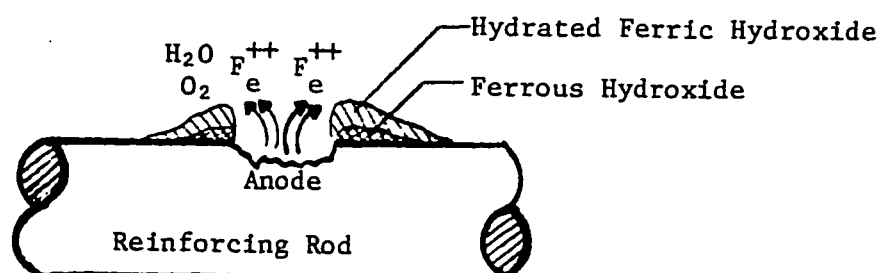


Fig.2.5: Formation of rust at the anodic region

interrelated reactions, then the rate of the overall process is determined by the rate of the slowest of these component reactions under a particular condition. In most cases the primary reaction at the anode is much faster than the secondary reactions and the dominating influence on the later is oxygen concentration of the media. The oxygen supply at the cathode is therefore usually regarded the dominant influence on the rate of the overall corrosion process.

2.3 FACTORS INFLUENCING CORROSION OF STEEL IN CONCRETE

2.3.1 Chemical Environment in Concrete

As pointed out in the opening paragraph of this chapter, the chemical environment of concrete is exceedingly well suited to prohibit corrosion activity along the embedded reinforcement. Steel is protected from corrosion by concrete due to the passivating effect of the alkalis within it. The alkalinity of concrete (pH 12.5 or greater) is mainly caused by the presence of calcium hydroxide (Ca(OH)_2) as a result of the hydration of the calcium silicate phase of the cement. The calcium hydroxide is partly dissolved in the water within the concrete pores and exists there as a super saturated Ca(OH)_2 solution, and partly it is precipitated in the

form of calcium hydroxide crystals, which are embedded within the set cement. The initial alkalinity of the concrete is at least that of saturated lime water (pH of about 12.5 depending upon the temperature). In addition, the seemingly relatively small amounts of sodium and potassium oxides in cement further enhance the initial alkalinity of the concrete (pH values of 13.2 and higher have been reported)⁴¹. In the presence of water soluble alkaline products, moisture and oxygen a thin protective film of gamma ferric oxide (Fe_2O_3) is formed⁴² on the steel surface which inhibits corrosion even though an electrical potential may exist on the rebar surface due to any of the heterogenities elucidated in section 2.2. If it can be ensured that this impervious, dense, protective film is maintained on the entire surface of the reinforcement, there is a strong chance that corrosion will not take place.

However, this passivating film is disrupted in either of the two ways :

- (1) By penetration of carbon dioxide dissolved in the atmospheric air through the concrete. This results in the formation of carbonate and bicarbonates of calcium ($\text{Ca}(\text{OH})_2 + \text{CO}_2 + \text{H}_2\text{O} \longrightarrow \text{CaCO}_3 + 2\text{H}_2\text{O}$) reducing the pH value from 12.5 to somewhat below 9⁴³.

When pH value is below 10 the passivating film becomes weak and is impaired no longer assuring protection against corrosion.

- (2) By the presence of sufficient chloride ions at the steel surface, chloride ion being a specific and unique destroyer of the protective oxide film⁴⁴.

2.3.2 Effect of Carbonation and Chloride Ion

The process of carbonation progresses slowly and only in exceptionally porous concretes the penetration depth reaches 1 or more inches⁴⁵. In normal structural concrete, however, the depth of carbonation does not progress much further than 0.2 to 0.3 inch⁴⁶. Measurements in structures up to 10 years old showed atmospheric carbonation to a depth of 2 to 11 mm (0.073 to 0.43) inches) with only a single exception⁴⁷. This shows that at least half of the usual depth of cover would remain unaffected and the loss of alkalinity due to carbonation is not an active problem. An exception to this comment is the lightweight aggregate concretes where investigations⁴⁸ at Building Research Station, U.K., have shown that carbonation upto 1 inch can be found after only six years with a 1:9 cement:aggregate mix and upto 0.5 inch for a 1:5 mix.

However, the presence of chloride ion at or near the steel concrete interface has a uniquely allround damaging effect on the degree of protection afforded by concrete to the embedded steel. Firstly, chloride ions impair the passivating film of ferric oxide by converting it to soluble ferric chloride and thereby releasing ferrous ions at the anode⁴⁹. Secondly, chloride salts being strong electrolytes increase the electric conductance of concrete and by allowing the corrosion current to increase, accelerate the rate of rusting⁵⁰. Thirdly, calcium chloride ions depress⁵¹ the pH value of the cement pore fluid reducing the protective quality of the passivating film which depends on the alkalinity or pH of the environment (although sodium chloride will have the reverse effect of increasing the pH value). Finally, there is a view that chloride ions have an adverse effect on the rate of polymerization of the calcium silicates thus hindering the long-term strength improvement and impermeability of the concrete,⁵² the concrete shrinkage is increased,⁵³ and in the case of low C₃A the resistance to sulphate is impaired⁵⁴.

In the Gulf States chlorides are present in the concrete from several different sources. Firstly, soluble chlorides are introduced from the salt contaminated aggregate and

sea-dredged or beach sands which are rich in salt-content. Secondly, from the mix water (chloride content: 0.14%). Thirdly, from continual exposure to marine atmosphere characterised by salt-laden air borne moisture, salt-laden dew, wind and spray charged with sea salts. Fourthly, from ground and groundwater charged with salts which aided by capillary action find an easy ingress to concrete matrix. And finally, possibly as part of a concrete admixture. In the USA the most profuse source is the use of calcium chloride as a de-icing agent on bridge decks and highway pavements. In other cold countries calcium chloride is also used as an accelerator.

A more detailed discussion of the chloride effect on corrosion is included in chapter 3 wherein are also included the results of the lab and field investigations on this aspect.

2.3.3 Effect of Moisture Content and Oxygen

Even when the protective film is not perfect the reinforcing steel does not corrode to any significant extent unless moisture and oxygen are present near the paste-steel interface. The presence of moisture and oxygen is not only a necessary prerequisite for the initiation of corrosion, the extent of moisture and oxygen supply is the primary

rate determining factor in the corrosion process³⁷⁶⁵⁵⁶.. This is largely due to the effect of moisture content in reducing the resistivity of concrete thereby facilitating the flow of corrosion current and due to the role of oxygen as a cathode depolarizer (equation 3) and as an essential element for the reaction at the cathode (equation 3).

Stratfull⁵⁷ in comparing electrical resistivity of concrete with concrete deterioration produced by corrosion of reinforcement, found a nearly linear inverse relationship upto a resistivity value of 60,000 ohm-cm beyond which corrosion apparently does not occur. While resistivity is influenced by chloride content, it is more sensitive to the moisture content.

Dissolved oxygen in solutions is another factor that has a very definite effect on corrosion rates⁵⁸. In nearly neutral solutions, as in many natural waters, the concentration of dissolved oxygen is the controlling factor that determines corrosion rates. This fact is demonstrated⁵⁹ in Figure 2.6 which shows the effect of dissolved oxygen concentration on the corrosion of mild steel in slowly moving water containing 165 ppm CaCl_2 . The influence of dissolved oxygen on the corrosion process is illustrated by the qualitative curves shown in Figure 2.7. These curves

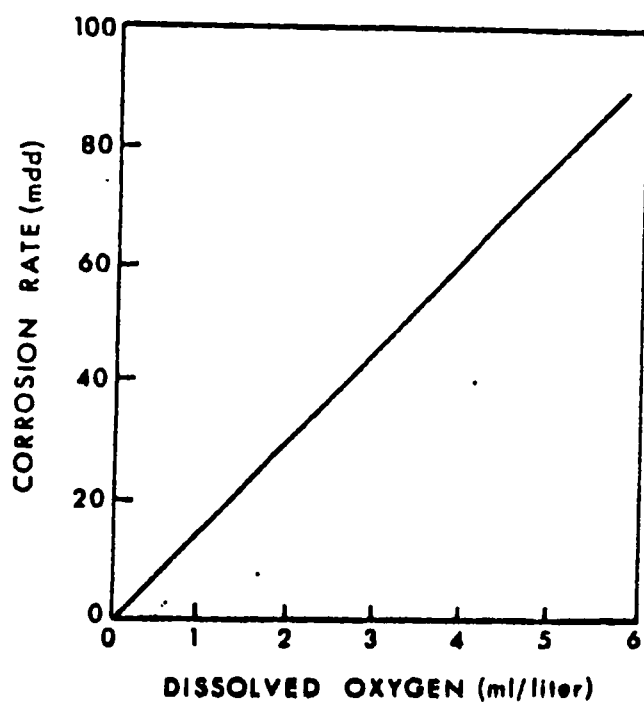


Fig. 2.6 : Effect of oxygen concentration on corrosion of mild steel

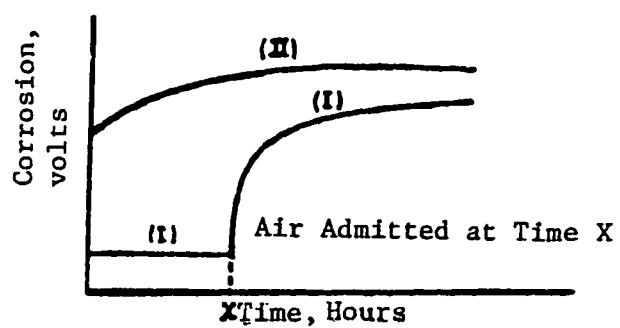


Fig.2.7 : Iron specimens immersed in 0.1 N. Na OH solution

obtained from experiments by Mayne, Mentor, and Pryor⁵⁹ are representative of iron specimens immersed in sodium hydroxide solutions. Curve I represents the corrosion rate in a de-aerated solution; the sharp break in the curve takes place at time X when air containing oxygen was admitted into the solution. Curve II represents the corrosion rate in a similar solution except that the system was in contact with air throughout the entire experiment.

Differential aeration, that is, the unequal distribution of oxygen over the steel surface, will set up anodic and cathodic regions^{37, 86, 0}. Regions of the metal least accessible to oxygen become anodic, and regions readily accessible to oxygen become cathodic. Thus the presence of oxygen in varying concentrations along the reinforcement will tend to increase the probability of corrosion. The influence of differential aeration emphasizes the deleterious effects of cracking and of porous concrete; two conditions which allow the penetration of oxygen to local areas of the reinforcement.

In considering the significant role played by the presence of oxygen, it should be said that steel in reinforced concrete completely and permanently immersed at some depth in sea water rich in salt, will not corrode at any appreciable

rate, as oxygen supply is virtually excluded. Reinforced concrete piles removed from Los Angeles Harbor after 30 years of immersion and use showed the steel below tide level to be quite free of corrosion⁶¹.

2.3.4 Effect of Concrete Quality

Properties of concrete that influence the alkalinity and the quantity or uptake and transmission of water, oxygen, and chlorides directly and profoundly affect corrosion of embedded steel. The alkalinity variations are governed to a small extent by the cement type and the admixtures and will be discussed in the following section. For the uptake and transmission of water, oxygen, chlorides and carbon dioxide the most significant index parameter related to the quality of concrete is its *permeability*. Low permeability, in dense and impervious concrete, retards the ingress of these corrosion promoting factors, and is attained by the use of low water-cement ratio, well compacted and well cured concrete. For w/c ratio above 0.5 concrete permeabilities increase exponentially with w/c ratio⁶⁸. The primary influence of curing is the effect that it has on concrete permeability. For example, the permeability coefficient for cement paste after 1 day of hydration will be more than 25000 times that for paste hydrated for 7 days⁶⁹. This

marked effect on practical concrete making is drastically mitigated by the influence of bleeding, segregation, degree of consolidation and the permeability of the aggregate.

The degree of consolidation of the concrete, especially in the vicinity of embedded steel, has a profound effect on corrosion. Insufficient consolidation provides channels of ingress for the ingredients of corrosion: water, chlorides, oxygen and carbon dioxide - and when occurring as "honey-combing" in the vicinity of the reinforcement promotes the formation of differential aeration corrosion cells. All these factors are discussed in greater detail in chapter 5.

2.3.5 Effect of Concrete Mixture Components

The primary effect of the aggregate on corrosion of embedded steel relates to its effect on permeability. All other factors being constant, permeability increases with increasing maximum size of the coarse aggregate for most mineral aggregate materials⁷⁰. This is due to the fact that most mineral aggregates have permeability coefficients 10 to 1000 times greater than the permeabilities of cement pastes in the usual range of w/c ratios⁷¹. Since aggregates typically constitute about 70% of the volume of concrete, it is evident that they play a major role in determining the permeability of concrete. This is especially true of

the Gulf situation where the aggregate is of marginal quality and has higher absorption.

The physical and chemical properties of cement are reported to be having some effect on steel corrosion. Fineness of the cement, for example, can influence corrosion to the extent that it affects bleeding and shrinkage. High alkali content can help to retard corrosion through its effect on pH. However, high alkali contents may lead to difficulties with alkali-aggregate reactions. In a practical sense the property of cement that has the greatest influence on corrosion is the tricalcium aluminate (C_3A) content²²⁷³. This factor is discussed in greater detail in chapter 5.

A wide variety of admixtures are incorporated into concrete mixes for various purposes including set retardation, set and early strength acceleration, water reduction, plasticizing, air entrainment. Most of them are organic materials that have no effect, per se, on corrosion. However, the water reducers, plasticizers and air entraining agents usually reduce w/c ratio and may therefore be somewhat beneficial in retarding corrosion. Calcium chloride ($CaCl_2$) is sometimes used as an accelerator, specially in cold weather concreting operations. The addition of chlorides in this form markedly accelerates corrosion.

The use of mix water in the Gulf region with a high content of chlorides (0.14 %) renders it a potential supplier of the chloride ion to the concrete thus enhancing the possibility of corrosion of reinforcement.

2.3.6 Influence of Thickness of Cover over Steel

Obviously the thickness of concrete cover over reinforcement is an important parameter in the mechanism of rebar corrosion; it provides the alkaline environment and acts as a barrier inhibiting ingress of chloride ions, oxygen, water and carbon dioxide. The imperviousness of the cover depends mainly on its thickness and on concrete quality. Since the chloride ion on entry into concrete reacts chemically to form partially insoluble calcium chloroaluminate, the effect of concrete cover is not a simple arithmetic relationship. This topic is discussed in greater depth in chapter 5.

2.3.7 Corrosion Models Based on Concrete Quality

The importance and the role of the various factors listed in the foregoing discussions can be best illustrated with the help of rebar corrosion models in concrete. These models are typical of the various physical characteristics of concrete. Figure 2.8 schematically represents a dense

and impervious concrete of low water-cement ratio, well compacted and well cured. Because of low permeability the rate at which oxygen, chloride ion and carbon dioxide can diffuse to the steel-paste interface will be very low. Also the electrical conductance within the paste system would be somewhat reduced, thereby further impeding the galvanic action. In addition because of the dense paste quality the number of capillary pores would be reduced and will be of smaller average diameter. Therefore, for any given, yet non-destructive stage of corrosion, the corrosion product would fill a larger number of capillary pores spreading over a relatively large distance from the corrosion site, which could slow diffusion of reactants and reaction products to and from the reaction site, thereby further stifling corrosion.

Figure 2.9 represents the opposite model - concrete of high w/c ratio, poor compaction and inadequate curing. Because of the porous nature of concrete, the destructive materials, chloride, oxygen, water and carbon dioxide can diffuse rapidly towards the corrosion site. Since the capillary pores are large in size and numerous the "postulated protective zone" is significantly reduced.

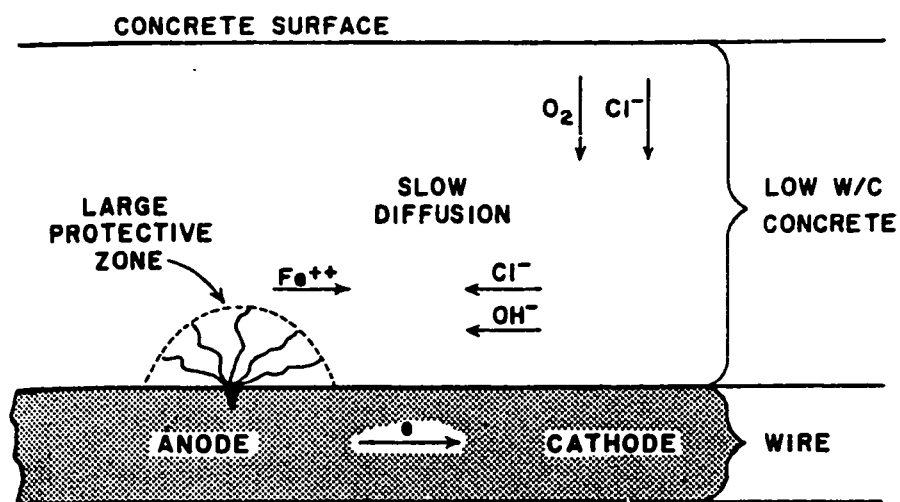


Fig 2.8 Schematic representation of corrosion in concrete of low water-cement ratio and well cured

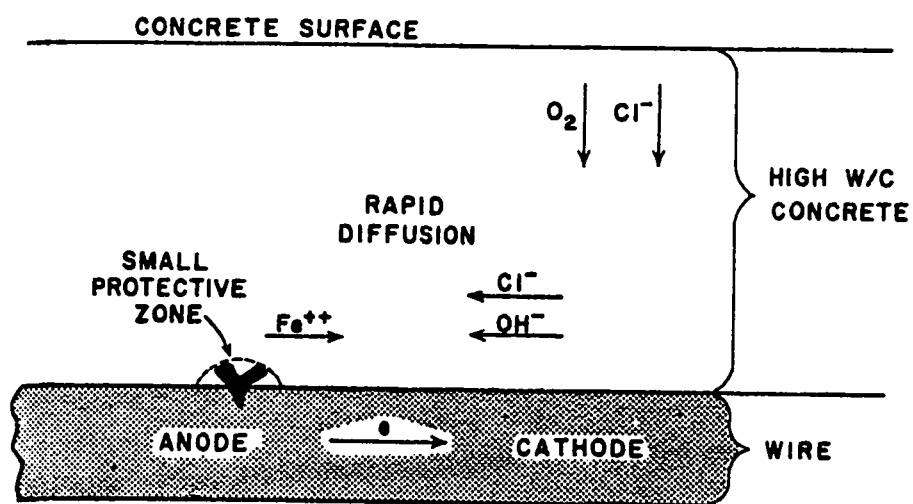


Fig. 2.9 : Schematic representation of corrosion in concrete of high water-cement ratio and poorly cured

The concept of "protective zone" is further emphasized in Figure 2.10. All conditions are similar to those shown in Figure 2.9 except for an air pocket trapped adjacent to the steel. In this case the reaction product is released into this pocket with little opportunity for the corrosion rate to be reduced by the development of the postulated protective zone. Monfore and Verbeck⁷⁶ in studying corrosion failure of prestressed concrete pipes observed that severe corrosion was most often observed at the site of such large voids adjacent to the steel wire. The author has also observed this phenomenon of highly accentuated corrosion in cavities adjacent to the steel reinforcement during his field studies. Plate 2.1 shows a typical illustration in practice of this model. Plate 2.2 is a yet another example which is photographed from the cores obtained from the author's laboratory samples.

2.4 MECHANISM OF CONCRETE CRACKING AND SPALLING

In field and lab investigations it was observed that the deterioration of concrete to the final state of complete and severe spalling progressed in four distinct stages which could be enumerated as follows :

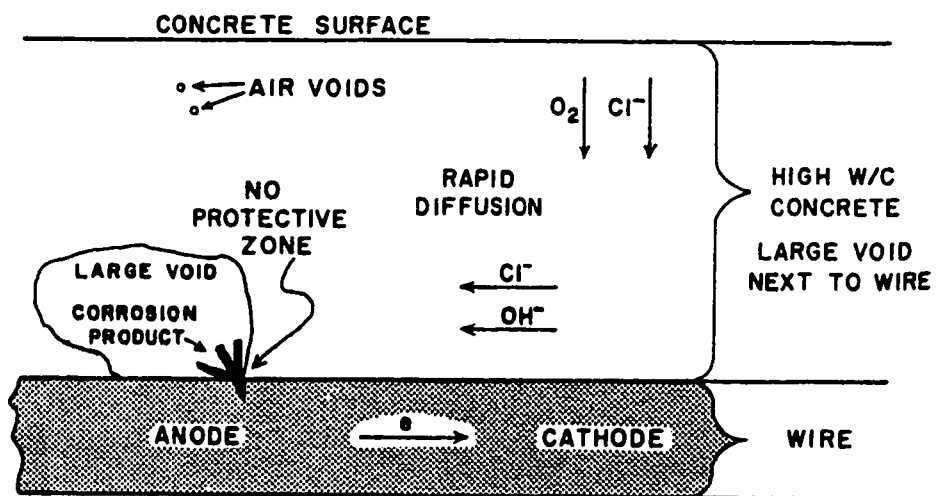


Fig. 2.10 : Schematic representation of corrosion in concrete of high water-cement ratio and with air pocket on steel

- Stage 1 : Appearance of a brown stain on the surface of concrete. At this stage no crack is visible and it appears that a brown coloured liquid has oozed out of the concrete at this spot and then evaporated leaving an iron stain.
- Stage 2 : Fine cracks appear near the stain and roughly parallel to steel underneath.
- Stage 3 : The cracks widen due to the expansion of the embedded steel when corroding until the cover concrete bursts under the tensile stresses.
- Stage 4 : The cover concrete spalls away leaving the reinforcing steel completely exposed.

Plate 2.3 shows the stages 1,2 and 3 where brown stains and fine and wide cracks are visible simultaneously as different stages of corrosion at different locations. Plate 2.4 shows stage 4 of concrete deterioration.

In columns and beams frequently cracks were found to develop along the reinforcement length (Plate 2.5) and were fairly wide. Plate 2.5 shows how concrete along the plane of reinforcement is pushed off to form a spall surface.

During the field studies the author noticed profuse spalling of concrete in structures 15-20 years old. In

almost all cases the spalls were a direct outcome of visible heavy steel corrosion and the concrete always spalled as large or small planes separating at the level of the reinforcement mat leaving the steel bars exposed. The explanation is that the volume of the products of corrosion is more than 4 times of the original metal and the resulting expansive tendency may develop tensile bursting pressure⁷⁴ within concrete as high as 4700 psi which far exceeds the tensile strength of concrete resulting in the fracture or rupture planes. Plate 2.6 shows the cracking along the bar in a reinforced concrete column in Dhahran area. The shattered concrete as a result of the expansive mechanical pressure can be clearly seen in the photograph.

The incidence, severity and the consequential manifestations of distress indicate that concrete spalling associated with advanced steel corrosion is undoubtedly the single most recurring and significant form of concrete deterioration noticeable in the Gulf area. This invariably leaves concrete in the affected area in crumbled form with significant loss of strength and cohesion and always leaves the steel reinforcement exposed. Corrective measures are difficult to adopt at this stage and the usual repair done by plastering the affected area with mortar is ineffective

with the spalling reappearing over the affected area within a short time.

Plate 2./a and Plate 2./b show a frequently recurring mechanism of spalling. Appearance of longitudinal and/or transverse cracks over the reinforcement mark the contours of a developing fracture plane and in a short time a plane of concrete as a whole scales off at the level of the reinforcement mat due to the expansive outward pressure exerted by the products of corrosion. This repeatedly observed mechanism gives credence to the existence of a plane of weakness at the level of the reinforcement mat⁷⁵. The rigid network of rebars blocks and retains at its level the heavier particles of fresh concrete at the time of normal sedimentation creating resistance for bleeding water to rise above this plane. This results in water gain in the immediate vicinity of rebar mat. High water content in this plane renders concrete porous and weak facilitating its enblock separation as a plane along the reinforcement mat. Such a plane of weakness would also form a potential porous area for the differential retention and diffusion of chloride-ions which penetrate to this level due to poor concrete quality and/or cracks. Any possible variations in the concentration of salt solutions in this plane would set up osmotic pressures

and electrochemical cells facilitating disruption and steel corrosion.

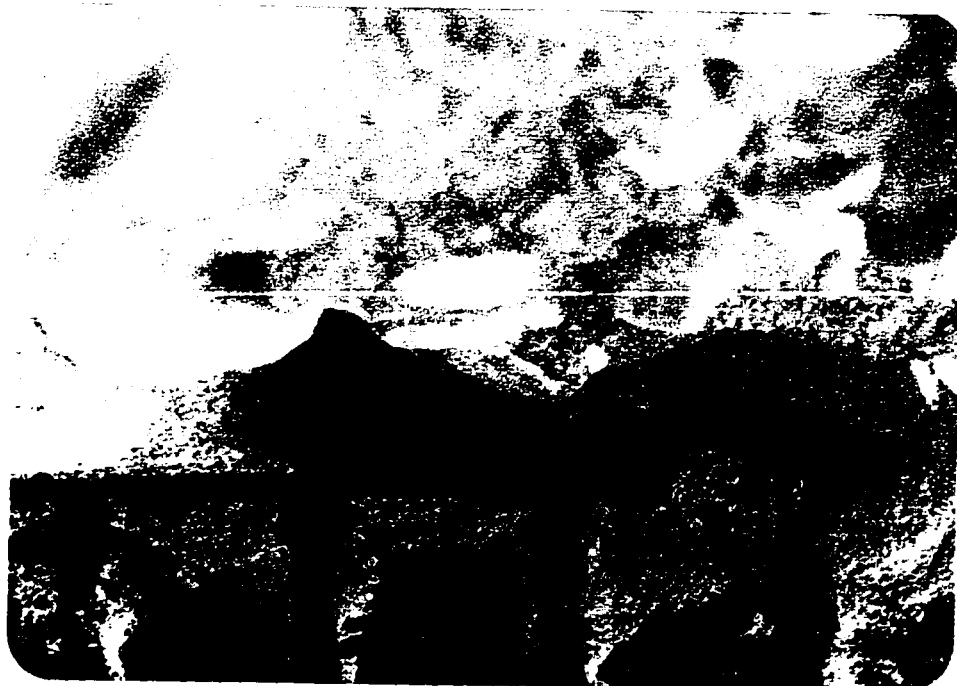


Plate 2.1 : Air pocket adjacent to the steel reinforcement enhances corrosion

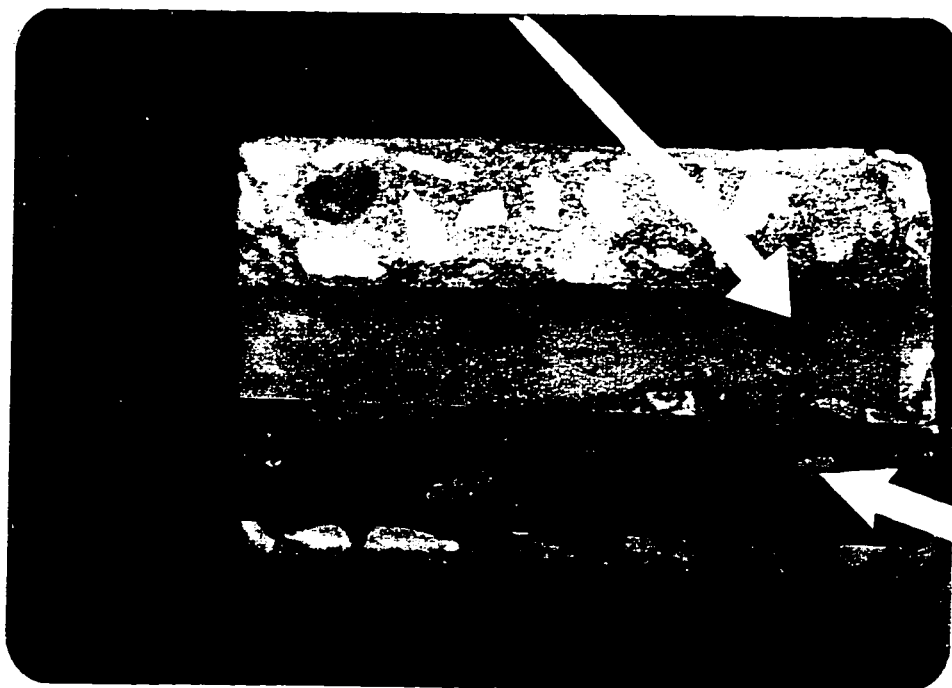


Plate 2.2 : Corrosion cell created at the airpocket adjacent to steel (laboratory sample)



Plate 2.3 : Brown stains, fine and wide cracks along the reinforcement

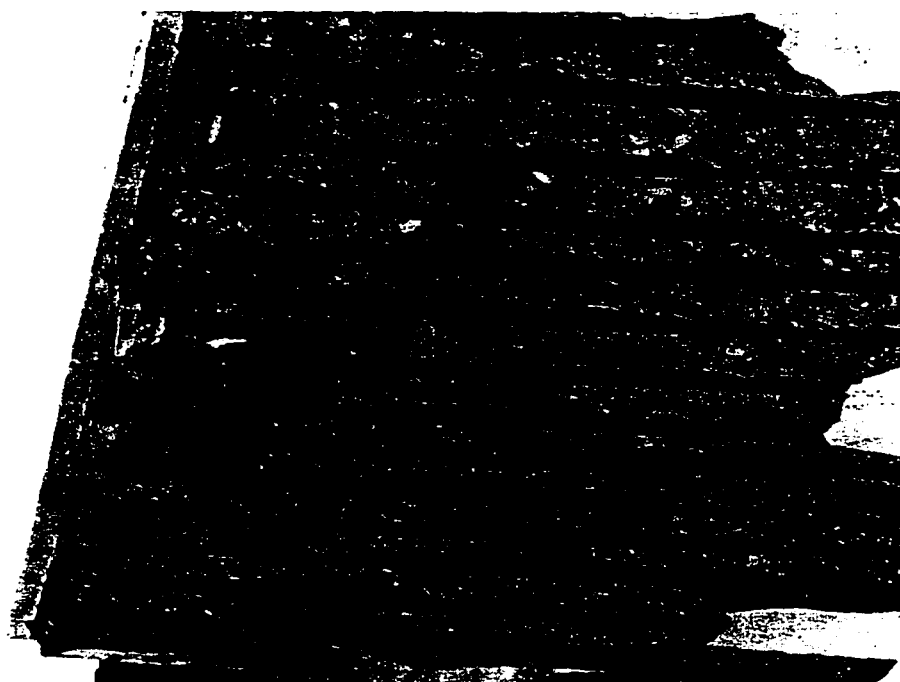


Plate 2.4 : Concrete cover spalled exposing steel reinforcement



Plate 2.5 : Cracks developed along corroded steel reinforcement



Plate 2.6 : Cracks and shattered concrete due to corrosion of reinforcement

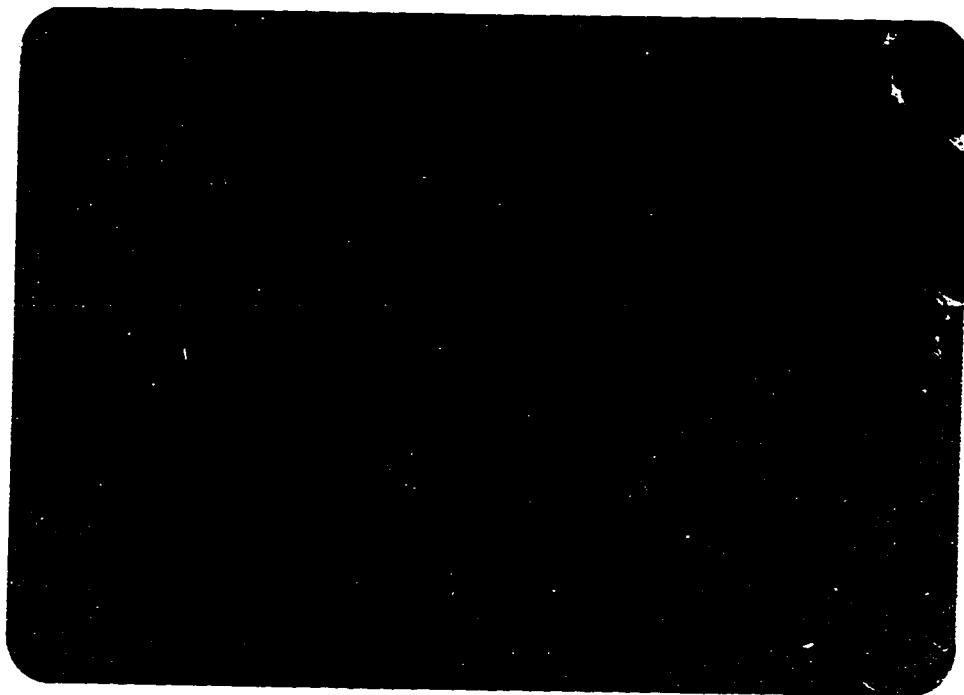


Plate 2.7(a) : Longitudinal cracks over reinforcement mark the contours of developing fracture plane

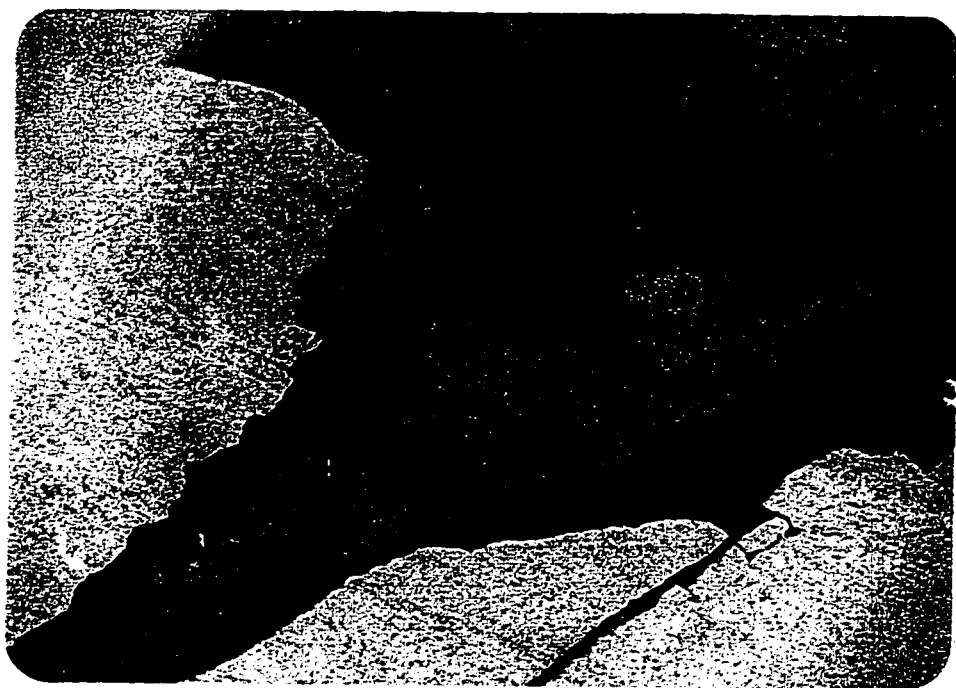


Plate 2.7 (b) : A close up of (2.8a) showing concrete scaling as a plane over the reinforcement mat

Chapter 3

METHODOLOGY OF INVESTIGATIONS

3.1 GENERAL APPROACH TO INVESTIGATIONS

In view of the significance of the field studies, this research was planned to be primarily based on collection, analysis and interpretation of field data from deteriorated field structures. However, selective laboratory investigations have also been undertaken to develop a better understanding of the operative parameters and their interrelationships with the mechanism of deterioration. The methodology adopted for these investigations is sequentially based on :

- (i) Condition surveys of selected concrete buildings in order to establish a reasonable overall picture of the extent and severity of concrete deterioration.
- (ii) Case studies related to special modes of deterioration employing destructive and non-destructive techniques as well as chemical examination of concrete samples obtained from the field structures.

- (iii) Analysis and interpretation of data.
- (iv) Selective laboratory investigations to develop a better understanding of certain related mechanisms and phenomena.

3.2 SELECTION OF FIELD STRUCTURES FOR SAMPLING

It was considered desirable to hold the age and the in-service exposure of structures reasonably constant in order to study the effect of other variables on steel corrosion and associated concrete spalling. Therefore, about 20 concrete structures of the age group 20-30 years were located in Dhahran-Dammam-Alkhobar areas which offered possibilities of extracting cores from roof slabs exposed on the upper surface and forming internal ceilings at the lower surface. Owners were approached for preliminary examinations to evaluate the possibilities of locating interesting areas from the viewpoint of corrosion deterioration. Some owners rejected sampling proposals, others agreed with restrictions regarding areas to be cored while some others were extremely helpful. Finally 10 buildings were chosen for coring which fell in the age group 22-27 years. All these buildings contained areas of corrosion deteriorations of varying degrees. Typical buildings from

which samples were removed are shown in Plates 3.1a,b.

3.3 CONDITION SURVEYS

The selected structures were condition surveyed and the observations during the inspection surveys were recorded in a survey proforma. The survey proforma was designed to record background information related to structure identification, construction type, site topography, exposure conditions, materials data and other relevant information about the construction techniques and the quality of workmanship and supervision. The proforma also recorded the general condition rating of the structure and the coring area on a five point scale, extent and type of deterioration. For the conditions of interesting areas, specially the coring areas, photographic documentation was invariably made.

Sampled area classification during coring process is as follows:

- A No visible cracks
- B No visible external cracks, cracks at the level of steel internally.
- C Visible external cracks
- D Severe external cracking associated with spalling
- F Severely spalled area as a result of heavy corrosion

Feedback at the initial stages of these surveys revealed difficulties in obtaining information on the initial mix design and other details of construction. Virtually no reliable information was forthcoming on constituent materials, compositional aspects of concrete and construction practices adopted. In the face of this difficulty initial concrete quality has been rated on a threepoint scale as poor, good, excellent on the basis of pulse velocity testing data obtained for these cores.

3.4 CORE DRILLING TECHNIQUE

A gasoline driven core drilling machine was used for extracting cores from interesting areas. A 3 inch diameter diamond bit was always used to obtain core size suitable for the 30-minute water absorption test according to British Standard BS 1881 Part 5 1970.

The core drilling machine was invariably carried to the roof and the bit was applied gradually with an ample supply of water to keep the temperature within limits. Plate 3.2 shows the core drilling machine.

Almost 60 cores were obtained from the exposed roof slabs of ten buildings. Plate 3.3 shows the cored areas and Plate 3.4 shows a typical core.

All the cores carried the reinforcement for a detailed analysis of the extent of corrosion and the physical and chemical characteristics of the surrounding concrete. Plate 3.5 shows a collection of some cores from which data regarding cover to reinforcement, loss of metal, chloride and sulphate contents at steel-concrete interface, absorptive characteristics, ultrasonic pulse velocity and pH value have been obtained. The relevant data is contained in the supplementary volume to this thesis.

3.5 TESTS FOR CONCRETE QUALITY

Permeability and porosity of concrete being the most direct characteristics bearing on corrosion mechanism, porosity of field samples was determined using the 30-minute water absorption test recommended by British Standards Institute and described in detail in BS 1881 Part 5, 1970. Length correction factors were applied for variations in the sample length. In the absence of suitable permeability testing equipment the absorptive characteristics of concrete were evaluated on the basis of its performance under the test. It may be pointed out that in similar investigations carried out by Stratfull and others on bridge deck concretes in the USA less accurate absorption tests were

adopted.

Ultrasonic pulse velocity measurements were made on some of the core procured from the field. 82 KHz transducers were used on opposite flat faces of the cores where they were applied with the help of a sticking grease. The time for the transmission of the pulse was noted on the digital display and an accurate measurement of the sample length between the transducers was obtained to calculate the pulse velocity.

Concrete ratings were determined on the basis of the following classification recommended by Whitehurst¹²³:

Pulse velocity classification ft/sec.

Excellent	above 15,000
Good	14000 - 15000
Questionable	10000 - 12000
Poor	7000 - 10000

• 3.6 CHLORIDE DETERMINATIONS IN CONCRETE

Chloride content being the most influential parameter in corrosion mechanism, the determination of the soluble chloride content in the vicinity of the reinforcing steel provides an indication of the reason and probability of

active corrosion. The acquisition of concrete samples for chloride analysis was made in the following manner. The cores were sliced using a high speed dry cutting saw machine at the level of the steel reinforcement (Plate 3.6). The steel bar was then removed and identified with the relevant core for metal loss determination. Several small samples of mortar weighing approximately 2-3 gms were removed from along the steel concrete interface using a small chisel and hammer. An attempt was made to obtain enough mortar samples along each bar length to enable several chloride determinations to be made to provide a representative value to be related with the metal loss data for the removed bar. Only 2 to 3 g sample was needed for each test.

Berman¹²⁴ has developed a reliable method for chloride determination which has been described in detail by Clear.¹²⁵ Briefly, it involves digesting a weighed quantity of the pulverized sample in nitric acid, boiling, filtering, and titrating the filtrate with silver nitrate solution of known normality. The titration end point must be precisely determined and this is accomplished by means of a potentiometric method utilizing a chloride ion specific electrode.

However Browne¹²⁶ and Bolling have evaluated this method and found that boiling as well as the use of silver nitrate solution for titration are redundant and make the analysis unnecessarily time consuming. They have suggested a rapid yet accurate method which was adopted in these investigations. The author made at least forty chloride determinations using the titration and the simplified procedure suggested by Browne and Bolling and found no differences in the determinations. In view of this and the large number of chloride determination involved the author used Orion 407A specific ion meter with solid state specific chloride ion electrode and double junction reference electrode to measure the water soluble chlorides from a solution wherein the mortar was dissolved and kept soaking in distilled water for 48 hours. Proper calibration techniques were used with the meter for zero adjustment and preparing standards.

3.7 SULPHATE AND pH DETERMINATIONS IN CONCRETE

Sulphate content in the concrete mortars was determined in accordance with ASTM C114. Sulphate determinations are expressed as % SO_3 .

For the determination of pH 1 gm of mortar sample was added to 20 ml of distilled water resulting in a 5% suspension which was kept for 24 hours. pH meter was standardized with Buffer solution and a direct reading for the pH value was obtained from the Bekhman pH meter.

3.8 LOSS OF METAL DETERMINATION FOR REINFORCEMENT

Steel bars removed from the cores were cut off neatly at the two opposite faces in the machine shop to obtain exact lengths. These bars were labelled with steel markers and their original diameter identified. The bar length which varied in size were then dipped in concentrated hydrochloric acid for approximately 15-20 minutes. Subsequently these bars were washed with water and then dried. These were then weighed and the percent loss was calculated as :

$$\% \text{ loss} = \left(\text{original wt/inch} - \frac{\text{weight of the bar}}{\text{length of the bar}} \right) \times 100$$

3.9 LABORATORY INVESTIGATIONS ON THE EFFECT OF CHLORIDE CONTENT ON CORROSION

In order to investigate the effect of chloride content on corrosion of rebars embedded in a rather poor quality concrete, small slabs 12" x 8" x 1½" were made from a

1.2 : 2:3 mix using a w/c ratio of 0.65. One ϕ 12 plain bar was embedded in each slab with a clear cover of $\frac{1}{2}$ inch.

Chloride contents of 0, 3.1, 6.2, 12.4 and 24.8

NaCl gm per litre of mixing water were added to each set consisting of three slabs. These samples were cured for 7 days and then were stored in air for observations of relative corrosion severity.

Corrosion activity was monitored daily using a Cu-CuSO₄ half cell (described in section 3.11). Half cell potential readings were obtained at three locations and average values were plotted to give a representative activity along the length of the bar.

3.10 LABORATORY INVESTIGATIONS ON THE EFFECT OF CEMENT TYPE ON CORROSION ACTIVITY

12" x 8" 1 $\frac{1}{2}$ " slab samples were made using Type I (C₃A 9%) and Type V (C₃A 2%) cements to monitor corrosion activity on embedded ϕ 12 bars. The mix used was 1.2 : 2 : 3 with a w/c ratio of 0.65. The 28-day strength of the concrete was found to be 2700 psi. 6.2 NaCl gm/l of sodium chloride was added in the mix water to create a corrosive environment in which to evaluate the corrosion response of these two frequently used cement types in the Gulf region. The

corrosion was monitored with the help of Cu-CuSO₄ half cell which recorded readings daily for the first 10 days, then every three days for next 10 days, then every week.

3.11 MONITORING CORROSION ACTIVITY WITH COPPER-COPPER SULPHATE HALF CELL

As discussed previously, the process of corrosion of steel in concrete involves the formation of galvanic corrosion cells due to the electrochemical processes of oxidation at anodic areas and reduction at cathodic areas on the embedded steel. Therefore, corrosion activity can be identified by its effect on the electrical potential of a reference electrode (half-cell) inserted in the galvanic corrosion circuit. A number of reference electrodes are suitable for evaluating corrosion of steel in concrete, but either the copper-copper sulphate (CSE) or the saturated calomel (SCE) half-cell are most commonly used. By convention, potential readings are taken to be negative values.

The readings are only statistically related to corrosion activity. Potential readings more negative than -0.35 V relative to the copper-copper sulphate half cell (CSE) have been found to be indicative of active corrosion 95 per cent of the time. Potentials in the range -0.20 to -0.35

V (CSE) are inconclusive as regards corrosion, and readings less negative than -0.20 V (CSE) are considered to be indicative of passive conditions.

The Cu-CuSO₄ half cell circuitry is shown in Figure 3.2. The half cell was fabricated in accordance with the requirements of the ASTM Test for Half Cell Potentials for Reinforcing Steel in Concrete (C876). The electrical junction device used was a sponge pre-wetted with low electrical resistance contact solution. The cell consisted of a rigid glass tube which is non reactive with copper or copper sulphate, a porous sponge plug which remains wet by capillary action and a pure copper rod immersed within the tube in a saturated solution of copper sulphate. The solution was prepared with reagent copper sulphate crystals dissolved in distilled deionized water. This solution was kept saturated with an excess of crystals lying at the bottom of the solution. The voltmeter used for the measurement of potentials was a very sensitive apparatus. The whole apparatus is shown in Plate 5.7.

3.12 CHLORIDE PENETRATION TEST

Chloride diffusion characteristics in concrete matrix were determined on some samples removed from a column

subjected to an intermittent splash of salt water. A core was sliced obtaining $\frac{1}{4}$ inch, $\frac{1}{2}$ inch and 1 inch slices in the manner shown in Figure 3.1. Mortar was removed from the faces of these slices and subjected to chloride determination in the manner explained in section 3.6. These determinations when plotted against the depth of the core provided the chloride migration data in cracked and uncracked concrete samples.

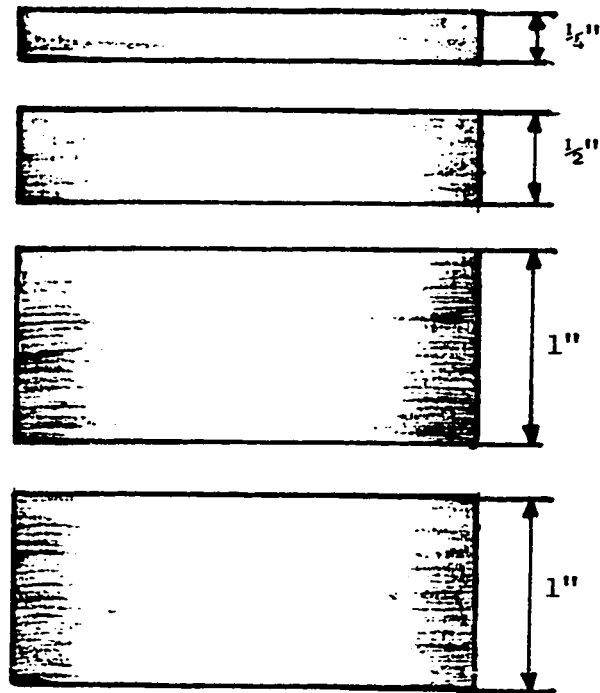


Fig.3.1 : Schematic diagram shows slices for chloride penetration test

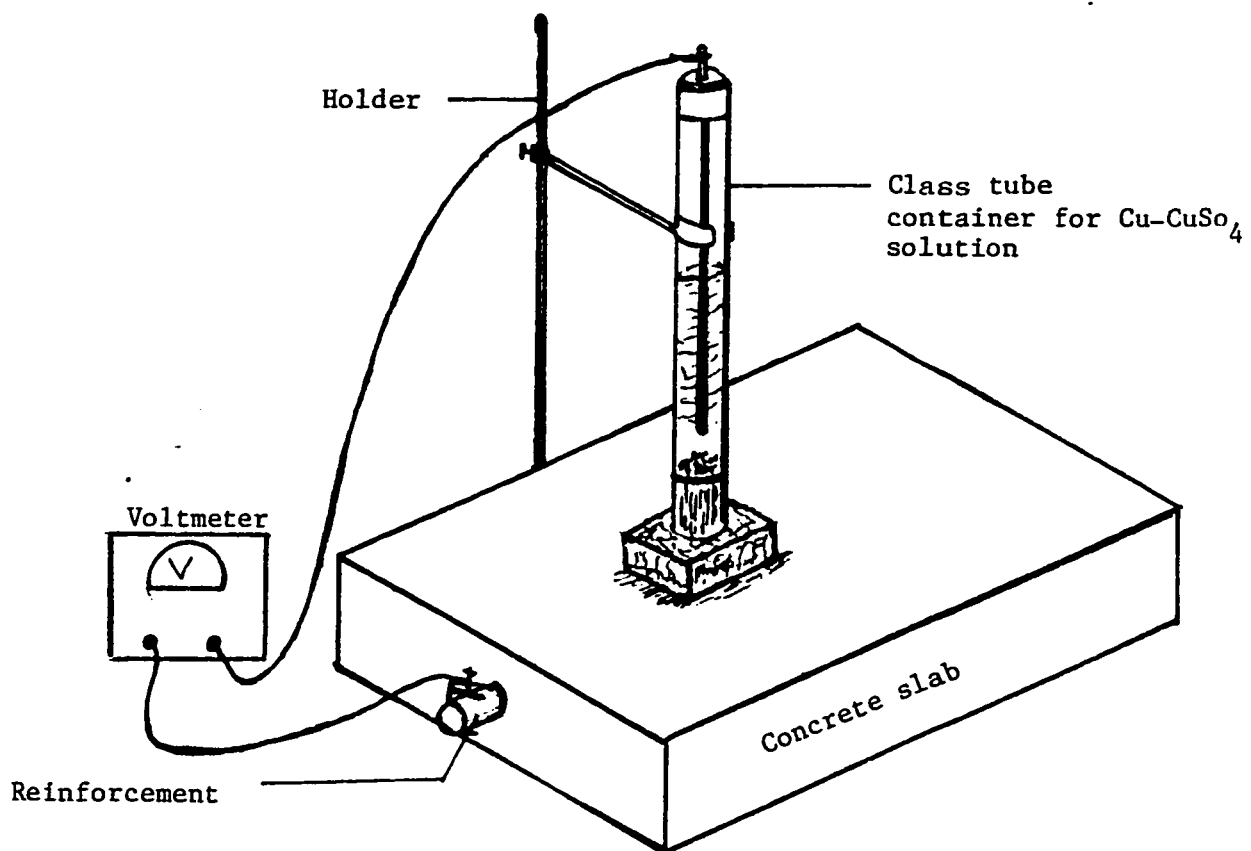
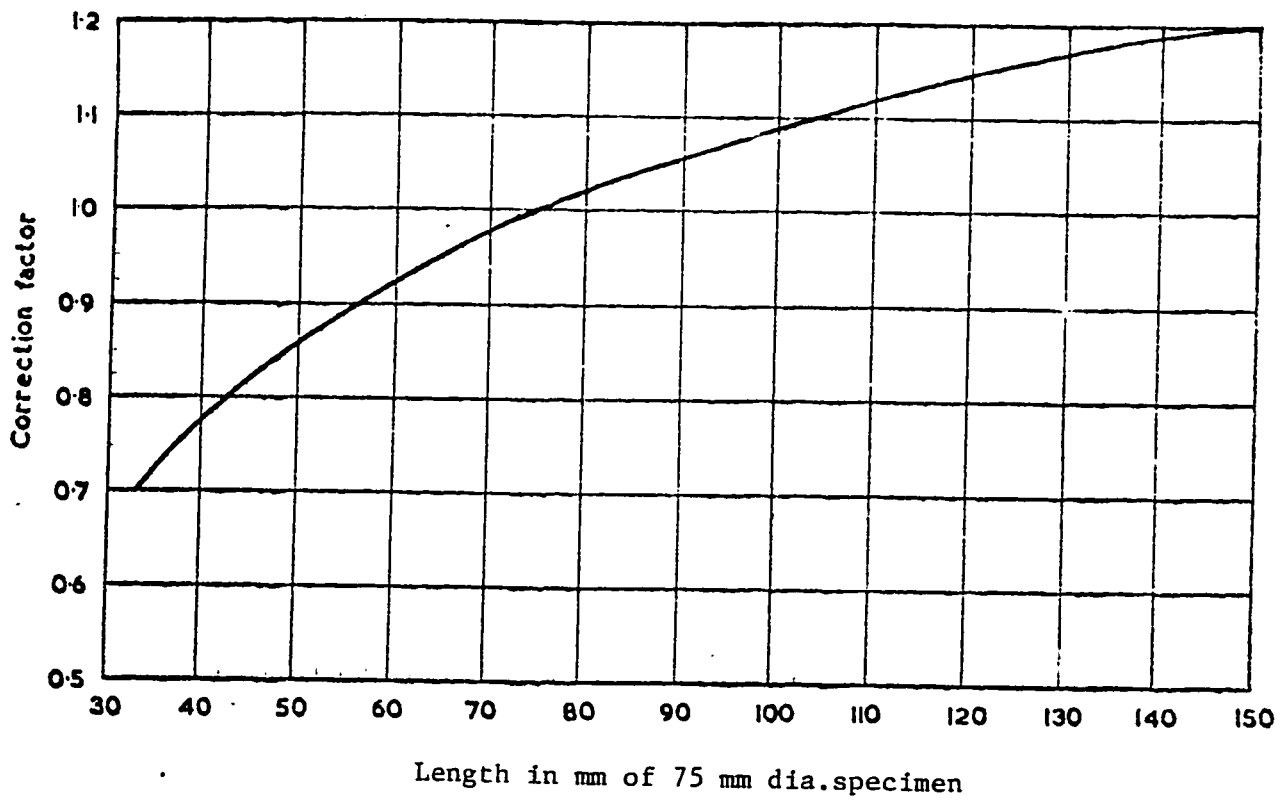


Fig. 3.2 : Copper-copper sulphate half cell constructed according to ASTM C876



Correction Factor for Water Absorption Test

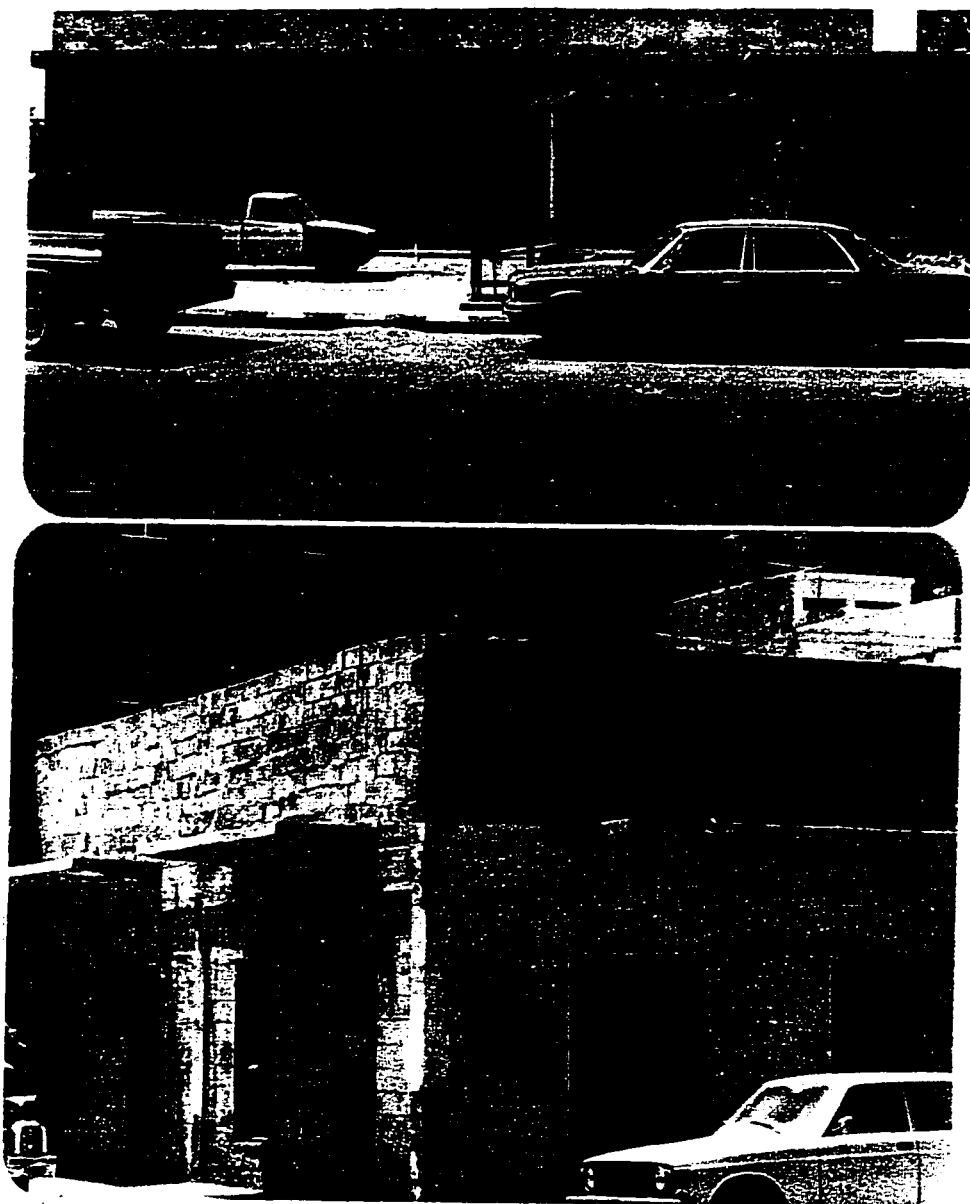


Plate 3.1a :Typical sampled buildings

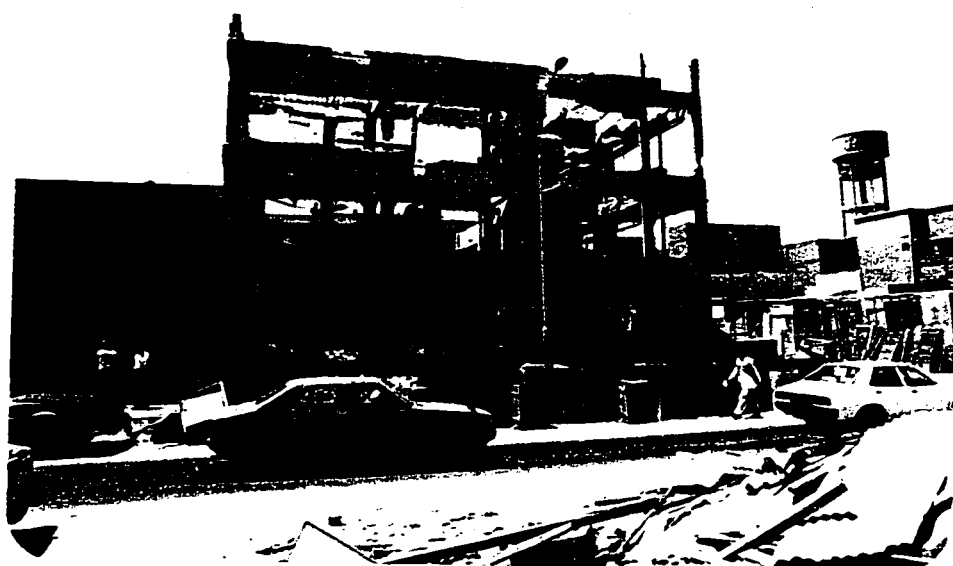
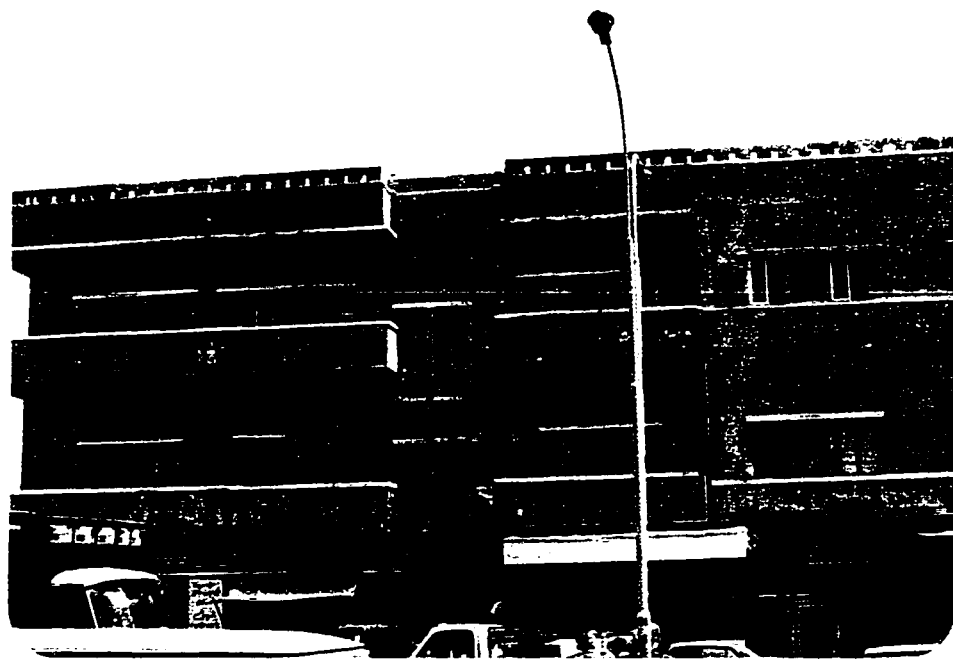


Plate 3.1b : Typical sampled buildings

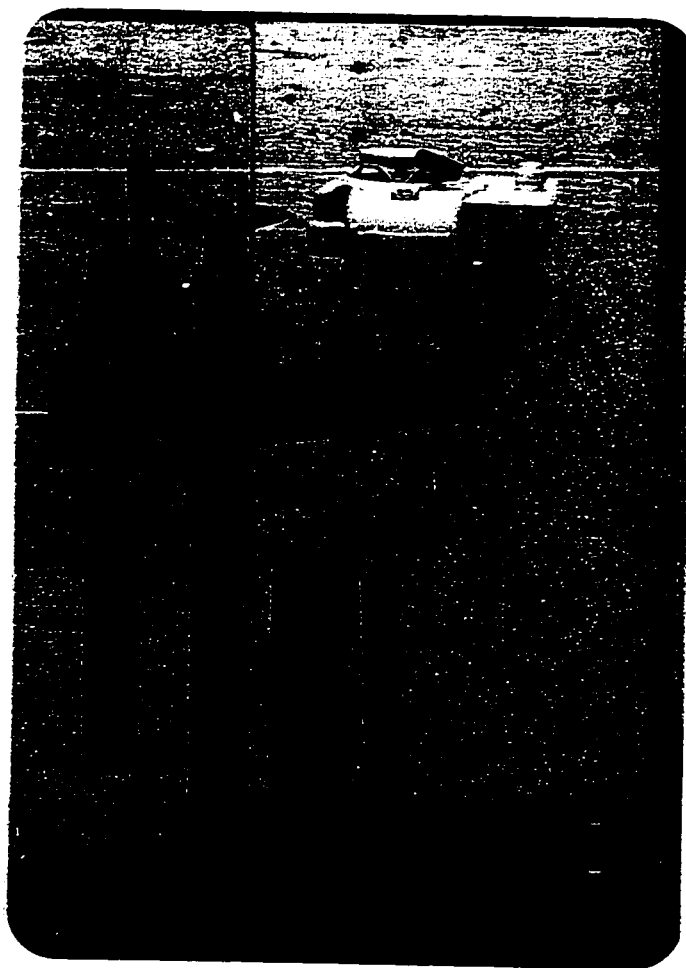


Plate 3.2 : Core drilling machine with diamond bits

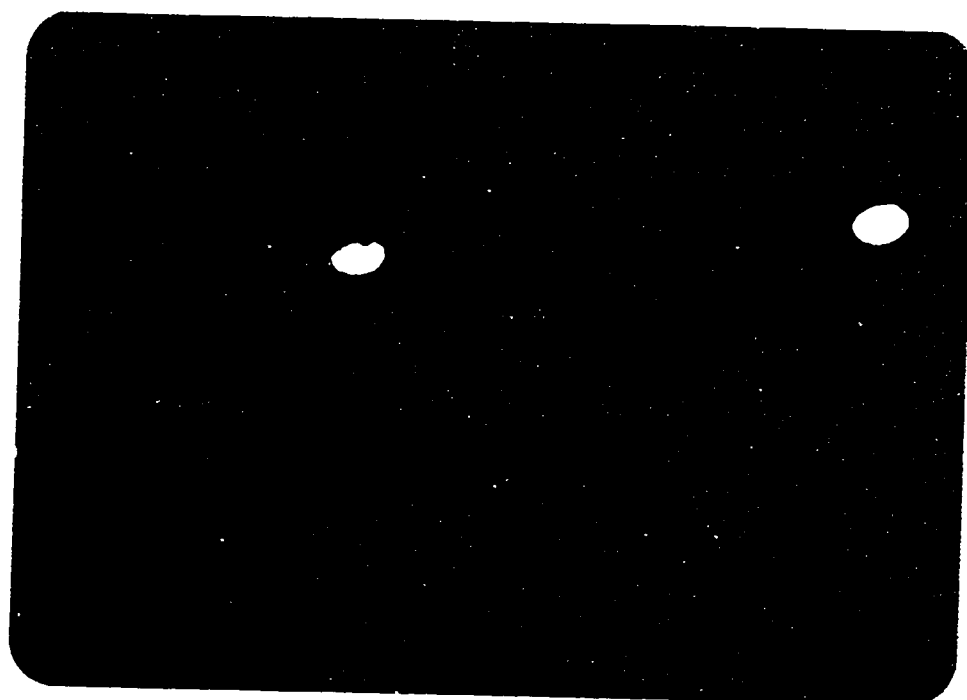
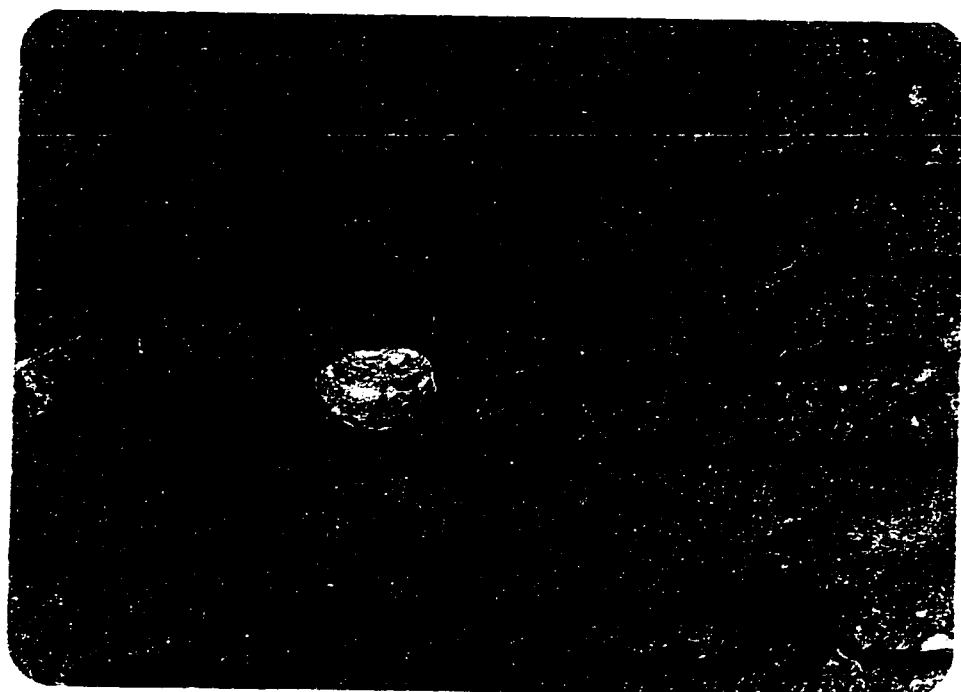


Plate 3.3 : Typical slabs with cores removed

Plate 3.4: Typical core obtained
from slabs

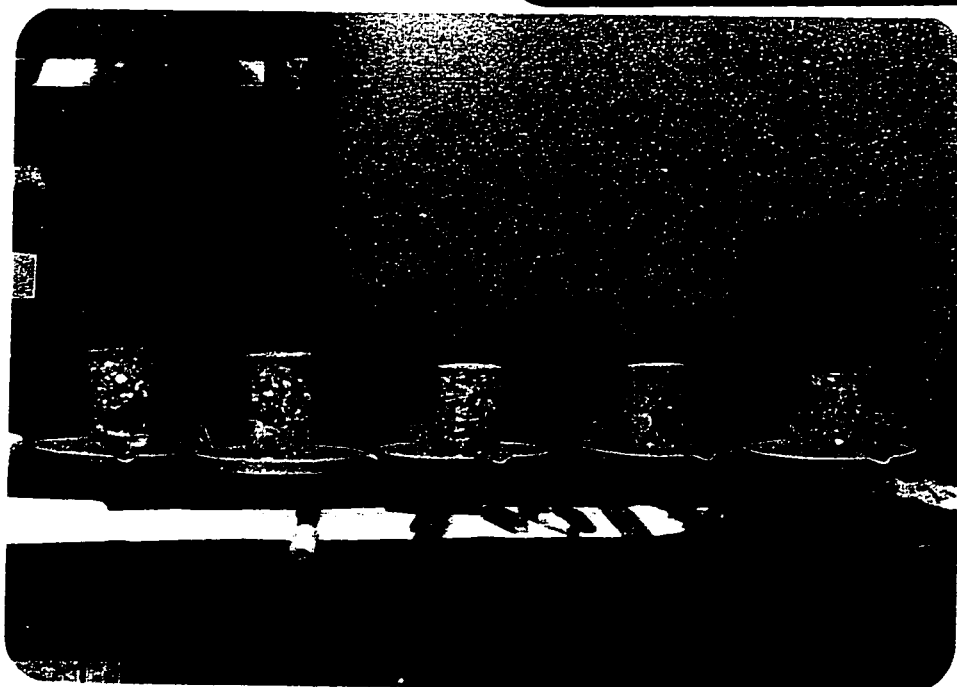


Plate 3.5 : Some cores for laboratory testing

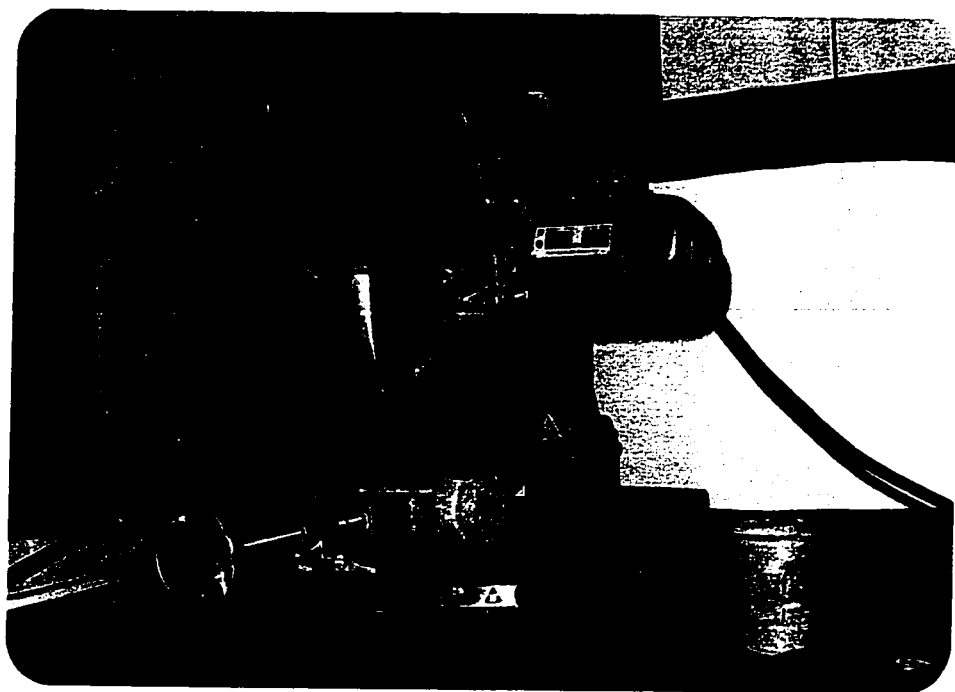


Plate 3.6 : Dry cutting and slicing machine

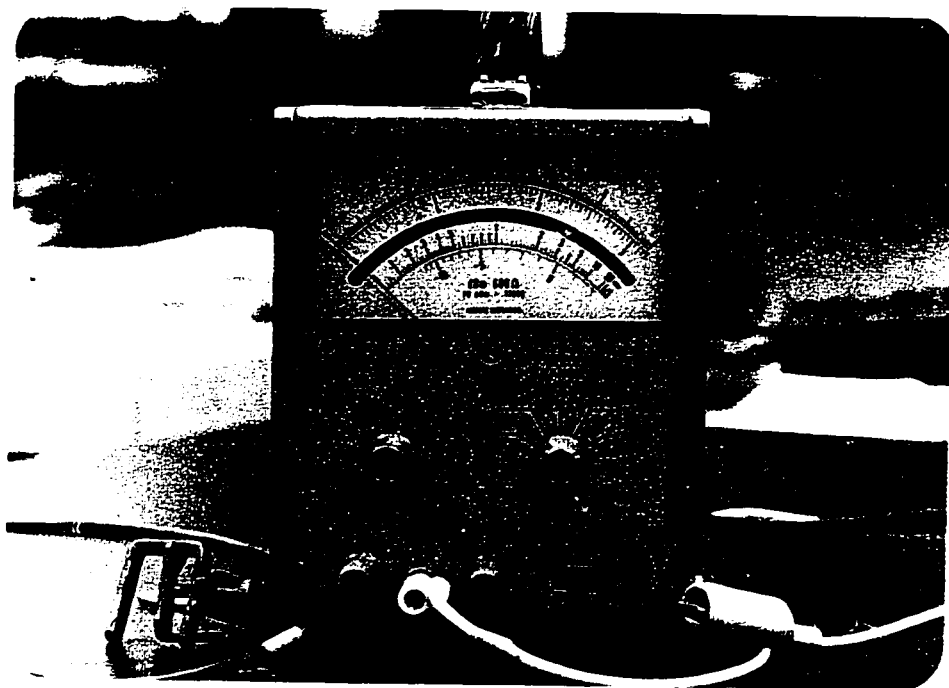


Plate 3.7 : Voltmeter used for measuring half cell potential volts

chapter 4

INFLUENCE OF CHLORIDES ON CORROSION

4.1 INTRODUCTION

While atmospheric carbonation is usually discounted^{76,77,78,79} as a serious cause of steel corrosion in ordinary structural concrete, the chlorides possess a distinctly marked potential for disrupting the passivity of steel in the alkaline concrete system. Baumel and Engell⁸⁰ have shown that in a saturated calcium hydroxide solution chloride concentration even smaller than 0.0035 percent has some adverse effect on steel passivity and it is by now acknowledged that, when the chloride ion is either mixed with or absorbed by concrete,^{8,1} this ion is primarily responsible for the overwhelming majority of the reported cases of corrosion of steel in concrete. Concrete construction in the coastal States of the Arabian Gulf is continually exposed to ground and atmosphere charged with salt, and aided by capillary action and high humidity conditions the salt contaminated ground water and the salt-laden airborne moisture and dew find an easy ingress in the concrete matrix; further the salts also pollute the mix water

and the aggregates thereby increasing the total salt content of the concrete environment in the Gulf area. The unusually high incidence of corrosion against this backdrop of a highly salt-polluted environment puts chloride ion as the most important cause of steel corrosion in this region.

There is virtually no organised and properly formulated field data from any part of the world which would provide some sort of a quantitative correlation between the chloride content and the extent of corrosion in a reasonable range of variations of these two parameters. This is because of the fact that there are several variables involved and to plot a correlation between chlorides and corrosion, the other influencing variables, such as concrete age, in-service environment, concrete quality and the concrete cover should be held reasonably constant. Data collection from field structures in such a format is usually extremely tedious. And yet the significance of such data can hardly be overemphasized. Only such information can provide a meaningful indication of the field performance of actual structures and will build up a useful storage of pragmatic and reliable estimate of concrete deterioration in the field under interactive causal factors. This information will act as the most reliable feedback for the formulation

of design and practice recommendations for concrete durability in the aggressive environment of the Gulf.

Because of these factors this study was deepened to present a reasonable interrelationship between chloride content of concrete and the extent of embedded steel corrosion. The objective of obtaining meaningful data for this correlation presented with the difficulty of obtaining concrete samples from field structures which are characterized by similar age, in-service environmental exposure, concrete quality and concrete cover to steel. A large number of buildings in the age range of 22-27 years were condition surveyed for selecting a sample population which could provide the requisite conditions. Finally concrete cores from slabs of 10 structures located in Dammam-Alkhobar habitations were obtained for a laboratory analysis. These samples were analyzed for chloride/sulphate contents at the steel concrete interface, metal loss, cover and pH values. In view of the acknowledged significance of the absorptive and moisture movement characteristics of concrete as an index of its vulnerability or otherwise in promoting or inhibiting rebar corrosion, concrete water absorption data was also collected for these samples using the 30-minute water absorption test (BS 1881 : Part V : 1970). The

technique used for chloride determination was refined over the existing usual methodology and extreme care was taken in determining the metal loss. A detailed description of these techniques is included in chapter 3. Lab investigations were also carried out to provide supportive data to field studies related with chloride vs. corrosion relationship.

In very many situations at least part of the chloride which causes steel corrosion permeates concrete from the environment. Two studies were carried out on cracked and uncracked field samples to investigate the diffusion characteristics of the chloride ion as it migrates into the concrete matrix.

From each of the large number of field samples steel bars were removed for close observation of the nature of corrosion and such condition studies were extended to other field structures also where reinforcement corrosion was visible. Extensive photographic and descriptive documentation has been built up from these condition studies.

4.2 ROLE OF CHLORIDES IN CAUSING CORROSION OF STEEL EMBEDDED IN CONCRETE

Before examining and evaluating the evidence of chloride

corrosion, it is useful to recall the various ways in which the chloride ion influences the promotion or inhibition of the corrosion mechanism in concrete. Although this topic has been briefly discussed in section 2.3.2 a broader discussion is called for in view of the importance of the phenomenon and its concentrated treatment in this chapter. The chloride ion has a multi-pronged influence in promoting corrosion.

Firstly, the presence of chloride ions* depresses the pH value of the cement pore fluid^{8 28 38 4}. Kurczyk and Schwietz^{8 4} have shown that without the addition of CaCl_2 the alkalinity of the aqueous phase in hydrating cement is represented by pH values of 12.31, 12.38, 12.41 and 12.43 at 1,2,3 and 4 hours respectively. The corresponding values decreased to 11.67, 11.77, 11.85 and 11.91 in the presence of calcium chloride. The effect of salts and chlorides on the pH values of calcium hydroxide solutions is also shown in Figure 4.1. Figure 4.2 shows a schematic presentation of the effect of both salt concentration and pH in a $\text{Ca}(\text{OH})_2$ solution. As the concentration of the salt increases, the protection afforded by the high alkaline solution ($\text{pH} = 12.6$) is reduced and rust development takes place at an increased rate. The reduction in pH tends to shift the steel of the

* It may be clarified that calcium chloride decreases the pH value of the cement pore fluid whereas the sodium chloride has the reverse effect of increasing the pH value.

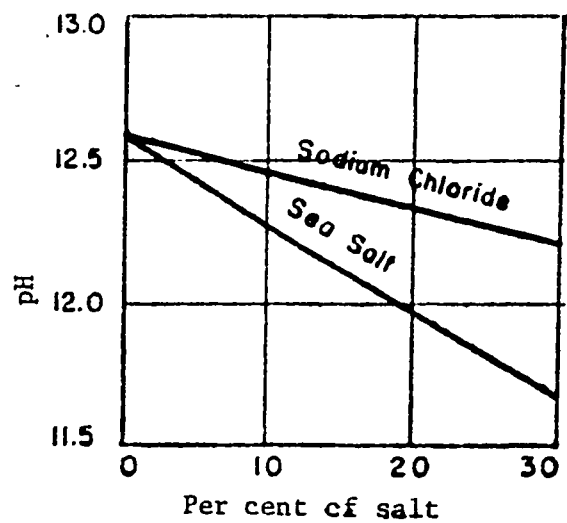


Figure 4.1 : Effect of salt concentration on pH values

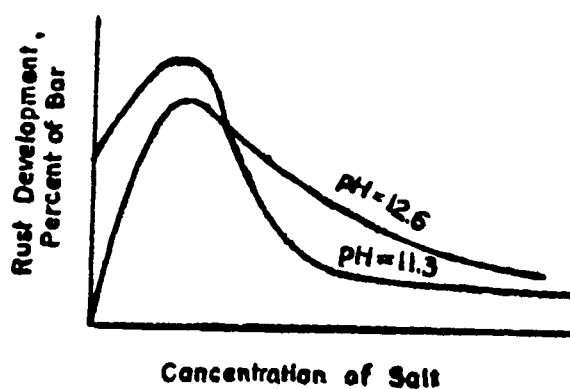


Figure 4.2 : Schematic presentation of the effect of salt concentration and pH in Ca(OH)_2 solution

reinforcement from the region of immunity to the region of active corrosion. This is explained by Figures 4.3 and 4.4 which show that for a metal immersed in an electrolyte solution, the likelihood of corrosion depends on the pH of the solution and the electrical redox potential of the metal. Figure 4.3 shows how the pH of a solution has a bearing on corrosion rates. Each of the three diagrams shown in Figure 4.3 represents a group of metal whose corrosion products exhibit similar solubility and protective film formation characteristics. Figure 4.3a represents those metals whose oxides are soluble in acids as well as alkalies (zinc, aluminum, lead, tin etc.). The pH value at which minimum corrosion occurs varies with the metal as indicated by the following examples : Al = 6.5, Pb = 8.0, Sn = 8.5 and Zn = 11.5. Figure 4.3b represents those metals whose oxides are easily soluble in acids but are insoluble in alkalies. Included in this category are nickel, cobalt, copper, chromium etc. and also iron. Lastly, Figure 4.3c represents the noble metals that are usually quite stable in acids and alkalies (gold, platinum, silver etc.).

Figure 4.4 which is a plot for iron or carbon steel between redox potential and the pH is known as Pourbaix plot after Professor M. Pourboux, the originator of this method

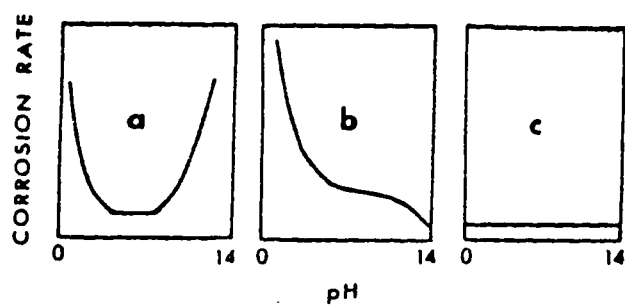


Figure 4.3 : Principle types of diagrams showing the relationship of corrosion rate of pH

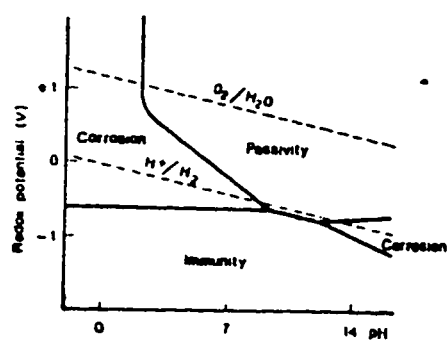


Figure 4.4 : Simplified pourbaix diagram for iron

of representation⁸⁵. This diagram shows the region in terms of redox potential and pH where the iron or carbon steel is in a state immunity, passivity or active corrosion. This diagram shows that the redox potential for the hydrogen electrode lies above the region of immunity for iron in both acid and alkaline solutions, implying that iron will dissolve with evolution of hydrogen in solutions of all pH values. However, in the pH interval 9.5 to 13.0 a layer of ferrous hydroxide forms on the metal surface thus conferring on steel immunity from corrosion in alkaline solutions in this range. It can be seen that the normal pH value for concrete (12.5) is included in the steel passivity region, demonstrating the natural corrosion protective action of Portland cement concretes. However, if the chlorides in conjunction with carbonation could decrease the pH value below 10, the steel would at once come in the active corrosion zone. Also, the integrity and protective quality of the passivating oxide film depend upon the alkalinity or pH of the environment. The lower the alkalinity or pH the weaker the protective quality of the passivating film⁸⁶.

Secondly, the chloride ions destroy the passivating film by converting the ferric oxide film to soluble ferric chloride and thereby releasing ferrous ions at the anode⁸⁷.

Thirdly, the accumulation of chloride salts along the steel-concrete interface being highly variable, these salts set up corrosion cells of the differential salt concentration type creating conditions for electrical potential to be set up giving rise to a large number of galvanic cells along the rebar length. Corrosion due to differential salt concentration has been dramatically illustrated by the distress of the San Mateo-Hayward Bridge in California;⁸⁸ built during the 1930's using a lean concrete, it absorbed salts from the sea spray. The concentration of these salts varied in different sections of the same members, due to the slightly different porosity, and this has allowed corrosion cells to operate. Extensive spalling of the concrete cover (1½ in) had occurred and the whole structure had to be repaired progressively with a dense gunite layer.

Fourthly, chloride salts, being strong electrolytes, increase the electric conductance of concrete. Investigations at San Mateo-Hayward Bridge showed⁸⁸ that the corrosion deterioration of concrete is almost a direct function of the concrete resistivity to electric current flow. Figure 4.5 depicts the resistivity vs. concrete deterioration relationship for the San Mateo-Hayward Bridge showing the rapid increase of deterioration with a reduction in

concrete resistivity.

Fifthly, the addition of calcium chloride in concrete increases the porosity of cement paste which enhances the ingress of aggressive media, specially the supply of moisture and oxygen, thereby enhancing the corrosion process. Porosity and pore size distribution changes which occur in the structure of the hydrated cement are well documented.^{8 9 9 0 9 1 9 2 9 3 9 4 9 5 9 6} Figure 4.6 compares the pore size characteristics of hydrated cement, formed at a water/cement ratio of 0.5 and hydrated for 7 days, with that containing 2% CaCl_2 . The substantial increase in porosity of CaCl_2 treated samples is due to the existence of a greater amount of smaller pores of size $10\text{-}50\text{\AA}$ ⁹².

Sixthly, chlorides increase the drying shrinkage which usually results in hairline cracking acting as a forerunner and feeder to rebar corrosion. Table 4.1 shows the drying shrinkage occurring in concrete containing 2% CaCl_2 , cured for 14 days and exposed to a relative humidity of $50 \pm 3\%$ for the periods indicated⁹⁷. The figures in brackets refer to the percentage increase in shrinkage compared with plain concrete. These results suggest that CaCl_2 produces significantly greater shrinkage than the

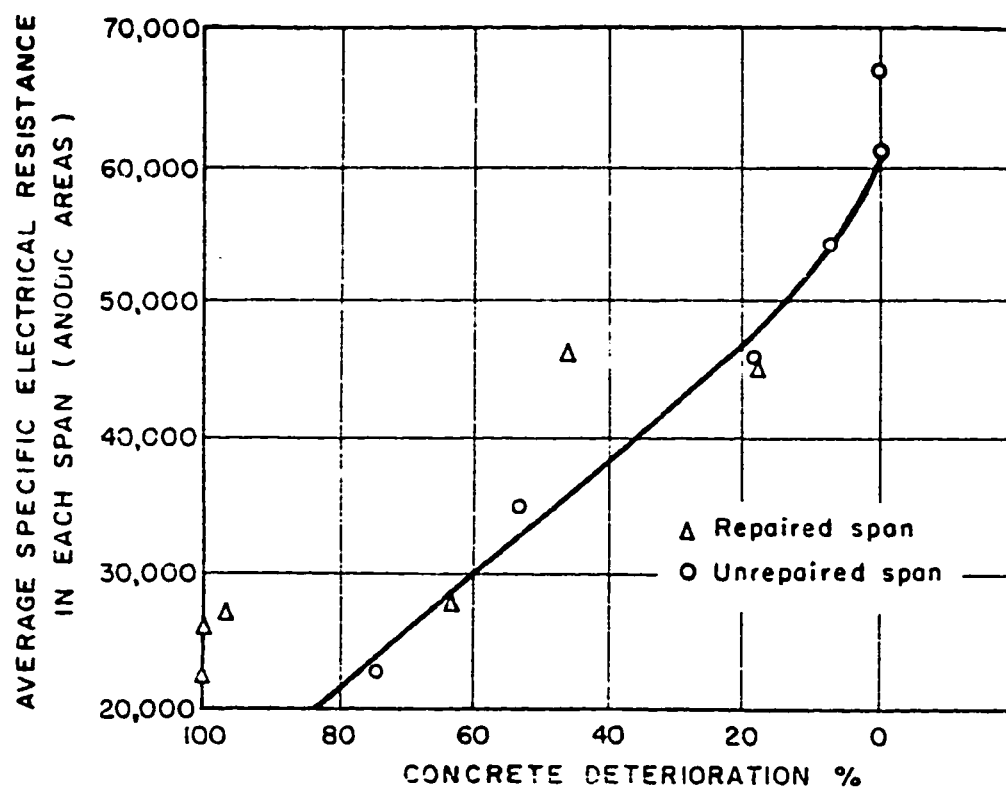


Fig. 4.5 : Specific electrical resistance against concrete deterioration

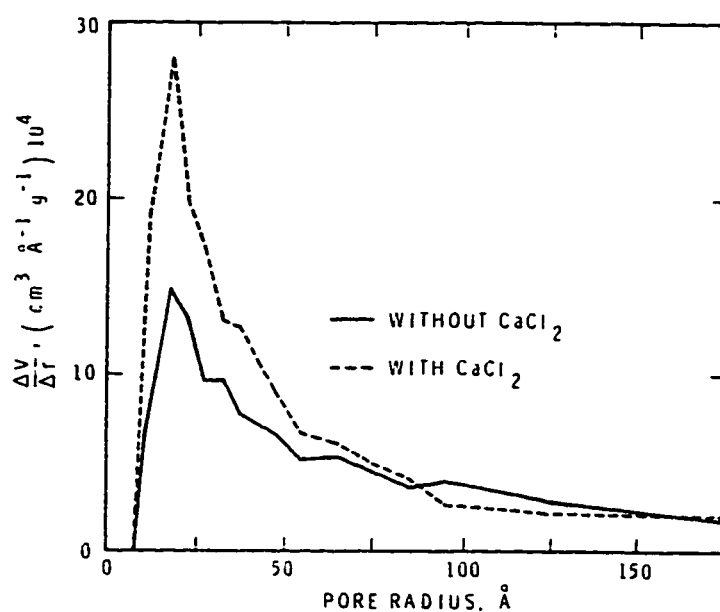


Fig. 4.6 : Pore-size distribution curves for portland cement paste hydrated for 7 days.

TABLE 4.1
Drying Shrinkage of Concrete Containing CaCl_2 ²⁷

<i>Admixture</i>	<i>Drying shrinkage (%) at varying periods of drying</i>					
	<i>7 days</i>	<i>14 days</i>	<i>28 days</i>	<i>56 days</i>	<i>84 days</i>	<i>168 days</i>
None	0.010	0.018	0.027	0.039	0.045	0.052
CaCl_2 (2%)	0.018 (80)	0.028 (55)	0.037 (37)	0.048 (23)	0.054 (20)	0.062 (20)
Ca lignosulphonate (0.25%)	0.015 (50)	0.023 (27)	0.032 (19)	0.044 (13)	0.048 (7)	0.049 (-6)
Ca lignosulphonate (0.25%) + CaCl_2 (0.10%)	0.018 (80)	0.024 (33)	0.034 (26)	0.044 (13)	0.048 (7)	0.053 (0)

plain concrete at all ages of drying.

However, the presence of salts in abnormally large quantities has been found to have a reverse effect on corrosion. Figure 4.7 shows that at high concentrations salts lower the corrosion rate. This is usually ascribed⁹⁸⁹⁹¹⁰⁰ to diminished oxygen solubility and the precipitating action of the negative salt radicals at high salt concentrations. The effect of salt concentration on oxygen solubility is shown in Figure 4.8 . However, the more influential parameter of the precipitating action of the negative salt radicals seems to have very little influence in the case of chlorides. The negative ion valencies reflect the precipitating powers of the salts. The relative precipitating powers of various negative ions have been investigated and found to have the following ratios¹⁰¹.

Chromates and bichromates	1000
Sulphates	670
Chlorides and nitrates	3

Since chlorides exhibit a low precipitating power as shown in the above table, a relatively high concentration of such salts is required for corrosion to be retarded. Therefore, the influence of chlorides is usually to enhance the

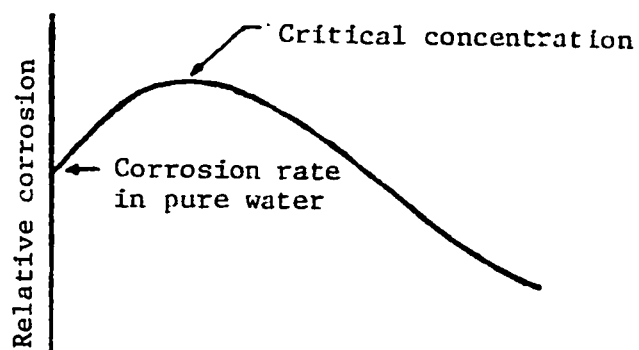
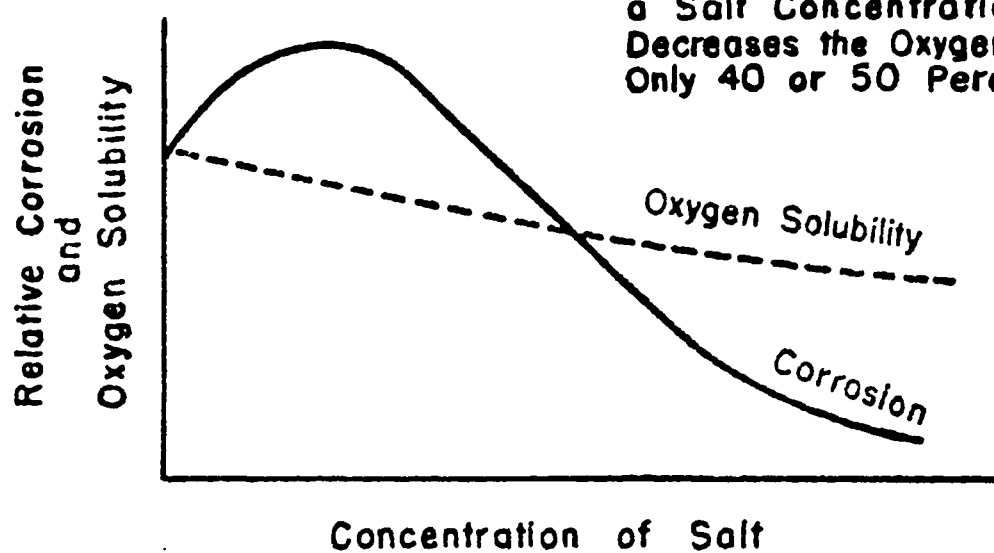


Fig. 4.7 : Concentration of salt

Fig. 4.8 : Corrosion Reduced to Nil at a Salt Concentration which Decreases the Oxygen Solubility Only 40 or 50 Percent.



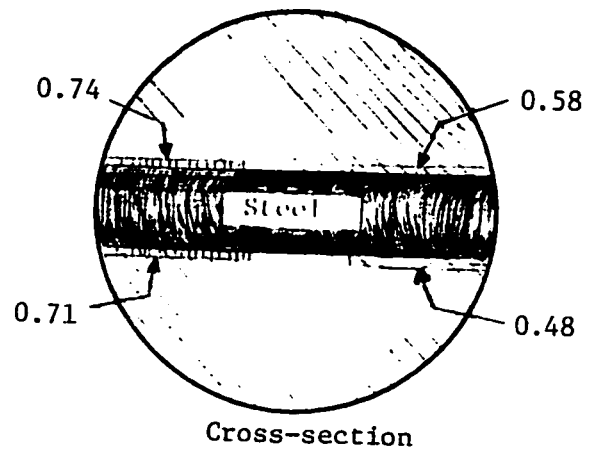
corrosion process unless they are present in abnormally high concentrations and in a uniformly distributive pattern throughout the cement paste matrix.

4.3 NATURE OF CORROSION

A large number of rebars were condition surveyed during this investigation and comprehensive condition records of the nature of corrosion were maintained. The record for each bar obtained from the cores included a visual description of corrosion conditions supplemented by photographic documentation. Two types of corrosion were observed. Firstly, where corrosion occurred within concrete that had not cracked or had only fine or moderate cracking along the reinforcement, the corrosion products were dark grey to brown in colour and the corrosion was typically spotty and discontinuous. This characteristic pattern of corrosion is visible in Plate 4.1 obtained from a building slab in Dammam. This slab had very little cracking before the building was demolished for a reconstruction programme. Removal of rust from the bar samples by emery paper or chemical treatment showed extensive pitting, along the whole length of the rusted bars. Pitting is a form of localized attack that results in holes and cavities. These are shown

in Plate 4.2 for some of the bars removed from the sampled concrete. Pitting is regarded as one of the most destructive and insidious forms of corrosion¹⁰². It causes metal to fail by perforation with only a small percentage weight loss of the entire part. As an example for the pitting shown in Plate 4.2 the loss of metal is only in the lower most bar very small per cent for a 20ft. length of the bar and yet the reduction in cross sectional area at the section shown is 50% . This reduction could have serious repercussions if section A is located in a critical stress region.

Bosich¹⁰³ says : "From the practical stand point most pitting failures are caused by the chloride ion. The chloride ions are present in varying degrees in most waters and solutions made with water. The chloride ion is characterized by a penetrating power". In the pitting type of corrosion which occurs on unexposed steel within the concrete small galvanic cells of the differential salt or oxygen concentration types, especially the former, are set up. An evidence of the differential salt type of cells was obtained in this investigation by carrying out tests determining the chloride content along the rebar and also along its perimeter. A typical chloride concentration variation profile along and around the bar is shown in Figure 4.9.



Chloride concentration as % of
mortar around the bar

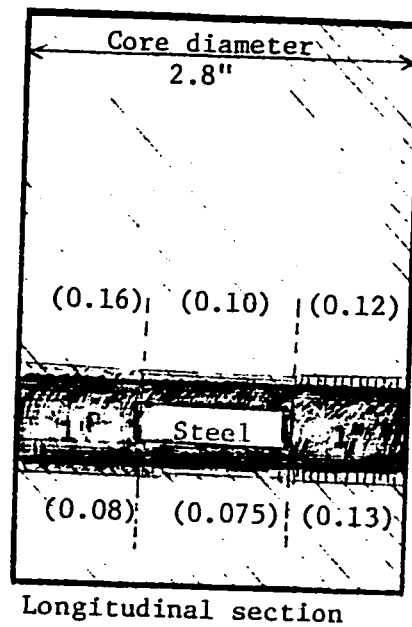


Fig. 4.9 : Variation of chloride

A local break through of the passivating gamma iron oxide film on the rebars by the chloride ion results in a small anodic site. The surrounding passive surface acts as a cathode and a galvanic cell is obtained. This renders pitting as a characteristic corrosion of unexposed steel in concrete where conditions are conducive to the setting up of differential concentration type of galvanic cells.

The second type of corrosion occurred in the exposed steel in which case the corrosion product was always dark brown, almost black in colour and was a typical stratified flake type as shown in Plate 4.3 . This plate depicts exposed steel in the underside of a residential building canopy in Dammam.

4.4 DISCUSSION OF TEST DATA RELATED TO THE INFLUENCE OF CHLORIDE ON CORROSION

4.4.1 Field Investigation

Despite the fact that chloride ion is considered the fount of steel corrosion in concrete structures there is virtually no data quantifying the relationship between chloride content in concrete and the resulting rebar corrosion. There are really substantive difficulties in collecting such a correlation data as it entails holding

several other variables, such as concrete quality, cover exposure and age constant. In an attempt to quantify this relationship the author sampled concrete from the exposed slabs of 10 in-service concrete framed buildings 22-27 years old and located in Dammam-Alkhobar habitations. Concrete cores were removed from the roof slabs using a portable diamond core drilling machine as described in chapter 3. Comprehensive observation recordings were made about the age, general condition ratings of the slabs and sampled concretes, extent and type of deterioration and the functional efficacy of the slabs from the point of view of attracting aggressive media during their in-service performance. Most of the cores were removed from slabs which were exposed on the upper side and formed the interior ceiling on the lower side. The methodology is detailed in chapter 3.

These large number of cores were subjected to 30-minute absorption tests (BS 1881 : Part V : 1970) and to a cover analysis. Samples which showed odd divergences were not used for this study. Samples which fell in categories to enable the other variables to be held constant have been tabulated in Table 4.2; these tables provide information regarding the buildings from which the cores have been obtained, the metal loss, cover, absorption. The methodology

of obtaining chloride content, metal loss, absorption etc. is explained in chapter 3.

Table 4.2 gives the metal loss versus chloride content data which has been plotted in Figure 4.10 for $\frac{1}{2}$ inch, 1 inch and 2 inch covers.

For $\frac{1}{2}$ inch cover chloride content versus metal loss data is plotted from eight concrete cores. The mean porosity as expressed on the basis of 30-minute absorption test is 3.702%. The maximum variation from the mean value is -16% and +18% for the eight points. The author holds that for concrete collected from field structures these variations are extremely reasonable and acceptable and this data indicates a fairly uniform quality of concrete for the observations plotted in Figure 4.10.

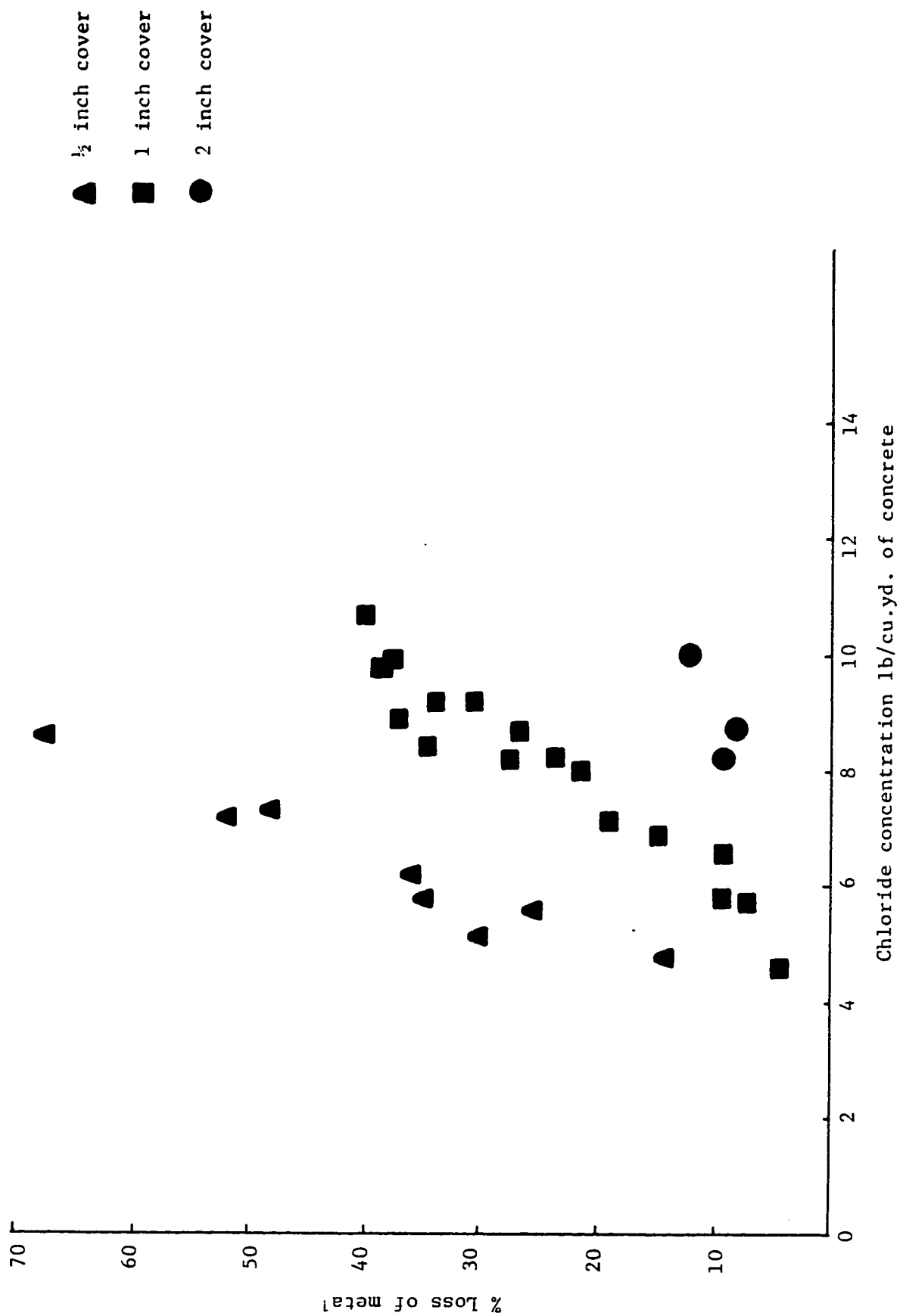
For 1 inch cover chloride and metal loss data is plotted from 19 cores. The mean value of porosity is 3.25 and the maximum variation is + 13.85% and -10.5%. This also entails a reasonably uniform concrete quality.

For the 2-inch cover chloride and metal loss data has been plotted for 3 cores. The mean porosity is 3.12 with maximum variation of -5.77% and + 3.2%.

TABLE 4.2
Chloride - Loss of Metal Data

Building No.	Core No.	Bar No.	Loss %	CL ⁻	Absorption	Cover
3	3	2	67.45	8.6	-	0.5
4	1	1	25.3	5.6	-	0.3
4	2	3	34.94	5.8	-	0.5
5	4	1	36.19	6.2	4.38	0.5
5	4	2	30.0	5.12	4.38	0.5
6	3	1	14.19	4.8	3.5	0.5
7	1	1	52.0	7.2	3.16	0.4
7	3	2	48.2	7.4	3.09	0.4
5	3	1	9.2	6.6	3.31	1.0
5	5	1	0	5.0	3.06	1.0
6	2	1	4.01	4.6	3.32	1.0
7	3	1	21.4	8.0	3.09	1.1
7	4	1	8.9	5.8	2.91	1.0
7	5	1	14.8	6.92	2.98	0.85
7	6	2	19.2	7.14	3.19	0.80
8	1	1	7.0	5.74	2.96	1.0
8	2	1	22.4	8.2	3.11	1.1
8	3	1	23.6	8.24	2.91	0.87
8	4	1	26.5	8.66	3.14	0.9
8	4	2	27.5	8.2	3.14	0.9
8	6	1	30.6	9.2	3.43	0.9
9	1	1	34.5	8.4	3.52	1.0
9	2	1	34.0	9.2	3.17	1.0
9	3	1	37.1	8.92	3.62	1.0
9	5	1	37.4	9.9	3.60	1.1
9	5	2	38.5	9.78	3.6	1.1
10	1	1	40.0	10.72	3.7	1.0
7	6	1	9.1	8.24	3.19	2.0
9	4	1	7.8	8.74	3.22	2.0
10	5	1	11.7	10.0	2.94	2.0

Fig. 4.10 : Relationship between chloride content and loss of metal



From the data plotted in Figure 4.10 the following broad conclusions can be drawn :

- (i) For the $\frac{1}{2}$ -inch and 1-inch cover concretes chloride ion has a definitive influence on corrosion and the consequent concrete cracking and spalling. An increase in chloride level generally indicates increased steel rusting and the consequent loss of metal.
- (ii) A comparison of loss of metal for $\frac{1}{2}$ -inch cover concretes with the corresponding loss of metal 1-inch cover concretes, shows that chloride content cannot be held as the sole criterion of corrosion severity. For these two sets of concretes for the same chloride concentration the severity of corrosion for the $\frac{1}{2}$ -inch set is more than twice that for the 1-inch cover set. This can be explained by a more stiffling barrier effect provided by the 1-inch cover in comparison to the $\frac{1}{2}$ -in cover.
- (iii) The aforesaid contention is further strengthened by the trend of the corrosion data for the 2-inch cover rebars. It is seen that even for high values of chloride concentrations the corrosion is very nominal,

the loss of metal not exceeding 12% in any of the 3 concretes analyzed. This happens because oxygen supply is stifled and moisture uptake to the level of steel is choked by the 2-inch thick cover.

- (iv) The definitive effect of concrete cover to reinforcement, which is discussed in greater detail in chapter 5 is clearly seen from the trend of the plots. The slope of the "data trend" markedly decreases from $\frac{1}{2}$ -inch to 1-inch to 2-inch cover plots. This indicates that for an equal amount of chloride concentration the loss of metal increases markedly with decreasing concrete cover. This obviously is explained by the weakening of the barrier or shielding effect to the ingress of corrosion promoting media with the reduction in cover.

4.4.2 Laboratory Investigations

A series of laboratory investigations were carried out to determine the effect of chloride concentration on corrosion. In the laboratory the corrosion activity was monitored on the basis of the copper-copper sulphate half cell potentials. Concrete specimens 8" x 12" x 1½" in the form of small slabs, each containing a Ø.12 plain bar (Plate 4.4) were manufactured. The added chloride was in

the form of NaCl and five mixes were prepared using 0, 3.1, 6.2, 12.4 and 24.8 gm/l sodium chloride. These values correspond to 0, 3.76, 7.52, 15.0 and 30 lb chloride per cubic yard of concrete. For each chloride concentration 3 specimens were prepared, two of which were stored in air at room temperature and humidity and one was partially submerged in water. The concrete mix was 1.2 : 2:3/cement : Sand : Coarse aggregate by weight and the w/c ratio was 0.65. The bars had a clear cover of $\frac{1}{2}$ inch and for each slab potentials were measured on three locations as shown in Figure 4.11. The methodology of Cu-CuSO₄ monitoring of corrosion is described in detail in chapter 3.

Figure 4.12 shows the half cell potentials plotted against time for the various aforesaid chloride concentrations. It is seen that the potentials increase showing an enhanced corrosion activity with the increase in chloride concentration. After 110 days of storage cores were obtained from the samples to observe the state of corrosion on the rebars. The observed state of the bars is described below and is also recorded in a sketched format on the right side of Figure 4.12b.

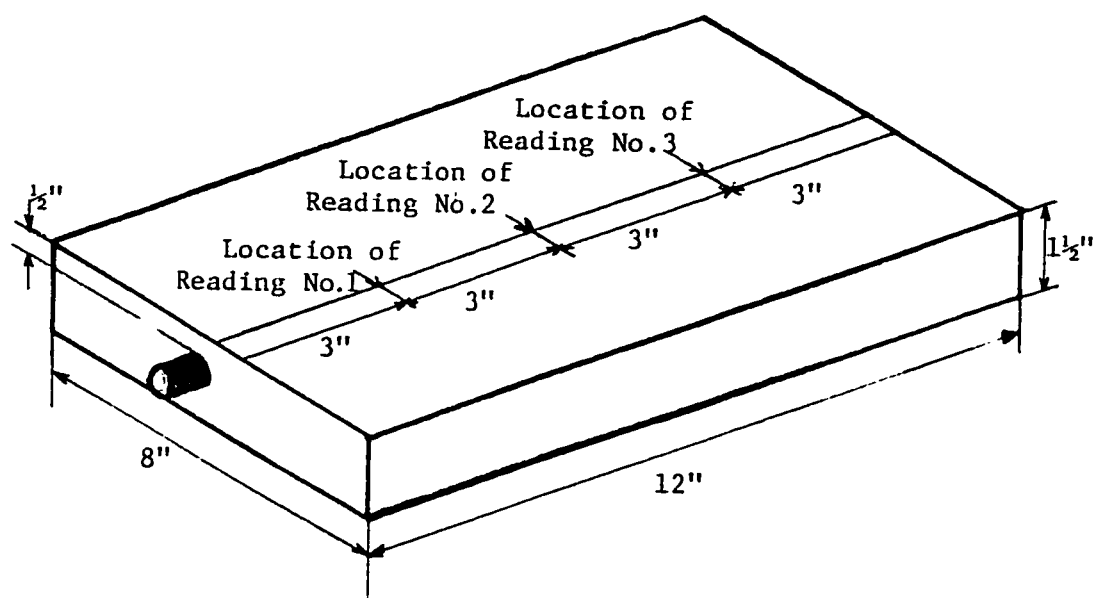


Fig. 4.11 : Laboratory slabs for monitoring corrosion by half cell device

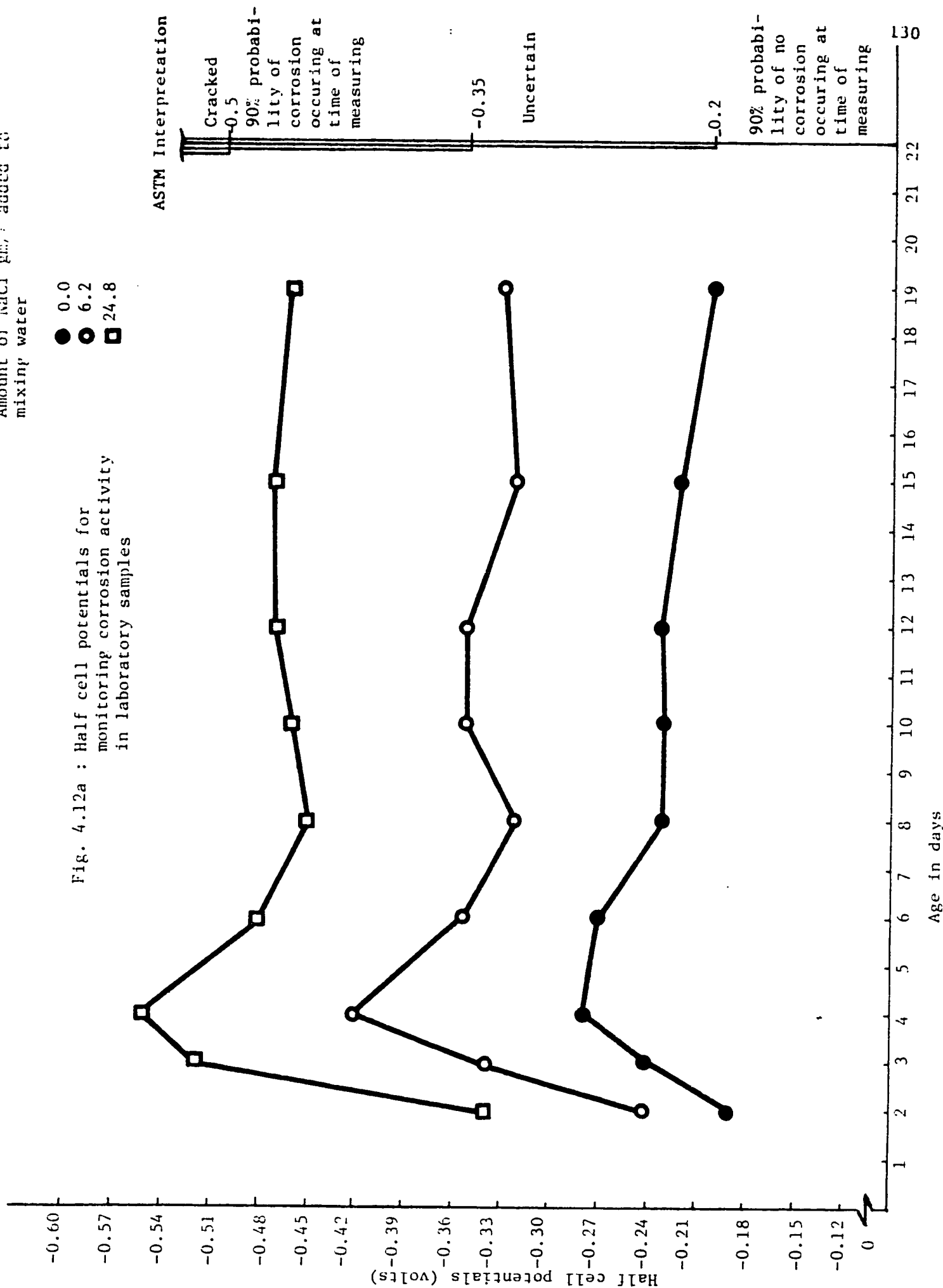
- 0% gm/l NaCl (curve 1) : There was no general rusting although one spot of pitting was observed along an air cavity in concrete.
- 3.1% gm/l NaCl (curve 2) : Pitting noticeable and brown corrosion stain in some parts of the bar although no substantive corrosion.
- 6.2% gm/l NaCl (curve 3) : Pitting was noticeable along small air pockets in concrete, brown corrosion stains were found to be deeper than for curve 2.
- 12.4% gm/l NaCl (curve 4) : Marked pitting in much larger area showing onset of corrosion with discontinuous small spotty corrosion products visible all along the bar.
- 24.8% gm/l NaCl (curve 5) : Cracking of concrete initiated.

For the last case (24.8% chloride) a fine crack has appeared along the length of the bar which is shown in a close up photograph in Plate 4.5. This shows the significance of high chloride content in a porous concrete ($w/c = 0.65$).

Amount of NaCl gm./: added to
mixing water

- 0.0
- 6.2
- 24.8

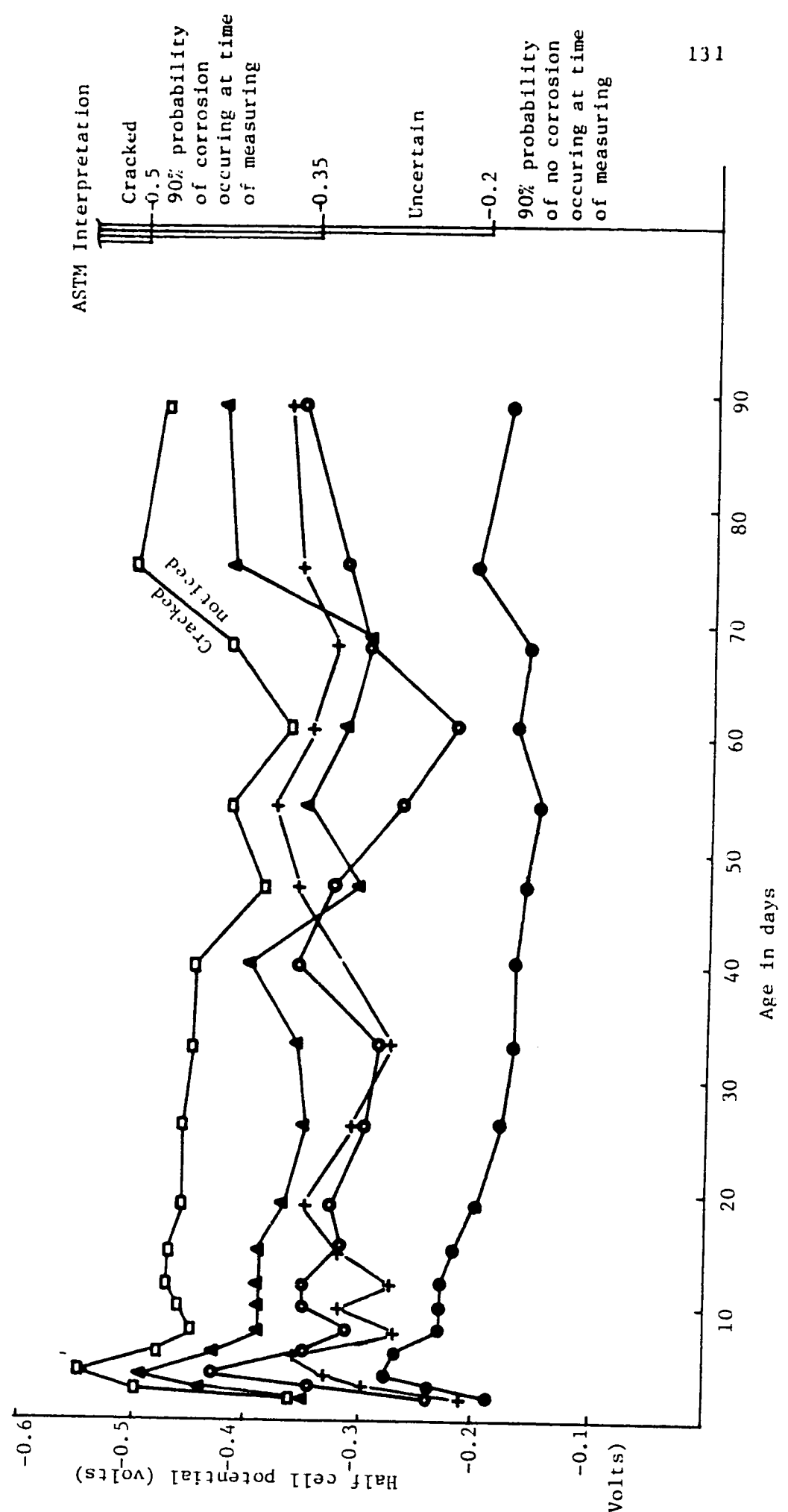
Fig. 4.12a : Half cell potentials for
monitoring corrosion activity
in laboratory samples



Amount of NaCl gm./l added to mixing water

- 0.0
- + 3.1
- 6.2
- ▲ 12.4
- 24.8

Fig. 4.12b : Half cell potentials for monitoring corrosion activity in laboratory samples



A 30 lb/cu. yd. chloride content in a 2700 lb/sq.in. concrete (w/c = 0.65) having high absorption cracked within 3 months of its casting in a rather temperate climatic environment where the temperature was around 24°C without variations and the humidity low.

On far right of Figure 4.12 is sketched the interpretive legend of half cell potentials in terms of corrosion activity according to ASTM Standard C876-77. A comparison shows a good correspondence from -0.35 potentials upwards. However, in the uncertain range of -0.2 to -0.35, definite pitting has been observed from these tests, specially along very small air pockets in concrete.

Figure 4.12 shows the chloride concentration and half cell potentials plotted for the 90 days age. This plot also indicates an increase in corrosion activity with the increase in chloride concentration.

4.5 CHLORIDE ACCUMULATION

The data obtained on chloride accumulation in the present investigation is compared with similar data obtained by other investigators in Table 4.3. Gewertz¹⁰⁴ et al, and Beaton et al, have reported data on chloride accumulation at steel-concrete interface for corrosion

TABLE 4.3

Average Values of Chloride Concentration (lb/cu.yd.)
Near Steel-Concrete Interface

San Mateo- Hayward Bridge Piling (submerged) for 37 years	San Mateo- Hayward Bridge Piling (Atmos- pherica- lly ex- posed for 28 years	San Mateo- Hayward Bridge	Califor- nia Bridge Decks	US Bridge Decks (8 Sta- tes)	US Bridge Decks	Gulf Area Samples Containing Corroded Steel Average Value from Concrete Samples
25	5	1.9	2.60	2.35	3.1	8.11

affected submerged and exposed piling and for the super-structure of San Mateo-Hayward Bridge. The data in columns 4, 5 and 6 is from bridge decks in the United States which had posed serious maintenance problems due to corrosion and concrete scaling as a result of chloride penetration from de-icing salt applications.

Spellman and Stratful have reported results of investigations on bridge decks less than 10 years old. The chloride levels at 1-inch depth from the surface ranged between 1.79 lb/cu.yd. to 12.5 lb/cu.yd. of concrete. At the level of the reinforcement mat the average chloride concentration was 2.6 lb/cu.yd. (column 4). A similar investigation on 1000 randomly selected bridge decks in eight States in the USA showed considerable variation but average values of chloride content at 2.35 lb/cu.yd. about 1-inch below the surface (column 5). In another cooperative investigation on bridge decks reported by the Bureau of Public Roads, several State highway departments and Portland Cement Association, the chloride content by weight of cement was found to be .72% near the surfaces of unspalled decks and 1.45 percent in unspalled concrete cores obtained from decks which showed spalling. Column 6 shows the result of this study in terms of average chloride ion accumulation

1-inch below the surface of the decking.

Obviously a direct comparison of values from different situations and from differently deteriorated samples of field structures can be misleading. But the common factors of chloride prevalence and the forms of deterioration render this comparison meaningful. Save for the pilings which were submerged for 37 years in sea water and understandably showed unusually high values, the chloride accumulation level in the Gulf concrete is the highest. It is even more than the accumulation in piling subject to 28 years of continual exposure to sea water.

In a laboratory investigation Ost and Monfore found chloride contents varying from 0 to about 30 lb. per cu. yd. after one year of testing. The chloride penetration depended on test parameters, specially the w/c ratio of the concrete mixture. In another study concrete samples absorbed upto approximately 45 lb of chloride ion per cu. yd. after 88 days of testing.

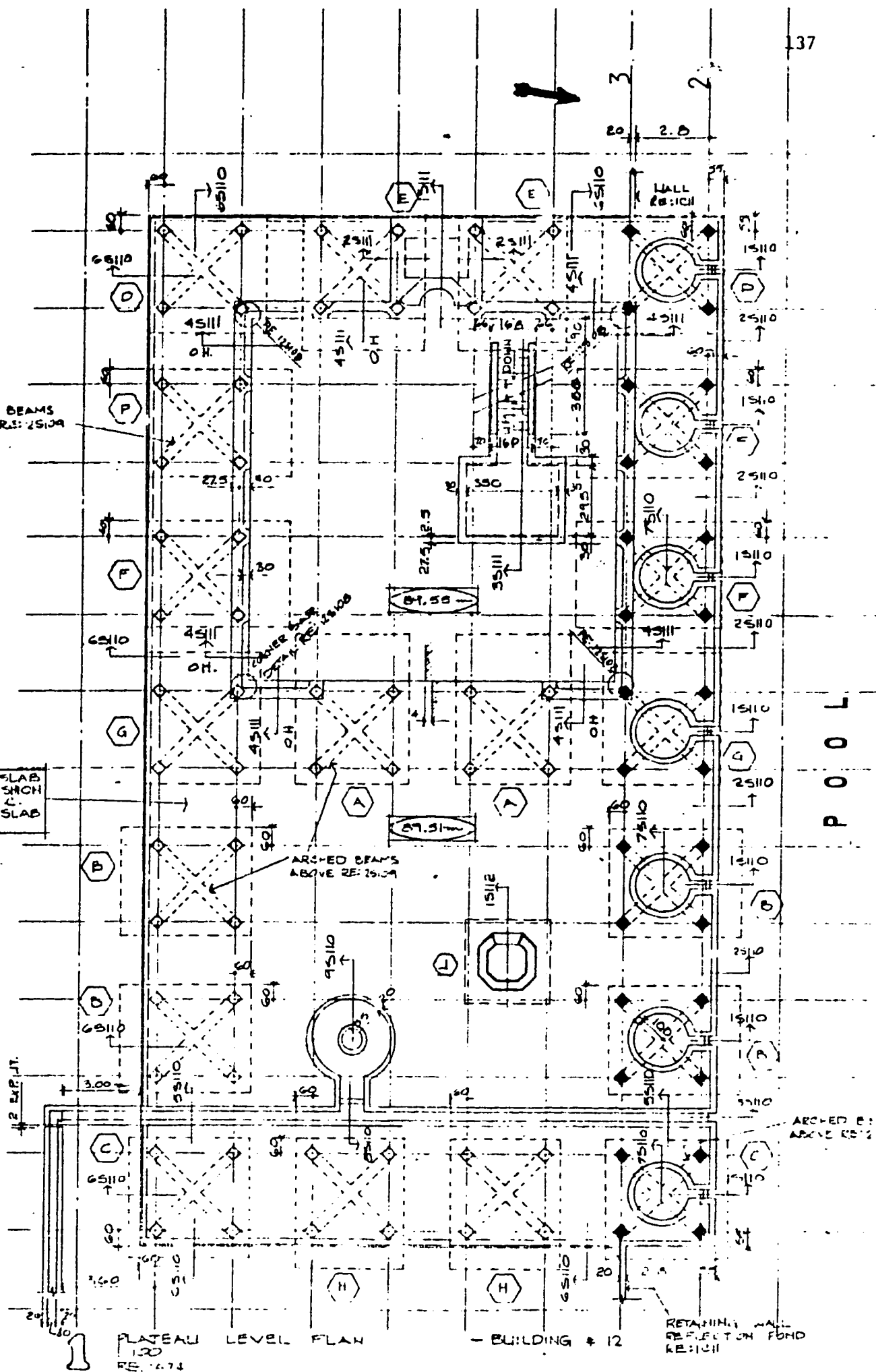
It is also of interest to note that the tests indicate the chloride accumulation mechanism to be more complex than the simple absorptive action of concrete. In a concrete piling which was submerged for 37 years, the computed

concentration of chloride was found to be greater than that of the sea water . Spellman and Stratfull have suggested a combined mechanism based on absorption and combination of desorption by evaporation and absorption by capillary action of the aggressive medium.

4.6 CHLORIDE PENETRATION

Wherever conditions are conducive chlorides easily permeate concrete from external environment or an applied source to cause severe steel corrosion. Corrosion of rebars due to the penetration of de-icing salts in bridge decks is a very commonly cited example of concrete deterioration due to chloride penetration. In order to evaluate the diffusion characteristics of chloride as it migrates into the concrete and the deterioration it eventually causes to structures two cases were investigated. Figure 4.13 shows the layout of a mosque in Dhahran which is surrounded on two sides by a pool filled with saline water. The mosque was constructed in 1972 and within 8 years all the twentyeight columns along plan lines (2) and (3) started showing marked corrosion of rebars and severe concrete spalling. Typical examples of the deterioration are shown in Plates 4.6 through 4.8. The lower 2-3 feet of

10cm. CONCRETE SLAB
ON 10cm. SAND CUSHION
W/ 3 @ 40cm. O.C.
BA. WAY ROUGH SLAB
ELEV 89.51m



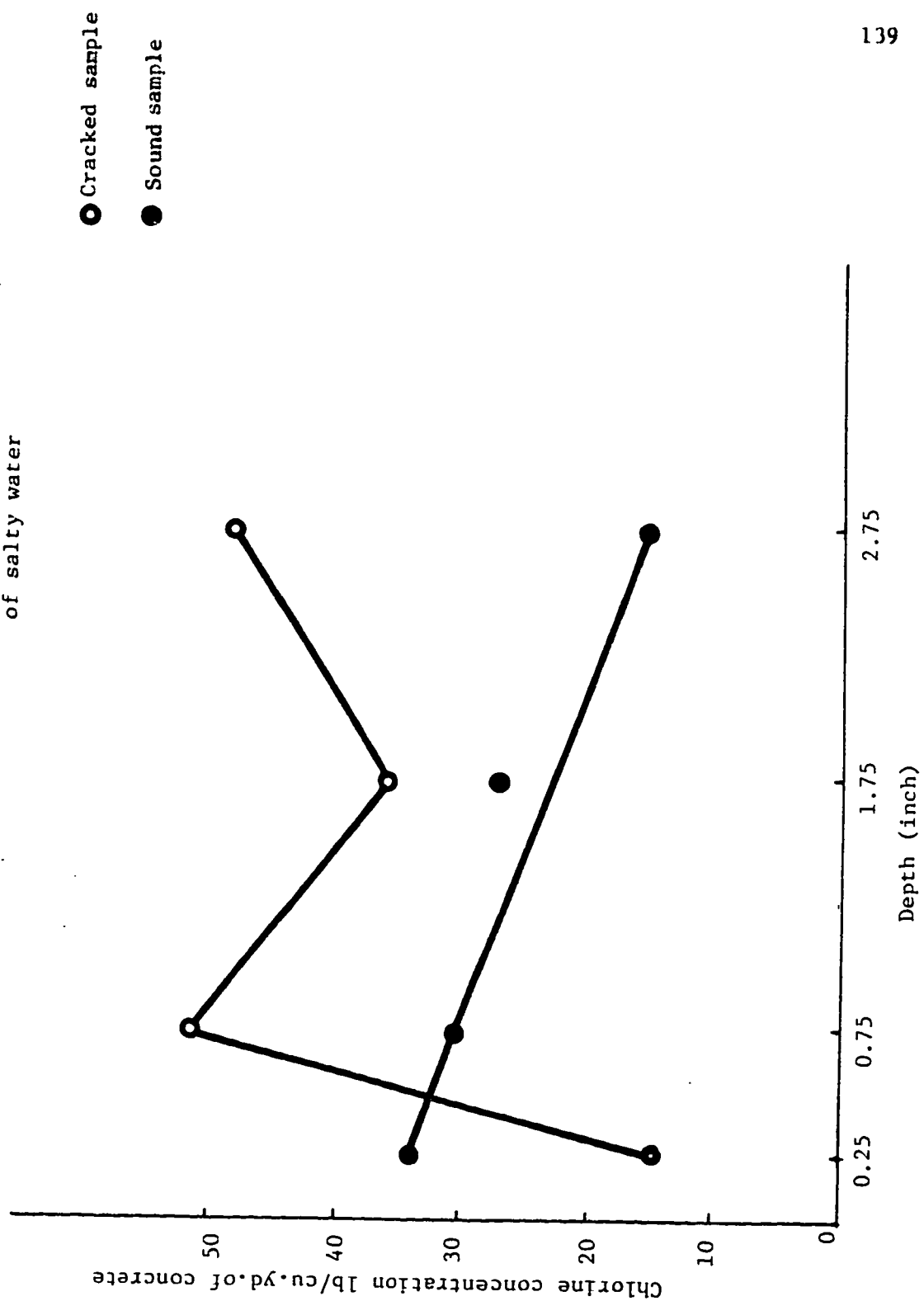
all the columns were badly spalled and in some cases vertical cracks of varying width parallel to the length of the main vertical column reinforcement ran upto a height of 10-15 feet (Plate 4.8). Tests on cores taken out from typical columns showed that the strength of concrete was only 2800 psi when it was intended to be 3000 psi.

Plate 4.9 shows the layout of a portion of another building in Dhahran. In this case also the deep vertical columns were situated along a pool filled with saline water. The deep columns and in this case even beams about ten feet high showed substantial steel corrosion and severe concrete cracking. Typical illustrations of steel corrosion and concrete cracking are shown in Plates 4.10 and 4.11.

It is obvious that the most significant reason for this deterioration was a profuse supply of chlorides through the wind generated spray and splash of salt-charged water on the 28 outer columns which stand within easy reach of this environment. Further, the differential salt and oxygen cells are much more activated by the cyclic wetting and drying conditions experienced by the columns.

Figure 4.14 presents the typical data for chloride penetration in one of the typical mosque columns with

Fig. 4.14 : Chloride penetration characteristic
for sample subjected to splash
of salty water



uncracked and cracked concrete covers. This data is compared with parallel situations elsewhere in Table 4.4. The general form of the penetration curve for the uncracked concrete in qualitative terms is similar to those reported for the atmospherically exposed sections of piling of San Mateo-Hayward Bridge and for bridge decks in the USA. All three examples show a maximum chloride build up close to the surface and then rapid decrease with depth. However, quantitatively speaking, there are significant differences. The maximum chloride content near the surface of sound concrete in bridge decks which had shown Cl^- concentration was 1.45% by weight of cement this decreased sharply to 0.17% at a depth of 2.75 in. In San Mateo-Hayward Bridge piling exposed to atmosphere maximum chloride content near the surface was about 6 lb/cu.yd. of concrete with decreased rapidly to 2.5 lb/cu.yd. at a depth of 2.75 in. In the case of the Dhahran structure the maximum chloride content near the surface was found to be 5.8% by weight of cement 34.8 lb/cu.yd. of concrete which also decreased rapidly and was found to be 2.6% by weight of cement 15.6 lb/cu.yd. of concrete at a depth of 2.75 in. This comparison as well as the data in Table 4.4 shows that the chloride content near the surface as well as inside the concrete was far higher

TABLE 4.4

Penetration of Chloride

Depth from ext. surface inch	Dahran Mosque		Bridge Deck			San Mateo-Hayward	
	CL ⁻ lb/cu.yd of concrete	CL ⁻ % weight of cement	Unspalled		Spalled CL ⁻ lb/cu. CL ⁻ % yd. of weight concrete of cement	CL ⁻ lb/ cu.yd. weight of concrete cement	CL ⁻ %
			CL ⁻ lb of concrete	CL ⁻ % weight of cement			
0.25	34.8	5.8		0.65	1.45		
0.75	30.0	5.0		0.362	0.97	5.8	
1.75	27.0	4.50		0.12	0.48	4.2	
2.75	15.6	2.60		0.06	0.17	2.5	

in the Dhahran structure than in the two cases cited here.

There are two plausible reasons for these high values. Firstly, the initial level of chloride in concrete which reached through salt contaminated aggregates and brackish mix water was higher than in the other two cases. Secondly, the column concrete permeability was high and the overall quality of the mosque column concrete was poor. Plate 4.12 shows the porous cover concrete for this column. For the mosque column for which chloride penetration data is presented here concrete absorption was measured at four points and is tabulated in Table 4.5. The average value of absorption is 5.18 which is higher than the average absorption value recorded from observations obtained from Dammam-Alkhobar area.

The fact that chloride content decreases rapidly with depth is brought out clearly by the chloride diffusion characteristics shown in Figure 4.14 and this emphasizes the role of the concrete cover in providing protection against chloride penetration and steel corrosion. It is reasonable to assume that one cause of the rapidly decreasing diffusion trend is the fact that the diffusion of chloride ion into the cement paste is accompanied by reaction of the chloride to form partially "insoluble" calcium chloroaluminate. This

TABLE 4.5

Absorption for Cores Obtained from Cover, Dhahran Mosque Column

Core No.	Length mm	Dry Wt. (gm)	Wet Weight (gm)	Length correction factor	Absorption %
1	71.12	572.3	601.0	0.98	4.91
2	67.2	549.2	576.9	0.96	4.84
3	86.36	713.1	751.3	1.04	5.57
4	105.41	894.9	939.0	1.1	5.42

reaction reduces the concentration of chloride ion and hence the tendency for inward diffusion is further reduced. Ost and Monfore¹¹² have reported that the migration characteristics of the chloride-ion into concrete matrix are greatly dependent on the w/c ratio of the concrete mix. Lower values of w/c ratio, without impairing concrete placeability, markedly reduce the chloride permeation resulting in a better protection of embedded steel.

The present investigation is related to the migration characteristics of chloride-ion in the cement-sand matrix. As most rocks from which the aggregates are won have greater permeabilities than good quality cement paste, the aggregate characteristics may have an important role in the chloride migration characteristics. This is specially significant in the Gulf region where the limestone and dolomite aggregates are specially susceptible in this respect.

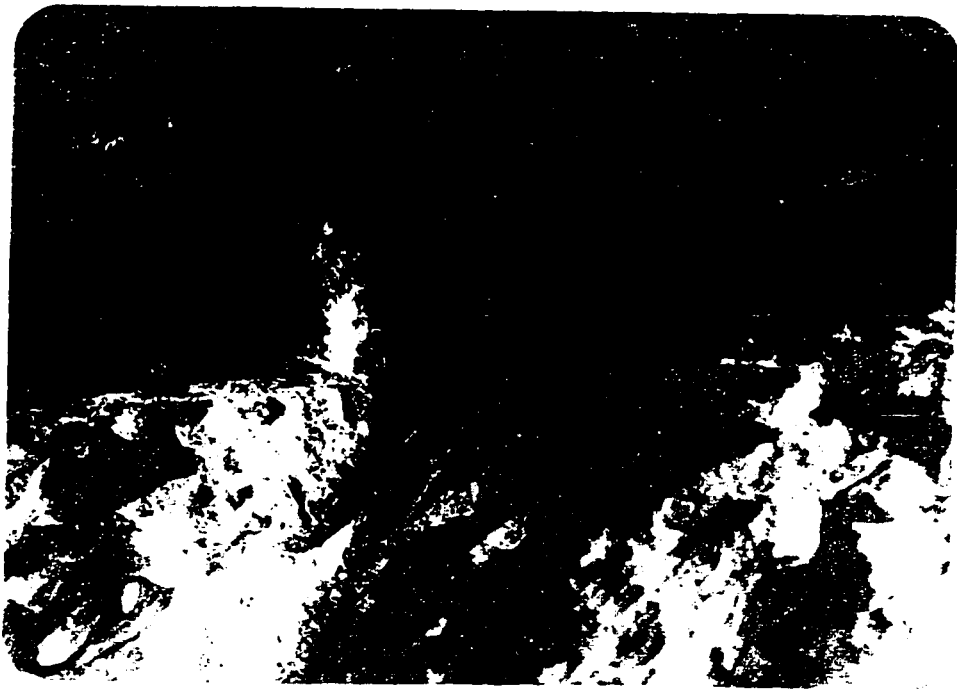


Plate 4.1 : Dark brown, and spotty corrosion product

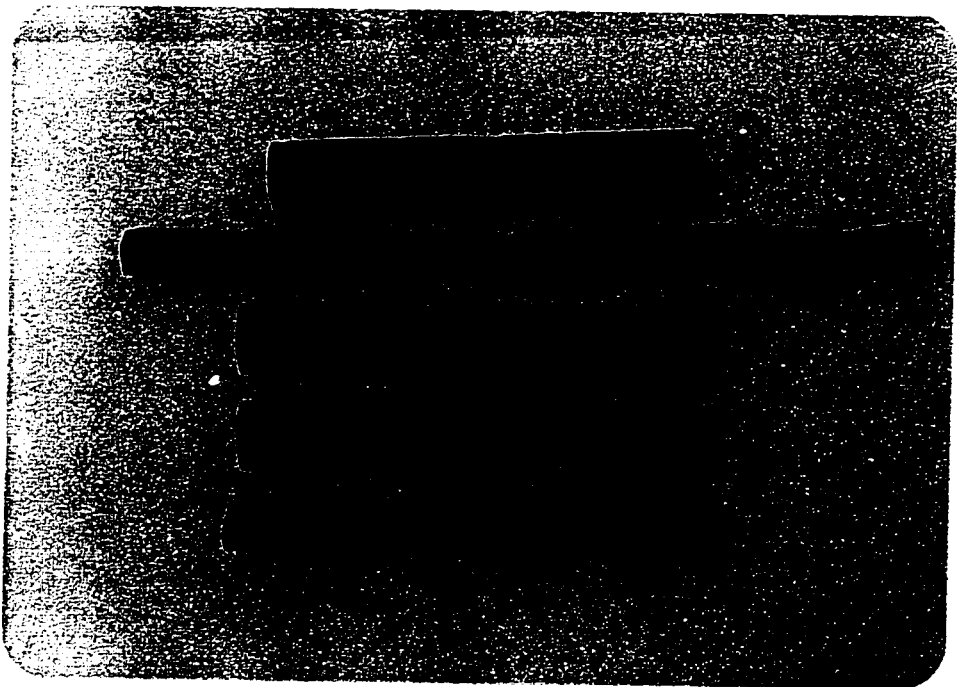


Plate 4.2 : Pitting corrosion of steel embedded in concrete

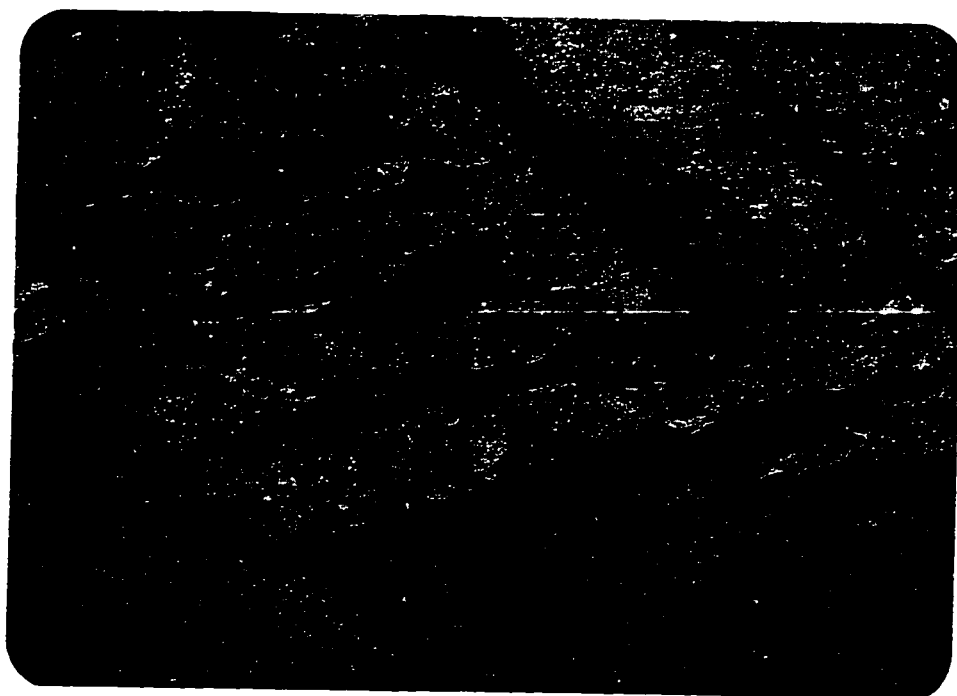


Plate 4.3 : Stratified and flaked corrosion material

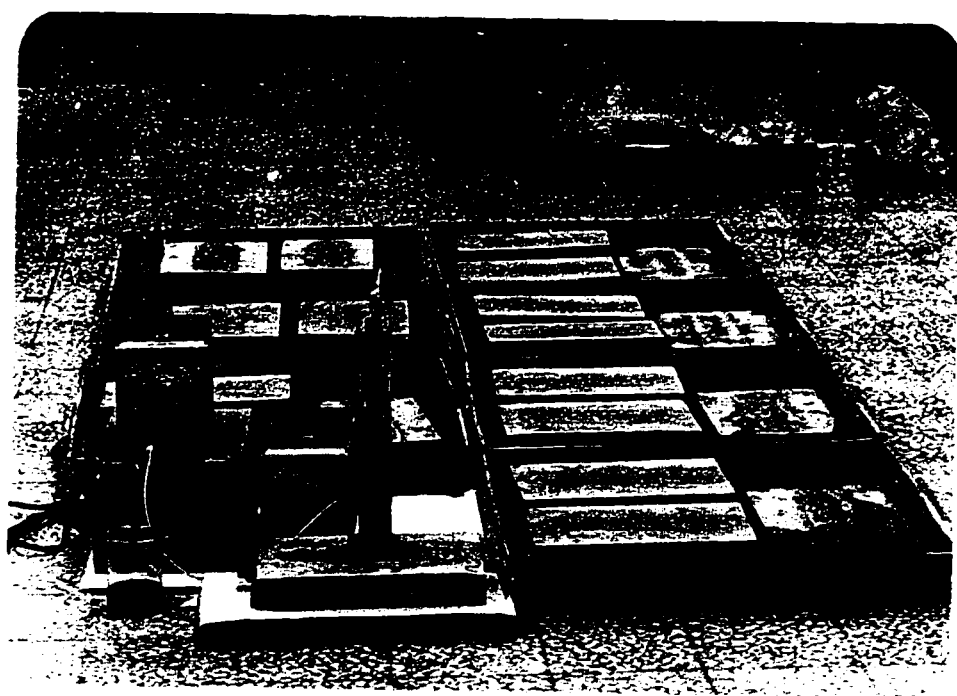


Plate 4.4 : Laboratory slabs for monitoring corrosion

Plate 4.5 : Crack developed along
the bar caused by
high chloride content

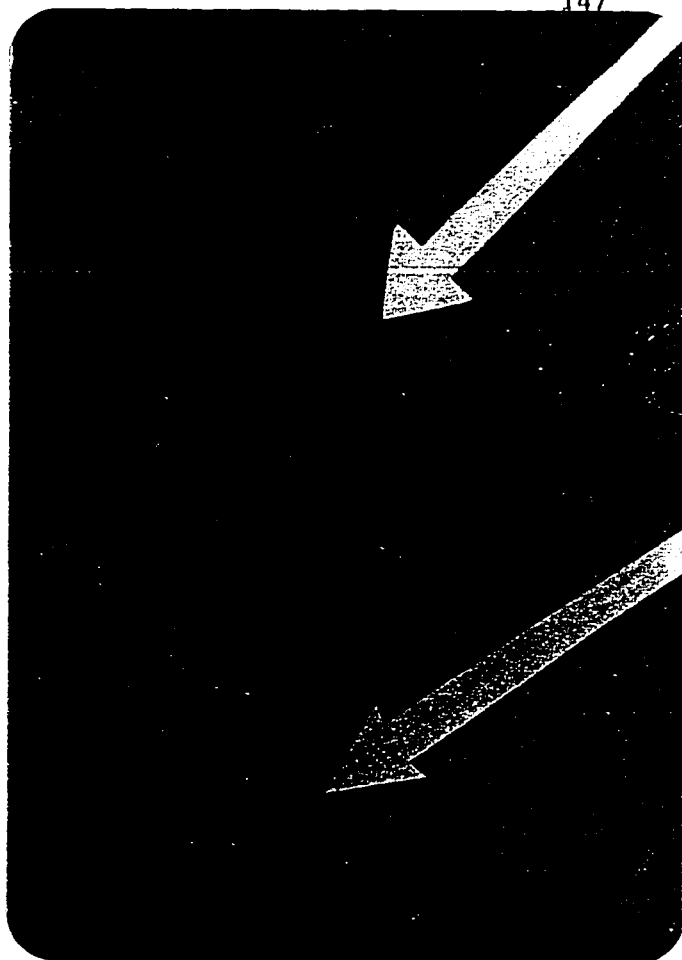


Plate 4.6 : Deterioration of columns
due to rebar corrosion

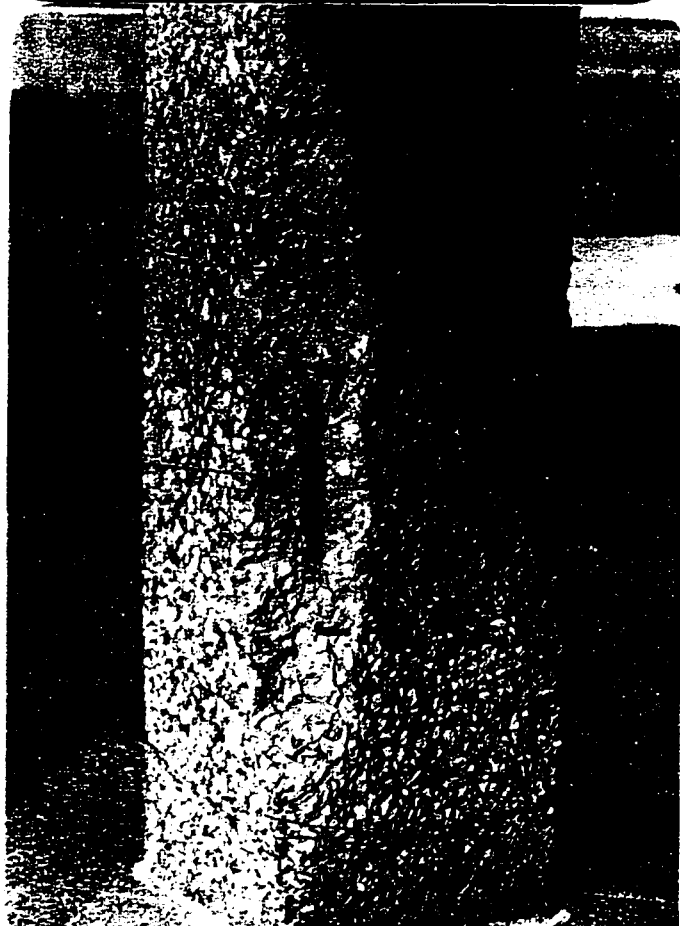
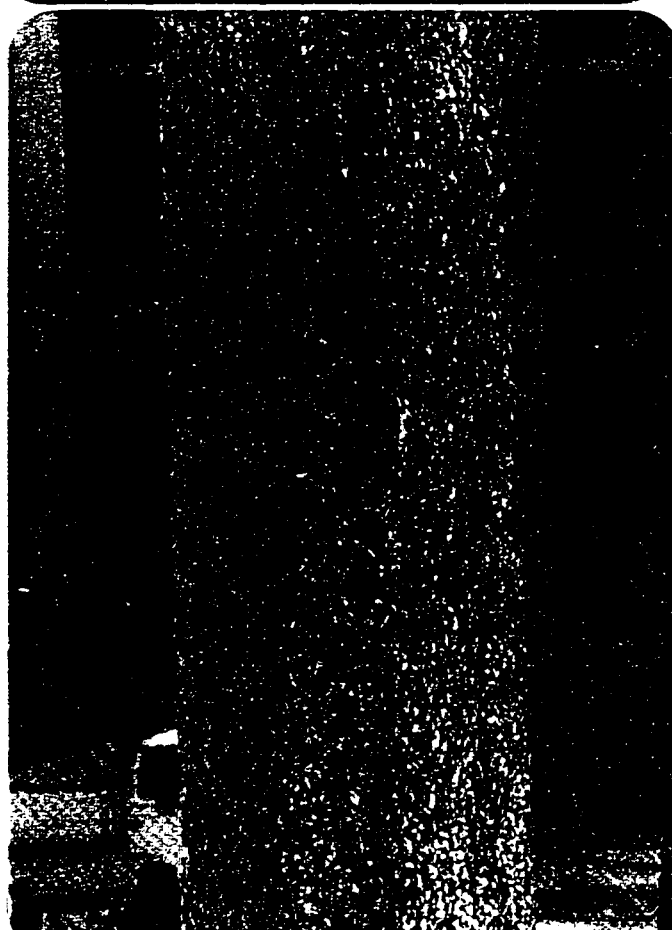


Plate 4.7: Cracked column above the
base due to corrosion of
reinforcement



Plate 4.8 : Cracks developed at
higher level



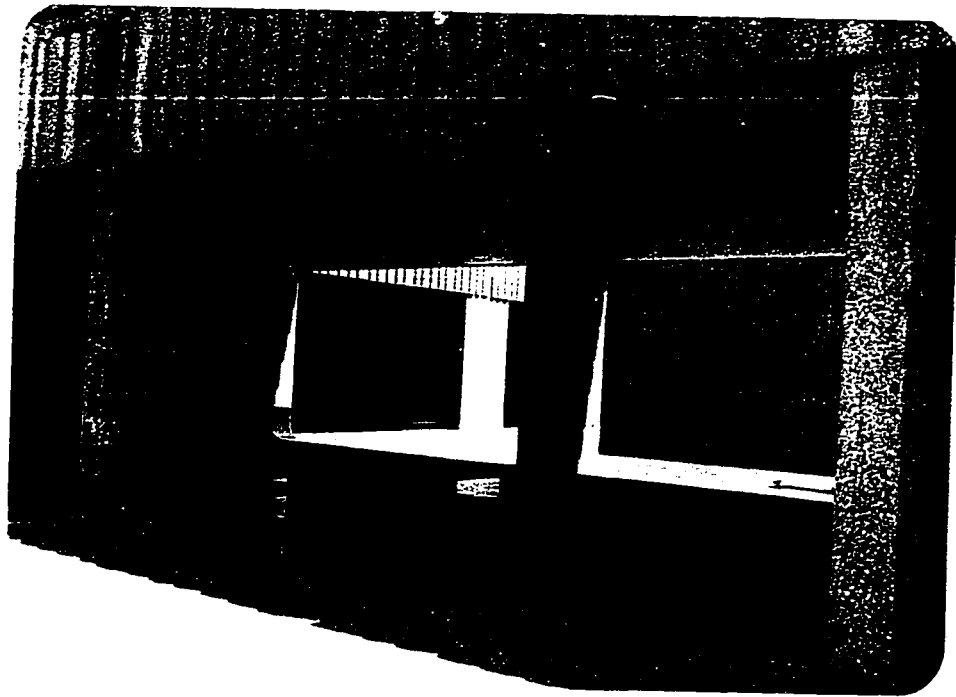


Plate 4.9 : Outlay of another building
cracks developed by corro-
sion of steel reinforcement

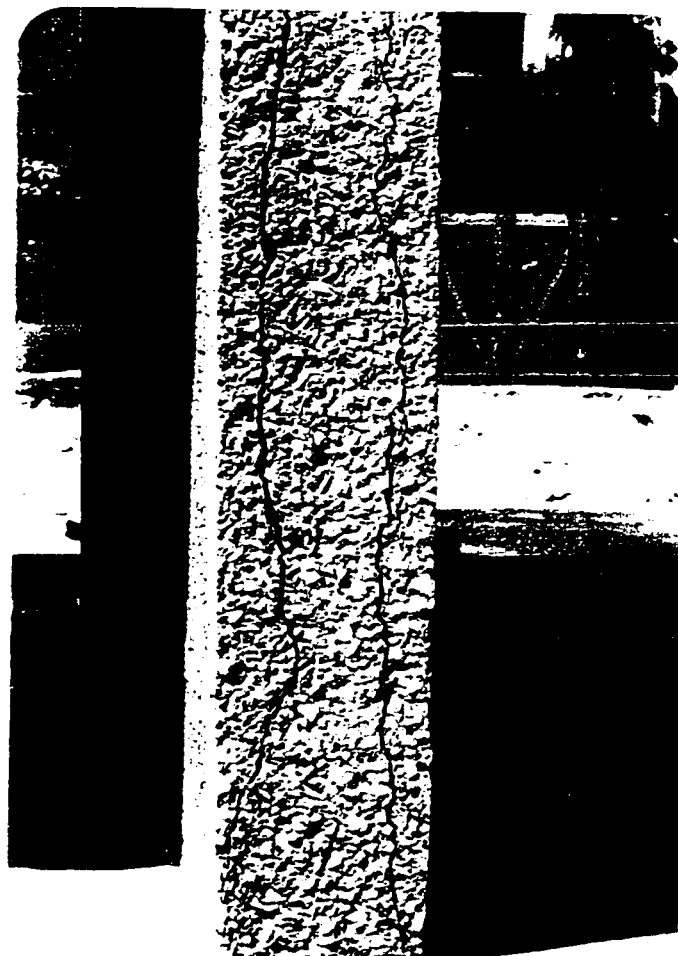


Plate 4.10: Longitudinal cracks in
column parallel to
reinforcement



Plate 4.11 : Wide and deep crack along reinforcement

chapter **5****EFFECT OF MATERIALS, CONCRETE QUALITY AND
CONSTRUCTION PRACTICES ON CORROSION OF
REINFORCEMENT****5.1 AGGREGATES**

The primary effects of the aggregate in concrete on corrosion of embedded steel relate to the degree of their chloride contamination and their effect on permeability and absorptive characteristics of concrete. Since aggregates typically constitute about 70 percent of concrete, it is evident that they play a major role in contributing salt if contaminated and also in determining the permeability and absorption of concrete. All other factors being constant, permeability increases with increasing maximum size of the coarse aggregate for most mineral aggregate materials . This is due to the fact that most mineral aggregates have permeability coefficients 10 to 1000 times greater than the permeabilities of cement pastes in the usual range of w/c ratios. It may, however, be noted that their contribution in the actual transmission of moisture,

chloride ion, oxygen and carbon dioxide from free surfaces to the vicinity of embedded steel is likely to be of less importance than might be implied from their effect on concrete permeability. The reason for this is that the more or less continuous capillary porosity in the paste phase is generally composed of much smaller pore diameters than those of the aggregates. Because of this, the aggregates are generally unable to compete with the paste for available moisture on the basis of capillarity. Thus, the capillary porosity of the paste phase constitutes the conduit system for the movement of moisture and dissolved solids and gases. But it is generally recognized that the main feature of the role played by the aggregates in the transport process is to replenish moisture to the paste during periods of drying.

The gradation of the aggregates may have a significant effect on corrosion of embedded steel if gradations are such as to result in segregation and bleeding of the concrete mixture. Segregation and bleeding will produce channels, promoting the ingress of water, chlorides, oxygen and carbon dioxide, and will produce voids in the vicinity of reinforcing bars which are single most significant promoters of the corrosion process as the occurrence of

areas of active corrosion at the location of these voids is commonly observed. Such poorly graded mixtures will also frequently display a porous concrete zone at the level of the reinforcing steel in heavily reinforced horizontal concrete slabs. This is due to the interference by the reinforcing mat to the sedimentation process that causes the segregation. This phenomenon results in a plane of weakness being created at the level of the reinforcement mat resulting in concrete spalling along planes (Plate 2.7).

Against the background of these general considerations the Gulf situation should be evaluated for the aggregate contribution to the corrosion deterioration. It is best to first state the relevant facts and then to sum up the derived conclusions based on these facts.

Figure 5.1 is a simplified map of the solid geology of the Arabian subcontinent¹⁰⁵. Igneous and metamorphic rocks normally associated with high strength and density, and low water absorption, prevail along the Red sea coast, in the Oman mountain and in small local concentrations in the south along the Arabian sea coast. Along the Gulf coast, however, where most recent constructional development has occurred, relatively young and weak sedimentary rocks

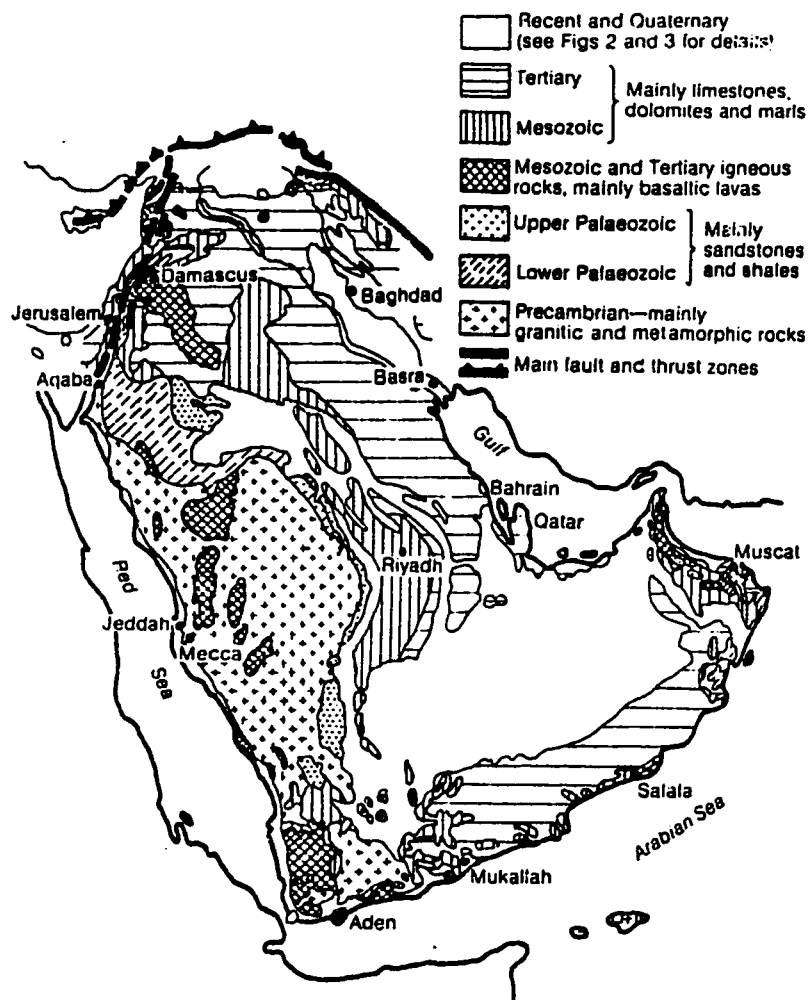


Fig. 5.1: Bedrock geology of Arabian subcontinent (after Beydoun and Dunnington)

predominate. From south of Dammam to Kuwait scarcely any useful aggregate materials can be obtained from the solid formations, but near Dammam, in Bahrain and in Qatar, restricted areas of Palaeogene (older Tertiary) limestones form virtually the only local sources of coarse aggregates¹⁰⁶. Neogene (younger Tertiary) limestones occur near Hofuf and along the southern Gulf coast, e.g. in Dubai. The Tertiary limestones, specially those of Neogene age found near Hofuf, tend to contain much weak and absorptive material. Older and stronger limestones (of Mesozoic age) exist near Ras al Khaimah and at less accessible points in the Oman range. They form an isolated source of better quality crushed aggregate which has been shipped to many parts of the Gulf. The geology of the Gulf States is shown in more detail in Figure 5.2, on which some of the main sources of materials have been identified.

5.1.1 Sources of Deleterious Impurities in Solid Formations and Sands of Arabian Peninsula:

Most of the Tertiary limestones are rich in magnesium and some are true dolomites. It is reported that dolomitic aggregates cause long term alkali-carbonate reaction in concrete resulting in expansion and cracking. Some of the local limestones contain chert or other forms of silica,

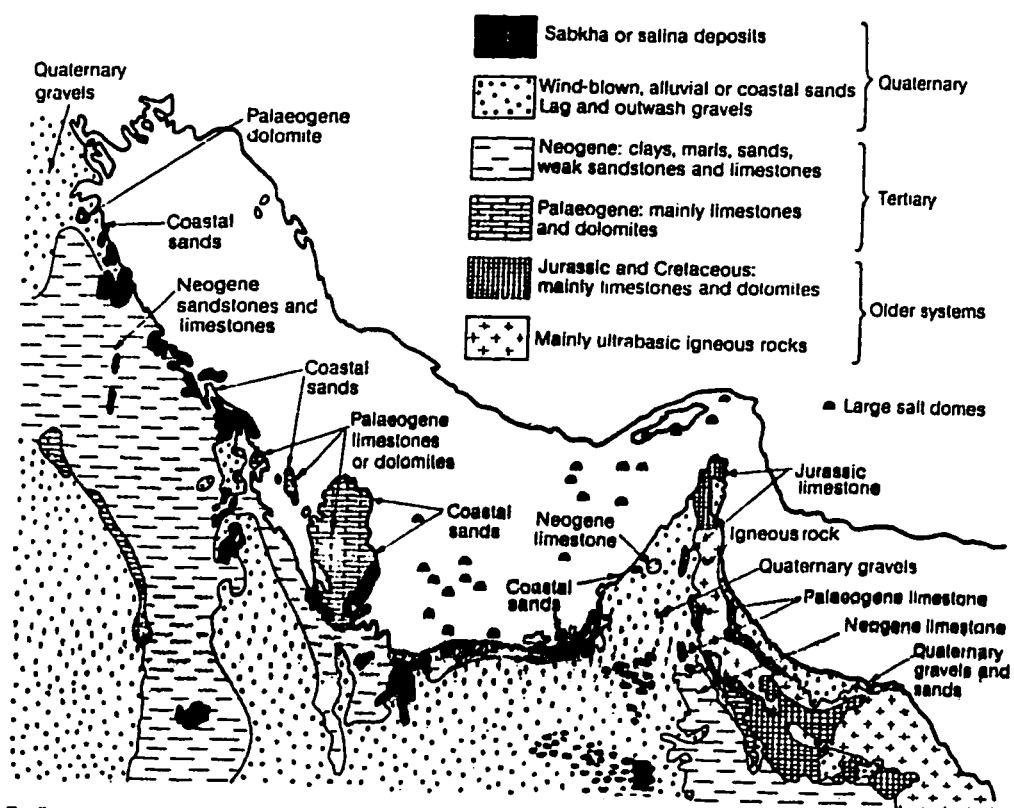


Fig. 5.2: Simplified geology of the Gulf States, with some important aggregate sources

with the consequent possibility of alkali-silica reaction resulting in profuse pattern cracking of concrete. These types of cracking in concrete are likely to act as fore-runners and feeders to corrosion promoting media into the concrete and usually indirectly contribute to the corrosion process.

However, the most direct source of corrosion is the salt contamination of aggregates. Common salt, gypsum and anhydrite occur at several horizons in the geological succession and in the numerous salt domes which lie in the Gulf, around its shores and in the Oman desert (Figure 5.2). These may constitute "built-in" sources of salt contamination, specially where they have been brought by erosion or geological structure into the zone of groundwater circulation.

Another source of salts are the "duricrusts" which characterize most of the regional outcrop sequences. These are formed by the climatic effect causing chemical changes due to excess of evaporation over rainfall resulting in strong upward leaching. Calcium sulphate (as strong "gypcrete" or weak "gypcrust") and sodium chloride ("saltcrust") are amongst the most widespread crusts found in the Arabian subcontinent. These crusts result in the build-up of

sulphate and chlorides.

A yet another salt source to the local aggregates are the numerous sabkhas which have developed from coastal lagoons and which have a high water table maintained by sea-water percolation. Sabkhas become natural evaporating pans saturated with brines, often at high surface temperatures, generating chloride, sulphate and carbonate minerals in their surface crusts. In typical Gulf coastal sabkhas sulphates and chlorides are reported to form around 40 and 28 percent respectively of the local soluble salt contents¹⁰⁷. It has been reported that wind blown sulphates and chlorides from sabkhas and salinas contaminate the dune sands; e.g. in Dubai 68% and 16% of dune sands analyzed within a distance of 0.5 km downwind of the coastal sabkha showed sulphates and chlorides in unacceptable high levels. Only at distances of 35 km and 16 km from the coastal sabkha were sulphates and chlorides found within working limits¹⁰⁸.

5.1.2 Comments on Principal Coarse Aggregate

Characteristics Related to Concrete Making :

Until recently, the limited sources of igneous and metamorphic rocks have not been much quarried for construction materials, partly because they tend to occur away from many centres of development and partly because

they are associated with high ground. However, in some areas, such as near Jeddah, the quarrying of igneous and metamorphic rocks is increasing. In the absence of suitable rock materials and coarse gravels much of the aggregate used in the Eastern Province comes locally from calcrete pavements. On the Western coast of the Gulf, these calcrete duricrusts are developed on weak limestone formations (Palaeogene or Neogene limestones) and constitute the only suitable source of crushed rock for use as aggregates. Unfortunately, carbonates are not the only minerals to accumulate in the calcrete. Gypsum and chlorides are often present, particularly, with a shallow saline water table. Table 5.1 shows the chloride content in crushed aggregate obtained from a quarry in Hofuf and the author's analysis on several samples collected from various quarries in the Eastern Province showed chloride (Cl) content varying from .03 gm/kg to 0.36 gm/kg. Clay and cherts with long term expansive trends are also reported to concentrate locally in calcretes.

Physical and mechanical properties of typical calcrete aggregates from the Gulf have been compiled by the author from various sources and are listed in Table 5.2 where they are also compared with typical Ras al Khaimah

TABLE 5.1

Contaminants in Coarse Aggregates
from Quarry in Hofuf

Chlorides	53 mg/litre
Total Carbonate	75 mg/litre
Calcium Carbonate	59 mg/litre
Total Alkalinity	36 mg/litre
pH Value	8.5

TABLE 5.2

Typical Properties of Some Carbonate
Aggregates from the Gulf

Source	Specific gravity (SSD basis)	Water absorption, %	Aggregate crushing value, %	Aggregate impact value	Los Angeles abrasion value, %	Uniaxial compressive strength (cylinder) MN/m ²	Magnesium sulphate soundness (ASTM C88) % loss	Sodium sulphate soundness (ASTM C88) % loss	Sulphate content* % dry weight SO ₃	Chloride content % dry weight Cl
<i>Neogene calcaretes/dolocretes</i>										
Eastern Saudi: mean	2.61	3.8	28	-	44	101	28	-	0.24	0.03
typical range	2.52-2.70	0.8-10.6	26-31	-	29-77	23-200	2-54	-	0.16-0.34	0.014-0.051
Dubai: mean	2.52	4.2	21	26	28	-	52	-	0.21	0.022
typical range	2.37-2.64	2.4-9.2	18-24	24-29	23-33	-	19-87	-	0.01-1.09	0.004-0.030
<i>Palaeoogene calcaretes/dolocretes</i>										
Eastern Saudi: mean	2.55	2.6	28	-	35	-	-	-	0.05	0.02
typical range	2.51-2.70	2.4-3.2	22-35	-	23-50	-	-	-	-	-
Qatar: mean	2.60	1.4	24	22	-	53	17	14	5.10	0.03
typical range	2.60-2.78	0.5-4.1	15-32	-	-	3-109	10-25	11-17	0.01-0.075	0.01-0.14
Qatar: mean	2.60	2.7	26	-	28	-	-	28	0.86	0.04
typical range	2.49-2.71	0.3-6.2	18-32	-	23-53	-	-	-	-	-
<i>Palaeoogene limestone/dolomite</i>										
Kuwait: mean	2.46	4.9	29	31	-	161	17	-	0.24	0.15
typical range	2.13-2.65	0.4-11.0	19-45	25-36	-	20-432	2-53	-	0.03-1.31	0.004-0.019
<i>Quaternary limestone</i>										
Ras al Khaimah: mean	2.70	0.5	24	22	31	-	1	0.5	0.025	0.01
typical range	2.6-2.74	0.3-0.7	23-25	-	26-37	-	0-3	0.4	0.01-0.04	0.003-0.02

* Liabile to variation with groundwater conditions

limestone which was in popular use as aggregate all over the Middle East sometime back. It is reasonable to conclude that many calcretes traditionally used in the area cannot be considered first class materials on purely physical and mechanical grounds apart from possible chemical problems but a few problems deserve special mention :

Firstly, a marked variation in properties such as crushing value, abrasion, soundness and absorption in aggregates obtained from very closely located quarries. In a study¹⁰⁹ carried out at UPM, 12 quarries were evaluated for aggregate characteristics in the Eastern Province of Saudi Arabia. This study showed that of the 12 quarries the best and the worst aggregates came from two quarries, both of which were located close to each other in the Hofuf area. The aggregate properties from these two quarries are listed in Table 5.3 which shows the marked variations in the abrasion, soundness and water absorption characteristics. Fookes and Higginbottom¹¹⁰ have quoted data from tests carried out by Engineering Geology Ltd., showing wide variations in the physical and mechanical properties of calcretes and dolocrete in Kuwait and Bahrain. This is explained in geological terms by the random and variable nature of the hardening process resulting in a lack of

consistency and marked changes at close distances in the veins of rocks forming the source material.

Secondly, limestones and most calcretes are very dusty on crushing (Plate 5.1). It has been observed in this study that chlorides and sulphates tend to become concentrated in the fine fraction and the dust obtained during crushing.

Tables 5.4 and 5.5 show the tests carried out in this study on laboratory aggregate samples and on the fine fraction and dust which accompanied these aggregates. It is seen that in certain cases the concentration of chloride and sulphate contents in the finer fraction and the dust is 5-6 times as much as in the lab samples of the aggregates. As this dust and finer fraction usually go into concrete on site as package along with the aggregate it is almost certain that, even if this finer fraction and dust constitute 5% by weight of the aggregate, the chloride content in concrete would be more than the value estimated on the basis of lab tests carried out on coarse aggregate fraction. The dust and fine fraction constitute a somewhat hidden source of substantive chloride supply to concrete which tends to be ignored by concrete technologists in the area. Further, the finer fraction and the dust creates a high water demand in concrete aggregates and shrinkage and other

TABLE 5.3

Sharp variation of Properties of Aggregates from two quarries located in Hofuf

Quarry No.	Absrasion % wear	Soundness % loss	Unit wt. (lb/ft ³)	Apparent sp.gr	Bulk sp. gr.(S ₁ S ₁ D)	Absorption %
1	48.52	20.81	87.99	2.71	2.49	5.49
2	22.95	1.26	91.48	2.70	2.64	1.39

TABLE 5.4

Chloride data for Dust and Fine Fraction
of Aggregates Passing Sieve No.50

Test No.	Coarse Aggregates			Fine Aggregates (Sand)		
	Location	Source No.	gm/kg	Location	Source No.	gm/kg
1	Dhahran	1	0.38	Half Moon	1	0.35
2	"	2	0.2	"	2	0.43
3	"	3	1.4	"	2	0.72
4	"	4	0.42	Dammam	1	0.14

TABLE 5.5

Chloride Content for Aggregates
According to BS812 Part IV 1976

Test No.	Coarse Aggregates			Fine Aggregates (Sand)		
	Location	Source No.	g/kg	Location	Source No.	g/kg
1	Dhahran	1	0.17	Halfmoon Bay	1	0.177
2	"	1	0.122	"	1	0.12
3	"	2	0.030	"	1	0.14
4	"	2	0.034	"	2	0.389
5	"	3	0.217	"	2	0.347
6	"	3	0.36	"	2	0.347
7	"	4	0.161	Dammam	1	0.028
8	"	4	0.154	"	2	0.014
				"	2	0.014

problems associated with a high water/cement ratio will be experienced with the induction of this dust.

Thirdly, the reported presence¹⁰ of chert as a silica source is most likely to generate alkali-silica reaction in concrete causing pattern cracking in the long run. Also, many Middle East limestones are dolomitic in nature, as in Bahrain and Qatar, where the calcium carbonate content is typically almost 50-55% remainder being mostly dolomite. Impure dolomitic limestones containing clay minerals have been known to generate alkali-carbonate reaction^{11,12} in which the rock undergoes de-dolomitization, the dolomite converting to calcite and brucite ($\text{Mg}(\text{OH})_2$) with resulting increase in volume and cracking. There are reported case histories¹¹³ of Bahrain dolomites being specially reactive in this manner in the presence of moderate to high sulphate and chloride contents.

The aggregate from Eastern Province also shows similar tendencies. In a study¹¹³ carried out at UPM, aggregate samples collected from quarries in Dhahran showed alkali-silica reactivity beyond the permissible limit. It is the author's belief that these reactions progress fairly slowly and the more familiar and intensive sulphate and

chloride attacks resulting in expansive concrete cracking and steel corrosion precede these reactions and become apparent in shorter time and in a more dramatic and devastating form. However, the role played by the short-term damage due to these reactions as feeders and harbingers to other aforesaid more apparent forms of deterioration remains masked by lack of attention and research directed toward this problem.

5.1.3 Sands of the Arabian Peninsula

The active wadi channels near areas of high relief have been mentioned as the most important sources of relatively trouble free aggregates. Leaching by flood water usually keeps chloride and sulphate levels acceptably low and the water deposited silica sands have a broad grading range which suits concrete making techniques. However, the active wadis in the Arabian peninsula are extremely limited and the deposits of some active wadis are approaching exhaustion.

Also, in general, the role of water in desert weathering is extremely limited and the absence of well graded water-deposited silica sands entails a heavy dependency on wind blown dunes and beach or coastal sands as sources of

fine aggregate.

Coastal sands, being more or less sorted by wave or current action, usually have narrower (and often finer) gradings than are preferred for concrete. A survey¹¹⁴ of beach sands in Qatar showed that, out of 365 grading analyses, 46% failed to conform completely to any of the BS882 grading zones, usually because they were too narrowly graded. Another study¹¹⁴ shows that out of 114 grading analyses of Dubai beach sands 62% fell within zone 4 (BS 882) and a further 26% were finer than in zone 4 (Figure 5.3). Figure 5.4 shows the typical narrow grading of the Half Moon beach sand.

The beach sands usually have smooth rounded grains but may also be absorptive, trap air, chlorides and sulphates in their porous, sometimes hollow particles being of carbonate origin consisting of the worn debris of young marine organisms. However, the most serious drawback of the beach sands is their contamination to unacceptable chloride and sulphate levels. Table 5.5 shows an analysis of chloride contents for the Half Moon beach sand. In all the samples tested the level of contamination works out to be beyond the accepted limits.

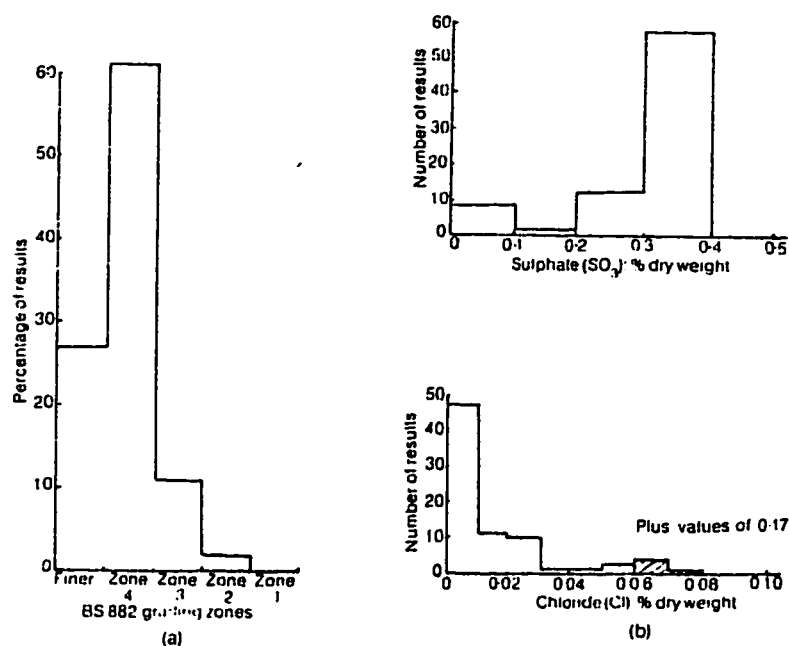


Fig. 5.3 : Some properties of Dubai sands:(a) beach sands-frequency distribution of gradings;(b) beach sands(from one typical section of beach) - frequency distribution of sulphate and chloride content; current working limits exceeded in hatched areas.

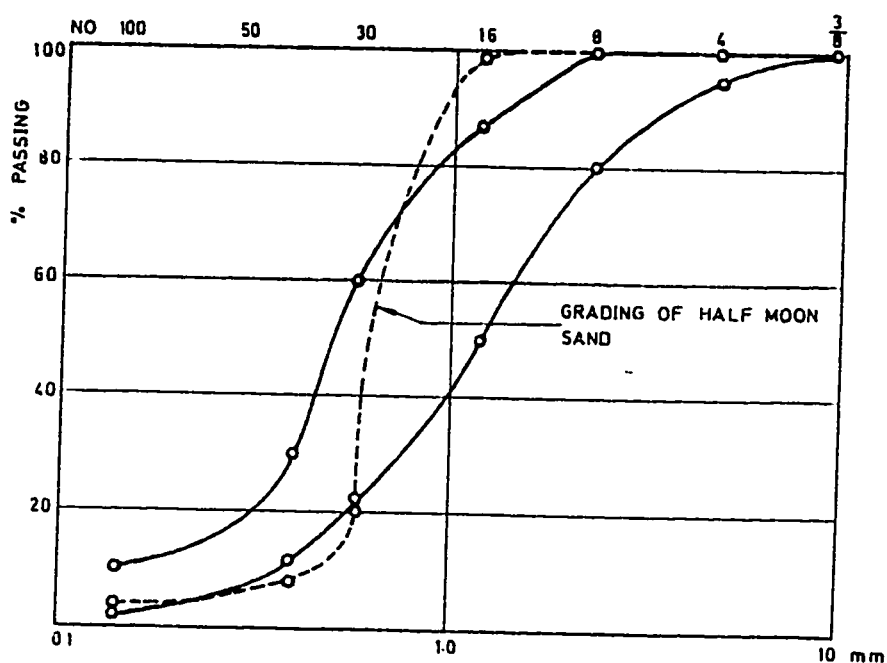


Fig. 5.4 : ASTM C 33 grading limits for fine aggregates

The wind-blown dune sands of desert interiors are generally viewed with disfavour because of their narrow and fine grading (mostly 50-600 μm) with silt contents often of about 10%. In the Eastern Province of Saudi Arabia dune sands are frequently used, although without much proper discretion exercised in their selection. Particles upto 1 mm diameter or more can be moved by the wind but will not be carried up the slopes of dunes. They may form thin deposits at their bases and in interdune planes but will still be narrowly graded and rarely exist in commercial quantities. It is, however, sometimes possible to select dunes or parts of dunes, with a better grading, or to blend dune sand with other material, such as crushed rock fines, assuming that this product is acceptable in respect of chlorides, sulphates and other contaminants.

Dune sands may also be heavily contaminated with chloride and sulphate salts when these salts are wind-blown from sabkhas and salinas. In a study in Dubai, dune sands were found to be heavily contaminated by sulphates and all the samples tested had acceptable limits only at a distance of 35 km from a coastal sabkha and an acceptable chloride limit distant 15 km from this sabkha.

The smooth rounded grains of dunes and coastal sands have a lubricating effect in a concrete mixture and contribute helpfully in improving the workability of concrete. However, any such advantage is largely offset by the fineness of the grain-size and the narrow grading of these sands. These two characteristics result in increased water demand and cement content with a tendency to bleed and segregate. These difficult parameters need proper skills and judicious mix design techniques to fix proper proportions of fine to coarse aggregate in order to avoid increased shrinkability associated with higher water demand and early thermal cracking associated with increased cement content.

In the face of some of the aforesaid difficulties with natural fine aggregates there is an increasing tendency in some areas to use crushed rock sands. These materials have been found to have a poor particle shape and a correspondingly adverse effect on concrete workability and density. However, their main drawback, specially when they are derived from calcretes in the Eastern Province, is that contaminants in the natural stone tend to become concentrated in the fine fraction during crushing. These crushed rock sands may contain as much as 25% of total impurities

such as gypsum flour (CaSO_4), chlorides, clays and salts. There is usually a high proportion of dust also which helps in enhancing water demand resulting in shrinkage and porosity and other problems associated with high w/c ratio.

5.2 CEMENT

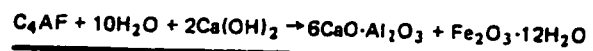
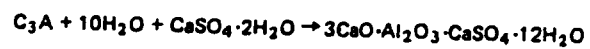
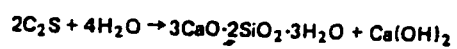
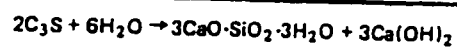
Inspite of a wide variety of cement compositions available to date only a limited number of relationships between cement composition and the progress of corrosion of steel in concrete have been established¹⁵. All Portland cements are known to produce sufficient alkalinity in their pore water as a result of the hydration process (Table 5.6) to place steel in a pH region (≈ 12 to 13.5) in which it is known to passify against corrosion.

The direct effect of *cement content* on corrosion of embedded steel has been reported to be negligible^{16,17}. However, the physical and chemical properties of cement are known to have an effect and the two reportedly most significant parameters in this category are :

- (i) Tricalcium aluminate (C_3A) content of cement
- (ii) Fineness of cement.

TABLE 5.6

**Hydration Reactions of
Portland Cement
(Simplified)**



The effect of C_3A content of cement on corrosion becomes a very important parameter in the Gulf area because of the fact that in the two types of cement commonly used in the Gulf States viz. Type I Portland cement and Type V Portland cement, the C_3A content varies widely. The local Saudi Type V Portland cement obtained from the Saudi Cement Factory at Al-Hasa was analyzed for chemical composition (Table 5.7) and the C_3A content was found to be only 2%. The aforesaid Saudi concern does not manufacture Type I cement. A sample of imported Type I cement (Japanese brand) was also analyzed for chemical composition (Table 5.7) and was found to contain 9% C_3A . This widely differing C_3A concentration raises the important question of its effect on corrosion of steel reinforcement in this area.

It is reported^{118,119,120} that chloride ions react with hydrated tricalcium sulfo-aluminate hydrate (shown in Table 5.6 as $C_3A \cdot CaSO_4 \cdot 12H_2O$) in cement paste to form tricalcium chloroaluminate compound ($C_3A \cdot CaCl_2 \cdot 10H_2O$) in solid solution with hydrated calcium aluminate. There are conflicting data on the extent of chloride combining chemically with $C_3A \cdot CaSO_4 \cdot 12H_2O$. However, Clear¹²¹ and Lewis¹²² report that when the total chloride content is near corrosion threshold level only 15 to 50 percent of

TABLE 5.7

Chemical Composition of the Cements (%) Weight

Type of Cement	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	CaO	MgO	SO ₃	K ₂ O	Na ₂ O	L.O.I	I.R	Total	Calculated Compounds			
												C ₃ S	GS	C ₃ A	C ₄ AF
Ordinary Portland (Type I)	21.6	5.42	3.16	64.07	1.51	1.94	0.83	0.27	0.98	0.07	99.9	50.6	24.16	9.02	9.61
Sulphate-resistance (Type V)	22.88	3.52	4.28	64.35	2.01	1.71	0.21	0.19	0.75	0.22	100.12	53.44	25.28	2.09	13.02

the chloride will combine, the balance remains as free uncombined chloride in the paste. Research carried out by Monfore and Verbeck¹¹⁸ and Roberts¹²⁰ indicate that as much as 75 to 90 percent of the chloride actually present would combine in the paste in the form of chloroaluminate compound and only the remaining exists in the paste as uncombined free soluble chloride. The important considerations bearing on the corrosion situation are two : *Firstly*, only uncombined soluble chlorides induce corrosion, while combined chloride is believed to have little effect. *Secondly*, the relative concentrations of combined and uncombined soluble chlorides in the paste would depend upon the *total amount of chloride present, the tricalcium aluminate content of the cement and the degree of hydration of the cement.*

In view of the very important role of C_3A content in cement in determining the concentration of corrosion inducing uncombined soluble chloride in cement paste, the comparative positions of Type I and Type V cements from the point of view of corrosion deterioration in concrete call for special attention. Laboratory investigations were, therefore, carried out on Type I and Type V cements to study the C_3A content effect on corrosion.

Tests were carried out to study the direct reaction of added CaCl_2 with Type I and Type V cements with the progress of hydration at room temperature. Type V cement used was obtained from Saudi Cement Factory at Al-Hasa and contained 2% C_3A . Type I cement was an imported Japanese brand containing 9% C_3A . The detailed chemical compositions of both types are shown in Table 5.7 and 1.85% of $\text{CaCl}_2 \cdot 2\text{H}_2\text{O}$ by weight of cement was added to a cement paste of w/c ratio of 1. These mixes of Type I and Type V cements with water and CaCl_2 were continually shaken for 10 days, were filtered every day and the filtrates analyzed for the free chloride content. The results are shown in Table 5.8 which are plotted in Figure 5.5. It is seen that Type I cement by virtue of its higher C_3A content is more effective in removing soluble free chloride by combining it chemically than the sulphate resisting cement Type V.

To confirm the effect of cement composition-soluble free chloride interactive relationship on corrosion, tests were carried out on two sets of concrete slabs embedded with # 4 rebars (Plate 4.4). One set was cast with Type I and the other set was cast with Type V. 6.2g/l of sodium chloride was mixed with the mixing water. The concrete mix was 1.2: 2: 3 with w/c ratio 0.65. Corrosion

Fig. 5.5 : Reaction of CaCl_2 solution with Portland cement Type I and Type V at 24°C

○ Type I cement C_3A 9%
 Δ Type V cement C_3A 2%

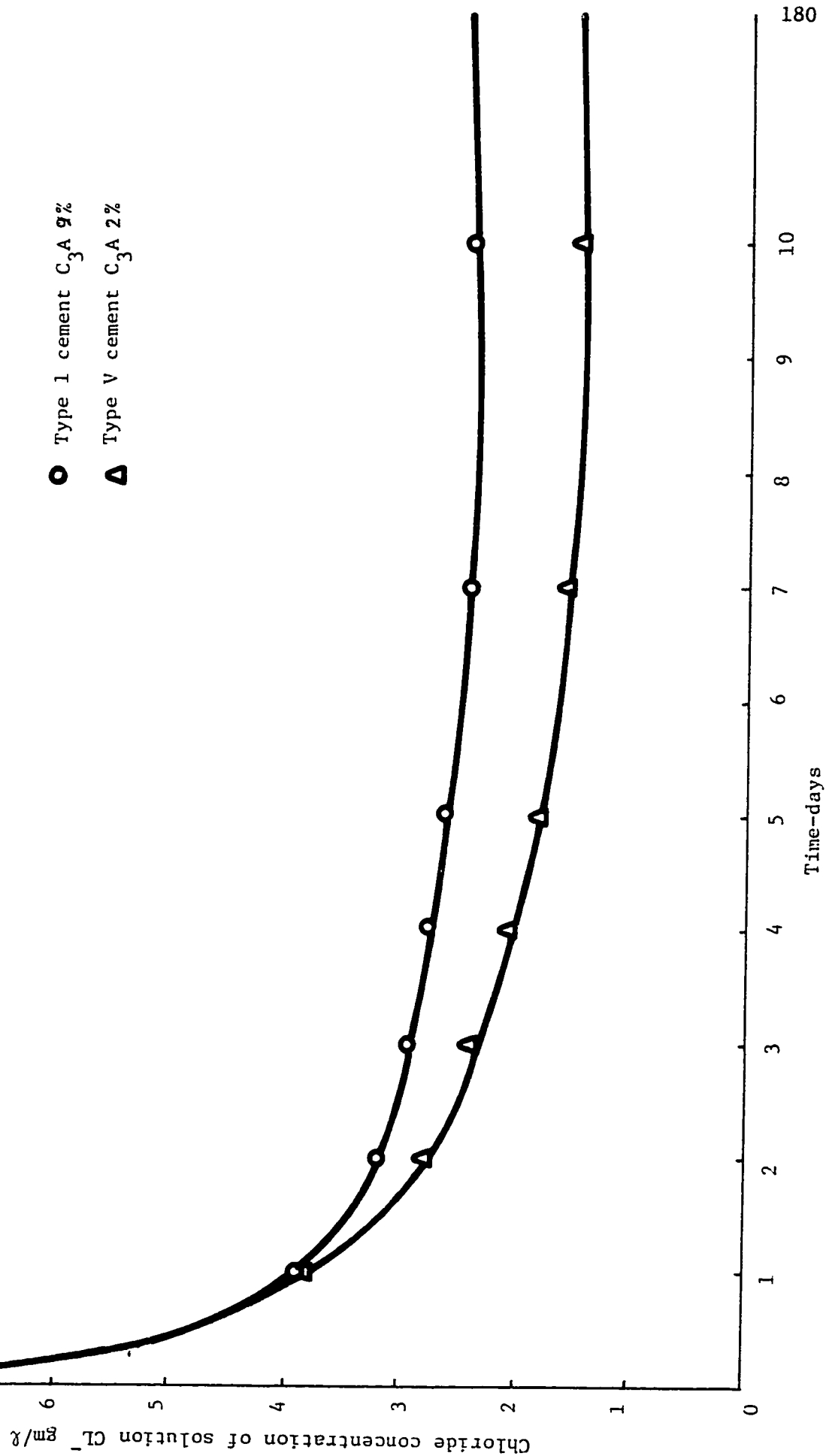


TABLE 5.8

1.85% $\text{CaCl}_2 \cdot 2\text{H}_2\text{O}$ By Wt. of Cement Add to Mixing Water(50 gm cement in 50 ml $\text{CaCl}_2 \cdot 2\text{H}_2\text{O}$ solution)

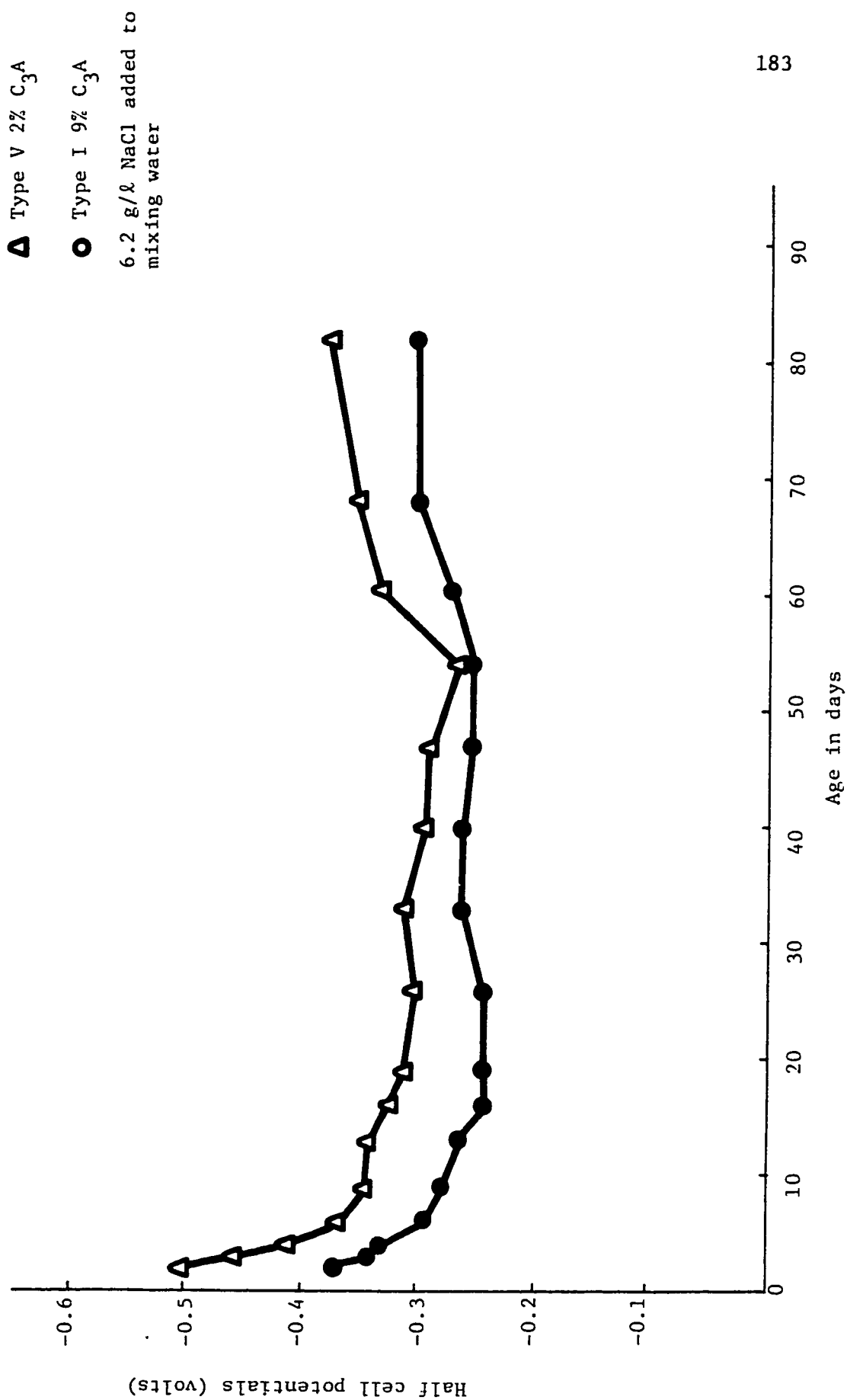
Cement Type	Chloride Concentration of Solution g/l-days							
	0 days	1	2	3	4	5	7	10
I	9.0	3.7	2.7	2.4	2.0	1.8	1.6	1.5
V	9.0	3.8	3.2	2.8	2.7	2.6	2.4	2.4

was monitored for about 3 months with a Cu-CuSO₄ half cell. The results of these tests are shown in Figure 5.6. This figure confirms the effect of type of cement on free soluble chloride and its apparent effect on corrosion. Type I cement appears to be more protective against corrosion than Type V.

This effect is exhibited on reinforced concrete samples in Figure 5.7 which has been extracted from Verbeck's research¹²¹ related to the influence of C₃A content of cement on corrosion. This data shows that concretes showed higher spalling and cracking when cements with less C₃A content were used.

This data brings us to a discussion of the practice in the Kingdom and also in other Gulf States wherein there is an increasing trend to use Type V cement for the sub as well as for the superstructures of the concrete construction in order to obviate the deterioration due to sulphate attack on concrete. In fact in recognition of the sulphate deterioration of concrete in this region, the major output of the local cement industry is of Type V cement. Two conflicting factors in this situation make a clear cut judgement in this matter difficult. It is true that the deterioration situation of concrete in soils and groundwaters

Fig. 5.6 : Half cell potential monitoring of corrosion activity for Type I and Type V Portland cements



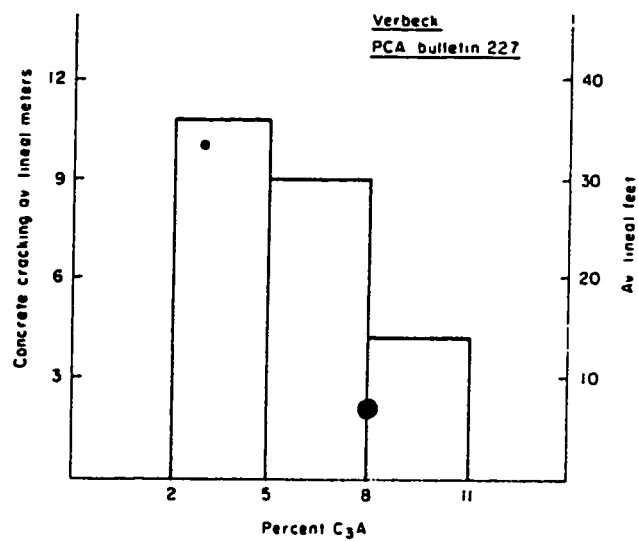


Fig. 5.7 : Effect of C_3A content on cracking of concrete due to steel corrosion.

containing abundance of sulphate salts is truly grave and warrant an unreservedly strong recommendation for the use of Type V cement. However, in the case of superstructures the situation is complicated by the fact that concrete is not directly attacked by the solid sulphate salts, but only by their solutions in water¹²². An analysis of the superstructure concretes cores shows a high sulphate content in these concretes. The sulphate content of the aggregates and mixing water is also high as shown in Tables 5.9 and 5.10. This undoubtedly puts an average sulphate content of the magnitude of 1.4 lb/cu.yd. from the constituent materials themselves. More sulphate salts may permeate concrete from salt charged ambient environment. These salts will, however, require a constant supply of moisture to mount an effective sulphate attack on concrete. This is apparently readily available through capillary action upto several feet from above the ground level and also from high ambient humidity and due to the exposed concrete surfaces. This appears to be the reason for sulphate attack being usually confined to a few feet above the ground level and to the external mortar and concrete. However, more research is necessary to evaluate the relative advantages and disadvantages of the use of Type V cement for

TABLE 5.9

Sulphate Content for Aggregates
(Fine Fraction Passing Sieve No. 50)

Test No.	Coarse Aggregates			Fine Aggregates (Sand)		
	Location	Source No.	% SO ₃	Location	Source No.	% SO ₃
1	Dhahran	1	6.05	Half Moon	1	4.248
2	"	2	4.446	"	1	4.108
3	"	2	4.643	"	2	6.355
4	"	3	3.169	Dammam	1	1.470
5	"	4	2.875			

TABLE 5.10
Sulphate Content for Tap Water

Test No.	Location	% SO ₃	gm/l
1	Safwa	.0538	0.538
2	Qatif	.0534	0.534
3	Dammam	.03859	0.386
4	Khobar	.0359	0.359
5	Dhahran		

superstructure construction in the Gulf States.

Fineness of cement increases the water demand of a concrete mix, thus increasing the net water to cement ratio (w/c) and hence the permeability of the concrete. A high fineness cement may also exhibit larger drying shrinkage than one of low fineness, hence increasing the probability of cracking in the hardened concrete, allowing aggressive agents such as salts, oxygen and moisture to penetrate more readily to the embedded reinforcing steel.

High alkali content in cements can help to retard corrosion by enhancing the pH of the pore water fluid of concrete and thus strengthening the passivating film on reinforcement. However, high alkali content may lead to difficulties with alkali-aggregate reactions.

5.3 MIX WATER

Table 5.11 shows the chloride content for ordinary tap water in the Dhahran-Dammam-Alkhobar area which is commonly used for mixing concrete.

Table 5.12 shows the contribution this component makes toward the total base level chlorides in a typical concrete mix. It is seen that for average and high

TABLE 5.11
Chloride Content for Tap Water

Test No.	Source	%CL ⁻	g/l
1	Safwa	0.18	1.8
2	Qatif	0.14	1.4
3	Dammam	0.15	1.5
4	Khobar	0.13	1.3
5	Dhahran	0.14	1.4

TABLE 5.12

Estimated Base Chloride Content for Local Material in Typical Concrete Mixes for 3/4" Max. Size of Aggregate Fine Sand (FM = 2.5) and for Consistency (3-4 inch Slump)

Mix 1 Lean Mix										Mix 2 Rich Mix												
w/c	Max. size of agg.	Water lb/cu. yd.	Cement lb/cu. yd.	Fine agg. lb/cu. yd.	Coarse agg. lb/cu. yd.	Total CL ⁻ lb/cu. yd.	w/c	Max. size of agg.	Water lb/cu. yd.	Cement lb/cu. yd.	Fine agg. lb/cu. yd.	Coarse agg. lb/cu. yd.	Total CL ⁻ lb/cu. yd.									
0.6 3/4														340	565	1200	1800	0.45 3/4	340	755	1040	1800
Ave. CL ⁻	CL ⁻ %	0.148		0.0175	0.0156			0.148			0.0175	0.0156										
con-	CL ⁻																					
tent lb		0.503		0.21	0.281	0.994		0.503		0.182	0.28	0.966										
High																						
CL ⁻	CL ⁻ %	0.18		0.039	0.036			0.18		0.039	0.036											
con-	CL ⁻																					
tent lb		0.612		0.468	0.648	1.73		0.612		0.406	0.65	1.67										

aggregate chloride contaminations the contribution of mix water is 51 and 35 percent respectively. In the two cases substantive induction of chloride salts occurs through mix water.

5.4 CONCRETE QUALITY

5.4.1 Influence of Mix Design Factors

5.4.1.1 Absorption and Water-Cement Ratio : In practice, compressive strength has universally become a convenient measure for concrete quality. This is largely because of the great deal of knowledge that has been accumulated from the use of the concrete crushing test as a control for concrete production. The prestige which has accrued to strength as a measure of concrete quality has in turn encouraged the use of non-destructive tests to provide an estimate of the strength of in situ concrete.

Nevertheless it has long been recognized that strength can only be an indirect measure of durability. Nurse has pointed out that it is possible to envisage concretes of equal strength but of radically different pore structure because of cement content or aggregate grading. Therefore, *other parameters governing the ease of movement of liquids and gases*

through the concrete would provide a better assessment. This *criterion* holds equally valid for the three most significant and well recognized threats to concrete durability viz corrosion of rebars, freeze-thaw phenomenon and the sulphate attack. The common denomination in each case being the resistance of impermeable concrete to the penetration and ingress of aggressive media. However, in the case of rebar corrosion this characteristic is all the more significant as less water can enter and remain in an impervious concrete making its electrical conductance low. From the durability point of view, therefore, permeability is the primary criterion of concrete quality and is undoubtedly the single most important characteristic of concrete which affects its durability performance in aggressive environments.

The paste permeability is not a simple function of its porosity but is governed by the pore system and the capillary porosity of the paste. In simpler terms the two operative characteristics of concrete which would affect its durability are : (a) concrete absorption and (b) the rate of moisture movement through concrete.

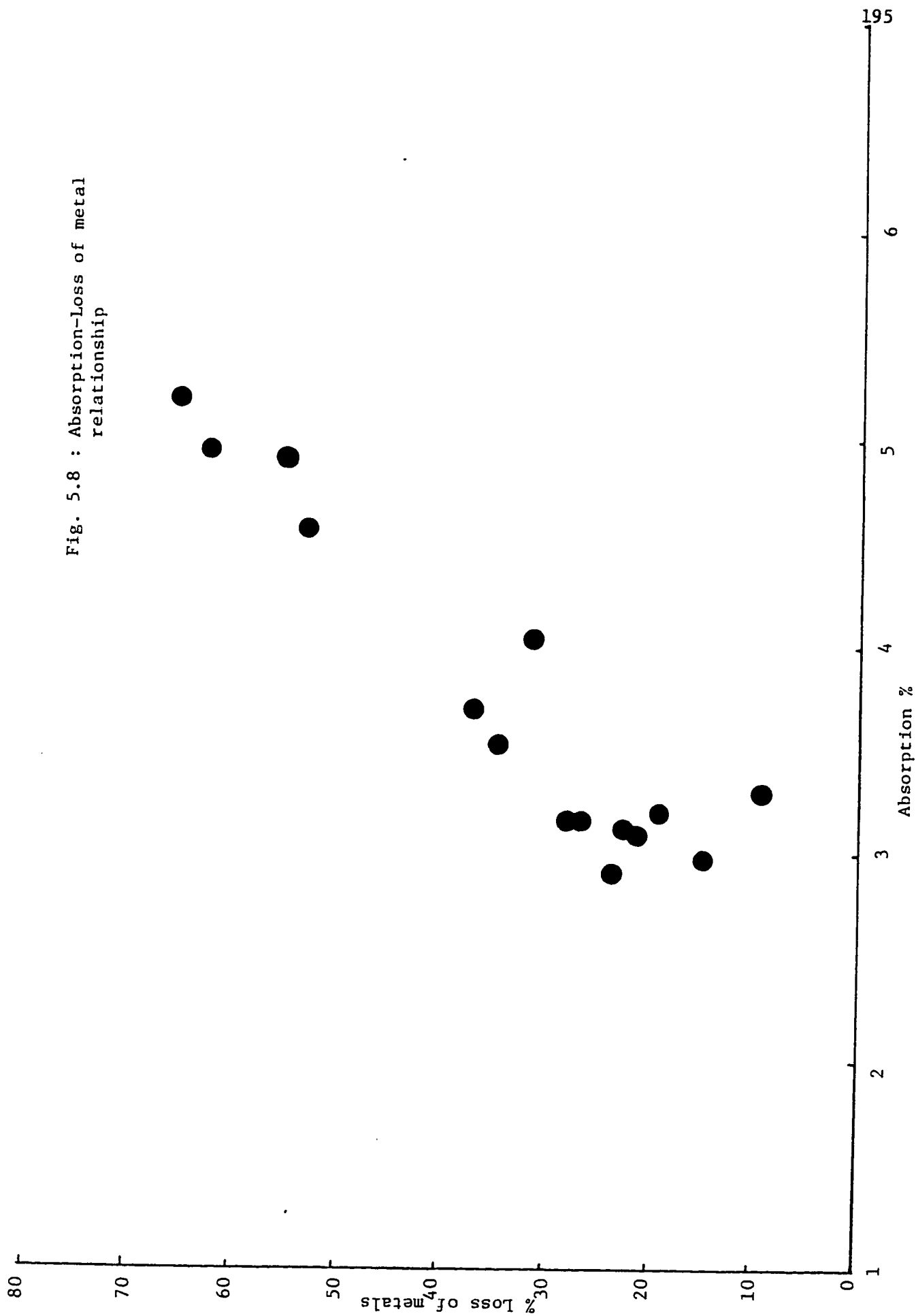
In the present study a correlation of concrete quality (in terms of its absorptive characteristics) and rebar corrosion (in terms of loss of metal) was sought to be

established. Field samples in the form of cores removed from ten buildings were subjected to 30-minute water-absorption test in accordance with BS 1881 Part V 1970 as detailed in chapter 3. The loss of metal was determined on bars removed from these cores and the relationship is plotted in Figure 5.8. While plotting this relationship an attempt was made to hold other parameters such as cover to reinforcement and chloride content affecting loss of metal within narrow limits of variation to obtain a meaningful relationship between absorption and loss of metal. The cover to concrete in these field samples varied in a range of 0.70 inch to 1.1 inch and chloride content varied in a narrow range specified with a standard mean deviation of 0.653 about a mean value of 8.06 lb/cu.yd. of concrete. It must be appreciated that in field studies such ranges of variation are inevitable. In fact a large number of observations were excluded from the plot to maintain as narrow a range of variation of the other affecting parameters as possible.

Figure 5.8 shows the plot of absorption against loss of metal. It is clear from the trend of data plotted that absorption characteristics of concrete very markedly affect the extent of corrosion. A 65% increase in absorption

TABLE 5.13
Absorption - Loss of Metal Data

Building No.	Core No.	Bar No.	Loss %	Absorption	CL ⁻	Cover
3	1	1	53.75	4.55	8.0	0.75
5	3	1	9.2	3.31	6.6	1.0
6	1	1	31.41	4.05	8.0	0.7
7	3	1	21.4	3.09	8.0	1.1
7	5	1	14.8	2.98	6.92	0.85
7	6	2	19.2	3.19	7.14	0.80
8	2	1	22.4	3.11	8.2	1.1
8	3	1	23.6	2.91	8.24	0.87
8	4	1	26.5	3.14	8.66	0.9
8	4	2	27.5	3.14	8.2	0.9
9	1	1	34.5	3.52	8.4	1.0
9	3	1	37.1	3.62	8.92	1.0
10	2	1	55.5	4.88	8.38	1.12
10	3	1	62.5	4.96	8.56	1.1
10	4	1	65.4	5.2	8.74	1.1



value from 3 to 5 increased the loss of metal threefold from an average value of 20% to 60%. It seems that from the corrosion point of view an absorption value of 2 to 2.5 will signify a good quality concrete, a value of 3 to 3.5 an average quality concrete and values greater than 3.5 will signify a poor quality concrete. It is also seen that of the concretes investigated 50% fall in the average category and the remaining 50% in the poor category. It must be noted that these concretes are 22-27 years of age and that 10-15% loss of metal will in most cases result in concrete cracking. This performance against a scale of satisfactory functioning for 50 years is by no means entirely satisfactory.

What constitutes acceptable values of absorption and permeability? It is difficult to answer this question in terms of permeability or absorption values confidently as there is virtually no data at all on this aspect. Whatever guidance is available in this direction is in terms of w/c ratio on the grounds that in a properly designed concrete mix the most significant factor affecting concrete absorption and permeability characteristics is the w/c ratio. In construction practice the two factors which affect this important parameter are curing and compaction.

Figure 5.9 shows the effect of w/c ratio on the permeability of concrete using 1/4 in. and 3 in. maximum size aggregates. Given sufficiently impermeable aggregate and a crack free matrix, the permeability of concrete is determined by the permeability of the paste phase, which can be reduced to extremely low values at w/c ratios below 0.4 (Figure 5.10). As deleterious ions such as Cl^- move through concrete by bulk flow and concentration diffusion, it is reasonable to assume that reduction in permeability will retard the progress of ions toward the steel, and hence increase the time necessary for initiation of corrosion. A similar barrier effect applies to the permeation of oxygen and moisture. Application of this concept can be seen in Figures 5.11a,b,c. Here 12 x 12 x 6 inch concrete slabs of varying w/c ratio having steel embedded at depths of 0.5 inch, 1 inch and 2 inches were subjected to a cycle consisting of 4 days ponding with a 3.5% NaCl solution followed by 10 days drying at 50% RH. The effect of w/c ratio is easily seen: Those slabs prepared at a w/c ratio of 0.6 exhibiting active corrosion much more rapidly than those prepared at a ratio of 0.45. Slabs prepared at the lowest w/c ratio (0.35) showed active corrosion (as measured by a sharp jump in the curve of potential vs time) only for

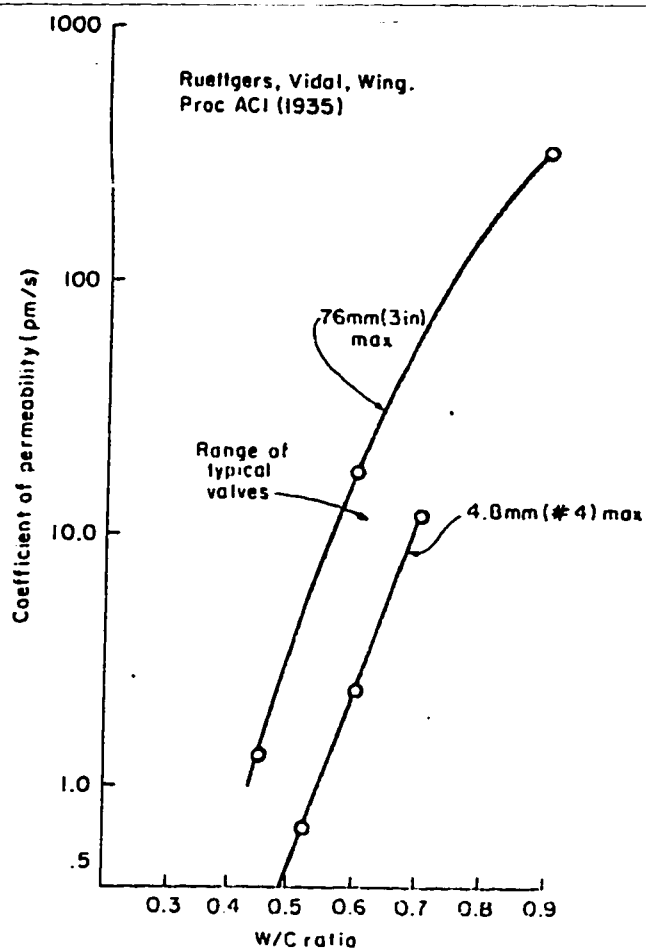


Fig. 5.9 : Permeability of concretes as influenced by w/c ratio.

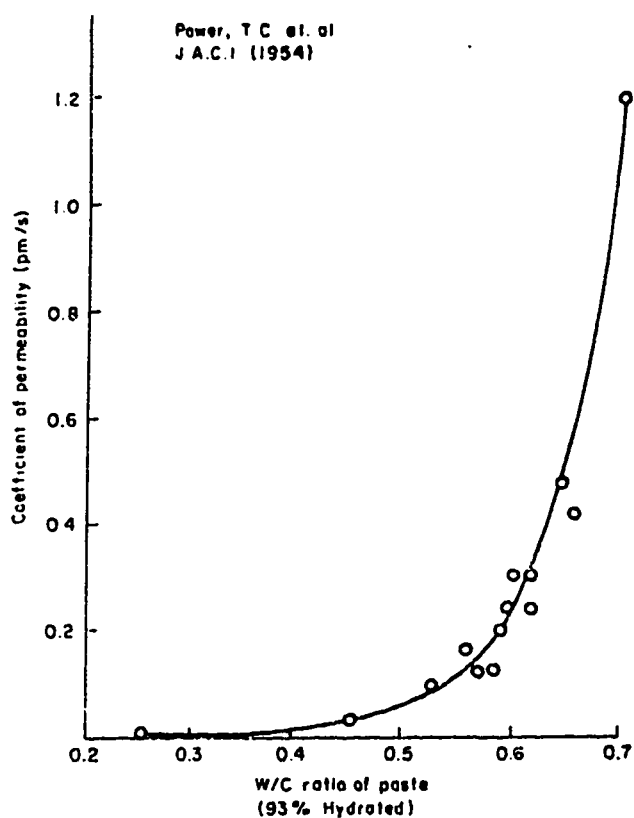


Fig. 5.10 : Permeability of mature cement pastes as influenced by w/c ratio.

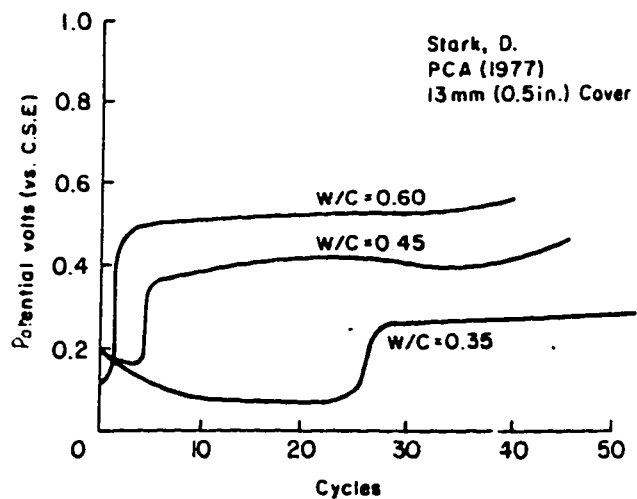


Fig. 5.11a - Potential vs time for various w/c ratios at 13 mm (0.5 inch) cover.

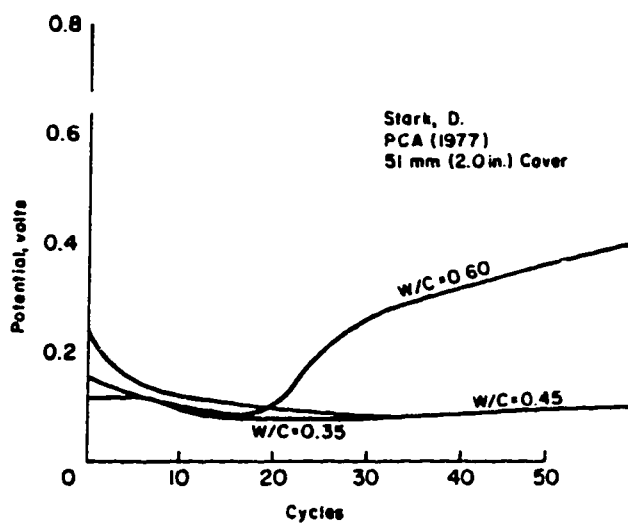


Fig. 5.11b - Potential vs time for various w/c ratios at 51 mm (2.0 inches) cover.

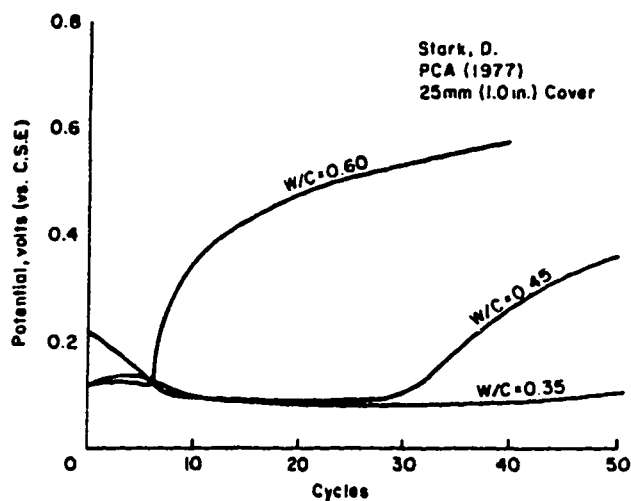


Fig. 5.11c - Potential vs time for various w/c ratios at 25 mm (1.0 inch) cover.

very thin cover. In another study exposure tests on 20 ft² slabs which were salted daily yielded similar results; concrete with a water-cement ratio of 0.45⁻ performed significantly better than concretes with water-cement ratios of 0.5 and 0.6, even with equal cement contents.

Tests carried out by the author on concrete slabs made with a w/c ratio of 0.65 resulted in active corrosion and the appearance of a crack over the reinforcement within 3 months of the casting.

Similar feedback is obtainable from field studies regarding the effect of water-cement ratio on the quality and durability performance of concrete. A part of the "Long-Time Study of Cement Performance in Concrete" consists of an investigation of the durability of reinforced concrete piles exposed to sea water; results after 15 to 17 years of exposure at locations in Massachusetts, Florida, and Southern California have been reported by Tyler¹³⁴. Durability was found to be largely a function of water-cement ratio of the three concrete mixes used. Specimens cast from concrete containing 7 bags of cement per cu. yd. (w/c ratio of about 5 gal per bag) performed best, and concrete containing 5 bags per cu. yd. (w/c ratio about 7 gal per

bag) performed poorest. It was concluded that a cover of 1.5 inches of even the 7 bag mix was insufficient in some cases to prevent rusting of the reinforcing steel and subsequent cracking of the concrete.

Spalling of concrete in the San Mateo-Hayward Bridge due to corrosion of reinforcing steel resulting from exposure to a marine atmosphere was carefully studied by Tremper, Beaton, and Stratfull¹³⁵. Factors contributing to the distress included water-cement ratios as high as 7.5 gal per bag and the resultant absorption of substantial quantities of chlorides.

Wakeman, Dockweiler, Stover, and Whiteneck¹³⁶ have described the experience of the Los Angeles Harbor Department in using concrete in marine environments. From their experience and the experience of others, they concluded that concrete used in reinforced concrete structures exposed to the action of sea water should be dense, impervious, relatively nonabsorbent, and have a minimum cement content of $6\frac{1}{2}$ and a maximum of $7\frac{1}{2}$ bags per cu. yd. Reinforcing steel, they advised, should have a minimum cover of 3 inches.

This discussion seems to suggest that the water-cement ratio should not exceed 0.40 for concrete exposed to brackish water or in contact with more than moderate concentrations of chlorides. If this water-cement ratio cannot be achieved due to workability problems the use of plasticizers may be invoked. In any case, it seems quite clear that the maximum water-cement ratio for durable concrete in the Gulf area should in no case increase beyond 0.45 *provided* the thickness of concrete cover to steel is increased by 0.5 inch over that postulated for a water-cement ratio of 0.40. A recommendation similar in nature has been made by ACI Committee 201 in its report entitled "Guide to Durable Concrete" ¹³⁷.

5.4.1.2 Cement Content : Cement content, for the most part, is important insofar as a higher cement content will enable a lower net w/c ratio to be used for the same workability. Clear¹³³ found little significant differences in protection of reinforcement when concretes were prepared at a *constant w/c ratio* and cement contents were varied from 564 lb/cu.yd. to 752 lb/cu.yd. However, it is possible that increased cement content may offer additional protection over and above that afforded by the reduction in w/c ratio if high C₃A content is used, as Cl⁻ may complex with

the C_3A hydrates in accordance with the discussion in the section 5.2 of this chapter.

5.4.1.3 Aggregate Factors : An overwhelming effect of w/c on permeability and concrete quality usually relegates other mix design factors to much lower importance. However, drastic reductions in the permeability and the quality of concrete are attributable to bleeding, segregation, water gain and the presence of underaggregate fissures . Crushed rock aggregates, specially angular or flakey, trap sizeable mobile water and air bubbles under their surfaces and produce zones which are very porous. This suggests the significance of the shape and size of the aggregate particles in influencing permeability. The Bureau of Reclamation Concrete Manual¹³⁷ confirms this and investigations by Carlson¹³⁸ show that lean mixes with a sand fineness modulus of 2.25 gave a permeability only 42% obtainable in a mix with a fineness modulus of 2.50. The use of smaller size aggregate and fine sand undoubtedly lowers permeability. However, in terms of mix design, such a reduction will atleast be partially offset by additional water demand to maintain a given workability. Further research is required to elucidate an interactive relationship between these two conflicting requirements.

5.5 INFLUENCE OF CONSTRUCTION PRACTICE FACTORS

We have seen how the quality of materials and concrete mix design can influence the degree of protection offered by concrete to embedded steel. The final set of variables which must be considered deals with the way the structure is designed, placed, finished and cured. These can be grouped under the heading of "Construction Practices". The design practice having the greatest influence on the corrosion process is the "clear cover". This factor being of very considerable importance is discussed in a separate section 5.6. In this section are discussed the effects of curing, consolidation and such practices and design factors which affect the degree of protection which concrete affords to reinforcement.

5.5.1 Curing

Adequate curing of concrete is very important in determining concrete quality from the point of view of durability but it is not always appreciated. The primary influence of curing is the effect that it has on permeability. For example, the permeability coefficient for cement paste after 1 day of hydration will be more than 25000 times that for paste hydrated for 7 days. Table 5.14 shows the effect

TABLE 5.14

Permeability to Water Rapidly
Reduces with Cement Hydration

Days of curing	Coefficient of permeability
fresh paste	1,150,000,000
1	36,300,000
2	2,050,000
3	191,000
4	23,000
5	5,900
7	1,380
12	195
24	48

of duration of curing on the water permeability of Portland *cement pastes*. The effect of curing on the permeability of concretes should be qualitatively similar but of course the permeability of *concrete* is also affected by bleeding, degree of proper consolidation, permeability of the aggregate, etc. Although concrete dries out slowly after completion of moist curing, the surface region dries rapidly with resulting cessation of hydration. Thus the most critical zone, that between the concrete surface and the paste-steel interface may have a permeability many-fold that of the centre-most concrete. During these investigations the author very often observed this phenomenon. Plate 5.2 shows the porous nature of the cover concrete used in the columns of a Dhahran mosque building erected with the help of imported skills.

5.5.2 Consolidation

The degree of consolidation of the concrete, especially in the vicinity of embedded steel, has a profound effect on corrosion. Insufficient consolidation provides channels of ingress for the ingredients of corrosion - water, chlorides, oxygen, and carbon dioxide - and when occurring as "honey combing" in the vicinity of the reinforcement promotes the formation of differential aeration corrosion

cells. For concrete with slump value of 3 inches or more, a good consolidation is easily achieved by internal vibration. However for heavily reinforced elements with lower slump concretes, normal construction practices cannot be relied upon, or else voids will form where concrete does not make adequate contact with the reinforcement. In one series of tests¹⁴⁰ the chloride content at a depth of 2 inches in concrete was about 12 times greater for concrete that had been compacted form 92 to 94 percent of maximum density than it was for maximum density concrete for the same mix design.

5.5.3 Cracking

An analysis of bridge deck deterioration data by Stark¹⁴¹ indicated that there was a definite correlation between the corrosion spalls and the presence of initial cracking in concrete. Even much before concrete is hardened and a structure is loaded shrinkage cracks, heat-of-hydration cracks and subsidence cracks appear on the surface. The governing factors are known, but these early cracks which mostly occur along the reinforcement are still the most difficult to control.

In most laboratory studies, specimens are unloaded and too small to develop realistic crack patterns; therefore corrosion of the steel must necessarily be due to diffusion of ions through the uncracked intact matrix towards the steel surface. In real life, however, salt solutions may readily penetrate cracks in large sections of concrete and cause rapid corrosion and spalling.

The causes of cracking can be traced to phenomena occurring in both fresh and hardened concrete. Dakhil¹⁴² et al, have shown that the fresh concrete cracking above reinforcement is attributed to restraint to settlement offered by the top bars. The magnitude of restraining stress is a function of concrete cover, bar size and slump (Figure 5.12). Once again, cover has the greatest influence, cracking of concrete slabs occurring in 96% of the slabs with 19 mm (3/4 inch) cover, but in only 20% of the slabs with 51 mm (2 inches) of cover. Increases in cracking are also noted for increases in slump and bar size. This can be attributed to the greater rate of settlement at higher slumps and the stress concentrations around larger diameter bars.

Cracking may also occur in hardened concrete, primarily due to shrinkage, thermal stresses, and possible

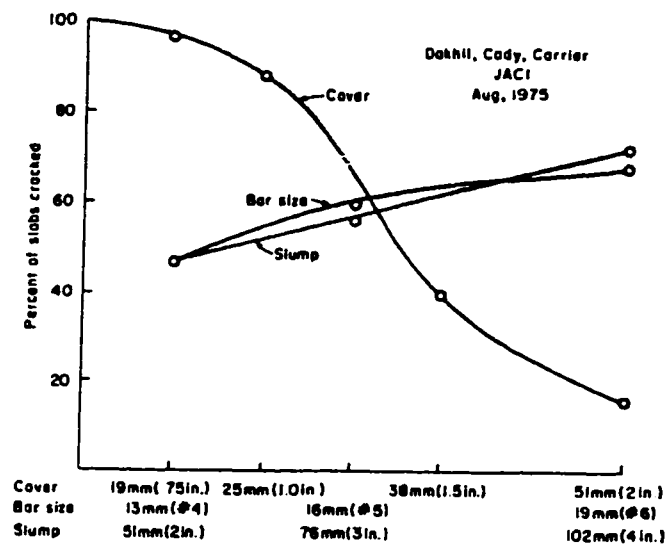


Fig. : 5.12- Cracking of concrete as influenced by cover, bar size, and slump.

structural loading effects. Shrinkage is caused by moisture loss from the face of a concrete element, which when restrained, leads to stresses which eventually exceed the tensile strength of the concrete and propagate cracks. Restraint can be either external, as for example, the restraint offered by a rigid beam, or internal, due to the much lower rate of shrinkage in the moist interior of a massive concrete member. Thermal stresses are usually relieved by expansion joints containing a resilient material, however, age hardening of the joint filler or plugging of the joint by foreign objects can restrain expansion and lead ultimately to cracking. Arrangement of the reinforcing mats can also affect cracking of concrete slabs.

Related to structural loading are the effects of flexural, shear or bond cracks. These cracks are usually 0.01mm wide when first visible to the naked eye. Two important parameters which affect their growth behaviour are : *effects of repeated loading* and *creep*. Experimental evidence suggests that the repeated loading effect is closely related to the variations in the tensile stress levels of the steel reinforcement. Under low tensile steel stresses repeated loading has been found to have almost negligible effect on crack widths. On the other hand an experimental study¹⁴³ showed that in a

reinforced concrete highway bridge the maximum crack widths increased by about 45% after repeated loading cycles in which the steel stress was varied from 18,300 psi at dead load to a maximum of 40,600 psi under live load. Such evidence makes out a case that there *must be a maximum permissible tensile stress from the point of view of concrete cracking, if the steel is to be prevented from corroding.* Creep effects under sustained loading would widen the cracks with time¹⁴⁴. With high strength deformed bars sustained load at steel stresses of 31,500 psi caused the maximum crack widths to increase by about 17%.

The obvious fact that the width of cracks on concrete surface is also likely to be governed by the thickness of the cover makes it clear that greater cover thickness adopted for steel protection, acts in the reverse direction when cracks develop. Also, contrary to the earlier indications the use of deformed bars show only a 10% improvement over the plain bars in controlling cracks.

The above discussion suggests that even under normal service conditions certain imperative operative factors will usually give rise to situations which will bring about cracking in concrete structures.

5.6 COVER TO REINFORCEMENT

Almost all studies are unanimous in attributing to concrete cover over reinforcement a decisive role in affording protection against rebar corrosion. The protection afforded by concrete cover against corrosion is partly due to its passivating alkalinity and partly due to its action as a barrier which minimizes or prevents the access of chemically aggressive substances to the protective alkaline film. Cover also provides a secondary inhibition of corrosion by preventing the supply of free oxygen to the "corrosion-cells".

In most cases the provision of an adequate cover appears to be one of the cheapest and most effective measure for inhibiting corrosion. Because of this most specifications have emphasized the need for its adequacy and have laid down standards for its provision.

In view of the importance of cover as a simple yet effective measure of controlling corrosion damage in reinforced concrete structures data was obtained and analyzed from field samples in an attempt to quantify its effectiveness. 26 cores drilled out from concrete slabs of frame buildings 22-27 years old were analyzed for cover

and metal loss data. The chloride content and absorption characteristics, being the other two important variables influencing metal loss, were kept within acceptable limits of variation for the cover metal loss data points plotted in Figure 5.13. All the field samples, results of which are recorded in this figure, have average chloride of 6.23 lb/ cu.yd. with standard deviation of 1.12 and average absorption of 3.38 with standard deviation of 0.48. This curve shows the dramatic effect of concrete cover thickness on metal loss. It is seen that for clear cover of 1 inch or greater the metal loss rapidly comes down to reasonable values. At a cover value of $1\frac{1}{2}$ inches the metal loss is extremely reasonable and this cover thickness seems to suggest effective protection against corrosion for the average quality of concrete obtained from field structures. The significant variation in the protection afforded by the varying thickness of the cover is also effectively demonstrated by the chloride-metal loss relationship shown in Figure 4.10 for $\frac{1}{2}$ -inch, 1-inch and 2-inch covers. It is seen that for the same quality of concrete and for the same chloride content the metal loss is significantly reduced with increase in cover thickness.

Fig. 5.13 : Clear cover - Loss of metal relationship
for solid reinforced concrete slabs
(Total thickness \approx 4.5 inches)

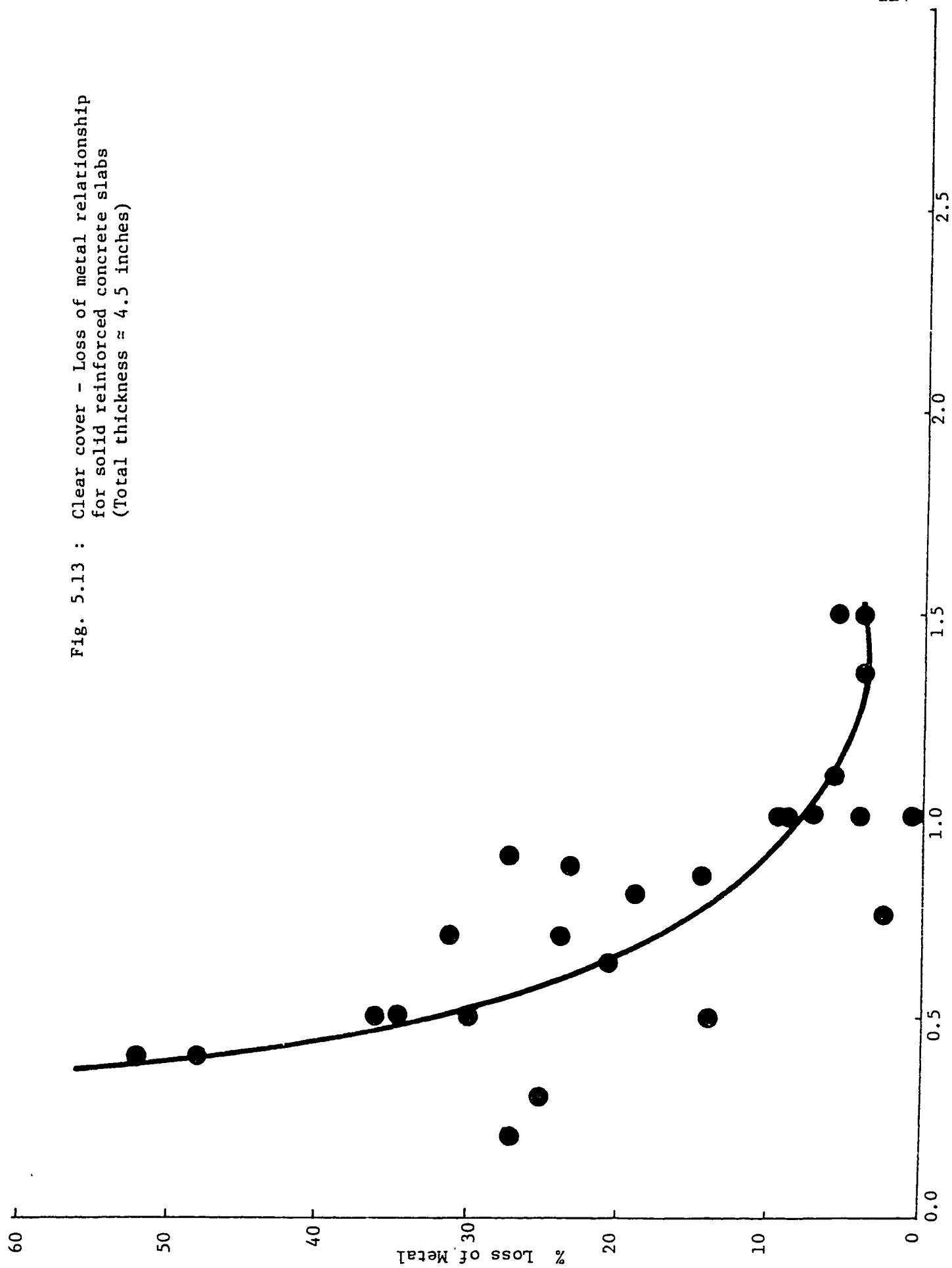


TABLE 5.15

Cover - Loss of Metal Data

Building No.	Core No.	Bar No.	Loss %	Cover	CL ⁻	Absorption
1	1	1	4.0	1.5	5.0	2.48
1	1	2	2.7	0.75	5.4	2.98
1	3	2	5.67	1.5	5.8	3.83
1	3	3	27.5	0.2	6.2	3.83
2	3	1	3.8	1.37	7.72	-
4	1	1	25.3	0.3	5.6	-
4	2	3	34.94	0.5	5.8	-
5	1	1	21.25	0.65	6.4	-
5	3	1	9.2	1.0	6.6	3.31
5	4	1	36.19	0.5	6.2	4.38
5	4	2	3.0	0.5	5.12	4.38
5	5	1	0	1.0	5.0	3.06
6	1	1	31.41	0.7	8.0	4.05
6	2	1	4.01	1.0	4.6	3.32
6	3	1	14.19	0.5	4.8	3.5
6	4	1	24.03	0.7	6.0	4.08
7	1	1	52.00	0.4	7.2	3.16
7	2	1	6.5	1.1	5.16	2.87
7	3	2	48.2	0.4	7.4	3.09
7	4	1	8.9	1.0	5.8	2.91
7	5	1	14.8	0.85	6.92	2.98
7	6	2	19.2	0.8	7.14	3.19
8	1	1	7.0	1.0	5.74	2.96
8	3	1	23.6	0.87	8.24	2.91
8	4	2	27.5	0.9	8.2	3.14

Another set of data obtained during this investigation which is relevant is that plotted in Figure 4.14 which shows the migration characteristics of the chloride ions into uncracked concrete for a mosque column which was exposed to the frequent splash and spray of chloride charged water. This figure shows a rapid decrease in the chloride content with depth. The fact that chloride levels become low at a comparatively large depth of 2.5 inches is explained by the highly porous quality of cover concrete of this column. Plate 5.3 shows the honey combed cover concrete used in this column. Normally, for a better quality concrete the decrease in chloride level can be expected to be much more rapid partly because of the barrier effect of the cover and partly because of the fact that diffusion of chloride ion in the cement paste is accompanied by reaction of the chloride to form "insoluble" calcium chloroaluminate. This reaction reduces the concentration of chloride ion and hence the tendency for inward diffusion is further reduced.

The effect of cover can also be seen by reference to Figures 5.11a, b,c, due to the research carried out by Stark at PCA. At low covers (Figure 5.11a), all concretes tested were unable to prevent the onset of corrosion for

more than 30 cycles of salt applications. At a cover of 2-inch (Figure 5.11b), only the slabs with w/c ratio of 0.6 show evidence of corrosion. In another study¹⁴⁵ use of thicker covers upto 3 inches offered even greater protection at only a small increase in cost. Field studies¹⁴⁶ by the Portland Cement Association in conjunction with four State highway departments indicated that corrosion spalls occurred only in those decks where specified cover was 2 inches or less. Research in Kansas¹⁴⁷ related to bridge deck deterioration showed a very high tendency for deterioration at covers of 1.5 inch or less and very little deterioration for covers of 2 inches or more. These findings agree with the author's findings reported above.

What are the various other considerations which should govern the specifications of adequate cover to reinforcement ? On the face of it, depth of cover of the reinforcing steel would seem to be a design problem rather than construction related. Generally speaking, this is true. However, the fact that design depth and actual depth of cover do not always agree is illustrated in Figure 5.14. Because of this differential in the specified and actual values there is an increasing trend in most jobs to specify a "plan cover". Surveys by the State of New York¹⁴⁸ and

FHWA¹⁴⁹ indicate that the average standard deviation of concrete cover is $\pm 3/8$ inch; thus to obtain an actual minimum cover 90% of the time, the plan cover must be $\frac{1}{2}$ to $5/8$ inch greater than the minimum cover desired. In another study¹⁵⁰ of 17 bridge decks in New Jersey, it was found that values of depth of cover were distributed approximately normally, with the mean value close to the specified value and standard deviation about 1 cm. Based on these findings, one can readily calculate specified target cover values to assure that a given percentage of the steel will have concrete cover equal to or greater than the desired minimum value. For example, if the minimum required cover is desired to be 5 cm, the percentage of steel having a cover of at least 5 cm. for various specified target cover values will be as given in Table 5.16.

The minimum required depth of clear cover with sound, uncracked concrete to prevent reinforcing bar corrosion is usually considered to be 2 inches (5 cm)^{151, 152, 153}. This observation is in very good agreement with the author's findings based on field studies and expressed in Figure 5.13. It is obvious that thicker covers should be more effective in preventing corrosion, but beyond 2 inches (5 cm) protection is reportedly not significantly improved.

TABLE 5.16

Percentage of Steel Protected (5 cm
minimum cover) for Various Target
Values of Cover

Target Value, cm	Steel Protected %
5.0	50
5.5	69
6.0	84
6.5	93
7.0	98
7.5	99

However, the required minimum depth of cover depends on the permeability (i.e. w/c ratio) of the concrete. Minimum cover depth of about 2 in (5 cm), 3 inches (7.5 cm) and 4 inches (10 cm) have been found to be necessary to protect reinforcing steel for w/c ratios of 0.4, 0.5 and 0.6 respectively^{154,155}.

The depth of cover of the reinforcing steel is of special significance in the case of horizontal slab concrete members (for example, bridge decks). Here, insufficient depths of cover have been observed to produce cracking of the concrete immediately above the steel reinforcing bars. *Even the highest quality concrete is of no value in retarding corrosion if cracks are available to admit chlorides, water, oxygen, and carbon dioxide.* Cracking results from the interference to subsidence of fresh concrete presented by the reinforcing bars. The existence of a zone of tensile stress in the plastic concrete above the reinforcing steel has been illustrated in photoplastic studies. It has been found that values of concrete cover of 5 cm or more are sufficient to prevent subsidence cracking. However, concrete slump and diameter of the reinforcing bar also have significant.

though considerably smaller, effects. In most of the updated Codes of Practices the cover specifications are usually hitched up with variable conditions of exposure, grade of concrete, maximum diameter of reinforcement, the maximum size of aggregate and the operative limits on crack widths. However, all modern specifications have emphasized one point: *That the protection of reinforcement depends more on the quality of the concrete, its compaction and impermeability, rather than on the distance between the outer face and the bar.*

The British Unified Code CP110 1972 for structural concrete specifies minimum values of concrete covers from 15 mm (0.6") to 60 mm (2.5") depending on the condition of exposure (mild, moderate, severe, very severe) and the grade of concrete.

The German Code of Practice DIN 1045 specifies the minimum values of concrete covers from 10 mm (0.4") to 40 mm (1.6") depending on the diameter of rebars and the environmental conditions.

The ACI requirements are well known. The International Recommendations for the design and construction of concrete structures issued by the European Committee for Concrete in 1970 specify absolute minimum values and values related to the diameter of the largest bar and the maximum size

of the aggregate.

However, the most comprehensive recommendations are those provided by the Model Code¹⁵⁶ for Structures in Concrete issued by the European Committee in 1975.

5.7 COMMENTS ON LOCAL CONSTRUCTION PRACTICES

Atleast in some instances, good and bad structures of the same age group stood side-by-side. This implies that part of the cause of concrete deterioration can be traced to failure in adopting sound design and construction practices and in implementing the requirements specified for good concrete quality and steel positioning. In an article entitled "Assessment of Concrete Durability" Nurse observes : " In general concrete is durable. Rapid failures are usually caused by overlooking known factors, either in specifying or making the concrete or in assessing the environment to which the concrete will be exposed". It is almost certain that one of the important reasons of low durability of concrete construction in the region is that the profession is committing the mistakes of overlooking known factors such as requisite specifications, sound construction practices based on the modern tenents of concrete technology and the implications of the super

aggressive environment to which concrete is exposed during its in-service performance in the Gulf region.

From the previous discussion it is obvious that the construction techniques in the region will have to contend with high chloride contents in concrete usually far in excess of the threshold value for corrosion. In view of this compulsive situation the only effective alternative to retard and contain corrosion is to check the secondary requirements of the process. This being the supply of moisture and oxygen, the aim should be to produce a high quality, dense and impervious concrete of requisite strength so as to provide an effective physical shielding against ingress and the design and construction practices should be effectively geared to this end.

In order to evaluate the construction practices with respect to the aforesaid objective, a survey of several sites was undertaken. All of these sites were residential housing colonies under construction. Most of the houses were meant for middle class citizens.

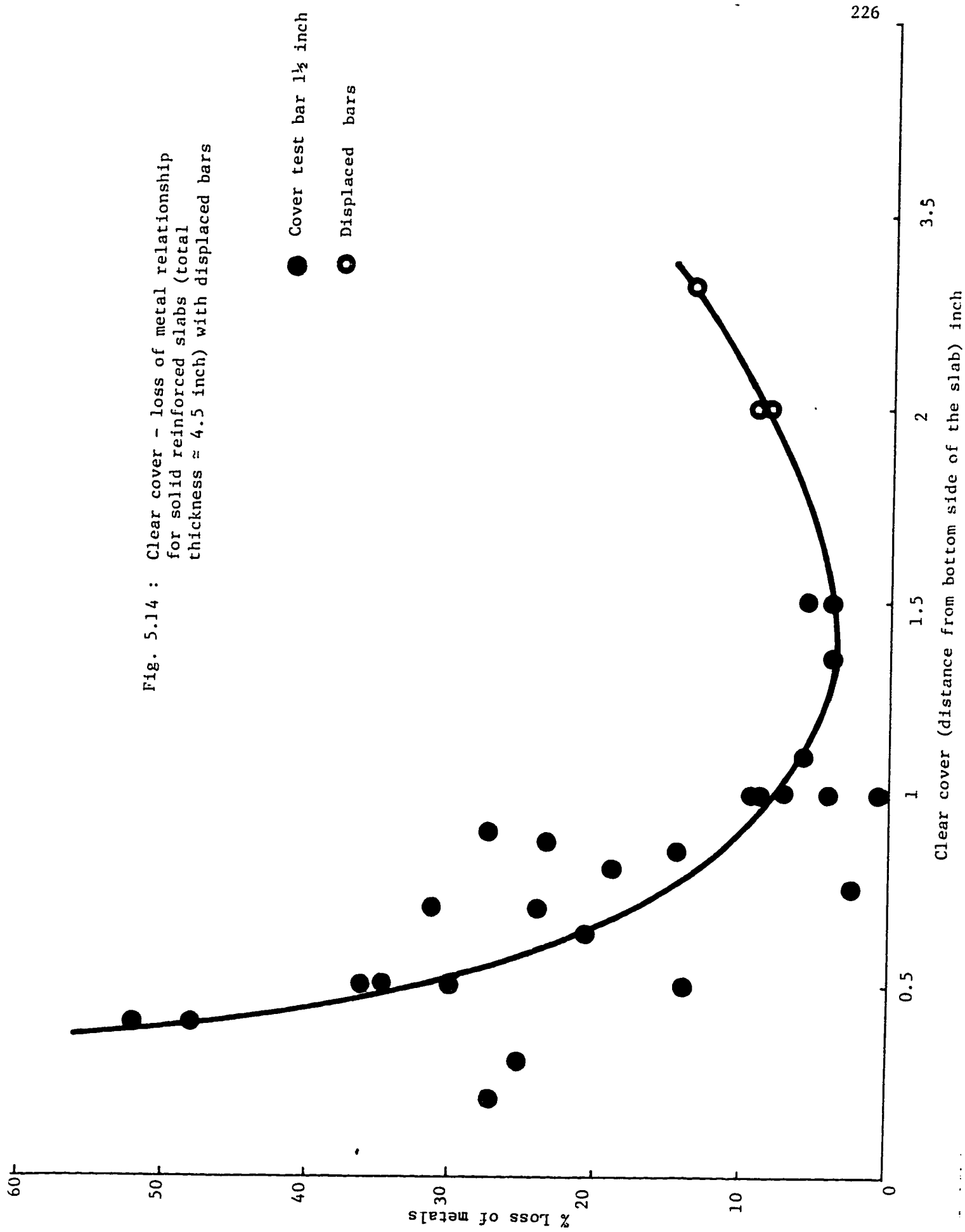
The recorded observations from these surveys are highly disappointing and show that the prevailing construction practices are totally out of gear with the grim

environmental situation and leave much to be desired. Starting from the point of mix design and material selection to finishing the surfaces there were hardly any considerations of rational principles of concrete technology; and almost all the classical situations which should be avoided were observable in abundance.

Firstly, the positioning of the reinforcement. There was a total absence of the use of bar spacers and chairs and there were unacceptable displacements of bar assemblies during placing operations. A typical example of this situation is shown in Plate 5.4. The tyre of the loaded trolley was found to displace reinforcement to absolutely unacceptable tolerances and would eventually result in a much reduced concrete cover to rebars than intended. At places there was excessive sagging or distortion of bars due to lack of rigidity of bar assembly. Another common streak was to overlook the basic fact that the stirrups and main bars are made of the same material. In many cases the cover to stirrup appeared to be almost negligible (Plate 5.5). It is almost certain that an oxidation of the stirrup will initiate concrete cracking in no time thus opening the way for the rapid and profuse corrosion of the main reinforcement.

A manifestation of the fact that these construction practices result in wrong positioning of reinforcement and in a lack of concrete cover to reinforcement was repeatedly available during the coring of concrete slabs from field studies. In atleast in a dozen of cases the cores extracted showed that the main reinforcement was 2 to 2½ inches above the lower surface in a 4½ - 5 inches thick slab. This is akin to either placing the main reinforcement near the neutral axis or providing an effective thickness to slab of only 2½ to 3 inches. Such a displaced location of the reinforcement made it more prone to the adverse effect of external environment causing high corrosion and consequent loss of metal. This position is shown in Figure 5.14.

In another study made at UPM for the building structures located in Dhahran-Dammam-Alkhubar area cover measurements were obtained for a number of spalled concrete regions. Figure 5.15 shows the result of this study. It was found that in 68% of the observed spalls the thickness of concrete cover was less than ½ inch, in 53% it was less than 3/8" and in 18 observations it was less than 1/4". There were 7 cases (9.2%) where there was almost no cover to steel. Plate 5.6 shows a typical column which shows no cover over stirrup and Plate 5.7 shows the situation in



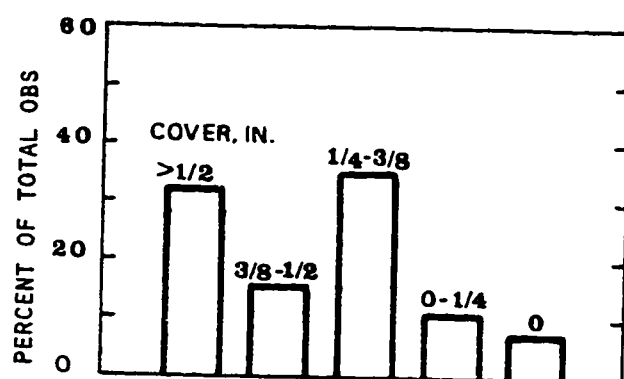


Fig. 5.15: Distribution of Concrete Cover over Reinforcement in Spalled Concrete

two newly constructed buildings where there is almost no cover to slab and beam reinforcements. Plate 5.8 shows that not only the cover to reinforcement is zero but the concrete has been so badly placed that large honey combs are present which even after plastering will provide excellent ingress channels for the corrosive media. These situations are no exceptions. They were found to recur very frequently in parts of new framed buildings.

Enquiries on sites showed that there were no mix design calculations to take into account some of the special characteristics of the mixture materials such as the fineness of the sands being used at these sites. The water/cement ratio, cement/aggregate ratio or fine aggregate/coarse aggregate ratio were never fixed skillfully and technically with the result that profuse segregation took place in concrete immediately after placing showing highly unstable concrete mixtures Shovelful of fine and coarse aggregates and bucketful of water were being added in a mixture (Plate 5.9) and the only overwhelming consideration seemed to be the ease in concrete placement. This very narrow interpretation of workability readily sacrificed the other components of this comprehensive term viz stability (absence of segregation and bleeding),

compactability and finishability. Due to high w/c ratio thin cement paste flowed clear of the aggregates as shown in Plate 5.10 and it required no effort to discern the separate identities of sand and coarse aggregates in the mix. Bleeding was excessive causing the mixture to act as a slop (Plate 5.11) and the finishing operation was being executed by masons splashing around in this slop (Plate 5.12). The excess water was found flowing freely over the surface of the sedimented aggregate particles (Plate 5.10).

The effects of these defective construction practices were visible within hours of the concrete placement. Hundreds of plastic shrinkage cracks opened up the same day the concrete was placed. Typical cracks are shown in Plate 5.13. In one case 530 shrinkage cracks measuring a total length of 415.6 ft in a roof area of 2476 ft² were observed to have opened up within 24 hours of concrete placement. The widest crack was as wide as 0.125 inch (Plate 5.14). These cracks provide easy ingress to corrosion media. Cady¹⁵⁷ observes: "Even the highest quality concrete is of no value in retarding corrosion if cracks are available to admit chlorides, water, oxygen and carbon dioxide". German studies¹⁵⁸ have shown that cracks greater than 0.1 mm (.004 inch) are associated with significant

amount of corrosion. In another study¹⁵⁹ observations on some cracked beams revealed significant localized corrosion in cracks greater than 0.55 mm (.02 inch) wide.

Another effect of the excessive bleeding was the deposition of a white crust (Plate 5.15) after the excess water evaporated. The evaporation also left the upper surface of the slab highly porous. Plate 5.16 shows the state of porosity of such a slab. This porosity will accelerate the permeation of aggressive media into the concrete.

Plate 5.1 : Dusty crushed limestone in
quarries in Eastern Province
of Saudi Arabia

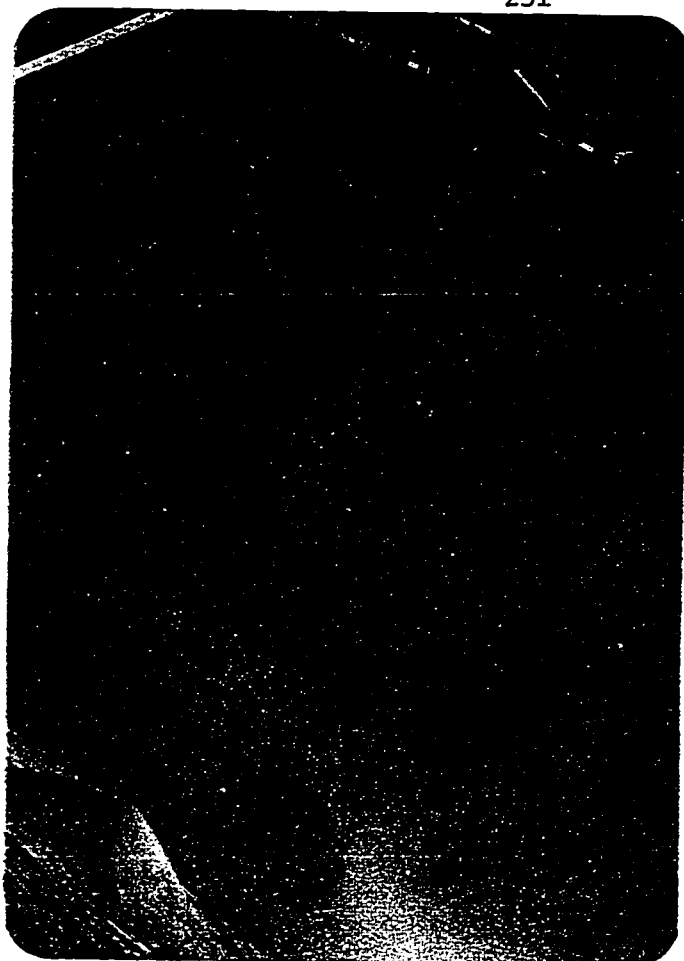


Plate 5.2 : Porous cover for column re-
inforcement causing corrosion

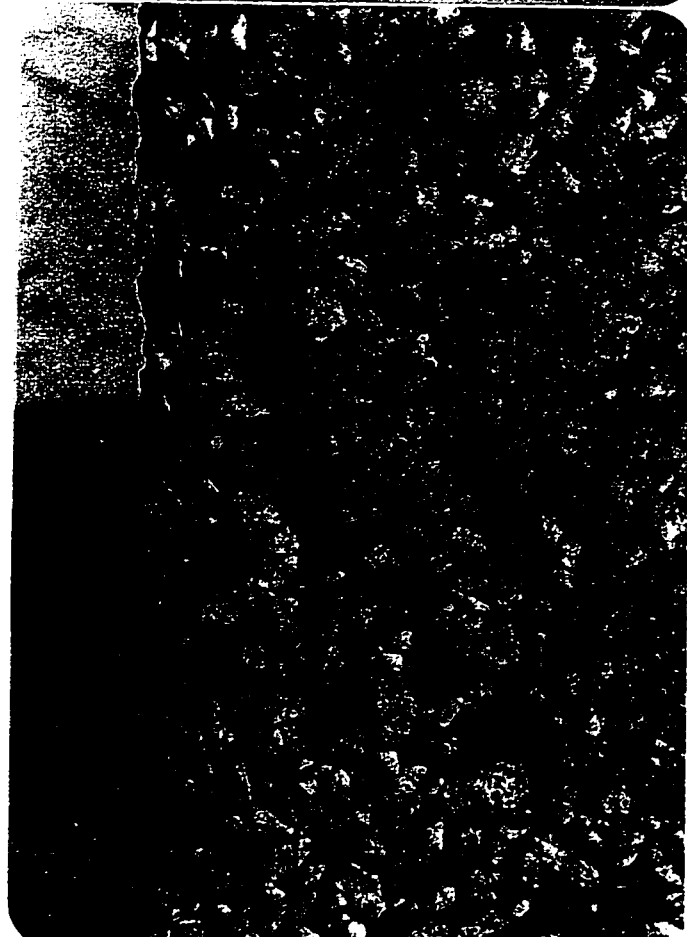




Plate 5.3 : Honey combed cover with thickness of 2.5"

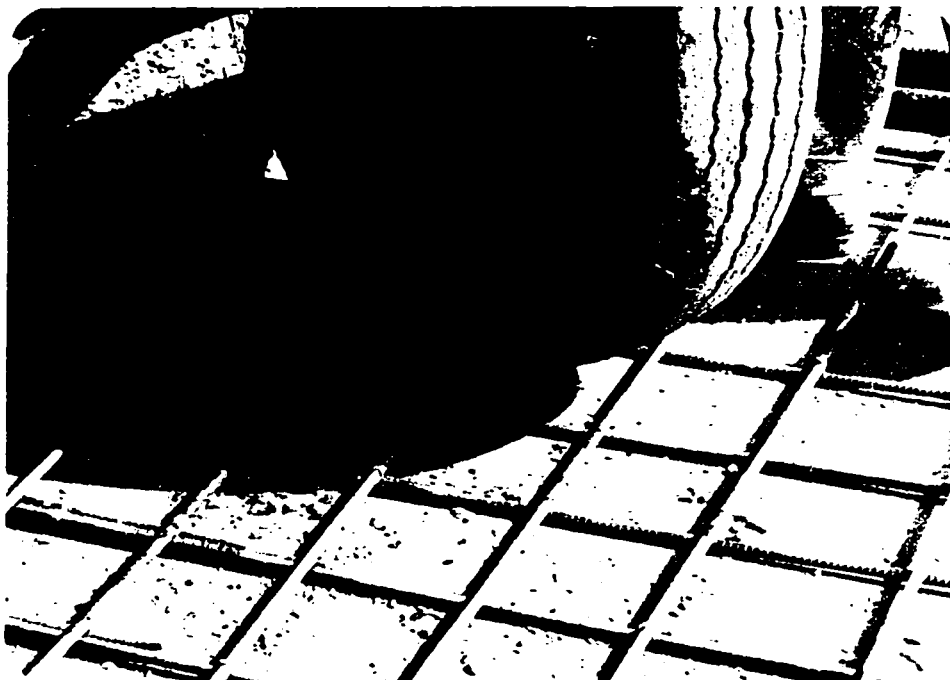
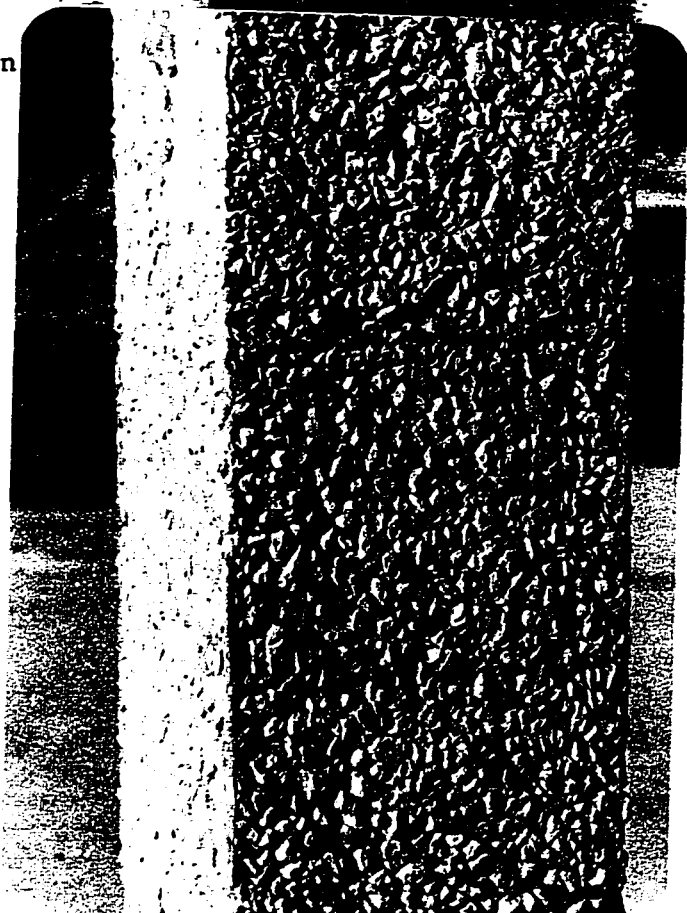


Plate 5.4 : Loaded trolley displaces reinforcement resulting in a much reduced concrete cover

Plate 5.5 : Corroded stirrup as a result
of lack of cover



Plate 5.6 : Lack of cover to stirrup in
a newly constructed column



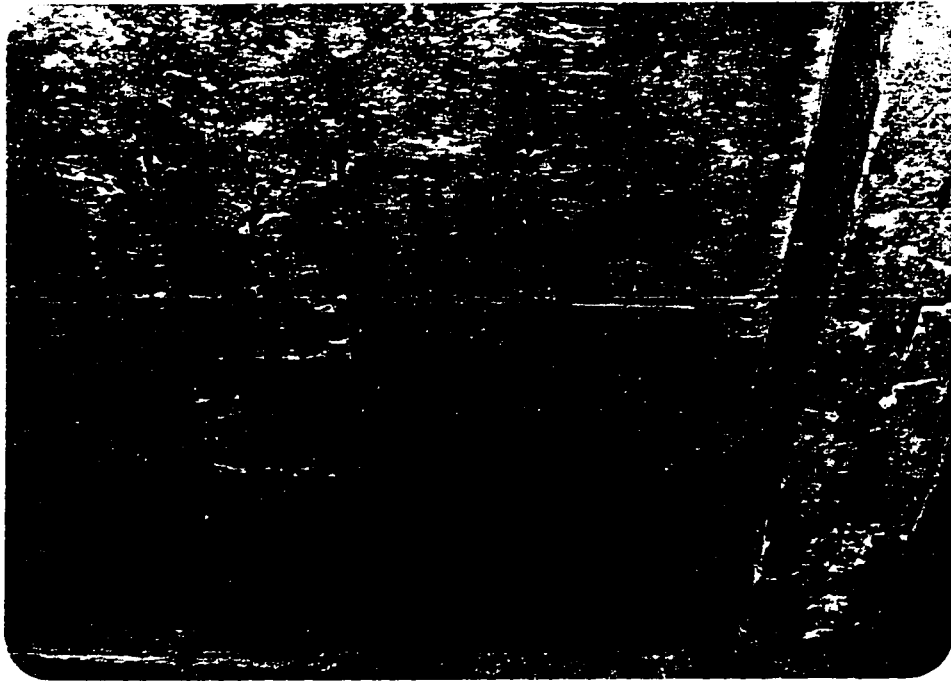


Plate 5.7 : Newly constructed slab with no cover provided for reinforcement

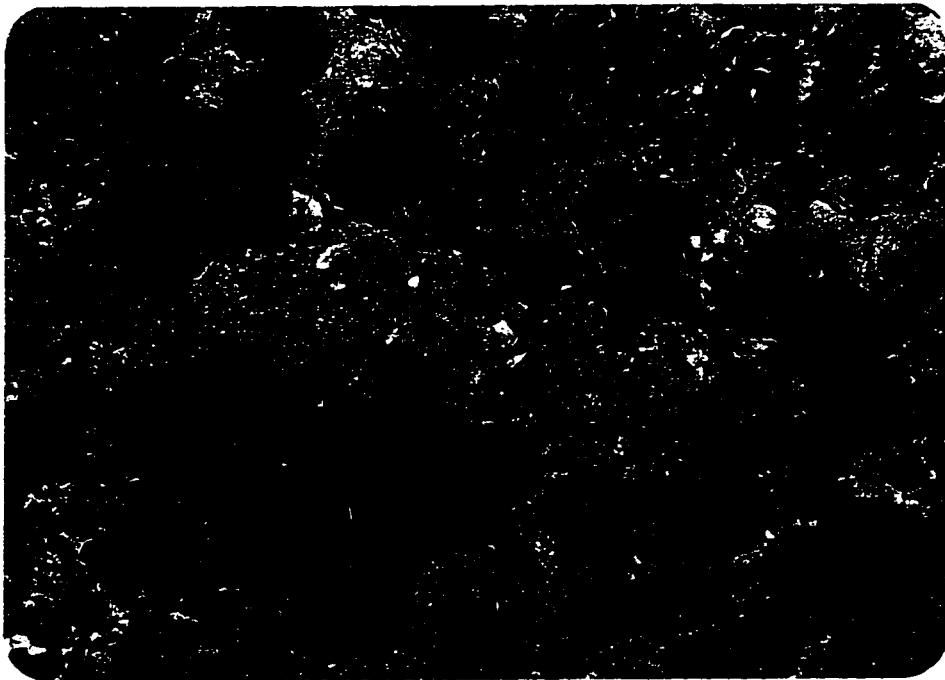


Plate 5.8 : Badly placed concrete results in honey combs and large voids

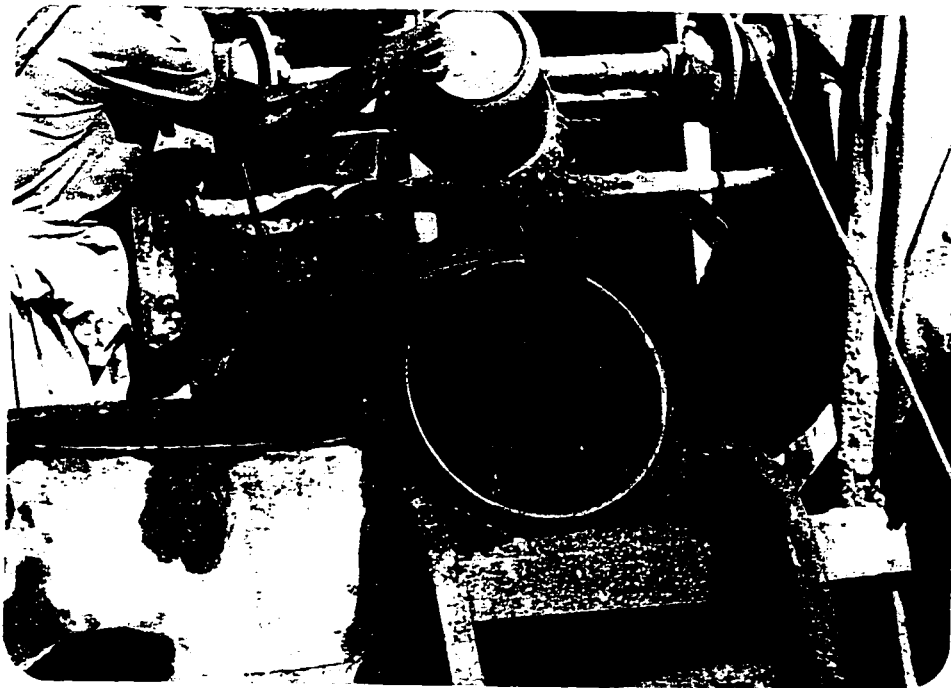


Plate 5.9 : Mixing process, no control for quantities



Plate 5.10 : Cement past flowed clear of the aggregates resulting in segregation

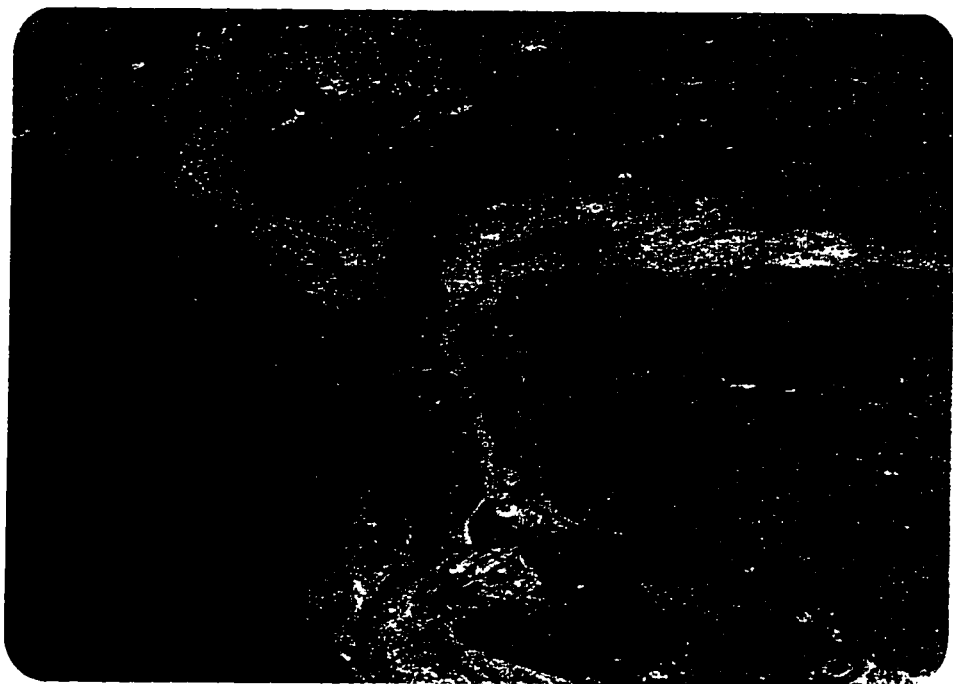


Plate 5.11 : Excessive bleeding after placement of concrete

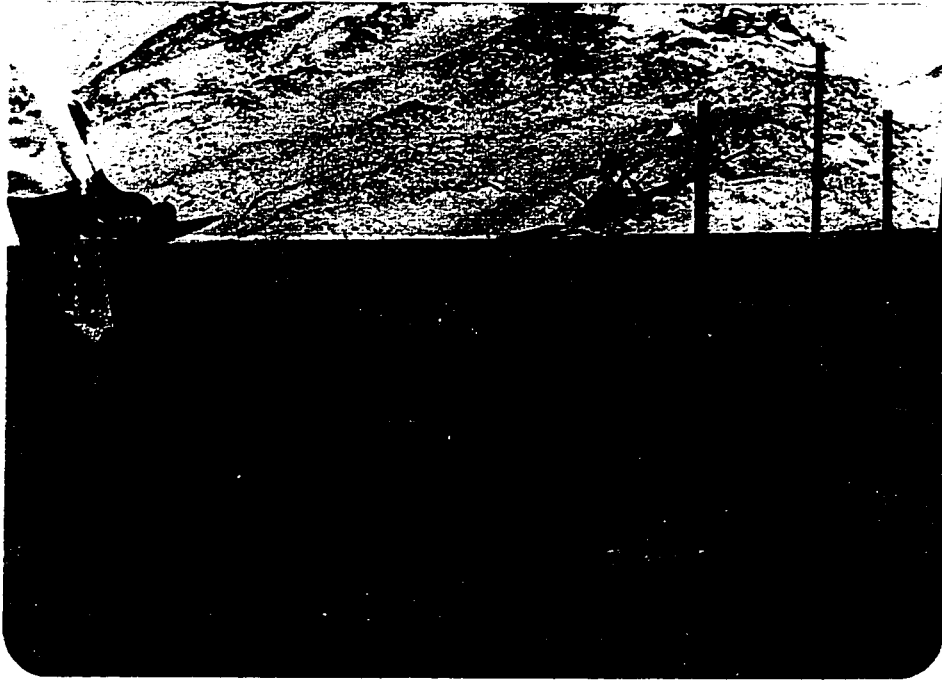


Plate 5.12 : Finishing operation executed by mason splashing around the slop

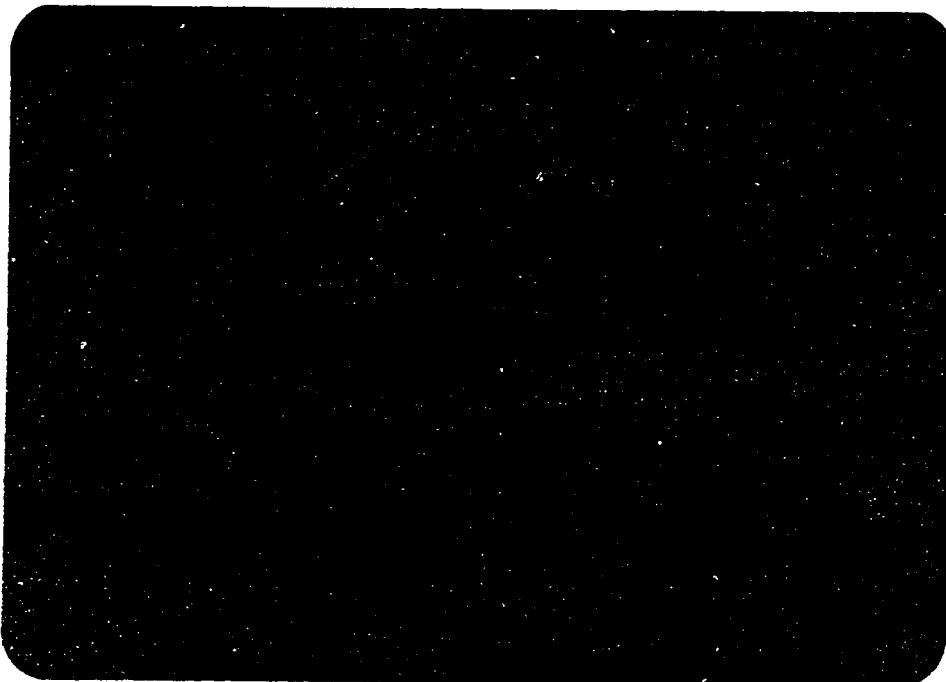


Plate 5.13 : Plastic shrinkage cracks opened within hours of the placement of concrete

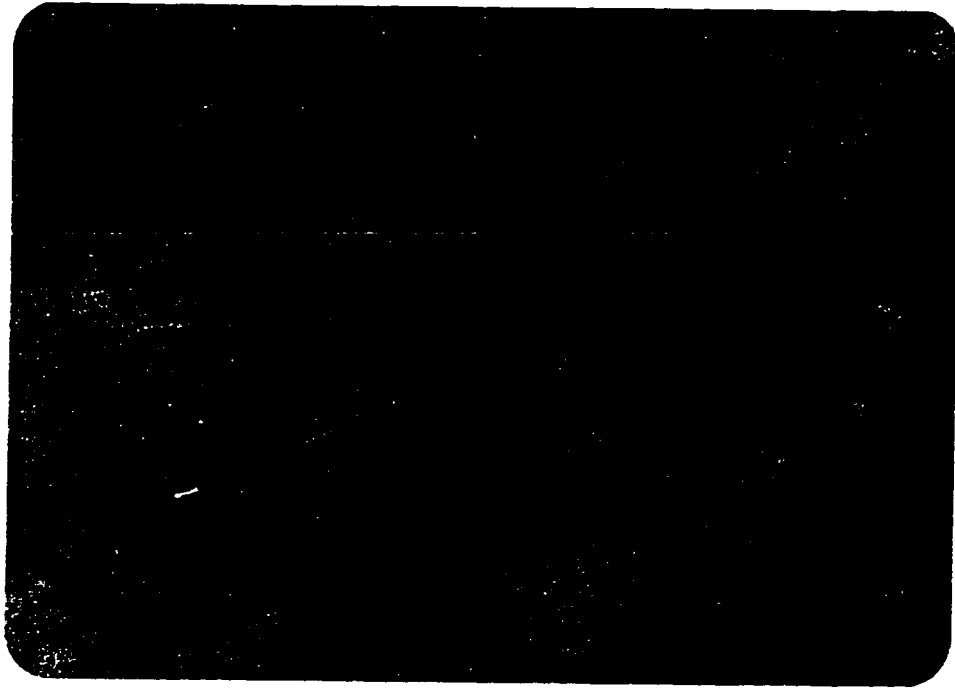


Plate 5.14 : Wide plastic shrinkage crack

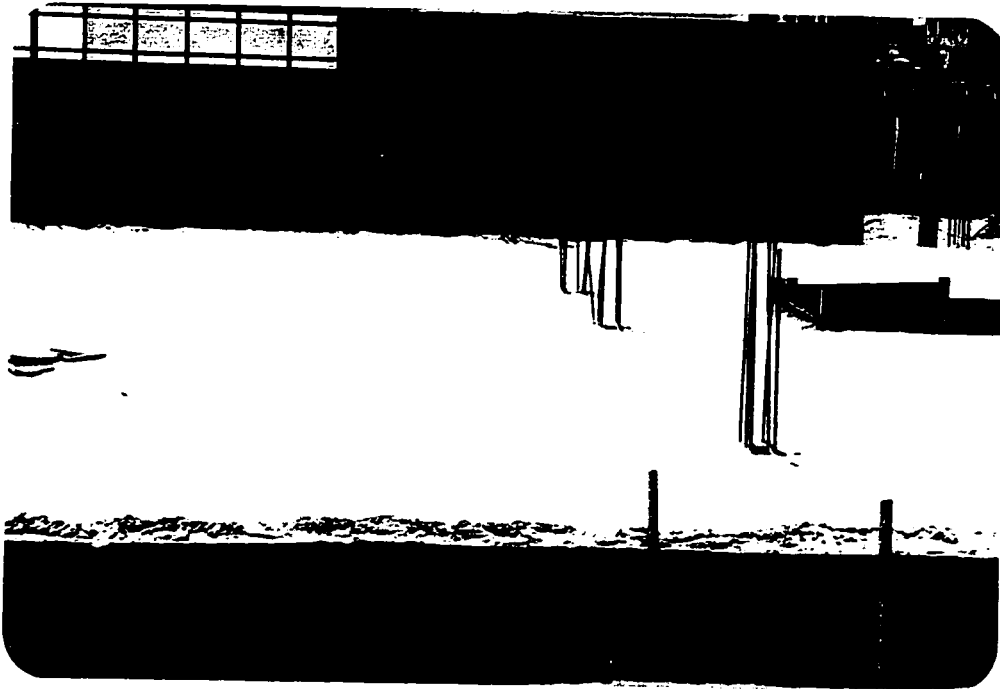


Plate 5.15 : Deposition of white crust after excess water evaporation and high content of dust with aggregate

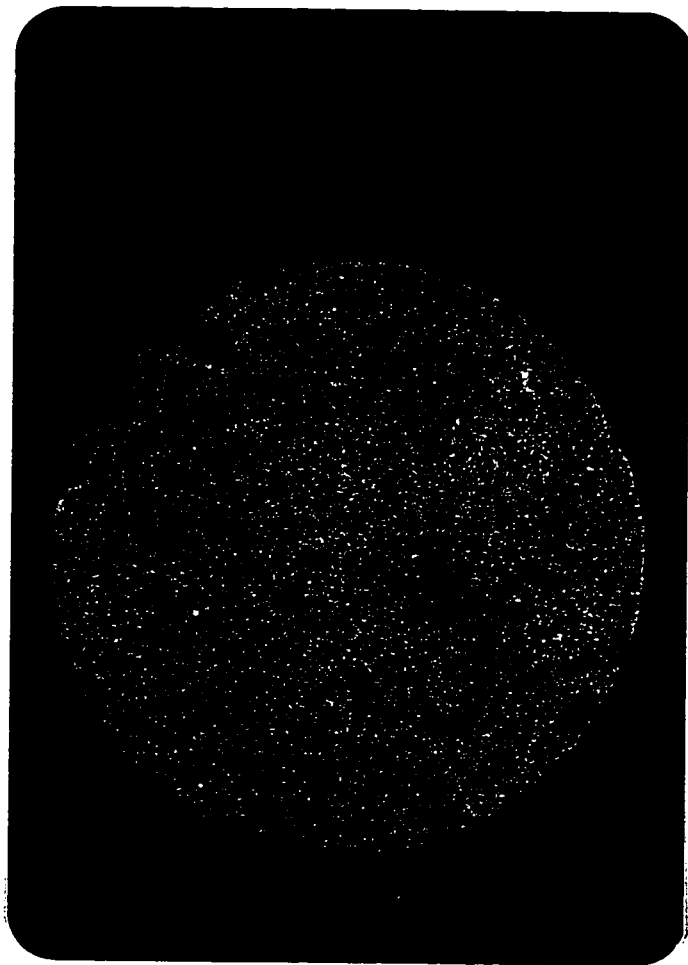


Plate 5.16 : Porous slab surface due to excessive bleeding of high w/c mix

Chapter 6**SOME SUGGESTIONS AND POSSIBILITIES
FOR THE SOLUTION OF CORROSION
DETERIORATION PROBLEM**

Figure 6.1 shows a schematic statement of the general requirements for corrosion or no corrosion situation. All recommendations and preventative measures should arise from these basic concepts. It seems that the broad framework for the prevention of corrosion of embedded steel in concrete will consist of the following essential elements:

- (i) Exclusion of chloride ion from concrete by limiting its initial inclusion and subsequent penetration.
- (ii) Cutting off the secondary requirement of the corrosion mechanism by choking the intrusion of oxygen and moisture.
- (iii) Use of corrosion resistant reinforcement.
- (iv) Applying a negative potential on anode thus superimposing a stronger current in the opposite direction.
- (v) Applying inhibitors in concrete which may counter the corrosion mechanism at a vital stage.

The possibilities and the limitations of each approach should be examined and evaluated from the point of view of adopting it for the Gulf region.

6.1 RECOMMENDATIONS REGARDING LIMITS ON CHLORIDE INCLUSION IN CONCRETE

The potential hazard of chlorides to concrete containing steel in a marine environment or in other salt-charged exposure conditions suggests a position that no chloride should be allowed in the concrete mix. This would reject the use of sea water as mixing water, salt contaminated aggregates or those which have been washed with sea or salt water, and admixtures containing chloride.

Considering this problem the ACI Committee 201 in its report entitled "Guide to Durable Concrete" comments that specifying a zero chloride content for the mix is impossible to realize in practice. The effects of chloride ion concentration on corrosion probability and rate are difficult to quantify because of the strong dependence on moisture content and availability of oxygen to depolarize the cathode. However, Hausman¹⁶⁰ seems to have established the lower limit for threshold chloride concentrations through experiments with oxygenated, saturated lime solutions.

He showed that as the pH increases above 11.5, chloride ion threshold concentrations required to initiate corrosion increase (Figure 6.2). Hausman also showed that the chloride ion activity at the corrosion threshold bears a constant ratio to hydroxyl ion activity of about 0.60. It should be reemphasized that while the threshold chloride concentration increases with increasing pH, corrosion of steel in concrete might not occur at the chloride ion threshold concentration predicted by Hausman due to somewhat more limited ion mobility and limitations on the rate of diffusion of oxygen to the cathode in concrete as compared to oxygenated, saturated lime solution. However, using Hausmann's results as lower limits it may be stated that the onset of corrosion may occur at chloride ion concentrations as low as about :

0.34 lb of chloride ion/cu.yd. of concrete for pH = 12.5

4.71 lb of chloride ion/cu.yd.of concrete for pH = 13.2.

pH values of normal concretes are higher than 12.5 and usually lie between 12.5 and 13.0. If an average value of pH of 12.75 is adopted this gives a threshold value of 1.75 lb of chloride ion per cu. yd. of concrete.

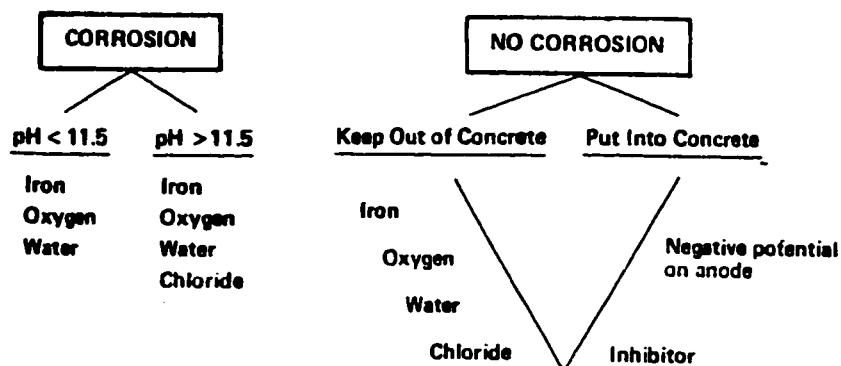


Fig. 6.1:-Basic general requirements if corrosion is to occur. Research is needed to establish more precisely the pH limits and the factors causing variations in pH in concrete

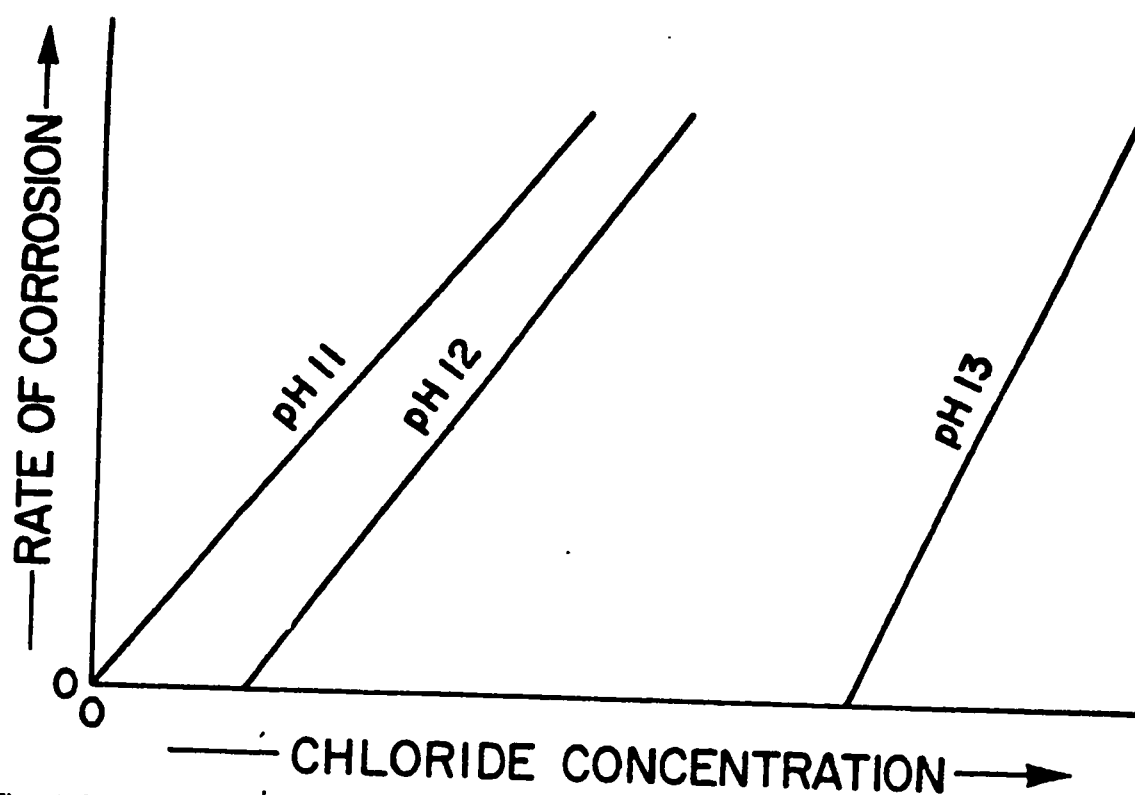


Fig. 6.2: Threshold chloride ion concentration required for corrosion to occur increases as the alkalinity, pH, of the cement paste liquid increases

Weiher and Segmuller¹⁶¹ also carried out laboratory electrochemical studies and report a minimum concentration of chloride ion as 0.4 per cent by weight of cement required to induce corrosion of embedded reinforcement.

With regard to corrosion threshold concentration of chloride ion in full-size concretes, research carried out by Lewis and by the Federal Highway Administration¹⁶² revealed that the threshold value for chloride content in concrete necessary for the corrosion of embedded steel can be as low as 0.15 per cent by weight of cement in the concrete mixture. For the typical range of concrete mixtures normally encountered (cement content ranging from 590 to 750 lb/cu.yd. of concrete) the 0.15 per cent criterion represents corrosion threshold chloride contents ranging from .88 to 1.12 lb of chloride ion per cu. yd. of concrete. These values lie between the values of 0.34 lb/cu.yd. of concrete (for pH = 12.5) and 1.75 lb/cu.yd. of concrete (for pH = 12.75) predicted by Hausmann as discussed earlier.

Detailed field studies of 21 bridge decks¹⁶³ of ages varying from 6 to 23 years, some showing signs of deterioration and others not, led Stratfull et al. to conclude that the quantity of chloride ion associated with the

incidence of active corrosion of steel in concrete is about 1 lb/cu.yd. of concrete. Again this figure is in close agreement with the data previously cited. Other bridge deck studies revealed similar corrosion threshold values.

In view of this feedback from lab research and field investigations ACI Committee 201 on Durability of Concrete in agreement with ACI Committee 222 on Corrosion of Metals in Concrete has made the following recommendations on the maximum amount of chloride ion expressed as per cent by weight of cement that can be allowed in concrete containing steel :

- * Prestressed Concrete : 0.06
- * Conventionally reinforced concrete located in a moist environment and exposed to chloride : 0.10
- * Conventionally reinforced concrete located in a moist environment but not exposed to chloride : 0.15
- * Above ground building construction where the concrete will stay dry(does not include locations where the concrete will be occasionally wetted - such as kitchens, parking garages and water front structures : No limit for protection against corrosion

These recommendations are, however, qualified by considerable discussion, since the effect of chloride on corrosion of steel is a complicated subject and the rate of corrosion is subject to a number of influences, including the nature of the chloride and the vagaries of exposure conditions. The availability of oxygen and moisture adjacent to steel, which in addition to chloride are established influencing parameters controlling corrosion and its speed, will vary with service exposure from one structure to another and between different parts of the same structure as well as with the quality of concrete and depth of cover to steel provided. Also, a meaningful discussion of limits should include the form in which chloride occurs.

Chloride in concrete may be in the free soluble form or may be chemically combined with other ingredients. Soluble chlorides induce corrosion, while combined chloride is believed to have little effect. It has been shown that when the total chloride content is near the corrosion threshold level, from 50 to 85 percent of it will be soluble¹⁶⁵.

There are exceptions, however. Some data indicates that a large percentage of calcium chloride admixture

combines chemically with the cement and thus is not available to induce corrosion . Some chloride-bearing aggregates have a high total chloride content but very little of it is soluble. On the other hand, some aggregates with high chloride content are known to have caused corrosion.

When considering the probability of corrosion, it is therefore logical to measure only the soluble chloride content of concrete, rather than total chloride. Tests for soluble chloride, however, are time-consuming and difficult to control. Factors such as sample size, boiling and/or soaking time, temperature, and quantity of distilled water used all affect the results. Therefore the test must be performed in a standardized manner. Conversely, the test for total chloride, which involves a nitric acid extraction, is not significantly affected by the above factors. Most interested parties, therefore, measure total (soluble plus combined chloride) and test for soluble chloride only when follow-up studies are desired.

If the total chloride is less than the allowable limit, obviously soluble chloride need not be measured. Should the total chloride content exceed this limit, additional information on the risk involved in using the material may

be obtained by performing a soluble chloride test. When this value is found to be above the limit, corrosion is likely if moisture and oxygen are readily available. If it is below the limit, the risk of corrosion is low.

The user should exercise good judgment in applying these limits, keeping in mind that other factors (moisture and oxygen) are always necessary for electrochemical corrosion.

No calcium chloride should be intentionally added to the mix in prestressed concrete or conventionally reinforced concrete which will be exposed to moisture and chlorides in service, even if the naturally occurring chlorides in the materials are less than the state limits.

In view of the heavy damage to concrete structures which has been reported due to chloride inclusion in concrete and the research results cited above, the 1979 relaxing amendment to CP110, which had earlier imposed virtually a complete ban on the use of chloride containing admixtures for reinforced concrete, has been severely criticized. The latest British Standards provision allow the following limits (expressed by weight of cement) :

*	Prestressed Concrete	:	0.10
*	Normal reinforced concrete made with ordinary Portland cement	:	0.36
*	Normal reinforced concrete made with sulphate resis- ting Portland cement	:	0.2

It is interesting to note that in 1957 the British Standard Code of Practice CP 114 : 1957 contained a permissible dosage of calcium chloride of 2 per cent by weight of cement (about 1 per cent of chloride ion by weight of cement) which was reduced to 1.5 per cent calcium chloride (0.75 per cent of chloride ion) in 1969 by the British Standard Code of Practice CP 116 : Pt.2: 1969. The corresponding British Standard for prestressed concrete CP115 : Part 2 : 1969 prohibited the use of calcium chloride for prestressed concrete and the subsequent British Code CP 110 issued in 1972 prohibited the use of calcium chloride for reinforced concrete also. However, the above permissible limits are a relaxation proposed in 1979 through an amendment to CP 110 : 1972.

The German Practice limits chloride content to 0.1% by weight for prestressed concrete and to 0.2% by weight of cement for normal reinforced concrete.

Against this background of chloride threshold values and limits, it is interesting to study the Gulf situation.

Table 6.1 lists the data relating to the estimated base chloride content in Gulf concretes for typical concrete mixtures on the basis of the author's investigations on chloride contents in the constituent materials of concrete. A lean and weak mix (Mix 1) and a rich and strong mix (Mix 2) are considered. The cement contents and w/c ratios for Mixes 1 and 2 are 565 lb and 0.60 and 755 lb and 0.45 respectively. The maximum size of aggregate is 3/4" and the consistency is medium (3-4 inch slump).

According to ACI Committee 201 recommendations the permissible chloride value is 0.1% by weight of cement. This value is compared in Table 5.12 with the total estimated content for various degrees of chloride contamination of local aggregates.

It is seen that for both for Mix 1 and Mix 2 the estimated base chloride content in the Gulf concrete exceeds the threshold value for the average and high aggregate contamination levels. For Mix 1 with high aggregate contamination level the estimated base value is 2.7 times the permissible value. For Mix 2 the corresponding

actual value is 2 times the permissible value.

TABLE 6.1

Comparison of Permissible and Possible
Actual Chloride Values in Concrete Mixes
in the Gulf Area (Data extracted from
Table 5.12)

	ACI Permissible values lb/cu. yd. of concrete	Chloride content lb/cu.yd. for ave. degree of agg. contamination	Chloride content lb/cu.yd. for high degree of agg. contamination
Mix 1	0.565	0.994	1.73
Mix 2	0.755	0.966	1.67

This position exists before an exposure of the structures to the high chloride ambient environment of the Gulf. Almost daily chloride salts are deposited on the exposed matrix of concrete through salt-laden humidity, salt-charged dew and salt-contaminated wind. These are likely to increase the chloride content with time rendering the chloride level in the Gulf concrete far ahead of the threshold value at which corrosion is initiated at the embedded steel concrete interface.

It is obvious that this situation of excessive chloride induction from base materials and environment poses substantive difficulties for a possible solution. On the face of it the problem appears compulsive leading to a conclusion that engineers will have to contend with high chloride contents in concrete and a possible solution should take this hard fact into account. However, theoretically the problem can be reduced or brought to an acceptable level by an effective consideration of the following measures :

- (i) Winning and production of fine and coarse aggregate is a selective and organised manner. Quarries be established which are equipped with skilled and qualified personnel and testing techniques for frequent and rapid evaluation of the aggregate source material. The aggregate processing plants should contain scalping screens, washing and dewatering equipment and protection of stockpiles from any subsequent recontamination by salts, silts and other impurities.
- (ii) Suitable uncontaminated aggregates be hauled from distant sources.
- (iii) Considerable research be carried out to evolve

aggregate beneficiating techniques suitable for the region.

(iv) Desalinated water be used for mixing and subsequent curing operations.

6.2 TECHNIQUES FOR CHOKING OFF THE PERMEATION OF CORROSION PROMOTING AGENTS (CHLORIDES, OXYGEN AND MOISTURE)

A variety of solutions along this approach have been proposed, some still in the experimental stage, others being incorporated into current practice. These include latex modified concrete,¹⁶⁶ polymer impregnated concrete,¹⁶⁶ internally sealed concrete,¹⁶⁷ sulfur infiltrated concrete,¹⁶⁸ overlays of "super concrete" and membranes^{169,170}. These solutions have met with varying degrees of success and have been proposed for incorporation into new construction as well as for arresting deterioration of in-place structures. These are discussed in greater detail later in this section.

While it is true that structures already exhibiting corrosion must either be protected or replaced, it is not generally recognized that a simpler, less expensive approach exists to the design of reinforced concrete with the aim of minimizing damage due to corrosion of reinforcement. This approach, which may be termed "quality concrete" has been

previously discussed,^{171,172,173} though recognition of its merits remains limited to a fairly small group of practitioners within the field of concrete technology. This being one of the simplest and the most straightforward solution to the problem is discussed first.

6.2.1 Quality Concrete

It is by now obvious that the mechanism of corrosion deterioration is directly related to the porosity of the cement paste. The protective properties of concrete are inhibited and destroyed if it allows the diffusion of oxygen and moisture and additional chlorides by virtue of its porosity. What is important are the twin characteristics of absorption and the ease of movement of liquids and gases within the matrix of concrete. A good quality concrete that is dense and impervious reduces the intrusion of oxygen, moisture and aggressive salts thereby choking off the corrosion process. The permeability of concrete is mainly determined by the permeability of the paste and it is well known that paste permeability is not a simple function of its porosity. It can only be correlated to an effective porosity concept and would mainly be governed by the pore system and the capillary porosity of the paste. Unfortunately there is virtually no data

on the permeable pore space characteristics of concrete and no relationship has been established between absorption and the water movement characteristics along the capillary paths in concrete. However, Power's¹⁷⁴ work has shown that at a certain w/c ratio the pores in the cement gel get segmented and become discontinuous. It seems that this is the point at which maximum resistance to corrosion may be obtained and that further reduction would not lead to much improvement. There seems to be at present no clear idea of what the w/c ratio is for this stage, but on the basis of the existing data it is regarded to lie between 0.4 and 0.45. This gives a concrete with an aggregate/cement ratio of about 4.5 and a minimum cement content of about 700 to 750 lb /cu.yd³ for maximum size of aggregate lying between 3/4 to 1 inch. Even in normal circumstances where aggregate gradation can be considerably optimized such a concrete will be somewhat difficult to place due to poor workability. FHWA researcher Dick Hay says that 2 inches of cover using concrete with a w/c ratio of 0.5 falls short of providing the requisite impermeability in the case of bridge decks. On the other hand 2 to 2½ inch of cover using concrete with a w/c ratio of 0.4 is found to provide effective protection to rebars. But since such concrete has poor workability,

it is rarely used in the field.

This difficulty of low workability is further aggravated in the Gulf region due to poor gradation of coarse aggregates and due to the fineness and narrow gradings of the dune or beach sands. These factors considerably increase the water demand because of the increased specific surface area due to the sand fineness and because the sand proportion cannot be decreased below a certain amount. A reasonable amount of sand is always required in a mix to avoid harshness due to the lack of cement-sand mortar which fills the voids between the coarse aggregates. The consequences of using a seriously undersanded grading can be very harmful because such gradings cannot produce workable and dense concrete and undersanding also results in bleeding and segregation. The extra water demand is never satisfied by the suggested optimum w/c ratio of 0.40 and will invariably result in a stiff concrete mixture accompanied by dry segregation. This results in honeycombing with consequent loss in durability and strength. Moreover, there is evidence to suggest that the workability of concrete has a very significant effect in providing protection against corrosion. Test data shows that the best protection is obtained when the bars

are evenly coated with cement paste and this is not possible with harsh mixes. The classical solution to this problem appears to be to use a richer mix with an aggregate-cement ratio smaller than 4.5 which may enhance the workability to acceptable limits. However, this will almost certainly increase the cement quantities to a level where cracking due to shrinkage and early thermal stresses would appear within weeks of concrete placement in the environment of the Gulf States.

This is a statement of the conflicting and difficult situation a concrete technologist faces in this region. Such a situation can only be handled by reorienting mix design concepts to suit the local situation, by invoking the latest developments in the field of concrete technology and by developing data to obtain a better understanding of the effect of binary proportioning of the typical Middle East sands and coarse aggregates on the properties of plastic and hardened concrete. Arbitrary gradings should be replaced by optimum proportioning leading to maximum compactibility. Also, as the grading characteristics of fine aggregate control the concrete-making techniques more than the grading characteristics of the coarse aggregate, more attention should be paid on the fine sand

gradings and their effective incorporation in the design of quality concrete mixtures. Further, concrete quality should be evaluated in terms of strength and density and specifications should be developed on the basis of the twin considerations of strength and durability and not on the basis of strength alone.

It may be worth pointing out the U.S. experience in the State of Iowa in this connection. The State Highway Department has successfully used the so-called "super concrete" for overlays on more than 200 bridge decks to prevent chloride penetration. This concrete usually has a w/c ratio as low as 0.32 and a slump as small as 3/4 inches. However the resulting stiff concrete is placed with the help of special vibrating screeds having low frequency and high amplitude. The concrete is vibrated to 98% of rodded density which is measured with a direct transmission nuclear gauge.

A recent development in concrete technology appears to be specially promising to obviate the poor workability situation obtainable with low w/c ratios. This is the appearance of a new generation of water reducing admixtures called "super plasticizers"^{127,128,129}, which by virtue of dispersing the cement agglomerates efficiently increase the workability

of normal w/c ratio concrete to a slump of 8 or more inches without excessive bleeding or segregation. The use of superplasticizers produces normal workability in a low water-cement ratio concrete and concretes made this way are noted for their high early and ultimate strengths, excellent durability and water tightness. With the use of these admixtures no adverse properties are reported. The strength/time trend and creep and shrinkage properties remain normal. Information from Japan indicates that the long-term durability is improved.

Another development which could be beneficially exploited is the increased use of ready mixed concrete which is more likely to be properly proportioned and designed than the insitu concrete.

A yet another aspect of concrete technology which is known and practiced in the developed concretes for the last 15 years due to its markedly beneficial effect on concrete durability has only been rarely used in this region. This is air-entrainment which materially alters the properties of both fresh and hardened concrete. Air-entrained concrete is considerably more plastic and workable than non-air entrained concrete. The paste is more

cohesive and can be placed with less tendency for bleeding and segregation. The concrete strength may be reduced somewhat by air entrainment, but the durability of the hardened concrete is improved considerably by increasing uniformity, decreased absorption and permeability and by the elimination of planes of weaknesses at the tops of lifts.

The importance of good design and construction practices in the production of quality control can hardly be over emphasized.

6.2.2 CONCRETE COVER TO REINFORCEMENT

On the basis of the results of this field investigations the author regards adequate cover to reinforcement as the cheapest, simplest and the most effective precaution against corrosion damage. It is suggested that the following broad suggestions for the minimum cover values may be considered for the Gulf area. The author is making this proposal on the premise that low w/c ratio concrete (0.4 to 0.45) of an average strength of 5000 psi will ordinarily be obtainable.

Building components which are permanently exposed to the salt-laden corrosion atmosphere 2.00

Building components which are protected against weather and the aggressive conditions of exposure 1"-1½"

Concrete components exposed to sea water and footings as well as other main structural members cast against the ground 3"

Corrections to these basic values may be applied as follows :

CORRECTION FOR				
Poor* Work- manship	Reinforcement Susceptible to corrosion	CONCRETE		Prefabricated components with strict control
		Less than 4000 psi	More than 6000 psi	
+0.25*	+0.25 in	+0.5 in	-0.25 in	-0.25 in

* may be omitted for well protected members

It may be mentioned here that the above proposals could be at risk in the absence of minimum standards of workmanship, control on tensile cracking and the use of dense, good quality aggregate. In the circumstances prevailing in this region this consideration should be duly allowed for. In fact a correction for workmanship becomes meaningless if

the concrete production technique suffers to the extent that on the whole a reasonably impervious, uncracked concrete is not obtainable.

6.2.3 Polymer Impregnated Concrete

Research has shown that remarkable improvements in the strength and durability properties of concrete can be attained by the formation of concrete - polymer materials.^{130,131,132} Polymer impregnated concrete consists of a Portland cement concrete impregnated with a monomer system that is subsequently polymerized in-situ with radiation or heat. For a concrete mix that produces specimens with a compressive strength of 5000 psi impregnation raises the strength to 20,000 psi. In normal concretes voids are filled up to 6-8 per cent by polymer weight. The higher strength and durability properties are ascribed to improved cement-to-aggregate bond, plugging of voids and healing of micro fractures within concrete thus minimizing permeability and fracture propagation flaws. Tests at the University of Texas showed that after 24 hours of water immersion the depth of water penetration was nil for PIC in comparison to an average penetration of about 3 inches in the conventional concrete specimens. PIC also shows considerable resistance

to sulphate attack. In a test series the conventional concrete sample showed severe cracking and expansion of 0.5% (failure) after 450 days exposure, while at 510 days the PIC sample remained free of cracks and showed essentially no expansion. Design values for PIC that cover the range of monomer systems used have been published by Coward⁷⁵.

PIC is about three times as expensive as conventional concrete but has four times the strength and longer life outlook. It is noteworthy that the monomers used for impregnation are petroleum derivatives and hence offer special promise for use in Gulf States. There are, however, difficulties involved in evolving processes and hardware for large scale applications. Several bridge decks have been impregnated in the USA for protection against chloride penetration. These are under study for performance.

A variant of concrete-polymer composites is partially impregnated concrete which is designed for durability only limiting the depth of impregnation to 1 inch thus saving the amount of monomer. Another variant is polymer-concrete.

6.2.4 Latex-Modified Concrete

One material whose use is becoming more widespread to effectively choke off the intrusion of corrosion promoting

media is latex-modified concrete (LMC): Portland cement concrete with a latex admixture. The LMC is prepared by mixing the latex additive (water-styrene butadiene emulsion), a thin white fluid, with Portland cement, water and aggregate. The w/c ratio, which includes water in the latex, is somewhat lower (0.3-0.4) than typical Portland cement concrete, but the mix is workable, not as stiff as low-slump concrete of comparable w/c ratio is. The LMC costs about \$ 100/yd³ more than conventional Portland cement concrete, hence the use of LMC till now is confined as overlays on slabs rather than for the production of full scale structural components.

Because of its strength and high impermeability more than 200 bridge decks have been overlaid with LMC to prevent chloride penetration. Lab studies with a powerful electron microscopic show water or salt cannot possibly penetrate any deeper than $\frac{1}{2}$ to $\frac{3}{4}$ inch because pores in the concrete below that level are fully plugged with polymer. A latex-modified concrete sample continuously immersed in a saturated salt solution for more than two years showed no sizeable penetration of chlorides. Cores were removed from a bridge deck overlaid in 1969 (Michigan) with 1 inch thick layer of LMC after three winters of heavy salting.

These cores showed slight chloride penetration in the top $\frac{1}{2}$ inch and no chloride penetration at depths greater than $\frac{3}{4}$ inch. Cores removed from several other bridge decks showed similar excellent performance and excellent bond between the latex concrete and the old concrete of the deck.

6.2.5 Epoxy-Modified Concrete

, The epoxy-modified Portland cement concrete is achieved by mixing a blend of water-dispersible epoxy resin and converter with the conventional components of Portland cement concrete. The water-dispersible epoxy binder system is furnished as two components. The reinforcing steel is painted with the epoxy immediately prior to concrete placement. This procedure does not result in a coating for the reinforcing bar since the epoxy is still liquid when the concrete is placed. Its purpose is to provide an epoxy rich concrete layer around bar.

6.2.6 Internally-Sealed Concrete

If the capillaries in concrete could be filled, chlorides, oxygen and moisture could not diffuse to the reinforcing steel. This premise led the FHA to the concept of internally sealed concrete.

Wax is added (in the form of 20 to 80 mesh wax beads) to the conventional concrete at the time of mixing and the concrete is conventionally cured for 28 days. The wax, however, does not fill all the voids which are usually filled with air and water. However, when the concrete is heated the wax flows in the voids helped by the pull of the capillary tension. When the concrete cools it is sealed internally and is virtually impermeable to salt. Difficulties are being felt with the heating process. Electric blankets, infrared heating and a few other systems have been tried but a satisfactory system is under R & D stage. Strength is reduced by 10-15% but the impermeability is considerably improved. After 417 daily applications of a 3% NaCl solution there was no chloride penetration greater than $\frac{1}{2}$ inch. In another test series a 3% salt solution ponded for 90 days atop a slab failed to penetrate the concrete while in the average conventional concrete much salt penetrated to $1\frac{1}{2}$ inch to 2 inches. No adverse effects have been reported in terms of shrinkage and bond characteristics.

6.3 USE OF CORROSION RESISTANT REINFORCEMENT

The trend towards using corrosion resistant reinforcement has already started, examples being precast-ing industry. Precast concrete cladding panels used in many buildings in the U.K. are $1\frac{1}{2}$ to $1\frac{3}{4}$ in. thick incorporating 6 and 8 mm diameter galvanized reinforcement with a maximum cover of $\frac{3}{4}$ in. For many buildings which are on sea front, such as the Bank of Hawaii in Honolulu, galvanized reinforcement is being currently used to avoid disfigurement of the concrete by rust, to decrease the risk of spalling and permit a reduction of concrete cover over rebars. In Bermuda, where no building is more than three-quarters of a mile from the sea, and the humidity and salt-laden air attack unprotected ferrous metals, the Department of Public Works has required the use of galvanized rebars in structures for the past 25 years. All indications are that the galvanized rebars in Bermuda have stood up well in building use. Titanium-alloy ties have been used to fix the precast concrete cladding to the shell roofs of the Sydney Opera House which is situated in Sydney Harbour in an aggressive marine environment. FHWA has constructed more than fifty bridge decks in USA using epoxy-coated rebars, and presently the use of epoxy-coated

rebars is a number one solution of corrosion problem with the FHWA.

6.3.1 Organic Coatings on Rebars

Although the use of organic coatings on rebars to prevent corrosion was recommended ^{176,177} as early as 1968-1972 the idea gained real acceptance only after a very comprehensive evaluation ¹⁷⁸ was made by the US National Bureau of Standards in collaboration with US Federal Highway Commission. The research programme included the performance evaluation of 47 different organic coatings from the standpoint of their chemical and physical durabilities, chloride permeabilities, protective qualities and the bond with concrete. As a result of this research programme it was concluded that the best performance was provided by epoxy coatings and that properly formulated and properly applied powder epoxy coatings adequately protected the steel reinforcement from rapid corrosion due to chloride attack. The powder epoxies performed better than liquid epoxies. The epoxy coated reinforcing bars, with film thicknesses below 10 mils (250 μm) developed bond strengths which were comparable to the bond between uncoated bars and concrete. When the film thickness was

increased to 25 mils (640 μ m) the bond strength decreased to 60% of that with uncoated bars.

Presently several manufactures in the USA are producing epoxy-coated bars. The technique consists of sand blasting the steel surface of the ordinary bar to provide good bond, then heating, and finally spraying powdered epoxy on the heated rebar surface where the epoxy melts and cures forming a hard, durable coating. FHWA has evaluated this measure against corrosion very highly and has listed it as its No.1 preference. FHWA engineers consider the use of epoxy coated bars to be a permanent solution to the problem of corrosion and spalling due to chloride attack as against the use of galvanized bars which only seems to buy additional time. As a result of their superior performance more than fifty bridges have been constructed in USA using epoxy coated bars.

Epoxy coated bars can be bent to shapes to meet engineering specifications without significant holidays. If there are any tears due to rough handling research shows corrosion will occur only in the area of tear. Also, small breaks are successfully touched up with a liquid epoxy coating. Bars remaining exposed in aggressive job sites for long

periods have shown no signs of deterioration.

The chief objection to epoxy coated bars is their cost which is about $2\frac{1}{2}$ to 3 times the cost of uncoated bars.

6.3.2 Metallic Coatings on Bars

A possible electrochemical approach has been to the clad the steel with another metal. If a more noble (electropositive) metal is used there is great risk of pitting attack on the underlying steel at any pinholes or other defects in the coating (although in practice nickel-clad steel has proved durable in American trials). If a layer of a more electronegative metal, such as zinc is used, the steel exposed at any defects in the coating is protected by the sacrificial action of the zinc. Galvanised steel reinforcement is finding increasing use where very thin concrete sections are being employed (thin domes, etc.), where for aesthetic reasons rust stains cannot be tolerated on fair-faced concrete, and for very severe marine exposure conditions, where chloride contamination is inevitable.

A galvanized rebar has a 4-mil thick coating of zinc on it and has the advantage over epoxy coated bars of lesser

cost and better response to rough handling on site. If a galvanized coating is broken or scratched, it usually heals itself : the galvanised protection of adjacent zinc prevents an attack of the exposed steel. The use of zinc coated bars has been encouraged by FHWA on experimental basis to monitor its performance under aggressive chloride penetration. The cost of galvanizing the reinforcement adds about 25% to the cost of reinforcement.

6.4 CATHODIC PROTECTION

Cathodic protection is a preventive measure where a negative potential is applied on anode thus superimposing a stronger current in the opposing direction. In effect corrosion is arrested by making steel reinforcement a cathode for d-c current. Cathodic protection has been applied in several corrosion situations^{179,180} and has been found to be effective. However, the methodology of the process is still in R and D stage.

6.5 CORROSION INHIBITORS

Corrosion inhibitors are admixtures that are used in concrete to keep the metal embedded in the concrete from corroding. Anodic inhibitors which function in an effort

to stifle the reaction at the anode are considered to be more effective than cathodic inhibitors which inhibit the cathodic reaction.

There is very little evidence to reveal the long term effectiveness of inhibitors in concrete. Recently preliminary good reports have been received about a calcium nitrite admixture. The effective agent in this compound is the nitrite ion, which is said to prevent ferrous ions from migrating away from the anodic areas, thus stopping the corrosion process at the source.

Chapter 7

CONCLUSIONS

Condition surveys indicate the corrosion deterioration of reinforcement associated with concrete spalling to be the number one concrete durability problem in the Eastern Province of Saudi Arabia. Other forms of deterioration such as cracking and loss of strength due to shrinkage, thermal gradients and sulphate attack and salt weathering were all noticed. These may have an accelerating interactive effect on corrosion deterioration by acting as feeders of corrosion promoting media to the concrete-steel interface. The root of the deterioration problem lies in the cumulative interaction of : (i) presence of chloride and sulphate salts, (ii) defective construction practices, (iii) adverse climatic environment, (iv) geomorphic conditions resulting in marginal aggregates and (v) general lack of appreciation of the problem.

Corrosion of reinforcement resulted in loss of metal and concrete spalling. Inspection of corroded steel showed pitting to be the dominant characteristic corrosion of

unexposed steel in concrete indicating differential concentration type of galvanic cells to be the causal factor. Data showed sharp variations in chloride contents of mortar along the steel-concrete interface confirming the setting up of the differential salt concentration corrosion cells. It is considered highly probable that differential oxygen cells were also operative. The typical corrosion product of exposed steel was dark brown in the form of black stratified flakes. Strong evidence to the effect that spalling of concrete is initiated at a plane of weakness at the level of the reinforcement mat due to water gain was forthcoming from a large number of observations during condition surveys.

Analysis of Gulf concrete showed a very high chloride concentration. The average value being about 8 lb/cu.yd. of concrete which is about eight times as much as allowed by ACI Committee 201 on Durability of Concrete. This is explained by high contributions from mix materials and subsequent ingress from salt-laden dew, moisture and other environmental factors.

Chloride ion and its concentration in concrete have a very definitive influence on corrosion deterioration

Analysis of field data indicates that for concrete covers of $\frac{1}{2}$ inch and 1 inch and for a given quality of concrete the loss of metal was almost linearly proportional to the chloride concentration in concrete. However, chloride content was not indicated to be the sole criterion of corrosion severity. The quality of the physical shielding effect due to concrete cover or the quality of concrete also had significant bearing on corrosion damage. Although a threshold chloride concentration value depends on several factors, an analysis of field data suggests that for the concretes investigated and for the exposure conditions of the Eastern Province the following threshold concentration values may be considered to be reasonable :

$\frac{1}{2}$ inch cover	2.5 lb/cu.yd. of concrete
1 inch cover	4 lb/cu.yd. of concrete
2 inch cover	6 lb/cu.yd. of concrete

During condition surveys it was observed that very frequently corrosion and spalls were directly attributable to insufficient concrete cover. From an analysis of field data for cover and loss of metal relationship it has clearly emerged that the extent of concrete cover is one of the most influential parameters affecting corrosion

deterioration performance of steel. In slabs 22-27 years old loss of metal is very high (30 to 50%) for $\frac{1}{2}$ inch cover, however it decreases sharply and becomes less than 10% for concrete cover of 1 inch or more. The chloride threshold concentration value decreased sharply from about 6 lb/cu.yd. for 2 inch cover to 2.5 lb/cu.yd. for $\frac{1}{2}$ inch cover.

Quality of concrete as expressed by its absorptive characteristics is also shown to have a marked effect on corrosion mechanism. Corrosion or loss of metal increases linearly with concrete absorption capacity. Concretes with 2.5% or less absorption in a 30-minute absorption test showed less than 10% loss of metal. Concretes with more than 4% absorption showed a high supportive tendency to corrosion. For these concretes the loss of metal in 20-25 years was as high as 60%.

Chloride migration characteristics in uncracked concrete showed a decrease in concentration with depth. The reduction in concentration, however, is far flatter than shown by field data from elsewhere. A probable explanation of this is high original base values in the concrete.

Laboratory data has been developed on the effectiveness of C_3A component of cements in removing free soluble chloride ions from the cement paste by chemically combining with the free chlorides. It is seen that a Type I cement with 9 % C_3A was more effective in reducing the chloride concentration in cement paste than a Type V cement with 2% C_3A . This was confirmed by a higher demonstrated corrosion activity by concrete slab samples made with Type V cement than by those made with Type I cement. This seems to lead toward a conclusion that Type I cement is likely to be more effective in countering corrosion where the presence of sulphates do not require a Type V sulphate resisting cement. However, analysis of field samples showed fairly high sulphate contents in the Gulf concrete making it difficult to formulate a clear recommendation. More research is required to elucidate this aspect of the problem.

The pH value for a rather poor quality concrete made for laboratory samples was found to be 12.5. pH values for samples removed from old buildings were found in the range of 8-9.

Fine and coarse aggregates from local quarries as well as typical mix water showed high degree of chloride and sulphate contaminations. A typical concrete mix gets a base value of chloride content 2 to 2.5 times the permissible ACI value.

Evaluation of coarse aggregates indicates poor physical and absorption characteristics and very sharp variations at close distances. Beach and dune sands are too fine and narrowly graded resulting in high water demand, shrinkage, bleeding and segregation. The local coarse aggregate is very dusty on crushing and this study shows that chlorides and sulphates tend to become heavily concentrated in the fine fraction and the dust obtained during crushing.

Recorded observations during condition inspections and surveys to the sites of construction show that one of the important reasons for the low durability of concrete construction in this region is the failure to adopt sound design and construction practices and to implement the requirements specified for concrete quality and steel positioning. Inadequate cover to reinforcement was frequently observed in old and new structures alike. On construction sites there was a total absence of the use of

bar spacers and chairs and unacceptable displacements of bar assemblies resulted during concrete placing operations. In concrete making operations on sites, the ease of placement seemed to be the governing consideration and this was invariably achieved by adopting high w/c ratios. Typically the aggregates in concrete mixtures were badly graded and with high w/c ratios this resulted in excessive segregation and bleeding. On most sites there was no control or supervision through qualified personnel.

It seems very likely that the construction techniques in the region will have to contend with high chloride contents in concrete usually far in excess of the threshold value for corrosion. In view of this, for improved durability of concrete construction it is essential to evaluate aggregate more critically, to adopt modified mix design techniques based on specifications for maximum w/c ratio, minimum cement content and optimum binary aggregate gradings leading to maximum compactibility. The aim should be to produce as dense and impervious concrete as possible in order to choke off the intrusion of oxygen and moisture to inhibit cathodic depolarization and thereby prevent corrosion.

Latest developments in concrete technology such as the use of super plasticizers with low w/c ratio to facilitate placement should be encouraged and invoked.

The use of epoxy-coated reinforcement appears to be a very promising possibility as a solution to the corrosion deterioration problem.

Research is needed to develop data relating concrete absorption and permeability characteristics with the strengths of typical concretes in the region, regarding optimisation of concrete mixtures from durability considerations, regarding the effect of adverse Gulf parameters on electrical resistivity properties of concrete and regarding the rate and amount of corrosion progress as affected by local exposure conditions.

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