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Structural Performance of Repaired Corroded Reinforced Concrete Beams

التصرف الإنشائي للكمرات الخرسانية المعالجة من الصدأ

By

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ABSTRACT

This research presents the results of a laboratory investigation of the flexural capacity such as strength, deflection and steel mechanical properties for both corroded and repaired corroded beams. Examination of the behavior of crack development was also examined.

Fourteen small scaled reinforced concrete beams were used in the testing program. Ten out of them were tested as simply supported beams subjected to two concentrated point loads. Two out of the ten as control beams two others as corroded beams and six of them as repaired beams. The other four were used to investigate the corrosion rate and the mechanicals properties for corroded steel bars.

An electrochemical system was used to achieve the corrosion level. The specimens were immersed in a 5 percent of sodium chloride solution for a period of 11 weeks. The beams were connected in parallel to +5 Volts with a capacity of 25 Amperes electric DC current generated by power supply which impressed an equal voltage on each beam to accelerate the corrosion process. Steel bars were immersed in the solution to act as a cathode and to force the steel reinforced concrete beam to act as an anode.

Six corroded beams out of the fourteen were structurally repaired after been corroded. Additional longitudinal steel reinforcement bars fixed with shear connectors were added to the flexural corroded reinforcement. Three types of cementitious and resin repairing materials were used to apply a new layer instead of the crashed deteriorated beams bottom concrete cover.

It was concluded that the flexural capacity of the corroded beams reduced by about 28 % compared with the control beams and showed a noted reduction in its ductility behavior during the flexural test. The flexural capacity of the repaired beams increased by about 47 % compared with the control beams. They showed good ductility behavior during the flexural test and performed as sound constructed beams regarding their flexural capacity, crack development and deflection.

It also concluded that types of repairing materials used for applying a new layer to corroded beams did not affect the flexural performance of repaired beams, in spite of that it may be important to inhabit the corrosion process in the future.

ملخص الرسالة

يقدم هذا البحث نتائج عملية للتصرف الانشائي مثل القوة و الهبوط و الخصائص الميكانيكة للحديد لكل من الكمرات الخرسانية التي تعاني من مشكلة صدأ الحديد بالاضافة للكمرات الخرسانية المعالجة من الصدأ. كما و يقدم تحليلا عن حالة التشققات التي تظهر.

اربعة عشر كمرة خرسانية تم استخدامها في برنامج الفحوصات، عشرة منها تم اختبارها ككمرات خرسانية مرتكزة على دعائم بسيطة و معرضة لقوتين مركزتين، كمرتين من العشرة تم اختبارهما ككمرات قياسية و كمرتين تم اختبارها ككمرات تعاني من الصدأ و ستة كمرات تم اختبارها ككمرات معالجة انشائيا من الصدأ. الأربعة كمرات المتبقية تم استخدامها لفحص درجة الصدأ و كذلك لتحديد الخواص الميكانيكية للحديد المعرض للصدأ.

للوصول للدرجة المطلوبة من الصدأ، تم استخدام نظام كهروكيميائي. تم غمر العينات بمحلول ملحي من كلوريدات الصوديوم بتركيز ٥% لمدة ١١ اسبوع. تم توصيل الكمرات بالتوازي بتيار كهربي بجهد ٥ فولت و شدة تيار قصوى بقدرة ٢٥ أمبير، و تم توليد الكهرباء بواسطة مولد يعطى كهرباء مستمرة لتسريع عملية الصدأ. قضبان من الحديد تم غمرها في المحلول لتعمل ككاثود و تجبر الكمرات لتعمل كأنود.

ستة كمرات من أصل الأربعة عشرة كمرة تم اصلاحها انشائيا بعد تعرضها للصدأ، حيث تم اضافة حديد سفلي جديد و تم تثبيته بقطع حديدية لتقاوم قوي القص، و تم استخدام ثلاث مواد اصلاح اسمنتية و ايبوكسية لاضافة طبقة جديدة للغطاء الخرساني المهترأ.

أظهرت النتائج أن القوة الانشائية لتحمل الكمرات المعرضة للصدأ لعزم الانحناء كانت أقل حوالي بنسبة ٢٨% من الكمرات القياسية و أظهرت ضعف في مرونتها أثناء الفحص. بينما زادت القوة الانشائية للكمرات المعالجة من الصدأ حوالي بنسبة ٤٧% مقارنة بالكمرات القياسية و أظهرت مرونة جيدة و تصرفا انشائيا مماثلا للكمرات التي لا تعاني من أي مشاكل بالنسبة لقوتها الانشائية و التشققات و الهبوط الناتج عن الفحص.

كما و أظهرت النتائج أن نوعية مواد الاصلاح المستخدمة في اضافة طبقة جديدة لم تترك أي تأثير على الأداء الانشائي للكمرات المعالجة من الصدأ، بالرغم من أهمية استخدام هذه المواد لمنع أي عملية صدأ مستقبلي قد ينشأ بعد عملية الأصلاح.

DEDICATIONS

This research is dedicated to my mother, my father, my wife, and my children for their endless love and support.

ACKNOWLEDGMENT

I would like to express my sincere appreciation to Assoc. Prof. Mohamed Ziara - Department of Civil Engineering, Faculty of Engineering, The Islamic University of Gaza- for his help and guidance in the preparation and development of this work. The constant encouragement, support and inspiration he offered were fundamental to the completion of this research.

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Finally I would like to thank everyone who gave advice or assistance that contributed to complete this research.

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Chapter 1: Introduction

1.1. General Background

Reinforced concrete is considered the most frequently used structural material, not only it has good mechanical proprieties after hardening, easy to use, etc. but also its dominant advantage that it is considered as an economic structural material.

In recent years, the common point view about concrete as a durable maintenance-free construction material has been changed. The insufficient consideration of durability during the design process, the inadequate execution and maintenance are some reasons that reinforced concrete structures did not perform as well as it was expected **(Kovacs, 2000)**.

Corrosion of concrete reinforcing steel is considered one of the most serious problems facing the reinforced concrete structures. More than 80% of reinforced concrete structural damages around the world are caused by the corrosion of steel **(Franciskovic and others, 2006)**.

The problem of reinforced concrete corrosion is consider also as an economical problem, some countries like U.S. and U.K. spent hundreds of millions of dollars yearly for repairing structural damages resulting from reinforced concrete corrosion problem. This problem is clearly manifested in many other courtiers, especially in the Middle East Area **(El-Reedy, 2008)**.

1.2. Corrosion Problem in Gaza Strip

Gaza strip is costal area which has 40 Km coastline on the Mediterranean Sea. This location with the associated environmental conditions may have a considerable influence on the deterioration of existing concrete structures, especially steel corrosion.

Although the problem of reinforced concrete corrosion common appears in Gaza Strip area from common practice, few researches had studied this problem. Some factors like inadequate concrete cover, quality of water or admixtures uses in the mix design of concrete, seashore climate affects, etc. may be some factors causing this problem.

A survey of forty case studies for assessment of existing damaged structures in Gaza Strip showed that the main cause of defects in existing buildings was reinforcement corrosion, with about 31% of the causes **(Abu Hamam, 2008)**.

Figure 1.1 show photos of some corroded reinforced concrete beams. These pictures were taken from a repairing of 104 shelters project at Rafah area in Gaza Strip, executed by the United Nations Relief and Works Agency (UNRWA) 12-2006.

Figure 1.1: Some Pictures of Corroded Reinforced Concrete Beams

(Repairing Of 104 Shelters Project at Rafah Area In Gaza Strip, Executed By UNRWA 12-2006)

1.3. Research Problem and Scope

This study will deal with the problem of corrosion of reinforced concrete beams. The aim of this research is to evaluate the efficiency of repairing techniques for solving this problem, from a structural point view.

In this research, repairing techniques will be applied for corroded reinforced concrete beams. These repaired corroded beams will be tested to evaluate their structural performance. Although many previous studies have discussed the reinforced concrete corrosion problem, this study will focus on the problem of structural performance of the repaired elements.

This research will cover small_scale corroded reinforced concrete beams with steel reinforcement and normal weight concrete. Pre-stressed and or high strength concrete elements will not be covered by this study.

1.4. Objectives

Research The main objective of this study is to determine the structural performance of repaired corroded reinforced concrete beams and how to assist engineers in properly applying successful repairing techniques in real life application.

To reach this goal, sub main objectives where determined as follows:

1. To impose corrosion for reinforcing steel bars embedded in reinforced concrete beams.
2. To determine the strength capacity of corroded reinforced concrete beams.

3. To apply a successful repairing technique for repairing the corroded reinforced concrete beams.
4. To determine the strength capacity of repaired corroded reinforced concrete beams.
5. To investigate the validity of using ACI code in determining the flexural capacity of repaired beams.

1.5. Methodology

To achieve the research objectives the following methodology was followed

1.5.1. Literature Review

In this section previous research works related to the subject of the undertaken research were reviewed to identify main concern aspects of the problem and its repair.

1.5.2. Testing program

The test program is developed based on full understanding of the problem. It is designed to achieve the research problem. Influencing factors, corrosion mechanisms, repair materials and techniques, testing setups, etc. are fully considered in this step.

1.5.3. Experimental Work

The details of the test program are addressed in this step. These include testing equipments, number and size of tested beams, corrosion mechanisms, casting, curing, materials, etc.

1.5.4. Results and Discussion

After finishing the experimental study, the test results are obtained, analyzed and discussed to investigate the targeted objectives.

1.5.5. Conclusions and Recommendations

Conclusions and recommendations reached based on the test results are proposed for use by engineers in Gaza Strip to decide on an optimum and effective way for repairing corroded RC beams in the real life application.

1.6. Thesis Organization

The thesis contains seven chapters organized as follows:

Chapter 1(Introduction)

This chapter gives a general background about the reinforced concrete structures corrosion problem, research problem and scope, objectives and methodology used to achieve the research objectives. Also it describes the structure of the thesis.

Chapter 2 (Corrosion of Steel in Concrete "Basics, Causes and Effects")

This chapter discusses the concept of corrosion of steel in concrete, its mechanism, types, causes and structural effects.

Chapter 3 (Repair of Corroded Reinforced Concrete Structures)

This chapter reviews the techniques of repairing corroded reinforced concrete structures, main steps to execute repair and the execution methods of repairing.

Chapter 4 (Material and Test Program)

This chapter contain description of testing program, it discuses general preparation used and difficulties face the test program, the materials used for constructing beams, concrete mix design job, repaired materials and specimen design. It also describes the experimental set-up, corrosion process set-up, beam repairing set-up and beams flexure testing procedure.

Chapter 5 (Test Results)

This chapter illustrates the test results including the electrochemical results, visual inspection and flexural test results.

Chapter 6 (Discussion of Results)

This chapter discusses the results and shows the results summary drawn from the test results chapter.

Chapter 7 (Conclusions and Recommendations)

This chapter includes the concluded remarks, main conclusions and recommendations drawn from this research.

Chapter 2: Corrosion of Steel in Concrete (Basics, Causes and Effects)

2.1. Introduction

Corrosion is an electrochemical process where a metal undergoes a reaction with chemical species in the environment to form a compound. The chemical species are principally oxygen and water. The corrosion of steel is the process that steel is oxidized at the anode and the electrons are released and flow to the cathode for the oxygen reduction reaction **(Zhang and Mailvaganam, 2006)**.

In the case of steel embedded in concrete, the concrete is a porous material containing water in the voids due to the process of curing or because of rainy weather or any weather with high relative humidity. Thus, the concrete will contain humidity, which is a common cause of corrosion.

Although, it is not necessary that steel bars embedded in concrete to be corroded, this happen because concrete has a high concentration of the oxides calcium, sodium, and magnesium. These oxides produce hydroxides that have a high alkalinity when water is added (pH 12–13). This alkalinity will produce a passive layer on the steel reinforcement surface; consisting of oxides and hydroxides for iron and part of cement. This layer is dense and prevents the occurrence of corrosion **(El-Reedy, 2008)**.

Figure 2.1 shows a diagram of passivation of steel in concrete under normal conditions **(AL-Ostaz, 2004)**.

Passivation of Steel in Concrete

Alkaline environment protects reinforcing steel from corrosion

Figure 2.1: Diagram of Passivation of Steel in Concert (AL-Ostaz, 2004)

This passive layer is, however, can be broken when carbon dioxide enters the concrete and reaches the steel-concrete interface. This is called carbonation.

Another powerful destroy of the steel passive layer is the present of chloride salt in concrete. Chloride ions are introduced into the concrete by marine spray, industrial brine, deicing agents, and chemical treatments. These chloride ions can reach the reinforcing steel by diffusing through the concrete or by penetrating cracks in the concrete **(AL-Ostaz, 2004)**.

2.2. Corrosion Mechanism

After the passive layer is broken down, rust will appear instantly on the steel bar's surface. The chemical reactions are the same in cases of carbonation or of chloride attack **(El-Reedy, 2008)**.

During the corrosion process, current flows in a closed loop. In addition to electrons flowing through the steel, an external current is carried through the pore solution of the concrete by the movement of charged ions to complete a closed loop. The external current consists of negatively charged hydroxyl ions moving from the cathode to the anode, and positively charged ferrous ions moving from the anode to the cathode **(Kepler and others, 2000)**.

Equations 2.1 through 2.4 describe the anode and cathode steel corrosion reactions as follow **(El-Reedy, 2008)**:

The anode reaction:

$$2Fe \longrightarrow 2F^{2+} + 4e^{-} \qquad (electrons\ pass\ to\ cathode) \qquad \text{Eq. 2.1}$$

The cathode reactions:

$$2H_2O + O_2 + 4e^{-} \longrightarrow 4OH^{-} \qquad (electrons\ pass\ to\ anode) \qquad \text{Eq. 2.2}$$

$$2Fe + 2H_2O + O_2 \longrightarrow 2Fe(OH)_2 \qquad (ferrous\ hydroxide) \qquad \text{Eq. 2.3}$$

$$2Fe(OH)2 \xrightarrow{O_2,H_2O} Fe_2O_3.nH_2O \qquad (hydrated\ ferric\, oxide\ "rust") \qquad \text{Eq. 2.4}$$

Figure 2.2 shows a diagram of rust formation on steel reinforcement in concrete **(AL-Ostaz, 2004)**.

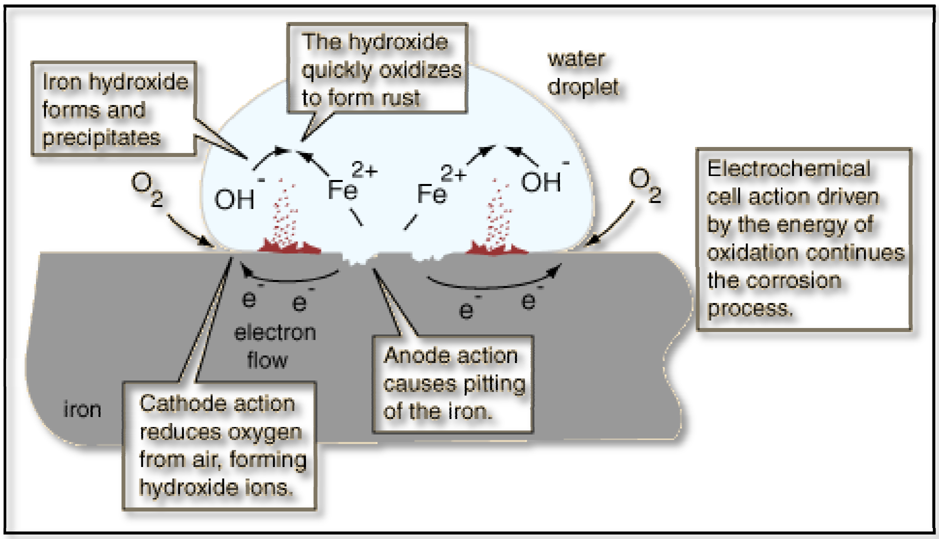


Figure 2.2 Diagram of Rust Formation on Steel Reinforcement in Concrete (AL-Ostaz, 2004)

2.3. Corrosion Electrochemistry (Cells and Half Cells)

The terms ‘anode’ and ‘cathode’ discussed in this chapter come from electrochemistry which is a basic of Daniell cell seen in Figures 2.3.

Daniell cell illustrates how chemical reactions produce electricity. The cell is composed to two ‘half cells’, copper in copper sulphate and zinc in zinc sulphate. The total voltage of the cell is determined by the metals used and by the nature and composition of the solutions **(Broomfield, 1997)**.

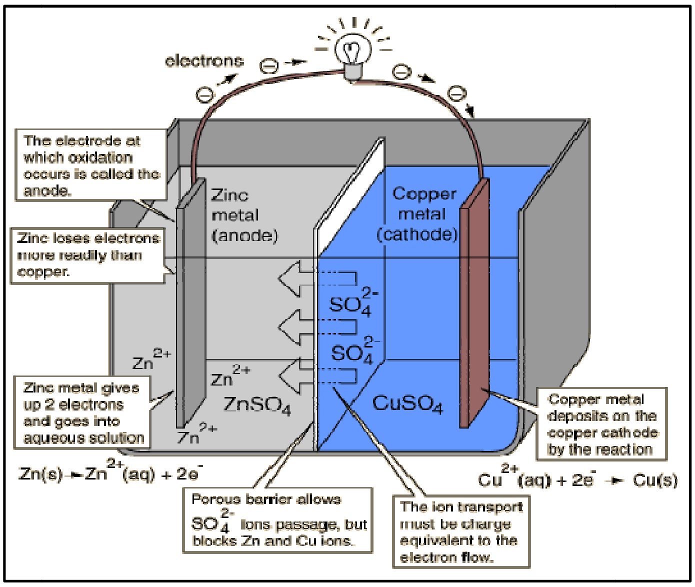


Figure 2.3: The Daniell Cell (Broomfield, 1997)

What is happening is that in each half cell the metal is dissolving and ions are precipitating. Copper is more resistant to this reaction than zinc so when connect the two solutions by a semi-permeable membrane (which allows charge to be exchanged through it but the ions cannot pass through). When connecting the two metals with a wire, the zinc goes into solution and the copper from the copper sulphate solution plates out (is deposited) on the copper electrode **(Broomfield, 1997)**.

Half cell potentials are a function of concentration as well as the metal and the solution. A more concentrated solution is generally more corrosive than a dilute one, so a current will flow in a cell made up of a single metal in two different concentrations of the same solution **(Broomfield, 1997)**.

When a metal such as steel is in an electrolyte (this is an aqueous solution which can carry ions such as water with some rock salt in solution) then a corrosion cell can be formed. A part of the steel in the electrolyte forms the anode and another part of the steel also in the same electrolyte forms the cathode. Corrosion in this case would be occurring at all the anode points which are dispersed around the steel. This gives the appearance of general or uniform corrosion **(Chess, 1998)**.

It can be considered the corrosion of steel in concrete as a concentration cell. That is the co-existence of passive and corroding areas on the same reinforcement bar forming a short-circuited galvanic cell with the corroding area as the anode and the passive surface as the cathode. The voltage of such a cell can reach as high as 0.5V or more, especially where chloride ions are present. The resulting current flow (which is directly proportional to the mass lost by the steel) is determined by the electrical resistance of the concrete and the anode and cathode reaction resistance **(Newman and Choo, 2003)**.

Figure 2.4 is a schematic of micro-corrosion cells on steel’s surface, regions labeled (A) are the anodic areas where metal is dissolving, regions labeled (C) are cathodic areas where no corrosion is occurring. The arrows represent the current flow **(Chess, 1998)**.

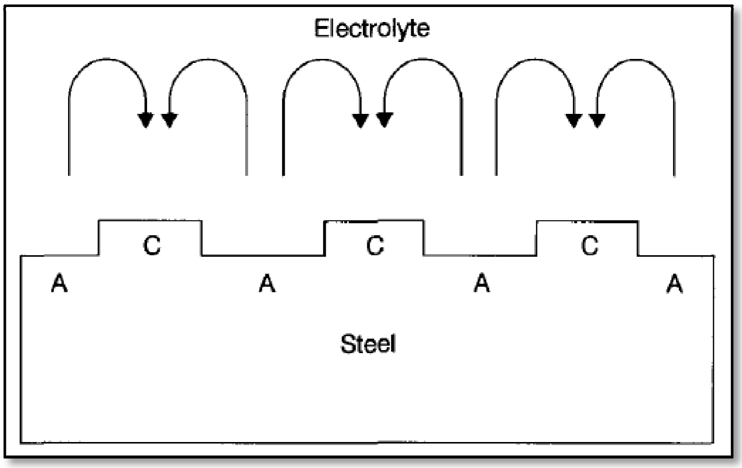


Figure 2.4: Schematic of Micro-Corrosion in Steel Reinforcement (Chess, 1998)

2.4. Black Rust

This type of corrosion occurs when there is a large distance between anode and cathode locations and also if oxygen is not available. This usually occurs in cases of buildings immersed in water or when a protective layer prevents presence of oxygen **(El-Reedy, 2008)**.

In this type of corrosion (known as ‘black’ or ‘green’ rust due to the color of the liquid seen on the rebar when first exposed to air after breakout) the iron as Fe^{2+} will stay in solution. This means that there will be no expansive forces to crack the concrete so corrosion may not be detected. It is potentially as there is no indication of corrosion by cracking and spalling of the concrete and the reinforcing steel may be severely weakened before corrosion is detected **(Broomfield, 1997)**.

2.5. Pit Formation

Corrosion in steel bars starts by forming a small pit, after that, the number of pits will increase with time and then the combination of these pits causes a uniform corrosion on the surface of the steel bars. This is obvious in the case of a steel reinforcement exposed to carbonation or chloride effects. The uniform corrosion and pitting corrosion are illustrated in Figures 2.5-6.

Many chemical reactions describe the formation of pits and, in some cases, these equations are complicated. But the general principle of pit corrosion is very simple, especially in cases of chloride attacks **(El-Reedy, 2008)**.

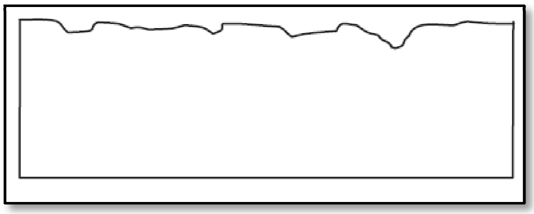


Figure 2.5: Uniform Corrosion (El-Reedy, 2008)

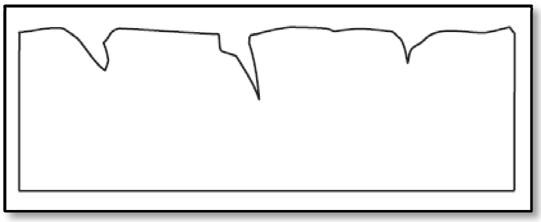


Figure 2.6: Pitting Corrosion (El-Reedy, 2008)

At some suitable site on the steel surface (often thought to be a void in the cement paste or a sulphide inclusion in the steel), the passive layer is more vulnerable to attack and an electrochemical potential difference attracts chloride ions. Corrosion is initiated and acids are formed; hydrogen sulphide from the sulphide (MnS) inclusion and HCl from the chloride ions if they are present. Iron dissolves (Equation 2.1), and the iron in solution reacts with water (Equations 2.5-6):

$$Fe^{2+} + H_2O \longrightarrow FeOH^- + H^+ \qquad \text{Eq. 2.5}$$

$$MnS + 2H^+ \longrightarrow H_2\mathrm{S} + \mathrm{Mn}^{2+} \qquad \text{Eq. 2.6}$$

A pit forms, rust may from over the pit, concentrating the acid (H^+), and excluding oxygen so that the iron stays in solution preventing the formation of a protective oxide layer and accelerating corrosion **(Broomfield, 1997)**.

2.6. Bacterial Corrosion

Bacteria are another cause of corrosion. Because bacteria exist in soil, the foundation is considered the main element exposed to this type of corrosion. These bacteria will convert sulfur and sulfides to sulfuric acid. The acid will attack the steel and then cause initiation of the corrosion process. Other bacteria that attack the sulfide exist in the steel reinforcement FeS due to reactions. This type of corrosion is often associated with a smell of hydrogen sulfide (rotten eggs) and smooth pitting with a black corrosion product when steel bars are exposed to soil saturated with water **(El-Reedy, 2008)**.

2.7. Stray Current Corrosion

Stray current corrosion (such as a nearby pipeline or DC electric railway lines) is a type of localized corrosion caused by the electrical circuits to the steel reinforcement. Corrosion takes place at the anode, the point where the current leaves the metal to return to the power source or to ground. Stray current corrosion is difficult to diagnose since the point of corrosion does not necessarily occur near the current source **(Singley, 1985)**.

2.8. Causes of Corrosion

2.8.1. Introduction

There are two main reasons for corrosion of steel in concrete: chloride attack and carbon dioxide penetration, which is called the carbonation process. There are also other reasons, such as the presence of certain chemicals inside the concrete and voids that affect the steel. Moreover, some acids, such as sulfate, will attack the concrete and cause concrete deterioration and corrosion of steel and then break the concrete alkalinity around the steel bars **(El-Reedy, 2008)**.

However, significant corrosion does not occur for steel in concrete that is either very dry or continuously saturated, because both air and water are necessary for corrosion to be initiated. Steel will remain corrosion-resistant in concrete if the concrete cover prevents air and water from reaching the embedded reinforcement **(Kepler and others, 2000)**.

2.8.2. Chloride Attack

Chlorides can attack concrete from more than one source. The first source is from inside the concrete during the casting process, the second is to move to concrete from outside to inside.

When casting takes place, chlorides exist in concrete as a result of the following **(El-Reedy, 2008)**:

1. Using seawater in the concrete mix.
2. Aggregate that contains chlorides that can be washed well.
3. Using additives that have higher chloride content than that defined in the specification.
4. Water used in the concrete mix that has a higher number of chloride ions than that allowed in the specifications.

Also chlorides can propagate inside concrete from the external environment by:

1. Concrete exposed to seawater spray or continuous exposure to salt water.
2. Using salt to melt ice.
3. Presence of chlorides in chemical substances that attack the concrete structure, such as salt storage.

The effect of chloride salts depends to some extent on the method of addition. If the chloride is present at the time of mixing, the calcium aluminates (C3A) phase of the cement will react with the chloride to some extent, chemically binding it as calcium chloroaluminate. In this form, the chloride is insoluble in the pore fluid and is not available to take part in damaging corrosion reactions. The ability of the cement to complex the chloride is limited, however, and depends on the type of cement. Sulfate resisting cement, for example, has a low C3A content and is therefore less able to complex the chlorides. In any case, experience suggests that if the chloride exceeds about 0.4% by mass of cement, the risk of corrosion increases. This does not automatically mean that concretes with chloride levels higher than this are likely to suffer severe reinforcement corrosion. This depends on the permeability of the concrete and on the depth of carbonation in relation to the cover provided to the steel reinforcement **(Newman and Choo, 2003).**

2.8.3. Carbonation

Carbonation is the result of the interaction of carbon dioxide gas in the atmosphere with the alkaline hydroxides in the concrete. Like many other gases carbon dioxide dissolved in water to form an acid. Unlike most other acids the carbonic acid does not attack the cement paste, but just neutralizes the alkalis in the pore water, mainly forming calcium carbonate that lines the pores as given in Equation 2.7 and 2.8 **(Broomfield, 1997)**:

$CO_2 + H_2O \longrightarrow \mathrm{H_2CO_3}$ (Gas Water Carbonic Acid) Eq. 2.7

$H_2CO_3 + Ca(OH)_2 \longrightarrow Ca\mathrm{CO_3} + 2\mathrm{H_2O}$ (Carbonic Acid Pore Solution) Eq. 2.8

Calcium hydroxyl exists in the concrete and increases the concrete alkalinity that maintains a pH level of 12–13; after carbonates attack inside the concrete and spread, it will form calcium carbonate. As seen in equation 2.7 and 2.8 the value of pH will be reduced to the level that causes the corrosion to the steel reinforcement **(El-Reedy, 2008)**.

Carbonation damage occurs most rapidly when there is small concrete cover over the reinforcing steel. Carbonation can occur even when the concrete cover depth to the reinforcing steel is high. This may be due to a very open pore structure where pores are well connected together and allow rapid CO2 ingress. It may also happen when alkaline reserves in content, high water cement ratio and poor curing of the concrete **(Broomfield, 1997)**.

2.9. Corrosion Rate

The corrosion rate is considered the most important factor in the corrosion process from a structural-safety perspective and in the preparation of the maintenance program for the structure. This factor is considered an economic factor of structural life, when the corrosion rate is very high, the probability of structure failure will increase rapidly and structural safety will be reduced rapidly **(El-Reedy, 2008)**.

During the last decades, many physical and mathematical models have been introduced to estimate the time of corrosion initiation and propagation. The first numerical model of this kind was developed by Collepardi, in which he employed the Fick's second law of diffusion and indicated that diffusion coefficient is one of the most important parameters in service life prediction. Tutti's model was one of the first attempts to predict the service life of RC structures. As shown in Figure 2.7, the concept of this model is to divide the service time of the structure into T_0 (t_{init}), as the time to corrosion initiation, and T_i (t_{prop}), as the time of corrosion propagation until failure occurs **(Alizadeh and others, 2006)**.

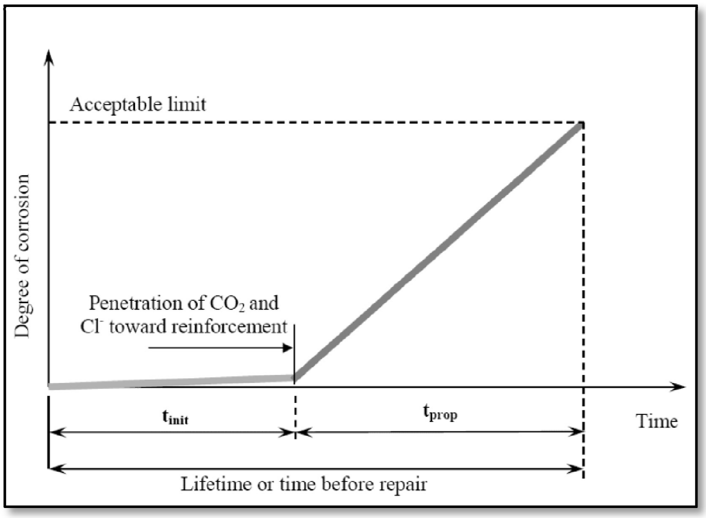
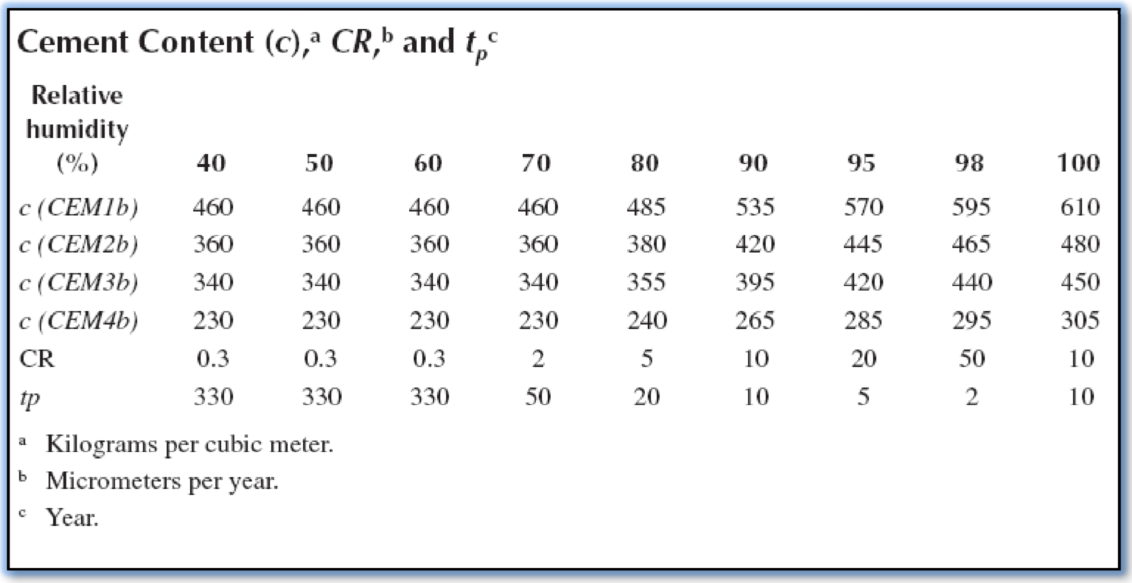


Figure 2.7: Tutti's Model for Corrosion Process of Steel in Concrete (Alizadeh and others, 2006)

On the other hand, typical corrosion rates of steel in various environmental situations have been reported in recent years. According to Ting (1989), the average corrosion rate, C_r, for passive steel in concrete attacked by chlorides is about 100µm/year **(Ting, 1989)**. According to Mori and Ellingwood, (1994), the typical corrosion rate, C_r, is a time-invariant random variable described by a lognormal distribution with mean C_r of 50 µm/year, and coefficient of variation V_{cr} of 50% **(Mori and Ellingwood, 1994)**.

Because the corrosion rate changes with environment, no accurate data are available to predict the real corrosion rate. However, there are many empirical formulas to calculate the corrosion rate, these formulas changes according to the corrosion reasons, for example in the case of carbonation attack. Table 2.1 gives values of the corrosion rate according to the relative humidity **(El-Reedy, 2008)**.

Table2.1: Corrosion Rate According to Relative Humidity (El-Reedy, 2008)



Cement Content (*c*),[a] *CR*,[b] and t_p[c]

Relative humidity (%)	40	50	60	70	80	90	95	98	100
c (CEM1b)	460	460	460	460	485	535	570	595	610
c (CEM2b)	360	360	360	360	380	420	445	465	480
c (CEM3b)	340	340	340	340	355	395	420	440	450
c (CEM4b)	230	230	230	230	240	265	285	295	305
CR	0.3	0.3	0.3	2	5	10	20	50	10
tp	330	330	330	50	20	10	5	2	10

[a] Kilograms per cubic meter.
[b] Micrometers per year.
[c] Year.

The corrosion rate depends on different factors, so if it possible to control these factors, the corrosion rate will be low. The main factor that affects the corrosion rate is the presence of water and oxygen, especially in the cathode zone shown in the in Figure 2.8 **(El-Reedy, 2008)**.

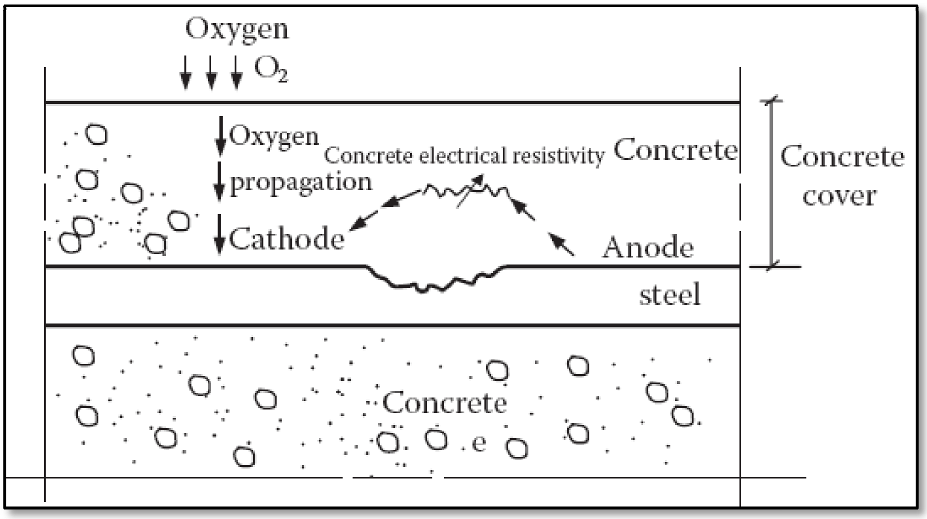


Figure 2.8: Factors Affecting Corrosion Rate (El-Reedy, 2008)

The second important factor affecting the corrosion rate is the moving of the ions inside the concrete voids around the steel reinforcement. If the speed of moving ions is very slight or prevented, the corrosion rate will be very slow also or, in the ideal case, prevented. This case may happen when the concrete around the steel bars has a high resistance to electrical conductivity between anode and cathode. The measurement of electrical resistivity to the concrete surrounding the steel reinforcement can give us an assumption of the corrosion rate and the chemical reaction rate **(El-Reedy, 2008)**.

2.10. Corrosion Structural Effects

Among the different deterioration mechanisms occurring in concrete structures, the corrosion of reinforcement is the most detrimental one. Their consequences, whether due to concrete carbonation or to an excessive chloride content in the concrete, can be classified into three main groups as seen in Figure 2.9 **(Rodriguez & Ortega, 2006)**:

1. Those, which affect the reinforcement section, reducing the effective area and ductility.
2. Those, which are related to concrete integrity.
3. Those, which affect the interaction concrete – reinforcement due to the bond reduction.

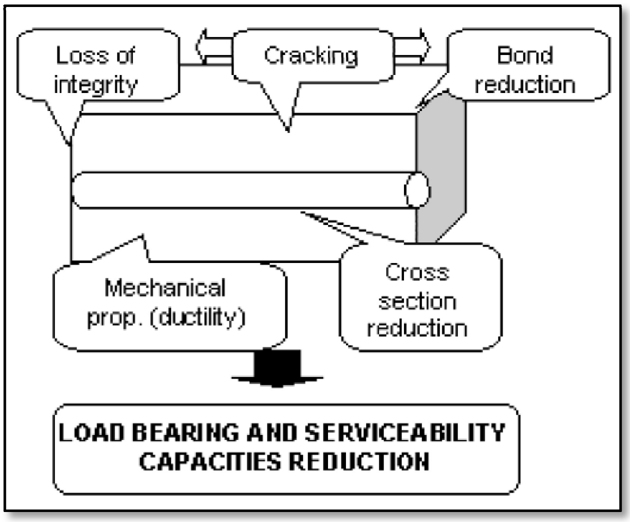


Figure 2.9: Reinforcement Corrosion Effects on Concrete Structures (Rodriguez & Ortega, 2006)

However, the most problems that occur because of corrosion of steel in concrete are due not only to the shortage in the steel section but also to fall of the concrete cover. Many studies and much research have been conducted to calculate the amount of corrosion occurring and causing the concrete cover to fall. It has been found that cracks may occur in cases of reduction of 0.1 mm from steel reinforcement sections and, in some cases, much less than 0.1 mm, depending on the distribution of oxides and the ability of concrete to withstand the stresses, as well as the distribution of steel **(El-Reedy, 2008)**.

The reason of concrete cover failing refer to the fact that the rust occupies a much larger volume than the original steel and causes the buildup of bursting forces at the surface of the reinforcement. Because concrete is weak in tension these bursting forces quickly cause the concrete to crack parallel to the reinforcement direction and eventually, to spall away from rebars. However, corrosion is a complex mixture of oxides, and hydroxides and hydrated oxides of steel have a volume ranging from twice to about six times that of the steel consumed to produce it. The magnitude of the rust incremental are various according to various steel oxides generated as shown in Figure 2.10 **(AL-Ostaz, 2004)**.

Note that the concrete cover in the corner is more prone to falling because it is a largely exposed area for the penetration of carbon dioxide or exposure to chlorides, as well as oxygen. Therefore, concrete cracks often happen faster in this situation **(El-Reedy, 2008)**.

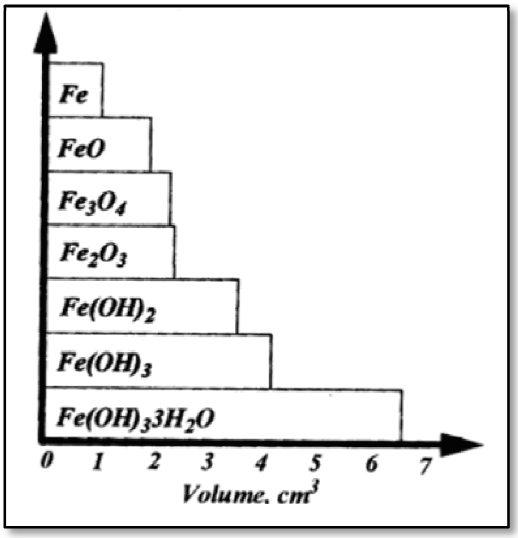


Figure 2.10: The Volume Change for Various Steel Oxides in Comparison With Original Volume of Fe Equal to 1 Cm3 (AL-Ostaz, 2004)

However, in the last decades many empirical formulas were integrated to calculate the following parameter:

1. The losses in the steel reinforcement due to corrosion.
2. The crack width of the concrete cover due to corrosion.
3. The pond between the concrete and steel reinforcement in the present of corrosion.

These parameters are mainly affected with the corrosion rate, corrosion cause (carbonation and or chlorides attack) and environmental situation surrounding the concrete **(Rodriguez and Ortega, 2006)**.

2.10.1. Losses in Steel Reinforcement Due to Corrosion

Weight-loss of the reinforcing steel could be calculated based on the current going through rebar. According to Faraday’s Law, the total weight loss of a reinforcing steel bar that is oxidized by the passage of electric charge can be expressed in Equation 2.9 as follows:

$$W_{loss} = [TC] \times \frac{EW}{F} \qquad \text{Eq. 2.9}$$

Where, W loss is the total weight loss of reinforcing steel (grams), TC is the total electric charge (amp-sec or coulombs); EW is the equivalent weight, indicating the mass of metal (grams) that is oxidized. For pure elements, the EW is given by EW=W/n; here W is the atomic weight of the element, and n is the valence of the element. For carbon steel, the EW is approximate 28 (grams). F is Faraday’s constant in electric charge (F=96490 coulombs, or amp-sec) **(Wang and others, 2000)**.

Equation 2.9 presents the losses of steel bars in electrolyte, in the case of reinforcement embedded in concrete the reduction in steel cross section can be expressed in equations 2.10 and 2.11 as follows **(Feliu and others, 1990)**:

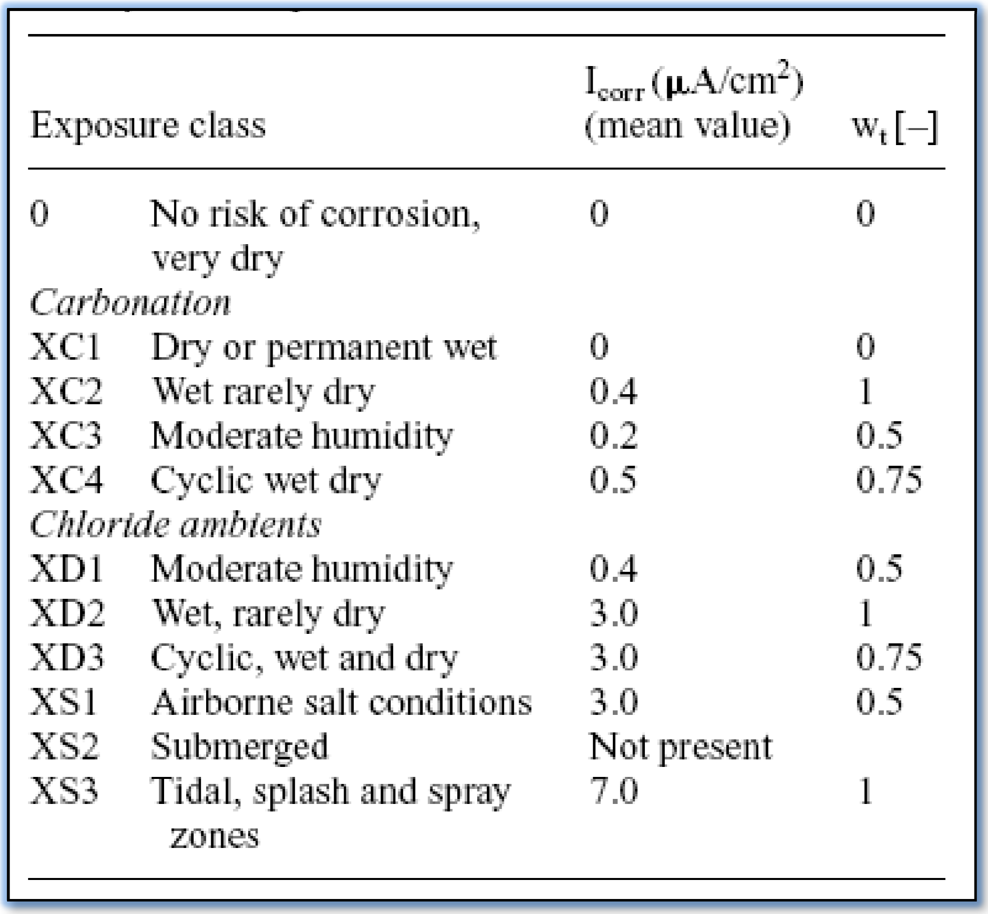
$$\phi = \phi_0 - \alpha P_x \qquad \text{Eq. 2.10}$$

Where ϕ is the reduction in cross section are of steel bar (cm^2), ϕ_0 is the cross section area of steel reinforcement before corrosion (cm^2), α is the “pitting factor”, the α values are different if the corrosion is homogeneous (α = 2) than for pitting corrosion (5< α <10), P_x is the corrosion rate (mm/year) which can be expressed in equation 2.11 as follow:

$$P_x = I_{corr} \times t \qquad \text{Eq. 2.11}$$

The determination of I_{corr} depends on the environment evolution. Thus, several strategies may be used for the determination of I_{corr} and the loss of section with time P_x, anyhow Table 2.2 gives values of I_{corr} by means of the wetness time **(Rodriguez and Ortega, 2006)**.

Table 2.2: Averaged Corrosion Currents (Icorr) and Wetness Periods (wt) (Rodriguez & Ortega, 2006)



Exposure class		I_{corr} ($\mu A/cm^2$) (mean value)	w_t [–]
0	No risk of corrosion, very dry	0	0
Carbonation			
XC1	Dry or permanent wet	0	0
XC2	Wet rarely dry	0.4	1
XC3	Moderate humidity	0.2	0.5
XC4	Cyclic wet dry	0.5	0.75
Chloride ambients			
XD1	Moderate humidity	0.4	0.5
XD2	Wet, rarely dry	3.0	1
XD3	Cyclic, wet and dry	3.0	0.75
XS1	Airborne salt conditions	3.0	0.5
XS2	Submerged	Not present	
XS3	Tidal, splash and spray zones	7.0	1

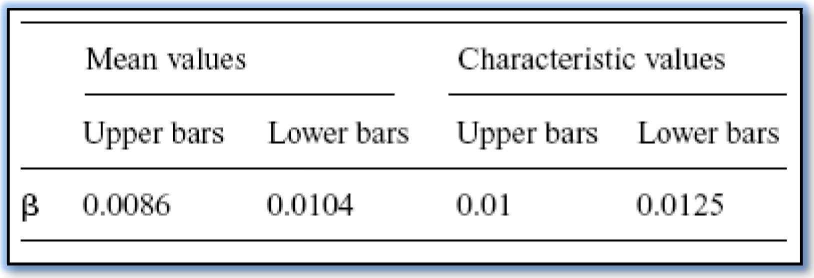
2.10.2. Crack Width of Concrete Cover Due To Corrosion

The oxides generated in the corrosion process provide a tensional state in the concrete cover that will produce final cracks, reducing consequently the cross section of the concrete element and therefore their load bearing capacity. Several empirical expressions have been developed, that can evaluate the crack width of the cover. The following function, as a direct function of the corrosion attack and several geometric and mechanical parameters can express the crack width of concrete cover due to corrosion as given in Equation 2.12 **(Alonso and others, 1998).**

$$w = 0.05 + \beta[P_x - P_{x0}] \qquad \text{Eq. 2.12}$$

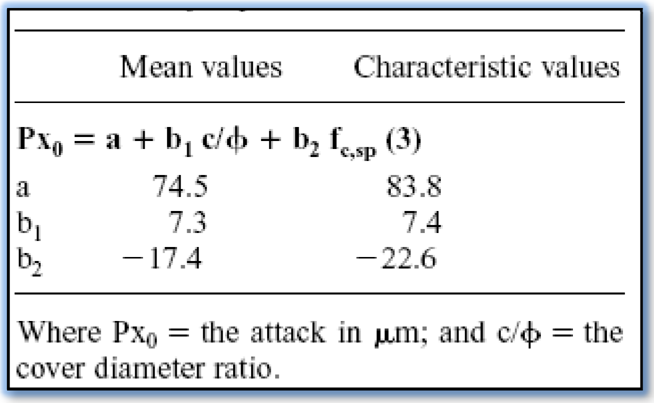
Where w is the crack width in mm ($w \leq 1.0mm$), β is a factor depending on the bar position (can be estimated from Table 2.3), P_x is the attack penetration in microns and P_{x0} is an attack corresponding to the crack initiation (can be estimated from Table 2.4); and.

Table 2.3: β Values for Crack Width Calculations (Alonso and others, 1998)



	Mean values		Characteristic values	
	Upper bars	Lower bars	Upper bars	Lower bars
β	0.0086	0.0104	0.01	0.0125

Table 2.4: P_{x0} Expressions for Crack Initiation (Alonso and others, 1998)



	Mean values	Characteristic values
$Px_0 = a + b_1 c/\phi + b_2 f_{c,sp}$ (3)		
a	74.5	83.8
b_1	7.3	7.4
b_2	−17.4	−22.6

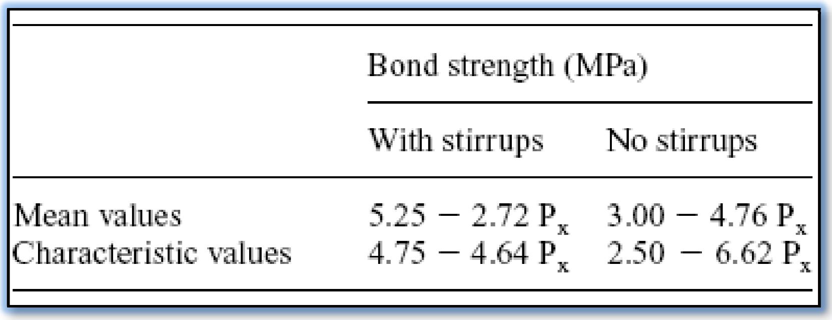
Where Px_0 = the attack in μm; and c/ϕ = the cover diameter ratio.

2.10.3. Losses of Bond between the Concrete and Steel Reinforcement Due To Corrosion

The concrete–steel bond is the responsible of the bar anchorage in the reinforced concrete member ends and this is responsible for the composite behavior of members. However, corrosion provokes a reduction in bond due to the cover cracking and stirrups corrosion. To calculate the concrete–steel bond, the following expression may be used **(Rodriguez and others, 2006)**:

1. For residual bond assessment. Table 2.5 shows empirical expressions that allow obtaining realistic residual bond values. All of them are expressed depending on the attack penetration P_x.

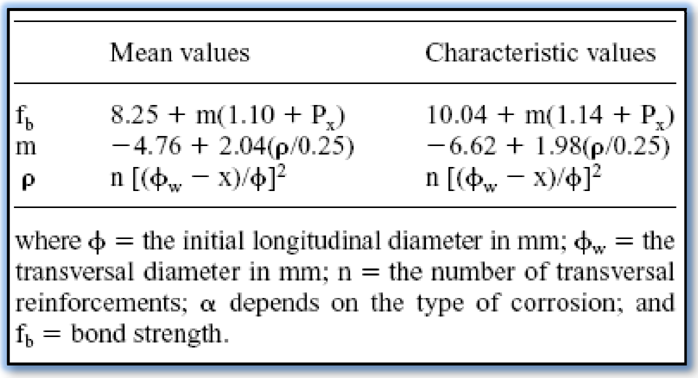
Table 2.5: Relationship between Bond and Px in mm (Rodriguez and others, 2006)



	Bond strength (MPa)	
	With stirrups	No stirrups
Mean values	$5.25 - 2.72 P_x$	$3.00 - 4.76 P_x$
Characteristic values	$4.75 - 4.64 P_x$	$2.50 - 6.62 P_x$

2. For intermediate cases where the amount of stirrups is low, below the actual minimum, or the stirrups capacity can be strongly reduced by corrosion effects, expressions of Table 2.6 may be applied.

Table 2.6: Bond Values for Cases of Intermediate Amount of Stirrups (Rodriguez and others, 2006)



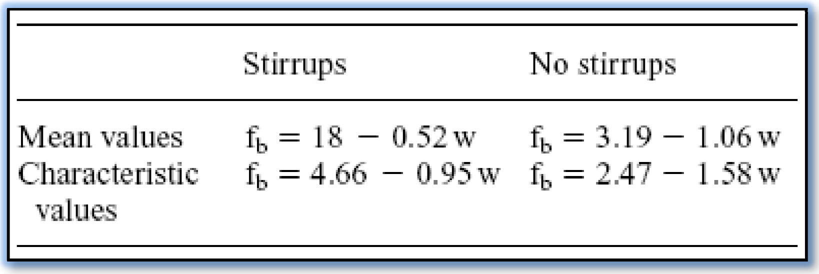
	Mean values	Characteristic values
f_b	$8.25 + m(1.10 + P_x)$	$10.04 + m(1.14 + P_x)$
m	$-4.76 + 2.04(\rho/0.25)$	$-6.62 + 1.98(\rho/0.25)$
ρ	$n\,[(\phi_w - x)/\phi]^2$	$n\,[(\phi_w - x)/\phi]^2$

where ϕ = the initial longitudinal diameter in mm; ϕ_w = the transversal diameter in mm; n = the number of transversal reinforcements; α depends on the type of corrosion; and f_b = bond strength.

These expressions are of application with P_x values between 0, 05 and 1mm with $\rho \leq 0.25$.

3. For relationship between bond and crack width, several expressions have been developed for relating the residual bond with the crack width expressions of Table 2.7 may be applied.

Table 2.7: Relationship between Bond fb (MPa) and crack width w (mm) (Rodriguez and others, 2006)



	Stirrups	No stirrups
Mean values	$f_b = 18 - 0.52\,w$	$f_b = 3.19 - 1.06\,w$
Characteristic values	$f_b = 4.66 - 0.95\,w$	$f_b = 2.47 - 1.58\,w$

2.11. Corrosion of Reinforced Concrete Structures in Gaza Strip

Reinforced concrete buildings in Gaza Strip had appeared since 1950 or earlier and have been used widely since 1970. Although few studies considered the causes of building damages in Gaza Strip, it is clear from common practice that the corrosion problem of reinforcing steel is considerable. This may refer to the fact that Gaza Strip is a costal area and have shore climate.

A recent survey study shows that the existing of buildings damages due to exposure conditions such as temperature, relative humidity, and concentration of salts in the atmosphere where the reasons of 49% for assessment requests in Gaza Strip. Furthermore, these conditions played an important role in deterioration of concrete and hence corrosion of reinforcing steel that constituted 31% of the damages in the surveyed cases **(Abu Hamam, 2008)**.

Figure 2.11 shows the reinforcing steel corrosion problem of many buildings in Gaza Strip.

Cracks along Steel Bars. **Local Spalling of Concrete Cover.**

Complete Spalling of Concrete Cover. **Corrosion of Exposed Steel Bars.**

Figure 2.11: Steel Reinforcement Corrosion in Different Defects Buildings in Gaza Strip (Abu Hamam, 2008)

Once the assessment of a damaged structure has been completed, decision of repair should be taken. In fact there are few local institutions and consulting firms in Gaza Strip having practical experience in assessment and evaluation of damages of existing structures. These include the Association of Engineering, the Ministry of Public Works and Housing, the Islamic University of Gaza, the United Nations Relief and Works Agency (UNRWA) in addition to some engineering consulting firms.

There are no unified standards or technical specifications used in repairing of reinforced steel corrosion used in Gaza Strip. Anyhow many rehabilitation projects deal with the reinforcing steel corrosion problem have been executed in Gaza strip. Rehabilitation of water tanks executed by the Palestinian Water Authority in cooperation of a local consultant office and Repairing of 104 Shelters Project at Rafah Area in Gaza Strip executed by UNRWA are some examples of these projects.

The main repairing steps used in these projects were as follows:

1. Removing contaminated cracked or defective concrete.
2. Cleaning of reinforcement and adding more reinforcing steel bars if needed.
3. Adding protection to the reinforcement.
4. Adding bonding agents to concrete and apply anew concrete layer by using normal concrete mortar or special repairing materials.

However these repairing methods depend on the engineering experiences and no structural performance testes were applied for these repairing methods to check the safety of the repaired structures.

2.12. Concluded Remarks

1. Corrosion of reinforcing steel imbedded in concrete is an electrochemical process and mainly caused by chloride attack and or carbon dioxide penetration.

2. There are many types of corrosion such as black rust, pit formation bacterial and stray current corrosion. These types consist according to the condition of corrosion.

3. Corrosion of steel reinforcement has a direct affect on the structural performance of the structural elements, that's because of many parameters such as:

   a) Losses in the steel reinforcement cross section or ductility.
   b) Losses of the concrete integrity.
   c) Losses in the pond between the concrete and steel reinforcement.

   However, many empirical formulas were integrated to calculate these illustrated parameters.

4. Gaza Strip is a costal area and suffering from the corrosion problem. A survey on 40 defects buildings in Gaza Strip shows that corrosion of reinforcing steel was the first reasons of damages which constituted 31% of the total reasons of buildings damages.

Therefore, it is important to investigate the corrosion problem and repairing techniques that might be used in repairing corrosion damaged structures in Gaza Strip. An experimental investigation would assist engineers in dealing with this problem and increase the level of confidence in used repairing techniques. One challenge of such investigation is to be able to induce corrosion onto reinforced concrete members.

Chapter 3: Repair of Corroded Reinforced Concrete Structures

3.1. Introduction:

Before approaching corroded reinforced concrete structures repairs, consideration must first be given to the cause of the problem. This is fundamental to the success or failure of the repair, and lack of adequate attention at this point can jeopardize the whole job.

To find the causes of deterioration a structural element, assessment methods and different measurement methods will be needed. These will help in deciding on the cause of corrosion and present and future deterioration of the structure **(Newman and Choo, 2003)**.

However when the cause of the problem becomes clear, one of the major issues facing any consultant or owner of a structure suffering from chloride or carbonation induced corrosion is what type of repair to undertake. In fact there are many repair techniques such as: coating and sealants, specialized patch repair materials, options for total or partial replacement, cathodic protection, chloride removal, realkalization and corrosion inhibitors. These can be applied to structures suffering different degrees of corrosion due to chloride attack, carbonation or a combination of the two.

The practical solution for most owners of individual corroding structures will be to take advice from structural engineering consultant or a corrosion specialist. Advice may also be sought from materials suppliers and applicators about their own particular system and consensus will be reached about the most effective repair to the structure based on local knowledge, experience and availability of materials and systems **(Broomfield, 1997)**.

To determine the way of repairs, many alternatives are available to the maintenance engineer in attempting to combat deterioration and keep the structure in its original condition. These may be grouped into three broad categories **(Vorster and Others, 1992)**:

- Protection.
- Patching.
- Rehabilitation.

All these alternatives are maintenance operations. While the work involved is not necessarily minor, it is limited by the fact that it does not involve any change to the original functional characteristics of the structure.

3.1.1. Protection

Protection is defined as the processes which use physical barriers such as waterproof overlays or membranes halt or slow the process of deterioration. Electrochemical methods can also be used to remove chlorides or upset the required chemical actions. These alternatives are viable if deterioration has not progressed beyond certain limits.

3.1.2. Patching

Patching is defined as the process whereby small localized areas of deterioration in the structure are repaired by removing the deteriorated concrete, cleaning any corroded reinforcing steel, and replacing the concrete with a substitute material. The objective is to reinstate the original characteristics of the structure as far as possible. There is no expectation of increased life and patching does not enhance the original functional characteristics of the structure.

3.1.3. Rehabilitation

Rehabilitation is more extensive than patching. Large areas of chloride and or carbonation contaminated or deteriorated concrete are systematically removed, the corroded steel is cleaned, and the area is repaired with materials similar to those originally used. The objective is to repair the area to a standard at least equal to that found in parts of the structure that are not contaminated.

The alternatives for reversing deterioration set out in Figure 3.1 **(Vorster and Others, 1992)** and the definitions of protection, patching and rehabilitation presented above may give the impression that any project may entail the selection of a single alternative. However, as most projects employ a mix of alternatives to achieve the desired result. This is particularly true insofar as renovation and rehabilitation are concerned. Most projects involve an element of both:

Renovation to bring the structure as close to current standards as possible and rehabilitation to systematically remove and repair areas where concrete and steel have been damaged by the ingress of chlorides and or carbonation.

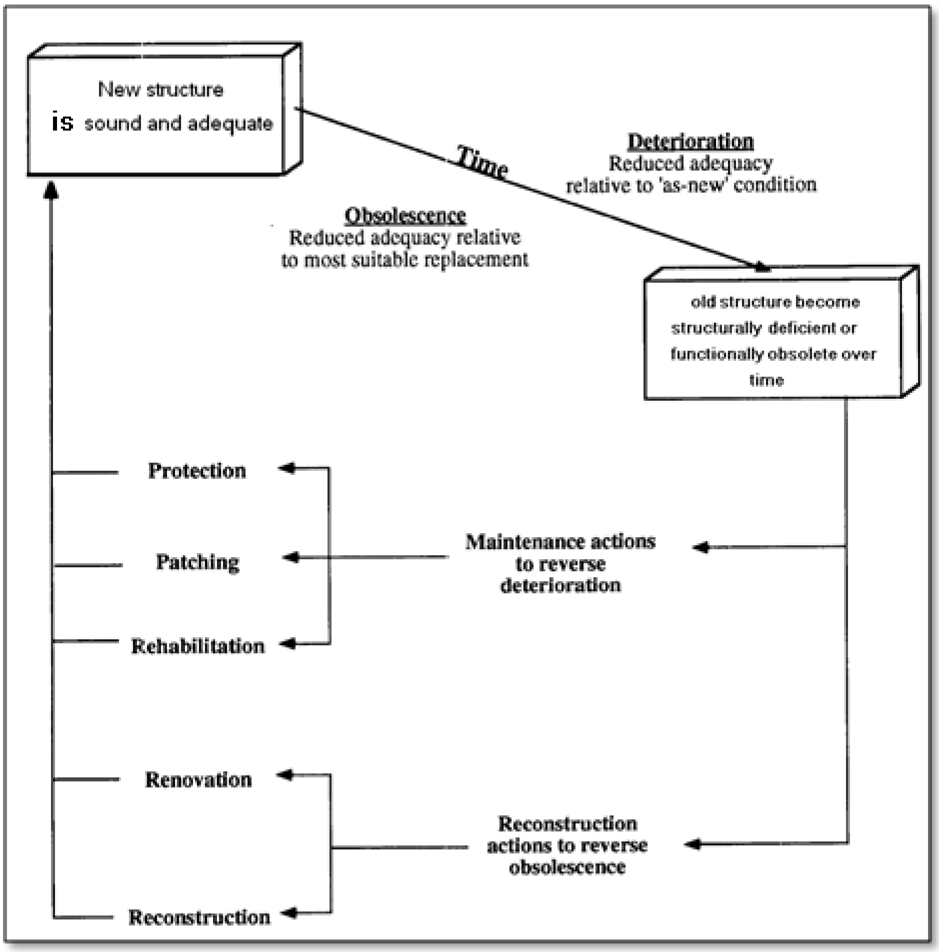


Figure 3.1: Alternatives for Reversing Deterioration and Obsolescence (Vorster and Others, 1992)

3.2. Main Steps to Executing Repair

There are several regular steps in the repair of all structures exposed to corrosion. The very critical first step is to strengthen the structure by performing structural analysis and designing a suitable location for the temporary support. The second step is to remove the cracked and delaminated concrete. It is important to clean the concrete surface and also the steel bars by removing rust. After rust is removed by brush or sand blasting, the steel bars should be painted with epoxy coating or replaced; then new concrete can be poured. The final step is to paint the concrete member as external protection. This is a brief description of the repair process. These steps will be explained in detail in the following sections **(El-Reedy, 2008)**.

3.2.1. Temporary Structure Strengthening

One of the most dangerous and important first steps necessary for the repair is selecting the temporary support, which depends on the following **(El-Reedy, 2008)**:

1. Evaluating the state of the whole structure.
2. Determining how to transfer loads in the building and its distribution.
3. Determining the volume of repair that will be done.
4. Determining the type of concrete member that will be repaired.

As mentioned earlier, the repair process must be carried out by a structural engineer with a high degree of experience who has the capability to perform the structure analysis and powerful knowledge of the load distribution in the structure, according to the kind of repair. The structural engineer has a responsibility to choose the right way to optimize the process of crushing and to determine the ability of the structure members to carry the loads that will be transferred to them. Therefore, the responsible engineer should design the temporary support based on data collected and the previous analysis and should be cautious in the phase of execution of temporary supports.

Choosing how to remove the defective parts will be based on the nature of the concrete member in the building as a whole; any member of breaking concrete has a detrimental impact on neighboring members because the process of breaking will produce a high level of vibration. The temporary members must be strong and designed to withstand loads and must be transported easily and safely to the defective area. The entire structure depends on the design and execution of the temporary supports and their ability to bear loads safely **(El-Reedy, 2008)**.

3.2.2. Removing Cracked Concrete

There are several ways to remove the part of the concrete that has cracks on its surface and shows the effects of steel corrosion. The choice of the method depends on the specification, budget and contractor’s preferences. If concrete is just starting to spall due to carbonation or if an electrochemical treatment like cathodic protection is planned, then a simple repair of removing unsound concrete, cleaning the rebar surface, squaring the edges and putting in a sound may be used.

If patching is required due to chloride corrosion then the most specifications recommend removing about 25 mm behind the steel and making sure that the concrete on the steel has no traces of chlorides after the repair process. The difference between good and bad repair procedures is obvious in Figure 3.2 **(Broomfield, 1997)**.

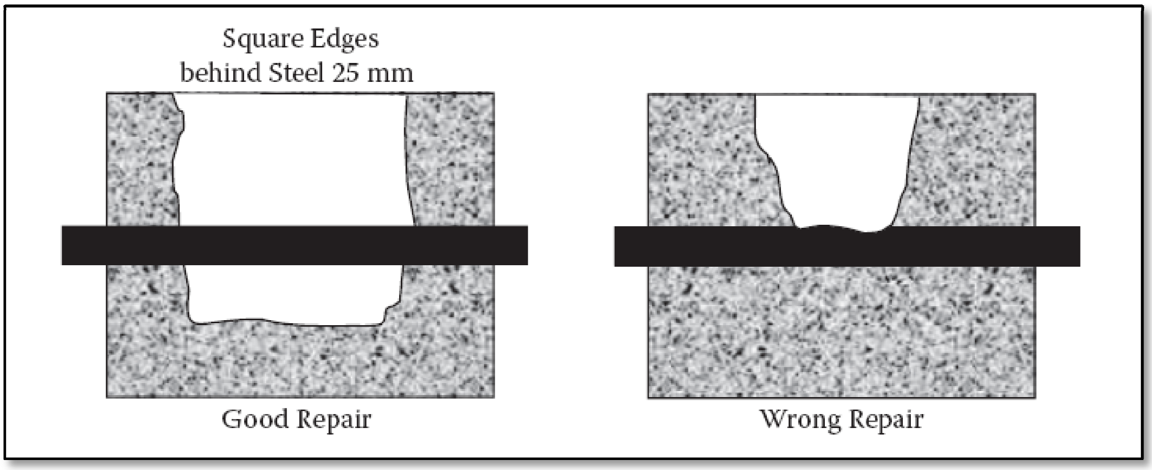


Figure 3.2 Differences between Good and Bad Repairs (Broomfield, 1997)

However, the method of removing of concrete cracks depends mainly on four factors as follows **(Vorster and Others, 1992)**:

1. Removal depth as seen in Figure 3.3.
2. Removal area.
3. Repairing methods.
4. Repair material specification.

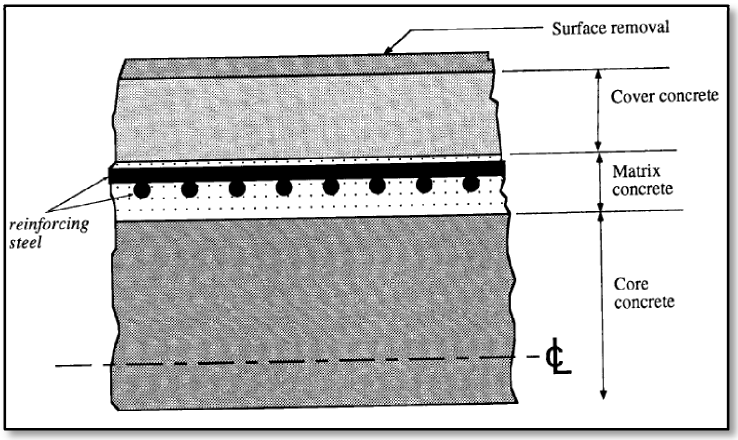


Figure 3.3: Depth Classification for Concrete Removal (Vorster and Others, 1992)

Depending on these factors, several methods are commonly used for breaking and removing the defective concrete and these are explained next.

3.2.2.1. *Manual Method*

One of the simplest and easiest methods is to use a hammer and chisel to remove defective concrete. This is considered one of the most inexpensive ways, but it is too slow compared with mechanical methods. However, mechanical methods produce high noise and vibration, have special requirements, and need trained labor. Using the manual method makes it difficult to spare concrete behind the steel. The method is used in the case of small spaces and is the preferred use in the event of corrosion due to carbonation and attacking chlorides from outside, when it is not necessary to break concrete behind steel. Any worker can manually break the concrete, but it is necessary to choose workers who have done repair work before as they must be sensitive in breaking the concrete to avoid causing cracks to the adjacent concrete members **(RILEM Committee, 1994)**.

3.2.2.2. *Pneumatic Hammer Methods*

The pneumatic breaker (frequently known as a jack-hammer but properly known as a paving breaker) is currently the most prevalent method for concrete removal in rehabilitation work. The breaker is hand held and powered by compressed air to deliver a series of high frequency blows which fracture the concrete in a small, easily controlled area. The production of pneumatic breakers depends on two factors; the size of the breaker and the skill of the operator.

Breakers are sized according to their weight. This can vary from 8 to over 120 pounds (4 to 55 kg). Heavier breakers are more productive because they are able to impart more energy with each blow. They are also more destructive. The International Union of Laboratories and Experts in Construction Materials (RILEM Committee) is typically limits the weight of breakers that can be used for selective concrete removal to less than 45 pounds (20 kg) to minimize residual cracking and preserve the bond between the residual concrete and the repair concrete in areas that are not removed. The angle of attack measured from the breaker's axis to the concrete surface is also frequently limited to 45 degrees for the same reasons. The skill of the operator is important with regard to both the quantity and quality of the work performed. This factor must not be overlooked when assessing the viability of the method. Some performance rates are about 0.025–0.25 m^3 per hour using hammers weighing 10–45 kg, respectively **(Vorster and Others, 1992)**.

The use of pneumatic hammers is more economical when a small, rather than large, area is to be removed (see Figure 3.4). A water gun is preferable for large areas as described in the following section **(El-Reedy, 2008)**.

Figure 3.4: Using a Pneumatic Hammer to Remove Wall Concrete Cover (El-Reedy, 2008)

3.2.2.3. *Water Jet*

This method has been commonly used around the world since it was introduced to the market in the 1970s **(Broomfield, 1997)**. It relies on the existence of water at the work site and on the removal of a suitable depth of concrete in a large area. It removes fragmented concrete, cleans steel bars, and removes part of the concrete behind the steel bars, as shown in Figures 3.5 and 3.6. The water jet is used manually by an experienced worker who has previously dealt with the hose, which is pushing water under high pressure or perhaps through the mechanical arm. Very high safety precautions need to be applied to the worker who uses it and the site around it.

The water used must not have any materials that affect the concrete, such as high chloride ions; in general, it must be potable water. The water gun consists of diesel engines and related through pressure pump and connected by hose that bears high water pressure from 30–70 MPa at the nozzle and at least 40 MPa. At least 40 MPa is required to cut the concrete and the rate of water consumption is about 50 L/minute. The performance rate of a water jet to break concrete is about 0.25 m^3/hour in the case of the use of a small pump and can reach up to about 1 m^3/hour if two pumps or one big pump is used **(Broomfield, 1997)**.

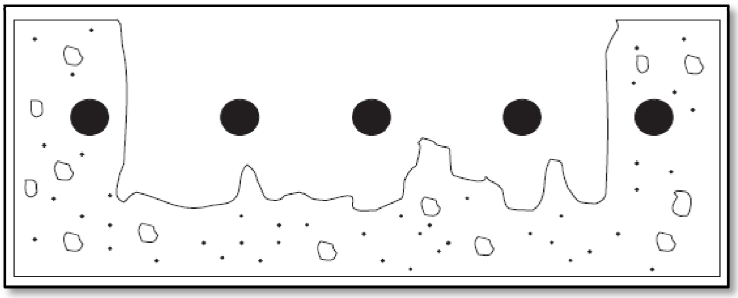


Figure 3.5: Shape of Delaminated Concrete When a Water Jet Is Used (Broomfield, 1997)

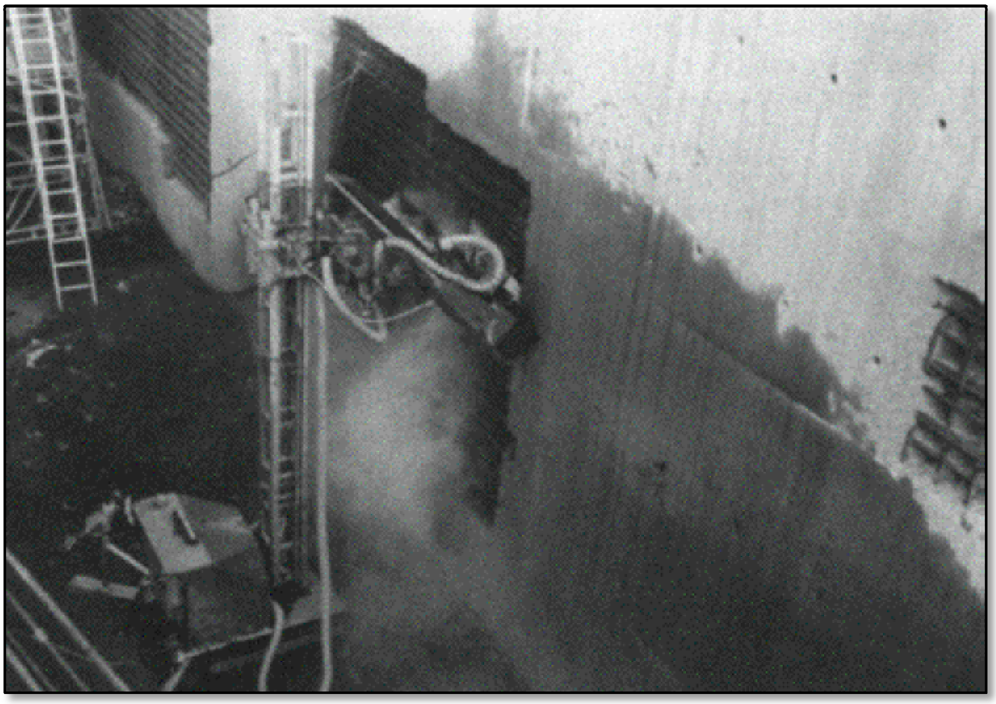


Figure 3.6: Hydro Jetting a Wall with a Robot Arm Controlled System (Broomfield, 1997)

3.2.2.4. Grinding Machine

This is used to remove concrete cover in the case of large, flat surfaces. An example is a bridge deck, as in Figure 3.7, but it must be done cautiously so that the process of breaking does not reach the steel. As in the case of any contact between the grinding machine and steel reinforcement, it will cut the steel bars and damage the machine. This method of breaking delaminated concrete is seldom used in the United Kingdom, because insulation film is often used to prevent the water, and it is used sparingly on U.S. bridges. The grinding machine is usually used after the water gun or the pneumatic hammer to obtain final concrete breakdown around and under the steel reinforcement. Therefore, one must take into account whether the thickness of the concrete cover is equal. The rate of removal of the concrete by this machine is very fast; it removes about 1 m^3/minute and its cutting part is 2 m in width **(El-Reedy, 2008)**.

3.2.3. Cleaning Concrete Surfaces and Steel Reinforcements

This phase removes any remaining broken concrete with a process of cleaning. At the same time, the process of assessing the steel and cleaning up and removing corrosion from the roof takes place.

Figure 3.7: Grinding Machine (El-Reedy, 2008)

3.2.3.1. Cleaning Concrete Surfaces

The stage of preparing a surface by pouring the new concrete is one of the most important stages of the repair process. Before application of the primer coating, which provides the bond between the existing old concrete and the new concrete for repair, It is frequently necessary to remove surface contaminants such as oil, rubber and rust from the work area in order to provide a sound, long-lasting bond between the existing structure and the new materials used to repair or rehabilitate the structure **(Vorster and Others, 1992)**.

When cement mortar or concrete is used in the repair, the concrete surface will be sprayed by water until saturation can be reached. The stage of saturation can be reached by spraying water on the surface for 24 hours or through wetting burlap. The water spray must be stopped and the burlap removed for about 1–2 hours, depending on weather conditions, until the surface is dry. It is then coated by a mix of water and cement only, which is called slurry and is applied by brush. Epoxy coating can also be used as an adhesive between new and old concrete; it is necessary to follow the manufacturer’s specifications and warnings **(El-Reedy, 2008)**.

3.2.3.2. Clean Steel Reinforcement Bars

Bar cleaning necessitates the removal of rust, chlorides and other unwanted material from the exposed reinforcing steel. The work follows the removal of matrix concrete and is extremely important as all the chloride contaminated rust and cement paste must be removed to stop corrosion from continuing in the backfilled concrete. Three methods are frequently used. These are **(Vorster and Others, 1992)**:

1. Sandblasting: The abrasive impact of sand particles under compressed air is an effective method for cleaning exposed steel because it is able to remove cement deposits and rust to leave a bare metal finish.
2. Wire Brushing: Powered rotary wire bristle brushes can be used to clean exposed rebar. Brushes are pneumatically or hydraulically driven and usually mounted on a small utility construction vehicle. Access to hidden and difficult-to-reach surfaces is restricted.
3. Hydro demolition: Sand arising from the hydro demolition process is propelled by the high pressure water to create an abrasive fluid able to clean rebar as an adjunct or parallel operation to the removal of matrix or core concrete by hydro demolition.

After cleaning the steel surface, the next step is to evaluate the steel reinforcement by measuring steel diameter. If the cross-sectional area of the steel bars is found to have a reduction equal to or more than 20%, additional reinforcing steel bars must be added. Before pouring new concrete, one must be sure that the development length between the new bars and the old steel bars is enough, as shown in Figure 3.8. It is usually preferable to link the steel by drilling new holes in the concrete and connecting the additional steel on concrete by putting the steel bars in the drilled hole filled with epoxy. However, in most cases the steel bars are completely corroded and need to be replaced **(RILEM Committee, 1994)**.

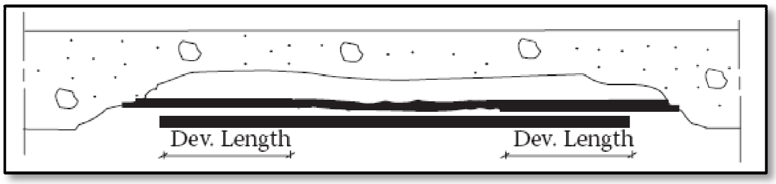


Figure 3.8: Installing Additional Steel (El-Reedy, 2008)

In the case of beams and slabs that need to add additional steel reinforcement bars, it is preferable to connect the steel bars with concrete by drilling new holes in the concrete and making the bond of the steel bars in the holes by using adhesive epoxies. For beam repairs, the additional steel bars are fixed in a column that supports this beam. In the case of slabs, the steel bars are fixed in the sides of the beam that is supporting the slab, as shown in Figure 3.9. The dowelling will be fixed in the beam side by drilling holes to a depth of around 70–80 mm; the dowel will be fixed in the holes by epoxy according to the supplier's assumptions and recommendations **(U.S. Army Corps of Engineers, 1995)**.

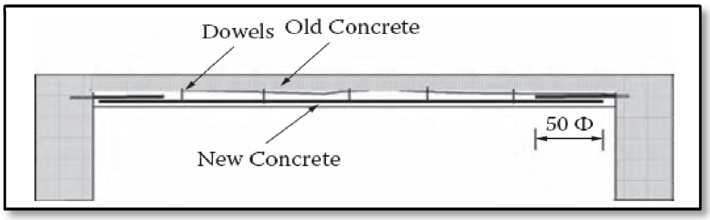


Figure 3.9: Steel Reinforcement Installation for Slab Repair (El-Reedy, 2008)

3.2.4. New Patch Concrete

After removing the damaged and contaminated concrete, it must be patched. Many proprietary patch materials around the world are existing. The pre-bagged materials are most likely to be applied, especially to small repair areas, but they are more expensive than conventional cement, aggregate and water mixes. If repair contractors must measure quantities and mix on site, it will save money at the risk of less consistency and higher risk of shrinkage, poor bonding, etc. specialized mix design can be carried out by concrete expert to provide pump able, pourable and trowel able mixes. In the United States and Canada, most of the contractors who work on bridge repair use their own concrete mix, which has secret mixing proportions based on the available materials in local markets. The ready-mix concrete factories can provide guarantees in case of corrosion due to carbonation, but if it is due to chlorides, they do not provide a guarantee for fear of the presence of chloride after the repair process is complete. Most manufacturers of materials used in mixing are field execution contractors, but when they supply materials only, they will provide all the information and technical recommendations for the execution, performance rates to calculate the required amount of the materials, and appropriate method of operation **(Broomfield, 1997)**.

3.3. Execution Methods

There are several ways to implement the repair process, which is entirely dependent on the type of structure member to be repaired and the materials used in the repair. These methods are described in the following sections.

3.3.1. Manual Method

The manual method is used in most cases, especially when small spaces are to be repaired (Figure 3.10). A wooden form can be made and then concrete can be poured into the damaged part, as in Figure 3.11, in the case of concrete columns or vertical walls. One must take into account an appropriate distance to cast the concrete in the wooden form easily. This method is commonly used for its efficiency and has little cost. It does not require expensive equipment, but extensive experience in fabrication, installing the wooden form, and the casting procedure is needed **(U.S. Army Corps of Engineers, 1995)**.

Figure 3.10: Manual Method for Repair (El-Reedy, 2008)

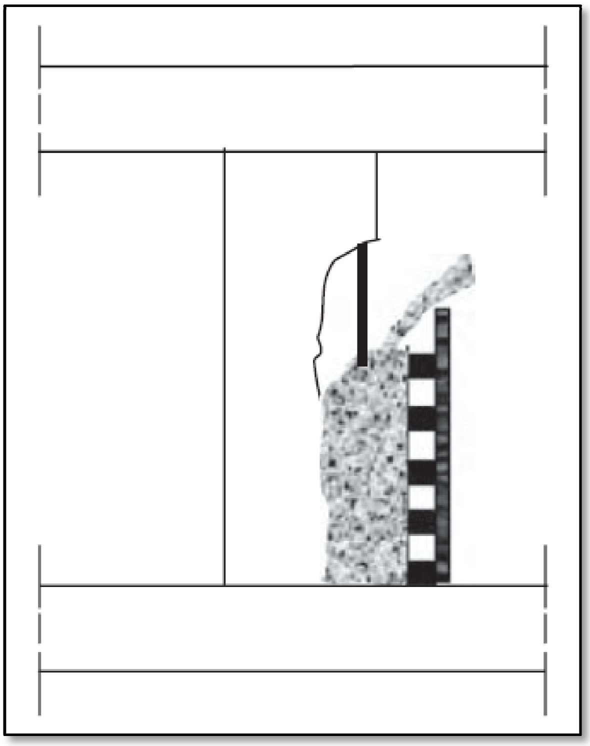


Figure 3.11: Casting Concrete On-Site (El-Reedy, 2008)

3.3.2. Grouted Preplaced Aggregate

As shown in Figure 3.12, the aggregate should be placed with gap grading in the area to be repaired. The next step is to make a grouting fluid by injecting it inside the aggregate by a pipe with pump to fill the gap between the aggregate and the grout.

This method of repair is used in repairing bridge supports and other special applications. This method requires special equipment such as the pipe injection, pump, and other special miscellaneous equipment. Therefore, one can conclude that this method is used by private companies with high potential **(El-Reedy, 2008)**.

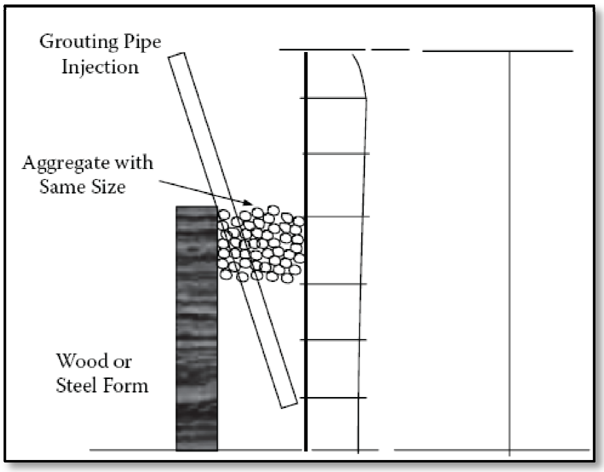


Figure 3.12: Injected Preplaced Aggregates (El-Reedy, 2008)

3.3.3. Shotcrete

Shotcrete is used when large surfaces need to be repaired, but the mix and components of concrete suitable for the use in shotcrete need special additives and specifications. As shown in Figure 3.13, health and safety precautions must be carefully followed for the worker using shotcrete, as it contains polymers and special additives. In the concrete mix design, the nominal maximum coarse aggregate size must be defined to be suitable to the shotcrete equipment's nozzle and pump to avoid any problem during concrete casting. Complete member casting is used when total reconstruction is required for the concrete member whose steel reinforcement bars have been depleted due to corrosion. Therefore, it is necessary to pour concrete for the complete member with full depth, as shown in Figure 3.14 **(U.S. Army Corps of Engineers, 1995)**.

Figure 3.13: Using Shotcrete (El-Reedy, 2008)

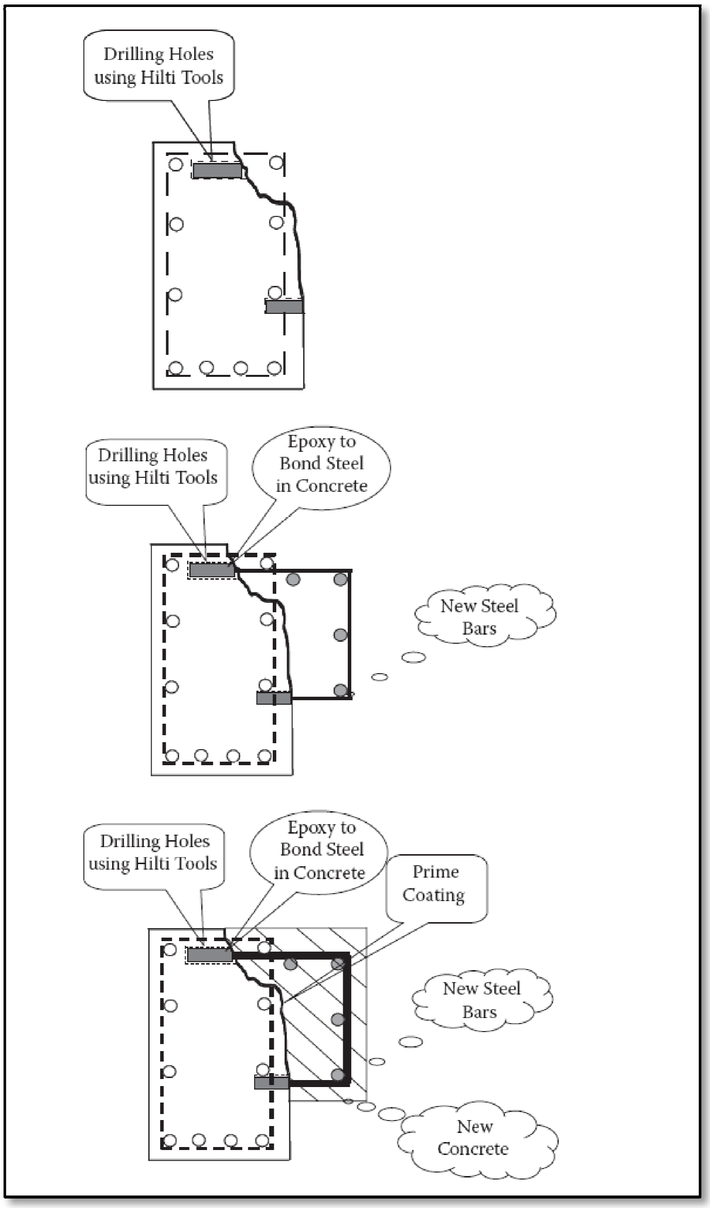


Figure 3.14: Steps of Repair (El-Reedy, 2008)

Frequently, in practice (e.g., when bathrooms are repaired), the concrete slab will be in very bad condition, so the ordinary repair procedure will not be efficient. In this case, the entire concrete slab will be demolished, a new steel reinforcement will be installed, and the concrete slab will be poured, as shown in Figure 3.14. A decision to use this method must take into consideration the whole building's condition. This method is usually easy to apply to the bathroom slab because, in residential buildings, it is usually designed as simply supported as it drops around 100 mm for plumping pipes **(El-Reedy, 2008)**.

3.4. Concluded Remarks

1. Many alternatives are available to the maintenance engineer in attempting to combat corroded deterioration buildings and keep the structure in its original condition. These may be grouped into three broad categories such as protection, patching and rehabilitation. However, before approaching corroded reinforced concrete structures repairs, consideration must first be given to the cause of the problem.
2. There are several regular steps in the repair of all structures exposed to corrosion. The first step is to strengthen the structure. The second step is to remove the cracked and delaminated concrete. The final step is to paint the concrete member as external protection.
3. To remove the cracked and delaminated concrete, many methods can be followed such as manual method, pneumatic hammer method, water jet and grinding machine.
4. After removing the cracked concrete and cleaning concrete and steel reinforcement's surface, new patch concrete is necessary. There are many execution methods for patching concrete such as manual method, grouted preplaced aggregate and shotcrete.
5. The methods of repairs illustrated in this chapter were used to be a guide line for repairing program used in this research as will be discussed in the following chapter.

Chapter 4: Materials and Test Program

4.1. Introduction

Concrete is a mixture of cement (usually Portland cement) and stone aggregate. When mixed with a small amount of water, the cement hydrated to form a microscopic opaque crystal lattice structure encapsulating and locking the aggregate into its rigid structure.

Typical concrete mixes have high resistance to compressive stresses; however, any appreciable tension (e.g. due to bending) will break the microscopic rigid lattice resulting in cracking and separation of the concrete. For this reason, typical non-reinforced concrete must be well supported to prevent the development of tension.

If a material with high strength in tension, such as steel, is placed in concrete, then the composite material -reinforced concrete- resists compression, bending and other direct tensile actions. A reinforced concrete section where the concrete resists the compression and steel resists the tension can be made into almost any shape and size for the construction industry.

The use of concrete evolved and steel reinforcement was added to counteract any tensile actions. Thus the susceptibility of the concrete to an aggressive environment increased with the addition of an unstable material which had the tendency to oxidize in the presence of oxygen. If the only solution is to use reinforced concrete, then certain precautions must be taken to ensure that the steel reinforcing bars do not impair the serviceability of the entire structure. The oxidation of the steel material is the direct cause of the cracks and defects in the concrete, which reduce the steel cross-sectional area and weaken the bond between the concrete and the reinforcement. All of these changes directly affect the response of the reinforced concrete members subjected to different loading conditions.

Very limited research has been conducted on the effects of corrosion on steel reinforcing bars embedded in concrete, and of the flexural behavior of reinforced concrete members with corroded bars. In recent years, substantial attention and interest has been expressed by the engineering sector concerning steel corrosion in reinforced concrete structures due to the occurrence of extensive corrosion in infrastructure elements worldwide and the associated large and increasing repair costs**.**

The present work consists of carrying out an accelerated corrosion experiments, understand and make reliable assessment of the effect of corrosion on beams. At the same time presents a method for repairing and strengthening these corroded beams.

The present research work consider a step in the direction, more studies on both fundamental and applied, are needed.

4.2. Test program

To reach the proposed thesis objectives, an experimental test program was designed. The main objectives of this test program were as follows:

1. To design and construct small scale beams.
2. To accelerate corrosion of beams.
3. To investigate the flexural capacity of these corroded beams.
4. Applying structurally repairing techniques for corroded beams.
5. To investigate the flexural behavior of the repaired beams.
6. And finally to make a comparison between un-corroded, corroded and repaired beams from structural capacity point of view.

To reach these objectives, an experimental test program consisted of designing, constructing, corroding, repairing and testing in flexural of fourteen reinforced concrete beams was carried out. Two of these beams were tested as control beams, two were tested as corroded beams while six beams were tested as repaired beams. The other four beams were used as standby beams and to investigate the corrosion rate for the corroded beams. In the following sections the test program details will be discussed.

4.2.1. Specimens Design

Fourteen beams with dimensions 1200x150x100 mm, shown in Figure 4.1 and Figure 4.2, were used in the test program.

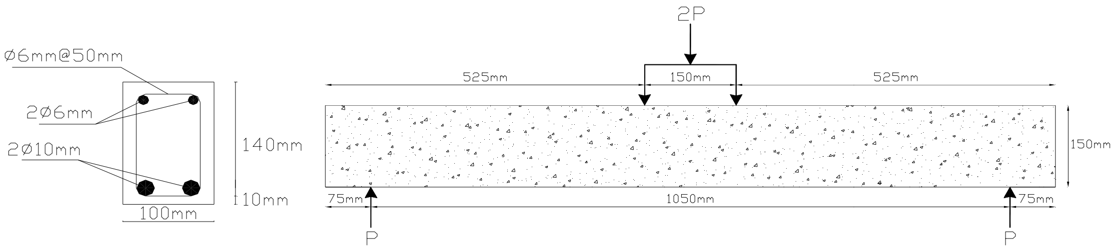


Figure 4.1: Section Details

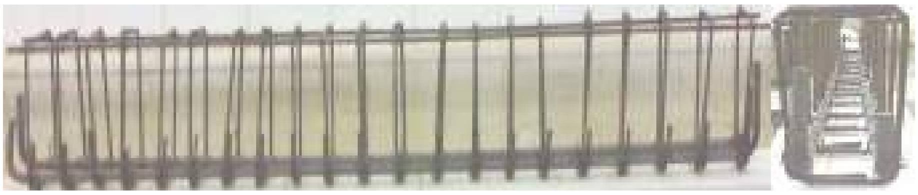


Figure 4.2: Steel Bars Cage

These beams were designed according to the ACI 318-08 **(ACI 318-08)**. A nominal bending moment $M_n = 9.51\times10^6$ N.mm was calculated. All design limitations to prevent shear failure according to the code were considered using $\phi 6mm$ stirrups @ 50 mm.

4.2.2. Corrosion Process Set-Up

In order to accelerate the corrosion process, an electrochemical system depends on the concept of Faraday’s second law, as discussed previously, was used.

The concept of accelerating the corrosion was to force steel reinforcement to act as anode in galvanic cell. That can be done by immerse beams in aqueous solution and connecting the steel reinforcement bars with positive DC current generator to act as anode while connecting the negative power supply to external steel rods immersed in the aqueous solution to act as cathode. This consists electric circuit and force steel ions to translate from anode to cathode as discussed previously.

Before finalizing the corrosion system setup, a pilot study was done, different current values were applied for four specimens, and the steel embedded in the concrete were visually inspected weekly for each different current. The most effective current which accelerate the corrosion process was used as will be discussed in the experimental work section.

4.2.3. Beam Repairing Set-Up

The repairing of corroded beams depended on the concept of patch repair as it was discussed in Section 3.4 and Section 3.7. However, after the end of the corrosion process, six corroded beams were repaired. The main idea of repairing of corroded beams was cutting the unsound concrete and cleaning both the concrete surface and steel reinforcement bars. Additional longitudinal steel reinforcement’s bars were added and fixed with shear connectors. These shear connectors were used for fixing the new reinforcement with the corroded reinforcement and work also as stirrups to prevent both diagonal and inter laminar shear failure. Figure 4.3 shows shear connectors and new steel bars section.

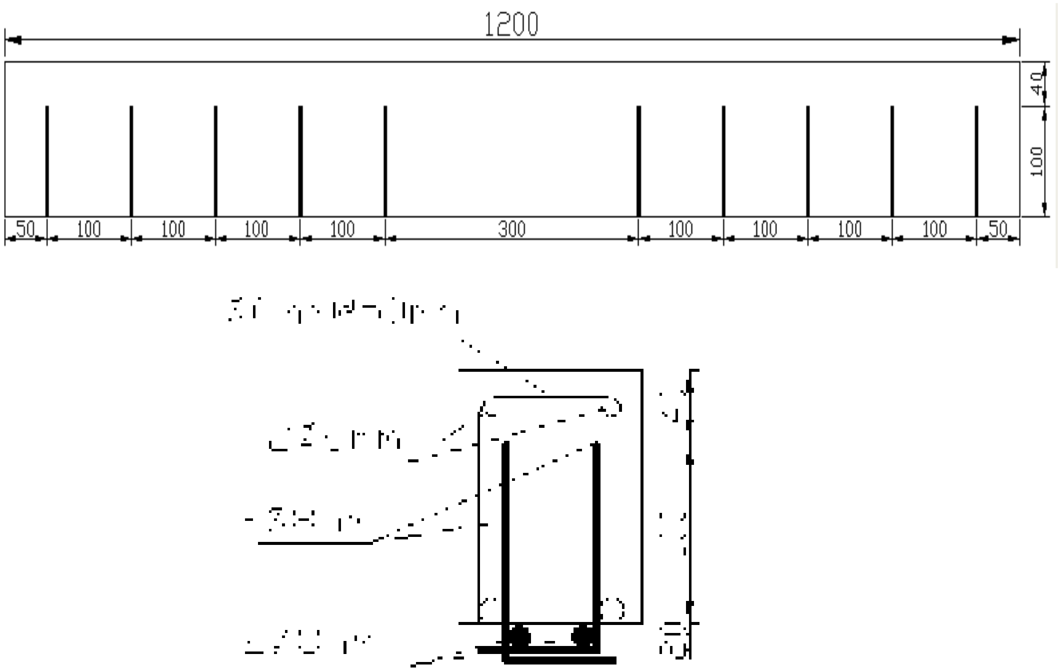


Figure 4.3: Shear Connectors and New Steel Bars Section

The second step was applying a new layer for the bottom sides of beams, for this purpose three repair materials were applied in order to study the effect of the used repairing material layers on the flexural capacity and the crack development of the repaired beams. The properties and specifications of these materials will be discussed in the experimental work Section.

4.2.4. Beam Flexure Testing

Control, corroded and repaired beams were tested as simply supported beams with two concentrated loads using the flexural testing machine as will be discussed later, the deflection was recorded for each load increment.

Another parameter was to record the crack development for beams during the flexural test.

4.3. General Preparation and Difficulties

The main difficulty during this work was how to develop an electrochemical system in order to accelerate the corrosion process. In fact many previous studies discussed similar systems. **Fazio, 1996** and **Kovacs, 2000** are some researches who dealt with such systems. For example Figure 4.4 shows Kovacs technique used for accelerating the reinforcement corrosion process.

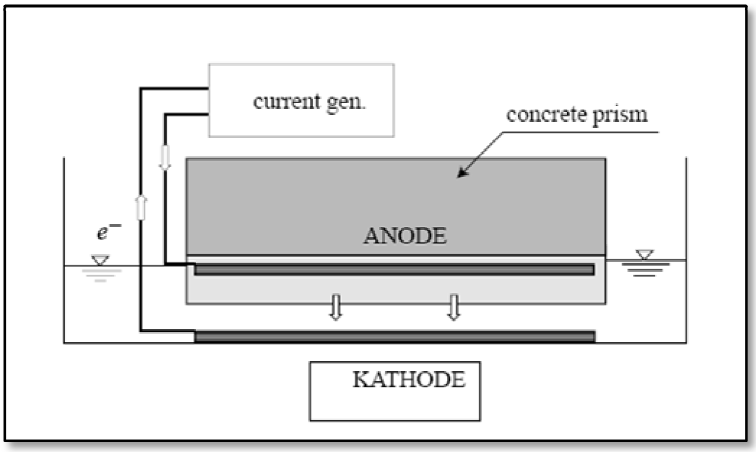


Figure 4.4: Drawing Of Steel Reinforcement Corrosion Accelerating Technique (Kovacs, 2000)

Although these systems depend on Faraday’s second law as discussed previously, the situation in these cases were different than it has been stated in Faraday’s second law. These differences discussed in Table 4.1:

Table 4.1: Differences between Faraday’s Second Law and Corrosion Accelerated Process

Item	Faraday’s Second Law	Electrochemical Reinforcement Corrosion Acceleration Process
1	Ions transfer freely through the electrolyte	Ions of steel transfer through the concrete cover and then the electrolyte
2	Faraday’s Second Law deals with pure elements	Reinforcement steel is an alloy consisting of many elements, not only pure steel
3	Existence of oxygen is not a condition	Existence of oxygen is necessary to complete the corrosion process

For these reasons, Faraday’s second law does not apply in the case of electrochemical reinforcement corrosion acceleration process.

The second difficulty that there was no machine in Gaza strip that can test the flexural capacity of reinforced beams specimens, a manual hydraulic jack with a compression strength capacity of 50 tons connected with a steel frame as seen in Figure 4.5 was used. A pressure gauge with an accuracy of ±1 KN was connected to the hydraulic jack and then the machine was calibrated by a standered loading cell.

Figure 4.5: flexural test machine (I.U.G. Laborite's Machine)

Another difficulty was the absence of a ready concrete mix design to satisfy the specifications, although there were many mixes used in the local market, but all of them use empirical mixes and no one satisfies a known specification especially in the term of fine aggregate specification That was because the fine aggregate used was normal sand which found in Gaza which does not meet any known specification.

To overcome this problem a new concrete mix was designed. Crushed stone sand was sieved on 0.3 mm sieve to reduce the percent of fines retrained on sieve 0.15 and 0.075 mm, and then mix with normal sand to satisfy the specification.

The material, equipment and the carried out experimental works are discussed in the following sections in details.

4.4. Experimental works

4.4.1. Materials

Material properties which used in the test program such as concrete, steel and repair materials discussed as follows:

4.4.1.1. Concrete

Concrete consist of these four main materials: coarse aggregates, fine aggregates, cement and water.

4.4.1.2. Coarse Aggregates

According to the local market surveying, three types of coarse aggregate were found. Table 4.2 illustrates the sieve analysis and the properties of these types.

Table 4.2: Coarse Aggregate Types, Sieves and Properties

Sample Description	Type1	Type2	Type3
SIEVE SIZE (mm)	% Passing	% Passing	% Passing
37.500	100	100	100
25.000	100	100	100
19.000	46.74	99.2	100
12.500	3.33	56.3	100
9.500	1.42	13.22	94.3
4.750	1.06	3.41	25.24
2.630	1.06	2.03	6.63
Dry unit weight (Kg/m3)	1436	1506	1486
Dry specific gravity	2.68	2.65	2.63
Saturated specific gravity	2.71	2.65	2.63
Absorption %	1.12%	2.42%	3%

To achieve the ASTM C33-03 standard requirements for coarse aggregate, a mix design of these three types was prepared as shown in Table 4.3 and Figure 4.6.

Table 4.3: Coarse Aggregate Sieve and Analysis According To ASTM C33-03

	Aggregate Kind	Type1	Type2	Type3
	% Percent	7.00%	68.00%	25.00%
Sample Description	Coarse Aggregate		ASTM C 33-03	
	Mix Of Type (1&2&3)		Min	max
SIEVE SIZE (mm)	% Passing		% Passing	% Passing
37.500	100.0		100.0	100.0
25.000	100.0		100.0	100.0
19.000	95.7		90.0	100.0
12.500	63.5			
9.500	32.7		20.0	55.0
4.750	8.7		0.0	10.0
2.630	3.1		0.0	5.0
Unit Weight (KG/m3)	1496			
Dry Specific Gravity	2.65			
Moisture Content %	0.14			
Absorption %	2.47			

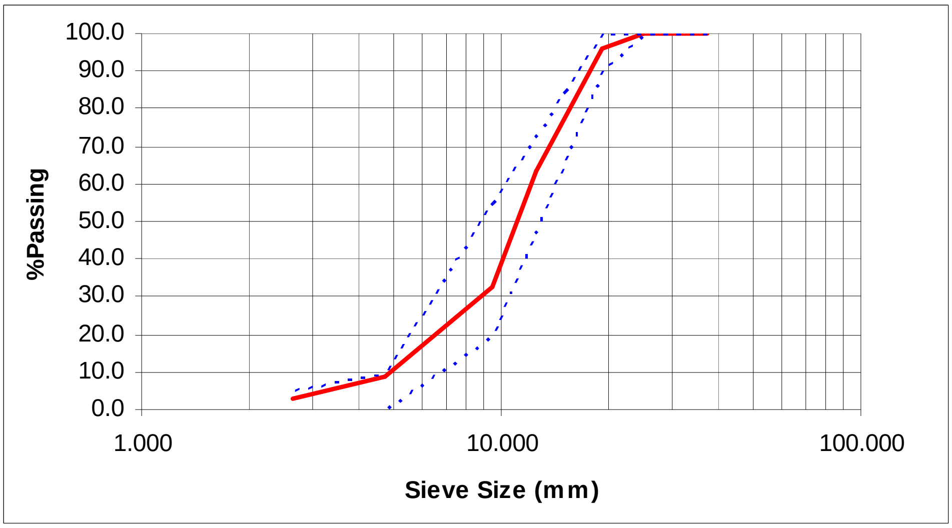


Figure 4.6: Coarse Aggregate Sieve Analysis According To ASTM C33-03 Limitation

4.4.1.3. Fine Aggregates

According to the local market surveying, two types of fine aggregates were found, Table 4.4 illustrates the sieve analysis and the properties of these types.

Table 4.4: Fine Aggregate Types, Sieve and Properties

Sample Description	Type 1 (Crushed Stone Sand)	Type 2 (Natural Sand)
SIEVE SIZE (mm)	% Passing	% Passing
9.500	99.91	100
4.750	99.03	100
2.360	81.92	100
1.180	45.93	100
0.600	22.30	99.6
0.300	6.97	47.8
0.150	4.15	2.04
0.075	3.26	0.34
Dry specific gravity	2.65	2.60

To achieve the ASTM C33-03 standard for fine aggregate, mix design of these two types was prepared as shown in Table 4.5 and Figure 4.7.

Table 4.5: Fine Aggregate Sieve and Analysis According To ASTM C33-03

	Type1	Type 2		
Percent	70.00%	30.00%		
Sample Description	Mix Of Type 1&2		ASTM C33-03 Fine AGG.	
			min	max
SIEVE SIZE (mm)	% Passing		% Passing	% Passing
9.500	99.94		100.0	100.0
4.750	99.32		95.0	100.0
2.360	87.34		80.0	100.0
1.180	62.15		50.0	85.0
0.600	45.49		25.0	60.0
0.300	19.22		5.0	30.0
0.150	3.51		0.0	10.0
0.075	2.38		0.0	4.0
Dry Specific Gravity	2.635			
Moisture Content%	0.20			
Absorption%	1.80			

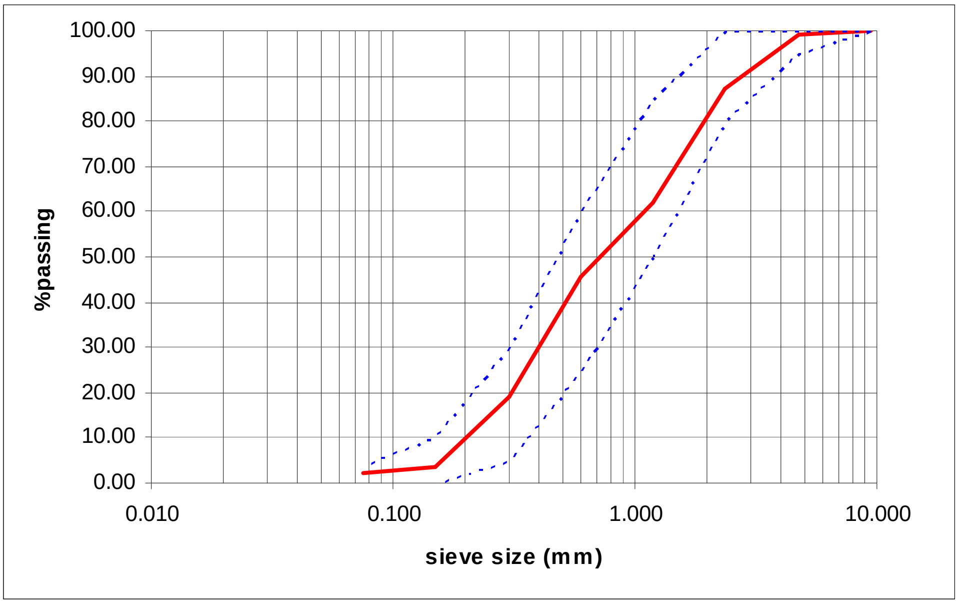


Figure 4.7: Fine Aggregate Sieve Analysis According To ASTM C33-03 Limitation

4.4.1.4. Cement

Portland cement type I was used. The cement was obtained from local market and kept in dry location. The cement source was Silo Nisher.

4.4.1.5. Water

Tap water, potable without any salts or chemical was used in the study. The water source was the soil and material lab in Islamic University of Gaza.

4.4.1.6. Concrete Mix Design

A concrete mix was designed to obtain 28-day compressive strength $f_{c'}$= 25MPa, 70-100 mm slump, a maximum aggregate size of 19 mm, and w/c ratio of 0.61. Table 4.6 illustrates the mix design proportions for each cubic meter of concrete.

Table 4.6: Mix Design Proportion for Each Cubic Meter of Concrete

No.	**Material Kind**	**Weight / m^3 (Kg)**	**Notes**
1	Coarse Aggregate	**930**	Coarse aggregate consist of the following proportions according to Table 4.2: • 65.1 (Kg) Type 1 • 632.4 (Kg) Type 2 • 232.5 (Kg) Type 3
2	Fine Aggregate	**892**	Fine aggregate consist of the following proportions according to Table 4.5: • 624.4 (Kg) Type 1 • 267.6 (Kg) Type 2
3	Cement	**336**	Portland cement type I.
4	Water	**241**	Potable water.

Normal weight concrete with the mix design proportions illustrated in table 4.6 was used. Three 10x10x10 mm concrete cubes from this mix design were tested to determine the compression strength of concrete, Table 4.7 illustrates these results.

Table 4.7: Sample Concrete Cubes Compression Strength Results

Sample No.	Dimension (mm)			Weight (g)	28 days Failure Load (KN)	Stress (MPa) Cube	Stress (MPa) Cylinder (0.8* cube stress)
	length	Width	High				
1	101	99	100	2380	351	35.5	28.4
2	101	100	100	2465	353	35.0	28.0
3	100	100	100	2430	347	34.7	27.8
Average	**100.7**	**99.7**	**100**	**2425**	**350.3**	**35.1**	**28.1**

4.4.2. *Steel Bars*

The steel reinforcing bars used for the construction of the beams consisted of 6mm diameter steel bar were used for both stirrups and secondary top reinforcement. Ten mm diameter steel bar were used for main bottom reinforcement.

Al1 bars had a minimum specified yielding strength of 412 MPa . Samples from the 10 mm reinforcing bars were tested using the standard tension test, an average yielding strength of 545 MPa. Ultimate strength of 738 MPa and 30% elongation were obtained.

4.4.3. *New Layer of Repaired Materials*

For the purpose of applying a new layer for the corroded beams, and according to the local market survey three types of related repaired material were found. The three types which were manufactured by FOSROC Jordan Co. were used.

The following section gives more details about these materials as follows:

For further information about these repair material specifications and instructions see appendix No. 1.

4.4.3.1. *Repair Material Type 1*

Repair material Type 1, called (Lokset) is polyester anchoring resin used for installing a high strength corrosion resistance heavy duty anchoring. These anchoring include bolts, tendons or dowels in drilled or formed holes located in concrete. This repair material was used in beam repairing set-up for the purpose of fixing 8 mm steel anchoring.

4.4.3.2. Repair Material Type 2

Repair material Type 2, called (Nitobond-EP), is an epoxy resin concrete bonding agent used for bonding wet cementitious material to existing cementitious surface. This repair material was used in beam repairing set up for the purpose of bonding the new added layer of concrete to the existing corroded beams.

4.4.3.3. Repair Material Type 3

Repair material Type 3, called (Conbextra-HF), is a shrinkage compensated cementitious precision grout. It is a free flow precision grout. This repair material was used in beam repairing set-up for the purpose of adding a new layer to the existing corroded beams.

4.5. Beam Specimens Casting and Treatment

Form of wood for constructing beams was prepared as seen in Figure 4.8.

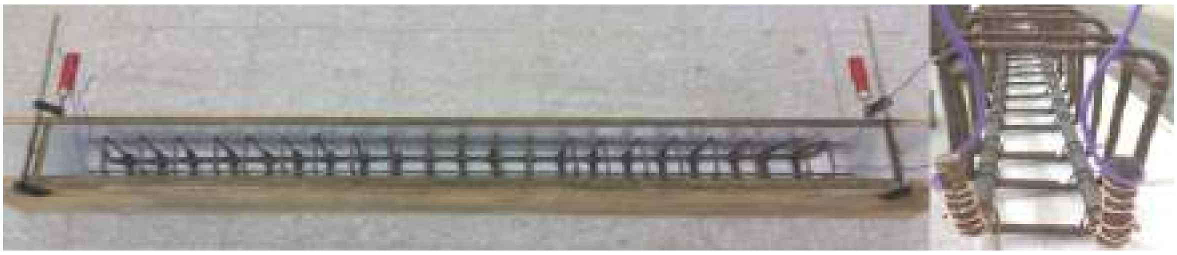


Figure 4.8: Wood Form and Reinforcement Details

The mixing of concrete was mixed in a conventional blade-type mixer according to ASTM C192 procedure. Figure 4.9 shows the casting process. Prior to the casting of the beams, an electric wire was attached to the reinforcement bars using a garden hose clamp. This wire was used to impress the necessary voltage for the corrosion process to proceed.

Figure 4.9: Casting Process

All beam specimens on its wood form base were moved to a curing basin after 24 hours from casting. The curing conditions were according to the ASTM C192 standard. Figure 4.10 shows the appearance of curing basin which used in this study.

Figure 4.10: Curing Basin Used In The Study

The samples remained in curing basin for 28 days. After that, twelve samples were transferred for the corrosion acceleration. The other two beams remained in curing condition until testing.

4.5.1. Corrosion Process Set-Up

The following is a list of the equipment used for the electrochemical test set-up:

1. A water basin (3x 0,85 x 0.7) m
2. Electric wires and electric clamps
3. Multimeter
4. AC 300 Watt computer power supply.

A basin filled with a sodium chloride solution (5 percent NaCl by weight of water) was prepared. The solution covered 30mm of the beams height (in which the flexural reinforcements exist). The electric wires were attached to the reinforcement bars with the help of a garden hose clamps. It was used to impress the necessary voltage for the corrosion process to proceed. The beams were connected in parallel to +5 Volts with a capacity of 25 Amperes electric DC current generated by power supply, which impressed an equal voltage on each beam. The negative power supply terminal was connected to twelve 10 mm steel rods immersed in the aqueous solution between the beams in order to facilitate the opening and closing of the electric circuit.

To ensure that enough oxygen needed for the corrosion mechanism exists, just 30mm of the beam height were immersed in the solution, on the other hand the sodium chloride solution was replaced twice every week. Table 4.8 illustrates the time schedule used for the corrosion set-up process.

Table 4.8: Time Schedule Used For the Corrosion Set Up Process

no	Beam Description	Beam Notation	Casting Date	Electrochemical Process (Starting Date)	Electrochemical process (Ending Date)
1	Control Beam 1	(CB1)	12-11-2007	15-12-2007	1-3-2008
2	Control Beam 2	(CB2)	12-11-2007	15-12-2007	1-3-2008
3	Corroded Beam 1	(Co.B1)	13-11-2007	15-12-2007	1-3-2008
4	Corroded Beam 2	(Co.B2)	13-11-2007	15-12-2007	1-3-2008
5	Repaired Type 1 Beam No. 1	(R1-B1)	14-11-2007	15-12-2007	1-3-2008
6	Repaired Type 1 Beam No. 2	(R1-B2)	14-11-2007	15-12-2007	1-3-2008
7	Repaired Type 2 Beam No. 1	(R2-B1)	15-11-2007	15-12-2007	1-3-2008
8	Repaired Type 2 Beam No. 2	(R2-B2)	15-11-2007	15-12-2007	1-3-2008
9	Repaired Type 3 Beam No. 1	(R3-B1)	16-11-2007	15-12-2007	1-3-2008
10	Repaired Type 3 Beam No. 2	(R3-B2)	16-11-2007	15-12-2007	1-3-2008
11	Standby Beam 1	(St.-B1)	17-11-2007	15-12-2007	1-3-2008
12	Standby Beam 2	(St.-B2)	17-11-2007	15-12-2007	1-3-2008
13	Standby Beam 3	(St.-B3)	18-11-2007	15-12-2007	1-3-2008
14	Standby Beam 4	(St.-B4)	18-11-2007	15-12-2007	1-3-2008

During the electrochemical corrosion process, the current passing through each beam was measured by the Multimeter every 10 days. The physical changes and crack development of beams during the corrosion process were obtained. Any how the standby beams were used to check the statues of flexural steel bars embedded in concrete.

Figure 4.11 shows the electrical connection of beams. While Figure 4.12 shows the rust developing on the beam surface at the end of electrochemical corrosion process.

Figure 4.11: Electrical Connection of Beams

Figure 4.12: Rust on Beams at the End of Corrosion Process

4.5.2. Beam Repairing works

Repairing process details were done as follows:

1. The concrete covering, the flexural reinforcement and loosely concrete was cut. The surface of concrete was cleaned well until reaching a sound concrete surface.
2. The corroded steel reinforcing bars were cleaned from rust using steel brush.
3. Twenty holes with 12mm diameter and 100 mm depth were drilled in the bottom side of beams.
4. These holes were cleaned with air compressor and then injected with repair material type1 (Lokset). Then 8 mm in diameter steel bars of length of 170 mm were installed in the holes (for further information about repair material type1 (Lokset) specifications and instructions see Appendix No. 1).
5. The beams were left for 24 hours, and then two 8 mm steel reinforcing bars were installed in the flexural zone and connected with the shear connectors as discussed in the test program. Figure 4.13 shows the installing of shear connectors during the experimental works.

Figure 4.13: Installing Of Shear Connectors

6. After installing the shear connectors and the steel bars reinforcement, 40 mm new concrete layer was added to each beam and three types of materials were used. Table 4.9 shoes the details of the applied repaired layer types and applying steps.

Table 4.9: Repair Material Details

No	Layer Description	Repairing Date	Beam Name	Repairing Steps
1	Normal weight concrete according to mix design proportions as shown Table 4.6, without any additions.	15-3-2008	(R1-B1) & (R1-B2)	▪ Concrete surface was cleaned from dust and unsound concrete. ▪ Surface of concrete were flooded with water, free water was removed. ▪ Beams were put in water after 24 hours from casting.
2	Normal weight concrete according to mix design proportions shown Table 4.6, adding repair material type 2 to the inter face between the new layer and the existing concrete.	16-3-2008	(R2-B1) & (R2-B2)	▪ Concrete surface was cleaned from dust or un sound concrete. ▪ Surface of concrete cleaned with water before 24 hours from applying repair material type 2, free water was removed. ▪ Repair material type 2 was well mixed and brushed to the concrete surface, the new concrete layer was caste after two house from brushing . ▪ Beam was put in water after 24 hours from casting.
3	Repairing material Type 3	17-3-2008	(R3-B1) & (R3-B2)	▪ Concrete surface was cleaned from dust or unsound concrete. ▪ Surface of concrete was flooded for 5 hours before casting the new layer, free water was removed. ▪ Repairing material Type 3 was prepared and cast to the existing concrete. ▪ Beam was put in water after 24 hours from casting.

Figures 4.14 and 4.15 show repaired materials while being installed.

Figure 4.14: Repair Material Type 2 Brushed On Beams

Figure 4.15: Repairing Material Type 3 While Mixing and Casting

After the end of repairing process, beams were kept in water for 28 days and then had been tested. Before testing process, beams had been brushed with a white colure and lines were drawn on beams every 75mm on the long and the height directions to obtain the crack development during the test.

4.5.3. *Beam Flexure Testing Works*

Beams specimens were tested as simply supported beams with two concentrated loads using the flexural testing machine discussed previously. Two 30 mm in diameter steel rods were used as edge simply supports while two steel bars with a semi-half circular section with radius of 40 mm were used to apply the middle concerted loads. A mechanical dia1 gauge was located at the mid-span to measure the deflection caused by changing of the applied load as seen in Figure 4.16. The load was increased with 0.785 KN increment and the corresponding deflection readings were recorded for each 4 KN load increment.

Figure 4.16: Deflection Dial Gage Fixation

During the test process the crack development was recorded and drawn on a paper we designed for this purpose.

Chapter 5: Test Results

5.1. Introduction

The main objective of this chapter is to report the results obtained during the electrochemical process, visual inspection and the flexure testing of the specimens.

The experimental data reported are classified under the following categories:

1. Electrochemical results.
2. Visual inspection results.
3. Flexural test results.

5.2. Electrochemical Results

The twelve concrete beams were placed in a water basin, each two beams were connected in parallel and the sex double beams were connected together in parallel to a power supply with +5 Volts-up to 25 Amperes electric DC current.

Readings were taken every 10 days of the impressed current for each beam. Table 5.1 illustrates the current readings which were taken (standby beams are not included).

Table 5.1: The Current Reading during the Electrochemical Process

no	Beam Name	Duration of corrosion (days)	Date Of Reading from the Starting of The Electrochemical (days)								**Beam Sub. Average**
			10	20	30	40	50	60	70	77	
			Current Reading (Amperes)								
1	(Co.B1) & (Co.B2)	77	1.70	1.55	1.54	1.62	1.66	1.60	1.66	1.75	**1.64**
2	(R1-B1) & (R1-B2)	77	1.79	1.70	1.77	1.88	1.79	1.71	1.75	2.00	**1.80**
3	(R2-B1) & (R2-B2)	77	1.66	1.58	1.66	1.87	1.70	1.65	1.65	1.50	**1.66**
4	(R3-B1) & (R3-B2)	77	1.69	1.48	1.33	1.57	1.69	1.65	1.70	1.10	**1.53**
Daily sub Average		**77**	**1.71**	**1.58**	**1.58**	**1.74**	**1.71**	**1.65**	**1.69**	**1.59**	**Total avg. current** **1.65**

Figure 5.1 shows graphically these current readings.

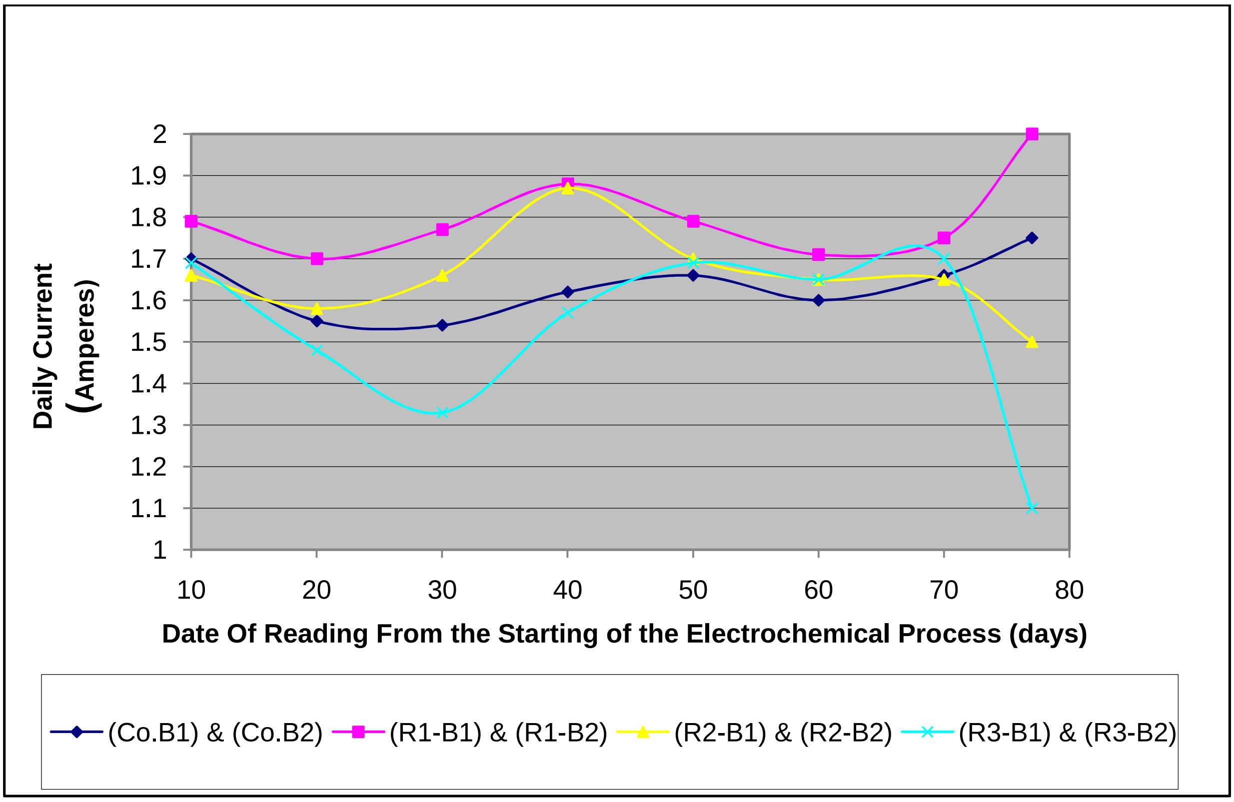


Figure 5.1: Beams Current Reading during the Electrochemical Process

5.3. Visual Inspection:

The visual inspection has been carried out after the corrosion process. Rust formed on beams, crack development due to the corrosion process and changes occurred in reinforcement due to corrosion are discussed as follows:

5.3.1. Rust Formed On Beams

After the end of electrochemical corrosion process, beams were left in air for fourteen days. This period was inessential to allow complete reaction between the iron ions with air to form the rust. Figures 5.2 and 5.3 show the rust accumulated on the beam bottom during and at the end of corrosion process.

Figure 5.2: Rust on Beams during the Corrosion Process

Figure 5.3: Rust on Beams At The End Of Corrosion Process

5.3.2. Crack Development Due To Corrosion

After fourteen days from the end of the electrochemical corrosion process, concrete surface were cleaned from rust, longitudinal cracks parallel to the flexural reinforcement had appeared. Figure 5.4 shows these highlighted cracks at the bottom and beam side parallel to the flexural reinforcement.

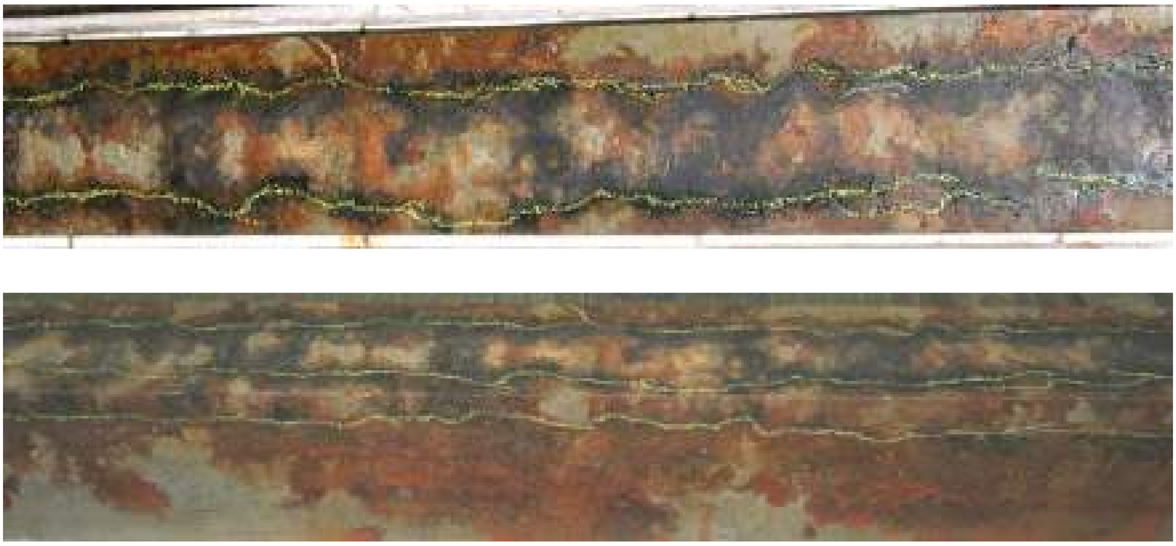


Figure 5.4: Cracks at the Bottoms Sides of Beams

5.3.3. Reinforcement Deterioration Due to Corrosion

After the end of corrosion process, the concrete covering the steel flexural reinforcement was removed. Both flexural and stirrups reinforcement were corroded and some of the stirrups were completely cutoff because of corrosion. Figure 5.5 shows the steel status due to corrosion.

Figure 5.5: Steel Status after Corrosion

Three corroded flexural steel bars were removed from a standby beam. It was difficult to determine the cross section area of these bars because they were deteriorated during corrosion process. The bars were tested using the standard tension to determine its elongation. An average elongation of 5% is obtained which is only 16.7% of the normal steel bar elongation.

Figure 5.6 shows this steel bar compares to non-corroded steel bar.

Figure 5.6: Corroded Steel Bar in Comparison with Non Corroded Steel Bar

5.4. Flexural Test Results

Figure 5.7 shows the load – mid span deflection for all beams.

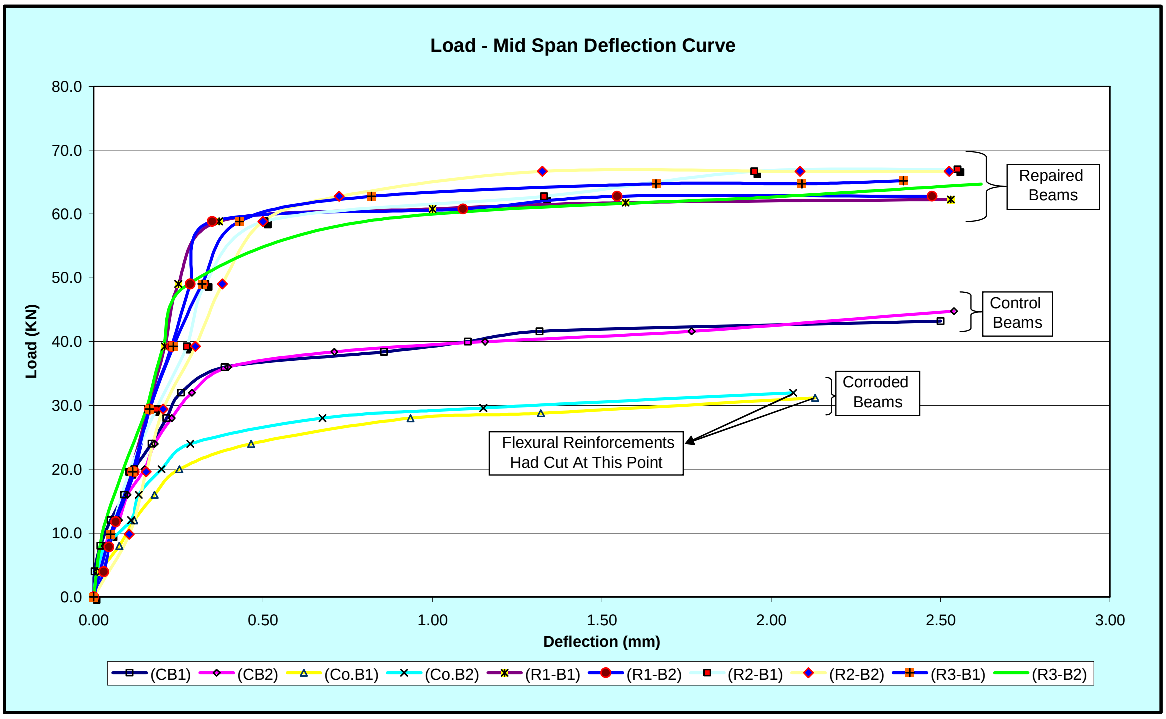


Figure 5.7: Load – Mid Span Deflection of Beams

Table 5.2 shows the maximum load and deflection for each beam at failure and the failure loads in percentage of design loads.

Table 5.2: The Maximum Load and Deflection for Each Beam

No	Beam Name	Beam Description	Failure Load (KN)	Deflection (mm)	Percentage Of Failure Load of Design Load (%)
1	(CB1)	Control Beam 1	43.20	2.50	102.2
2	(CB2)	Control Beam 2	44.80	2.54	106.0
3	(Co.B1)	Corroded Beam 1	31.20	2.13	73.8
4	(Co.B2)	Corroded Beam 2	32.00	2.07	75.7
5	(R1-B1)	Repaired Type 1 Beam No. 1	62.27	2.53	147.3
6	(R1-B2)	Repaired Type 1 Beam No. 2	62.76	2.48	148.5
7	(R2-B1)	Repaired Type 2 Beam No. 1	67.00	2.55	158.5
8	(R2-B2)	Repaired Type 2 Beam No. 2	66.69	2.53	157.8
9	(R3-B1)	Repaired Type 3 Beam No. 1	65.20	2.39	154.3
10	(R3-B2)	Repaired Type 3 Beam No. 2	64.70	2.62	153.1

5.5. Crack Development Due To Flexural Test

Figures 5.8-17 shows the crack developments of each tested beam at failure.

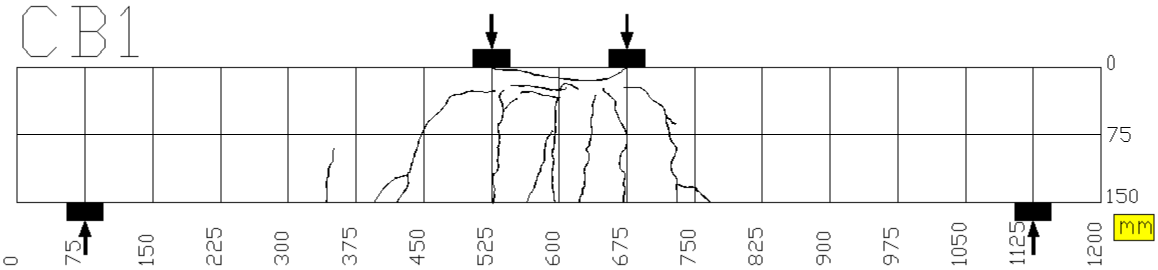


Figure 5.8: Crack Pattern for Beam (CB1) At Failure

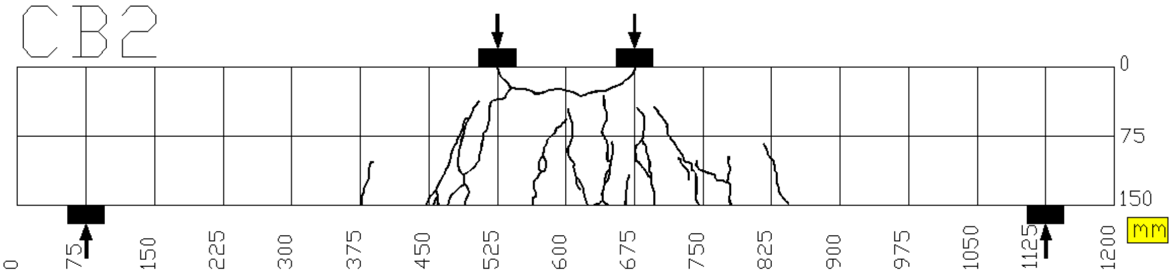


Figure 5.9: Crack Pattern for Beam (CB2) At Failure

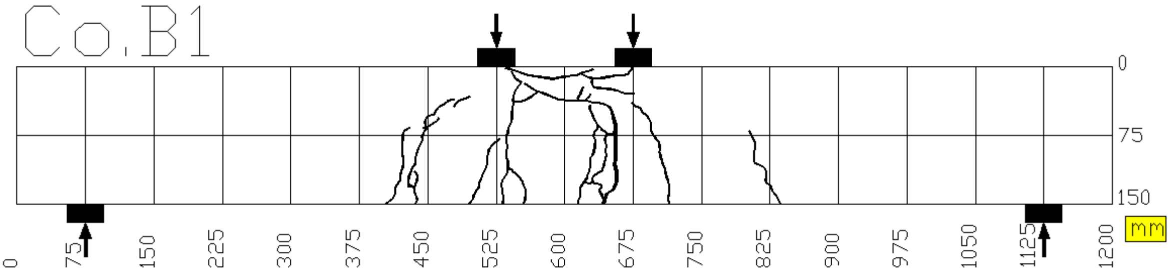


Figure 5.10: Crack Pattern for Beam (Co.B1) At Failure

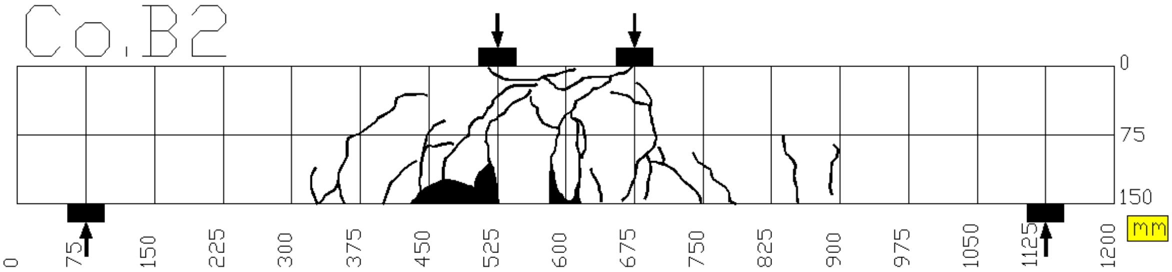


Figure 5.11: Crack Pattern for Beam (Co.B2) At Failure

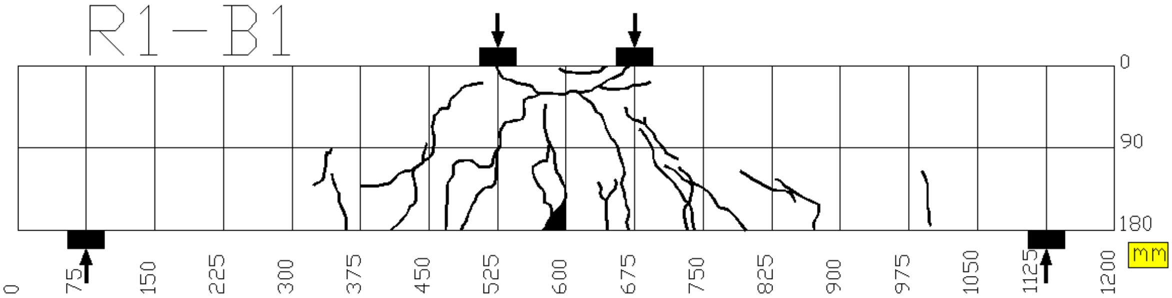


Figure 5.12: Crack Pattern for Beam (R1-B1) At Failure

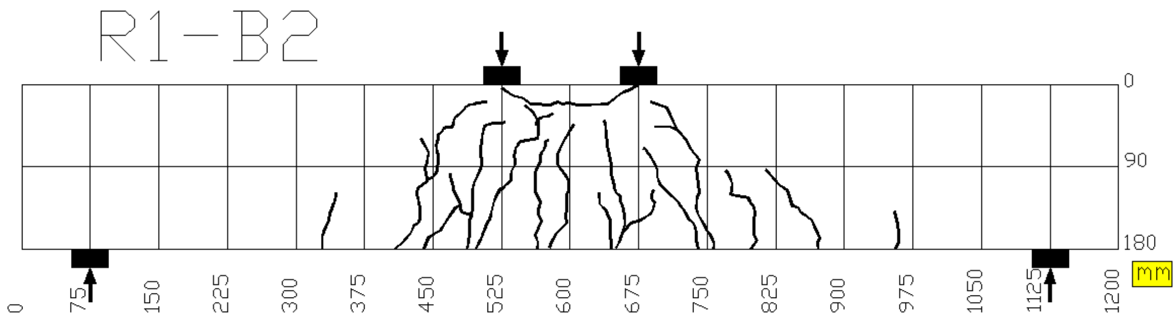


Figure 5.13: Crack Pattern for Beam (R1-B2) At Failure

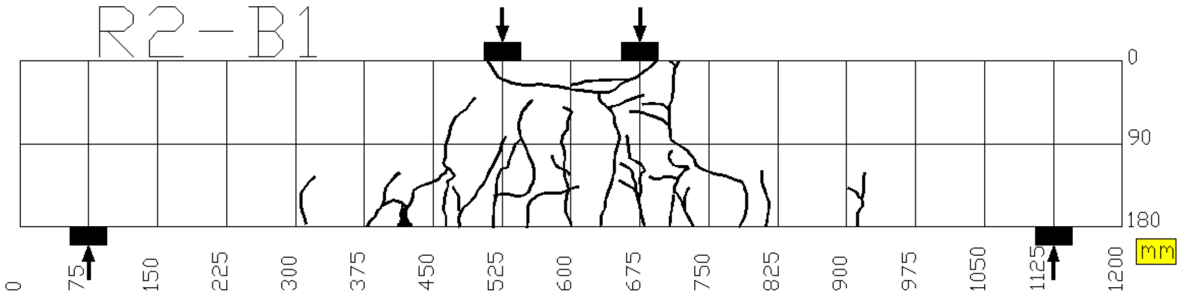


Figure 5.14: Crack Pattern for Beam (R2-B1) At Failure

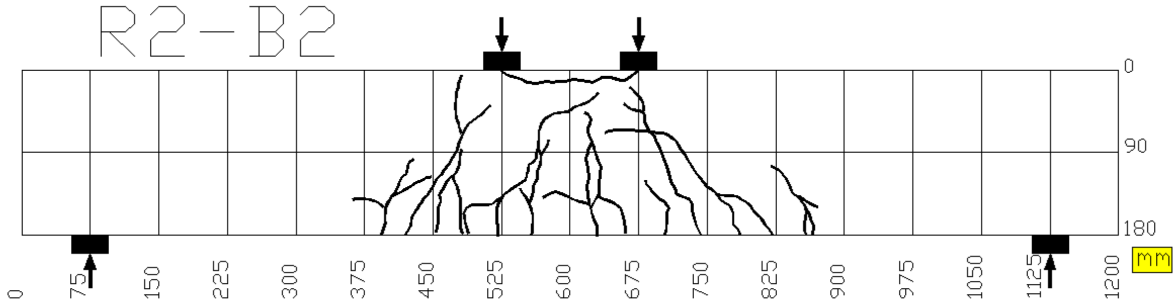


Figure 5.15: Crack Pattern for Beam (R2-B2) At Failure

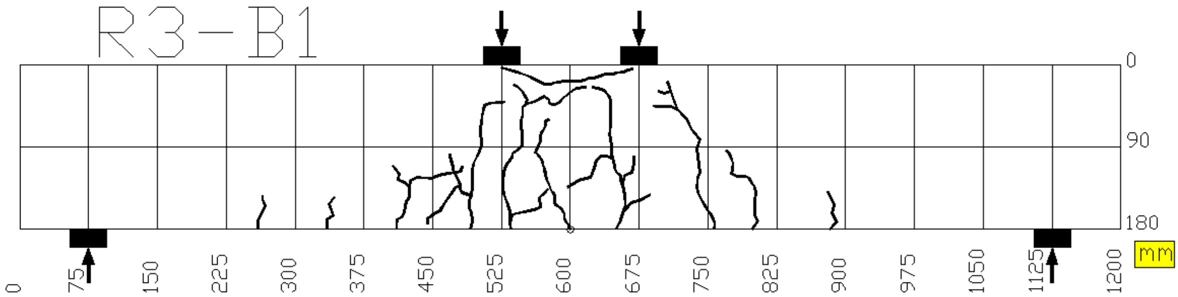


Figure 5.16: Crack Pattern for Beam (R3-B1) At Failure

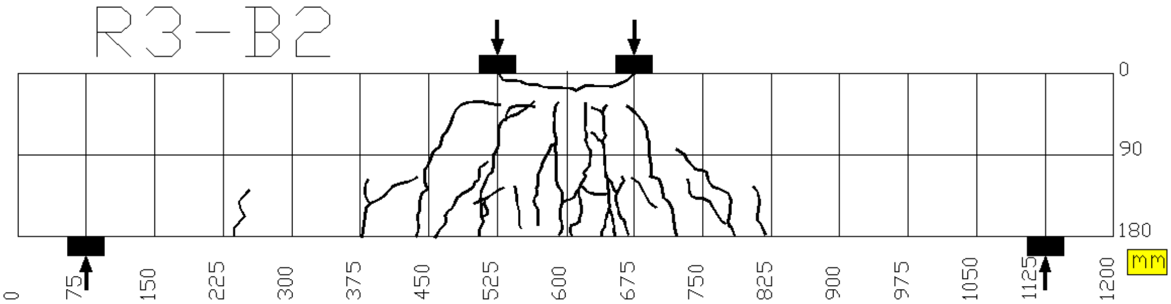


Figure 5.17: Crack Pattern for Beam (R3-B2) At Failure

Figures 5.18 through 5.22 shows photos for the crack developments of some selected beams at failure:

Figure 5.18: Crack Pattern Photo for Beam (CB2) At Failure

Figure 5.19: Crack Pattern Photo for Beam (Co.B1) At Failure

Figure 5.20: Crack Pattern Photo for Beam (R1-B1) At Failure

Figure 5.21: Crack Pattern Photo for Beam (R2-B1) At Failure

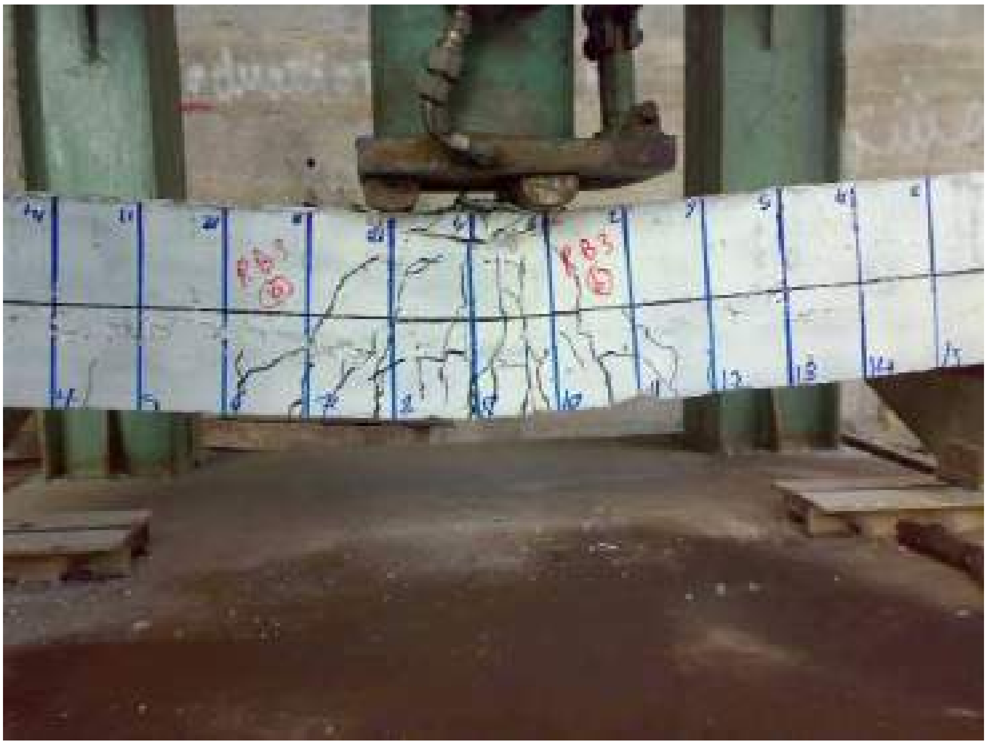


Figure 5.22: Crack Pattern Photo for Beam (R3-B2) At Failure

Chapter 6: Discussion of Results

6.1. Introduction

This chapter discusses the experimental results and the conclusions which are drawn based on these results. The main objective of this laboratory investigation is to study the effect of the corrosion process on the flexural capacity of simply supported beams subjected to two concentrated loads and structural performance of them after repairing.

Electrochemical current results, corrosion effects and flexural test results will be discussed.

6.2. Current Reading during Electrochemical Process

According to Faraday’s second law, the corrosion causing current is proportional to the corrosion rate. However, the situation in steel embedded in concrete is different as discussed in section 4.2.

During the process of accelerating the corrosion of beams reinforcement, the impressed current for each beam was recorded every 10 days. Figure 6.1 shows the average current readings during the electrochemical process.

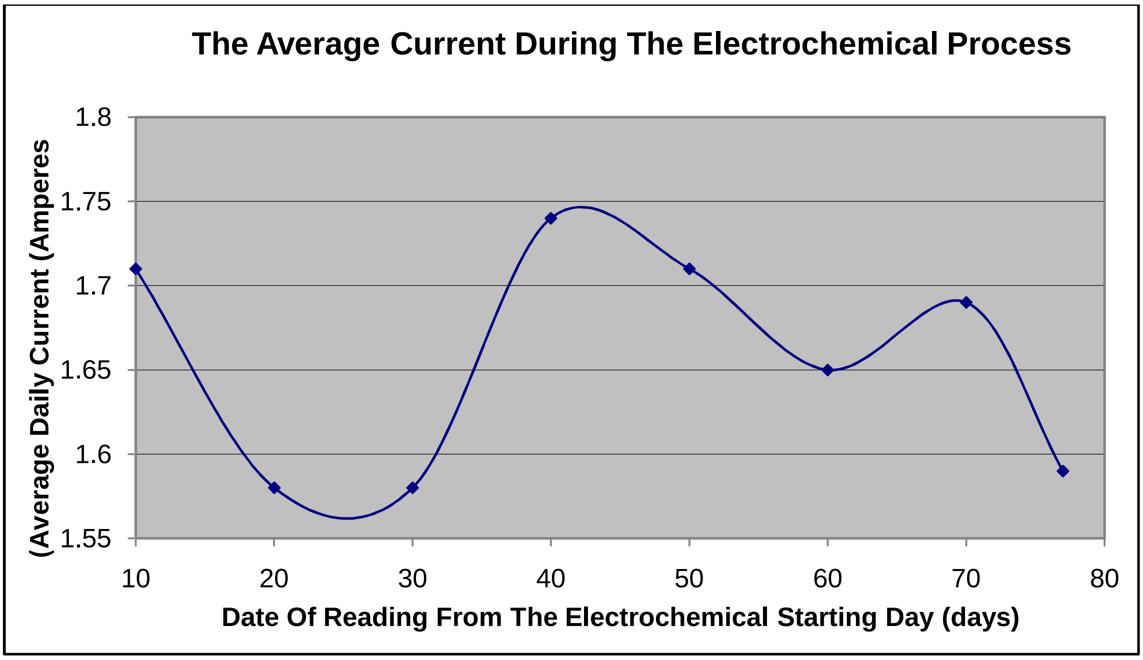


Figure 6.1: The Average Current during the Electrochemical Process

In fact these records are very close together with a standard deviation of 0.065, and no clear relation can be noted from them. These randomly changes may refer to many reasons. Concrete electrical resistance, steel factory components and the corrosion rate may be the main factors affecting these readings.

Anyhow, these records are necessary to understand this electrochemical process and can be used for further studies to understand the effect of impressing a direct current on the corrosion of steel embedded in concrete which was not covered by the scope of our study.

6.3. Crack Development of Beams due to the Corrosion Process

The visual inspections of corroded beams after the end of the corrosion process show longitudinal cracks parallel to the flexural reinforcement as seen in Figure 5.4.

In fact the orientations of these cracks are normal and refer to the fact that the rust occupies a much larger volume than the original steel. Because concrete is weak in tension these bursting forces quickly cause the concrete to crack parallel to the reinforcement direction. However, rust is a complex mixture of oxides, and hydroxides and hydrated oxides of steel have a volume ranging from twice to about six times that of the steel consumed to produce it. The magnitude of the rust generated are various according to various steel oxides generated.

6.4. The Influence of Corrosion on Steel Reinforcement

The steel tensile test is performed to establish the mechanical properties of the steel. In this investigation, the main objective is to establish any changes in the mechanical properties of the steel. Three corroded bars were tested using the standard tension test to determine its elongation. An average elongation of 5% is obtained which is only 16.7% of the normal steel bar elongation.

This result indicates that corrosion is not only reducing the reinforcement cross section area, but also is changing its mechanicals properties. The reduction in corroded reinforcement elongation by more than 83% compared with the normal steel reinforcement is considered a big problem facing corroded beams. This reduces the ductility of the steel and may cause a sudden failure for beams.

6.5. Flexural Beam Capacity Analysis

Figure 6.2 show the beam failure load capacity of beams, the following results can be noted:

Corroded beams show an average reduction by more than 28% of its failure loads compared with control beams. This indicates that the corrosion of reinforcement directly affect the flexural capacity of beams. That refers to the reduction in reinforcement cross section area due to corrosion.

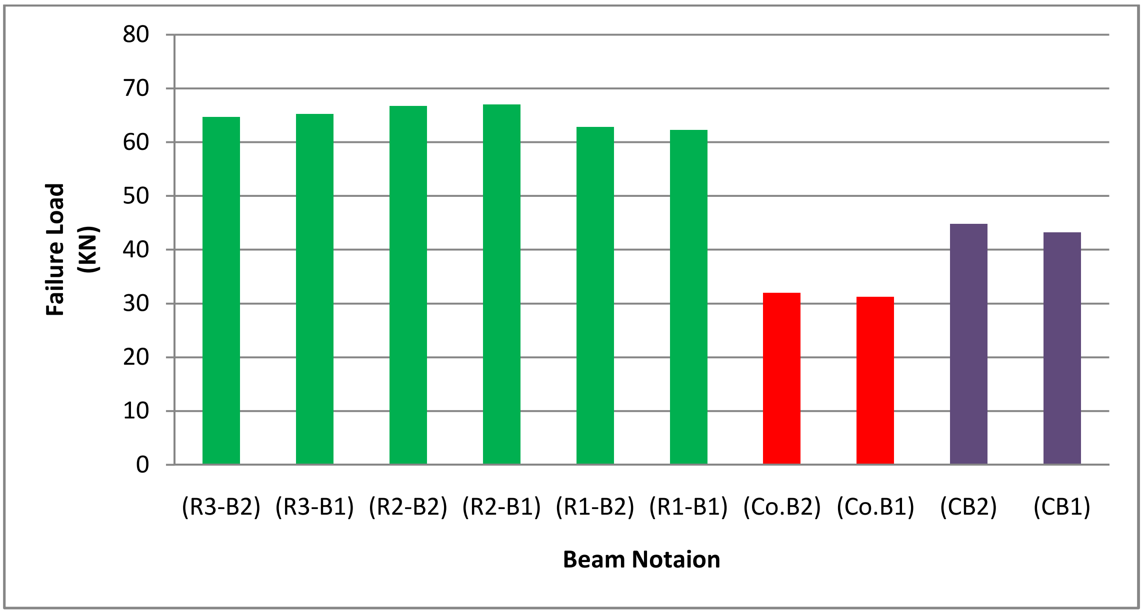


Figure 6.2: Beams Failure load

Repaired beams show an average increments by more than 47% of its failure load capacity compared with control beams and more than 104% compared with the corroded beams failure capacity. This indicates that the repairing process satisfies its aim to increase the flexural capacity of the corroded beams.

There was no significant difference in the flexural capacity of repaired beams according to its repaired material layer. This indicates that the applied concrete layer type does not affect the flexural capacity of beams. This refers to the fact that the tested new concrete layer was in the tension zone; however the use of these types of materials are important for other reasons. Applying an impermeable layer to protect the reinforcement from further corrosion and the durability of sound beams without corrosion may be the main parameter affecting the type of new materials layers.

6.6. Structural Analysis of Repaired Beams

To understand the behavior of repaired beams in the flexural tests, the following analysis was carried out:

- Average failure load for the corroded beams= $\frac{73.8+75.7}{2}$ = 74.75 KN Eq. 6.1

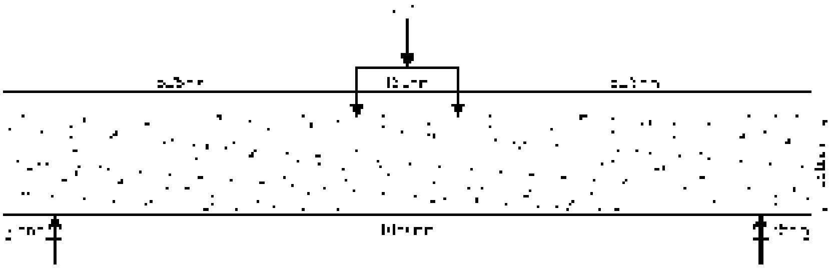


Figure 6.3: Section Details

- From Figure 6.3, $M_n = 525 \times P - 75 \times P$ (Equation 6.2)
- $P_{corroded-actual} = 31.6 \div 2 = 15.8 KN$

Where 2×P=average failure load for corroded beams (Equation 6.3)

- From Eq. 6.2 and 6.3 $M_{n\ corroded\text{-}actual} = 7110$ KN. mm
- $M_{n-actual} = T_{corroded} \times (d - \frac{a}{2})$

Where

- $d = 150 - 10 - 6 - 5 = 129$ mm
- $a = \dfrac{T}{0.85 \times f_{c'} \times b}$ (b = 100 mm, $f_{c'}$= 28.1 MPa)
- $T_{corroded} =$ 61192.81 N
- $M_{n-design-\mathrm{repaired}} = (T_{corroded} + T_{add}) \times (d_{repaired} - \dfrac{a_{repaired}}{2})$
- $T_{add} = A_s \times f_y = 100.53 \times 545 = 54789.4$ N
- $d_{repaired} = 180-24-8-4 = 144$ mm
- $a_{repaired} = \dfrac{(T_{corroded} + T_{add})}{0.85 \times f_{c'} \times b} = 48.56$ mm
- $M_{n-design-\mathrm{repaired}} = 13.885 \times 10^6$ N.mm
- From Equation 6.2, $P_{n\text{-}design\text{-}repaired} = 30.86$ KN
- From Table 5.2, the average P_{actual} for the repaired beams = 32.38 KN

This analysis indicates that the new capacity of repaired beams can be calculated according to the ACI-08 design codes.

This indication can be used by engineers to establish the required amount of new steel reinforcement needed to strengthen the corroded beams reinforcement.

6.7. Analysis of Deflection of Beams

From Figure 6.4 -which shows the deflections of beams during the flexural test- the following results, can be noted:

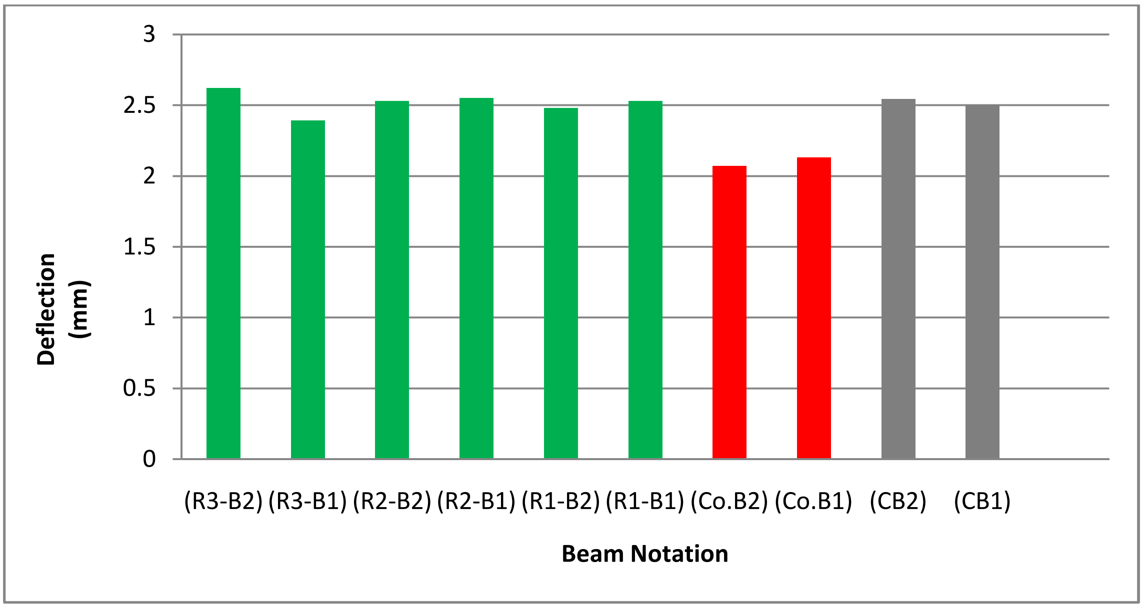


Figure 6.4: Deflection of beams at failure load

The average deflection of the corroded beams was 2.1 mm which is less than the control beam average deflection by more than 16%. Another note that the flexural steel bars of the corroded beams were cut during the flexural test that indicates bad ductility behavior for them and that is attributed to the mechanicals changes is occurring to the corroded beams flexural reinforcements as discussed in Section 6.4. However, this should not influence the ductility of the repaired beams since the new steel bars with good ductility were added in the proposed repaired techniques.

The average deflection of the repaired beams was 2.517mm and for the control beams were 2.52 mm, which indicates that the behavior of the repaired beams was very close to the control beams during the flexural test, which indicates that the repairing process satisfies its aim.

6.8. Analysis of Crack development due to the flexural tests

From Figures 5.8 through 5.17, the following results can be noted:

The crack development for the repaired beams shows normal flexural failure crack pattern compared with the control beams.

No separation was obtained during the flexural test between the old concrete and the new applied repair material patch layer for all of the applied repair materials. This indicates that the repaired beams perform as one element regardless the kind of repair layer used. This may be a result for using enough shear connectors which make the repaired beams perform as newly construction beams.

There were no shear or inter laminar shear crakes appearing on the repaired beams. This is due to using shear connectors as discussed earlier. These shear connectors work also as stirrups and prevent shear failure.

The corroded beams failure was due to flexural, that’s because the deterioration of steel was for the bottom steel reinforcement.

6.9. Result Summary

1. There were various changes of the current during the electrochemical process, these randomly changes refer to many reasons. Concrete electrical resistance, steel factory components and the corrosion rate may be the main factors affecting these readings.
2. The flexural results of the corroded beams reduced by percent of 28.2 % compared with the control beams results, and they were less ductile from the control beams. Since they gave less in deflection results and the steel reinforcement were cutoff during the test.
3. The flexural results of the repaired beams increased by a percent of 47.3% compared with the control beams results. They also, present a normal flexural test deflection and behavior during the test.

Table 6.1: Summary of the Flexural Test Results

No	Description	Count of Beams	Average Deflection at Failure (mm)	Average Load Value (2P) at Failure (KN)	Percentage Compared with the Designed Value of Control Beam(%)	Percentage Compared with the Designed Value of The Repaired Beams(%)
1	Control Beams	2	2.520	44.0	104	---
2	Corroded beams	2	2.100	31.6	75	---
3	Repaired beams	6	2.517	64.8	153	105

4. Shear failure was prevented using the shear connectors which act as stirrups.
5. Inter laminar shear failure at the interface between the old and the new concrete layer was prevented using the shear connectors.
6. The crack development of both control and repaired beams appears to behave normal for flexural failure.

Chapter 7: Conclusions and Recommendations

7.1. Conclusions

Several parameters were investigated in the undertaken research to understand the influence of the corrosion process on the flexural capacity of corroded beams, and the structural performance for the repaired corroded beams. An experimental test program consisted of designing, constructing, corroding, repairing and flexural testing of fourteen reinforced concrete beams was applied. Two of these beams were tested as control beams, two were tested as corroded beams while six of beams were tested as repaired beams. The other four beams were used as standby beams and to investigate the corrosion rate for the corroded beams.

In order to accelerate the corrosion process, an electrochemical system which depends on the concept of Faraday's second law was used. The aim of accelerating the corrosion was to force steel reinforcement to act as anode in a galvanic cell. Which can be done by immersing beams in an aqueous solution and connecting the steel reinforcement bars with positive DC current generator to act as anode while connecting the negative power supply to external steel rods immersed in the aqueous solution to act as cathode. This consist electric circuit and force steel ions to translate from anode and to cathode.

However, after the end of the corrosion process, six corroded beams were repaired. The main idea of repairing of corroded beams was cutting the unsound concrete and cleaning both the concrete surface and steel reinforcement bars. Additional flexural longitudinal steel reinforcement's bars were added and fixed with shear connectors. These shear connectors were used for fixing the new reinforcement with the corroded reinforcement and work also as stirrups and to prevent both diagonal and inter laminar shear failure. The second step was applying a new layer for the bottom sides of beams, for this purpose three repair materials were applied in order to study the effect of the used repairing materials layer on the flexural capacity and the crack development of the repaired beams.

Control, corroded and repaired beams were tested as simply supported beams with two concentrated loads. The flexural capacity, deflection and crack development of corroded, repaired and control beams were a part of the studies. Samples of the reinforcing bars were tested using the standard tension test to determine the properties of the corroded bars. The following conclusions are drawn:

1. Several longitudinal cracks formed parallel to the flexural reinforcement along both bottom and sides of corroded beams due to corrosion process.

2. The corroded steel bars experienced a reduction in its elongation by more than 83% compared with the normal steel bar elongation.

3. The flexural capacity of the corroded beams reduced by percent of 28.2 % compared with the control beams and showed a noted reduction in its ductility behavior during the flexural test, since the steel reinforcements was cutoff during the test at an average deflection of 2.1 mm.

4. The flexural capacity of the repaired beams increased by percent of 47.3% compared with the control beams and by a percent of 105% compared with the corroded beams.

5. The repaired beams showed good ductility behavior during the flexural test and performed as sound constructed beams with its flexural capacity and crack development.

6. The types of repairing materials used to applied new batch repairing layer to corroded beams did not affect the flexural performance of repaired beams; hence it may be important to inhabit the corrosion process in the future.

7.2. Recommendations for Future Research

1. Very few exponential investigations are available on the behavior of reinforced concrete corroded beams subjected to bending action. More experimental investigations are necessary to monitor and assess the structural condition in corroded concrete beams and investigating a repairing techniques to deal with the corrosion problem from structural point of view.
2. A more detailed study of the composite interaction between the steel and the concrete and the bond deterioration are required to understand the action of the expansion of the corrosion products and the reduction of the reinforcing bar section.
3. A larger number of specimens should be tested with the improvement of the electrochemical set-up to increase the corrosion rate and to decrease the corrosion time.
4. The increase in the number of specimens would permit to study the influence of different parameters such as the reinforcing bar diameter, the type of loading, the concrete cover thickness, the concrete strength and the steel yield strength.
5. More structural repairing techniques should be investigated and tested to develop structural strengthening methods for corroded elements and to enhance its flexural and shear capacity.

6. Further future research is needed to investigate the durability of the applied repairing techniques and how to prevent the future corrosion process.

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APPENDIX" 1": Repair Materials Specifications

Lokset*

Mix and place polyester resin anchoring grouts

Uses

High strength corrosion resistant heavy duty anchoring. These anchors include bolts, tendons or dowels in drilled or formed holes located in concrete masonry, brickwork or natural rock.

Permanent installation of reinforcement starter bars, foundation bolts, ballustrading, barriers and safety fences, railway tracks, ground anchors for towers, cranes, dock sills.

Advantages

- Ultra rapid strength gain
- Vibration resistant
- Corrosion resistant
- Tolerant of wet and damp conditions
- Can be placed underwater and in damp conditions
- Non expansive
- Pre-weighed components ensure consistent performance of the grout

Description

The Lokset range consists of pre-measured, two pack, filled polyester resin grouts. The two components are polyester resin and a catalysed filler. The range comprises three standard versions.

Lokset S40, which contains blended fine aggregates, has a gel time of 40 minutes at 20°C. The product is designed for use where annular gaps are up to 25 mm greater than the diameter of the fixing.

Lokset L40, which contains 2 mm aggregate, has a gel time of 40 minutes at 20°C. The product is designed for use in situations where annular gaps are 20-50 mm greater in diameter than the diameter of the fixing.

Lokset P40 contains thixotropic chemicals which allow the product to be pumped into overhead or horizontal holes. It has a gel time of 40 minutes at 20°C and should be used for annular gaps of up to 25 mm.

Lokset S20 and P20 have shorter gel times suited to cold weather working conditions.

Lokset S80 and P80 have longer gel times suited to hot weather working conditions.

This type of installation is a permanent fixing.

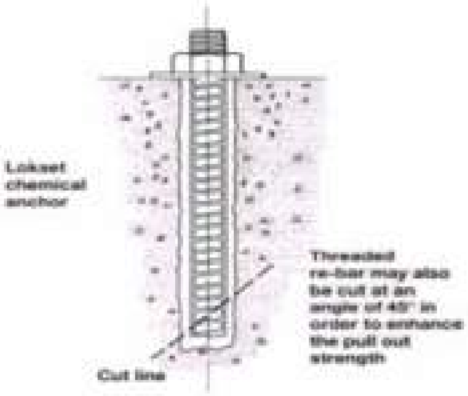

Figure 1: Threaded rebar with Lokset chemical anchor.

This type of installation is semi- permanent.

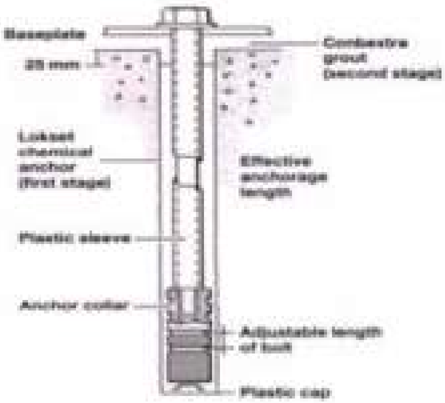

Figure 2: Hexagon headed bolt with removable nut and tie bar with Lokset chemical anchor.

Technical support

Fosroc offers a comprehensive range of high performance, high quality, construction products all backed by BS 5750 certification. Fosroc offers a technical support package to specifiers and contractors which includes computer-aided design (CAD) standard details as well as technical advice from staff with an unrivalled experience in the industry.

Design criteria

The version of Lokset grout to be used will depend upon ambient temperature and anchor conditions.

The high strength of the cured resin permits strong anchors to be created. The ultimate bond strength developed depends upon:

Strength of host material

Length of resin bond to bar

Hole preparation and formation

Type and dimension of bar

The following formula may be used to determine the minimum depth of installation for Type 1 rebar bolts, to ensure the shear stress within the concrete is kept within the limits set out in BS 8110.

Minimum hole depth (mm) $= \dfrac{0.6f_y}{f_b \mu D_h} \cdot \dfrac{\mu D_b^2}{4} = \dfrac{0.15\, f_y D_b^2}{f_b D_h}$

where:

f_y is characteristic yield strength of steel (N/mm²)

f_b is the bond stress for concrete approx. 2-3 (N/mm²)

D_h is the hole diameter (mm)

D_b is the bar diameter (mm)

This formula is used typically as shown in Table 1.

To estimate the force that one bar can take for a given length of embedment, the following formula can be used,
$F = 5.23\, D_s\, L_{emb}\, f_b$ (kN).

Typical values for 420 MPa steel and 25 MPa concrete are given in table 1.

Properties

Typical results			
Gel time			
Temperature °C:	10	20	30
Minutes:	100	40	15
	S40	**L40**	**P40**
Compressive strength (N/mm²)			
BS 6319 part 2 1983			
1 hour:	42	60	50
3 hours:	75	83	75
24 hours:	95	101	93
7 days:	103	106	102
Tensile strength (N/mm²)			
BS 6319 part 7 1985			
3 days:	12	14	11
7 days:	14	14	15
Flexural strength (N/mm²)			
BS 6319 part 3 1990			
3 days:	26	25	27
7 days:	33	29	30
Flash point:	29°C	29°C	29°C

Table 1

Bar Diameter (mm)	Hole Diameter (mm)	Tensile load on the steel bar, $F = 5.23\, D_s\, L_{emb}\, f_b /1000$ (kN)										For Full Steel Utilization (kN)	
		L_{emb} = 100 mm	120 mm	160 mm	200 mm	250 mm	300 mm	350 mm	400 mm	500 mm	600 mm	L_{emb} (mm)	F (kN)
8	12	12.5	15.0	20.1	FULL STEEL UTILIZATION							168	21.1
10	14	14.6	17.5	23.4	29.2							226	33.0
12	18	18.8	22.6	30.1	37.6	47.9						252	47.4
14	20	20.9	25.1	33.4	41.8	52.3	62.7					309	64.6
16	22	23.0	27.6	36.8	46.0	57.5	69.0	80.5				388	84.5
18	26	27.2	32.6	43.5	54.4	68.0	81.5	95.1				393	106.8
20	28	29.2	35.1	46.8	58.5	73.2	87.8	102.5	117.1			452	132.3
22	30	31.3	37.6	50.2	62.7	78.4	94.1	109.8	125.5	156.9		509	159.7
25	35	36.6	43.9	58.5	73.2	91.5	109.8	128.1	146.4	183.0		563	206.1

Specification clauses

Supplier specification

Mix and place grout where shown on the drawings shall be Lokset manufactured by Fosroc. It shall be used in accordance with the manufacturer's current application instructions.

Performance specification

Mix and place grout where shown on the drawings shall be a two component polyester resin system. When fully cured it shall exhibit a compressive strength in excess of 90 N/mm^2, a flexural strength in excess of 28 N/mm^2 and a tensile strength in excess of 14 N/mm^2 at 7 days.

The storage, handling and placement of the grout must be in strict accordance with the manufacturer's instructions.

Application instructions

Hole preparation and formation

Three methods of hole formation are possible.

1. Optimum performance of Lokset grouts requires rough sided, dust free holes. These can be made by using rotary percussive drills followed by oil-free air or water flushing.
2. Diamond drilled holes should be underreamed.
3. Cast holes should be of inverse dovetail configuration. If parallel sided holes are cast, they should be rough enough to provide an adequate mechanical key.

Bar preparation

1. All bars should be deformed. This will ensure good bond between bar and grout.
2. They should be degreased and any mill scale or flaky rust removed.

Mixing

Only a complete pack of resin and catalysed filler should be mixed in one operation. Mixing may be carried out manually or mechanically. When a smooth, even consistency is achieved, the grout is ready for use. It must be placed well within the gel time of the grout.

Placing

The mixed grout should be poured or pumped steadily into the prepared anchor holes. The anchor bar should then be pressed into the hole to the required depth. Slight agitation of the bar will greatly assist in achieving a complete bond.

The bar should be left undisturbed in the required position until the grout has hardened fully.

Cleaning

Any mixing drums, pumps etc should be cleaned within the pot life of the grout. Fosroc Solvent 102 is recommended for this purpose.

Limitations

1. At permanent operating temperatures above 40°C, creep of the cured grout may become significant.
2. Resin anchors should not be used where structural load bearing performances have to be maintained in anchors subjected to fire conditions.
3. For use at temperatures below 5°C, seek advice from Fosroc.

Estimating

Pack sizes

Lokset S40, S20:	Can of resin and plastic bag of catalysed filler 2.5 litres and 1.2 litres.
Lokset L40:	Can of resin and plastic bag of catalysed filler 8 litres.
Lokset P40, P20:	Can of resin and plastic bag of catalysed filler 2.5 litres and 1.2 litres.

Volume of Lokset Grout required in ml

$$= \frac{\mu}{4000} \cdot (D_h^2 - D_b^2) \cdot L_{emb}$$

No. of pulls using the standard gun and tubes of Fosroc can be easily estimated knowing that each pull equal to 8 ml.

Estimated volume and No. of pulls are given in table 2. (Wastage factor of 30% is allowed).

Table 2

Bar Diameter (mm)	Hole Diameter (mm)	Volume Required (ml) No. of Pulls: L = 100 mm	120 mm	160 mm	200 mm	250 mm	300 mm	350 mm	400 mm	500 mm	600 mm	For Full Utilization: L = mm	Volume (ml) No. of Pulls
8	12	8.2	9.8	13.1	16.3							168	13.8
		1.0	1.5	2.0	2.0								1.5
10	14	9.8	11.8	15.7	19.6			FULL STEEL UTILIZATION				226	22.2
		1.5	1.5	2.0	2.5								2.5
12	16	18.4	22.1	29.4	36.8	45.9						252	46.3
		2.5	3.0	4.0	5.0	6.0							6.0
14	20	20.8	25.0	33.3	41.7	52.1	62.5					309	64.4
		2.5	3.5	4.5	5.5	6.5	8.0						8.5
16	22	23.3	27.9	37.2	46.6	58.2	69.8	81.5				368	85.7
		3.0	3.5	5.0	6.0	7.5	9.0	10.5					8.5
18	26	35.9	43.1	57.5	71.9	89.8	107.8	125.5				393	141.3
		4.5	5.5	7.5	9.0	11.5	13.5	16.0					14.0
20	28	39.2	47.0	62.7	78.4	98.0	117.6	137.2	156.8			452	177.2
		5.0	6.0	8.0	10.0	12.5	15.0	17.5	20.0				18.0
22	30	42.5	51.0	68.0	84.9	106.2	127.4	148.7	170.0	212.4		509	216.2
		5.5	6.5	8.5	11.0	13.5	16.0	19.0	21.5	26.5			22.0
25	35	61.3	73.5	98.0	122.5	153.1	183.8	214.4	245.0	306.3		563	344.9
		8.0	9.5	12.5	15.5	19.5	23.0	27.0	31.0	38.5			34.5

Storage

Shelf life

All products have a shelf life of 12 months at 20°C if kept in a dry store in the original, unopened packs.

The shelf life will be reduced at higher ambient temperatures.

Storage conditions

Store in dry conditions away from high temperatures and high humidity. Keep away from sources of heat and naked flames.

Precautions

Health and safety

Some people are sensitive to resins and solvents. Avoid contact with skin and eyes. Ensure adequate ventilation and avoid inhalation of vapours. Wear suitable protective clothing, gloves and eye/face protection.

Barrier creams provide additional skin protection. Should accidental skin contact occur, wash immediately with a resin removing cream, followed by soap and water. In case of contact with eyes, rinse immediately with plenty of water and seek medical advice - do not induce vomiting.

For additional information see the relevant Material Safety Data Sheet.

Fire

Lokset products and Fosroc Solvent 102 are flammable. Keep away from sources of ignition - no smoking. In the event of fire extinguish with CO_2 or foam.

*** Denotes the trademark of Fosroc International Limited**

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Important note

Fosroc products are guaranteed against defective materials and manufacture and are sold subject to its standard terms and conditions of sale, copies of which may be obtained on request. Whilst Fosroc endeavours to ensure that any advice, recommendation, specification or information it may give is accurate and correct, it cannot, because it has no direct or continuous control over where or how its products are applied, accept any liability either directly or indirectly arising from the use of its products, whether or not in accordance with any advice, specification, recommendation or information given by it.

FOSROC

Nitobond EP*

Epoxy resin concrete bonding agent

Uses

For bonding fresh wet cementitious materials to existing cementitious surfaces. For use on horizontal surfaces or on vertical surfaces where mortar or concrete can be supported by formwork. The long 'open' life makes it suitable for use with formwork or where additional steel reinforcement has to be fitted. The product is ideal for roads, bridges, pavements, loading bays and factories, and for bonded or granolithic floor toppings. Nitobond EP is equally suited to internal and external applications.

Nitobond EP may also be used as part of a repair system where a substrate/repair barrier is required or where the substrate is likely to remain permanently damp or wet.

Advantages

- Can be applied on to dry or damp substrates
- Exhibits high mechanical strength
- Positive adhesion - exceeds that of the tensile strength of the host concrete
- Slow cure allows time to erect steel reinforcement and formwork
- Solvent-free - can be used in enclosed locations

Standards compliance

Nitobond EP complies with ASTM C881: Type II, Grade 2 Classes B&C.

Design criteria

Nitobond EP is designed with an overlay time of 24 hours at 20°C, 12 hours at 35°C and 5 hours at 45°C, making it more suitable for use where additional steel reinforcement and formwork has to be fitted or where temperatures are high. The minimum application temperature for Nitobond EP is 5°C. Consult the local Fosroc office for further information.

Properties

Test method	Typical result
Compressive strength (BS 6319, Pt 2):	50 N/mm^2 @ 7 days
Tensile strength (BS 6319, Pt 7):	20 N/mm^2 @ 7 days
Flexural strength (BS 6319, Pt 3):	35 N/mm^2 @ 7 days
Shear strength (BS 6319, Pt 4):	25 N/mm^2 @ 7 days
Adhesive bond to concrete:	In general, the bond will always exceed the tensile strength of the host concrete.

The following properties were measured at 20°C:

Pot life:	7 - 10 hours
Initial hardness:	48 hours
Full cure:	7 days
Maximum overlay time:	24 hours

Description

Nitobond EP is based on solvent-free epoxy resins containing pigments and fine fillers. It is supplied as a two-component material in pre-weighed quantities ready for on-site mixing and use. The 'base' component is white and the 'hardener' component is green, providing visual evidence that adequate mixing has been achieved.

Technical support

Fosroc offers a comprehensive range of high performance, high quality concrete repair and construction products. In addition, Fosroc offers a technical support package to specifiers, end-users and contractors, as well as on-site technical assistance in locations all over the world.

Specification clause

Epoxy bonding agent

The bonding agent shall be Nitobond EP, a two-component solvent-free epoxy resin. The two components shall be differentially pigmented in order to ensure visually that correct mixing has taken place prior to the application. The product shall achieve at 7 days 50 N/mm^2 compressive strength, 20 N/mm^2 tensile strength, 35 N/mm^2 flexural strength and 25 N/mm^2 shear strength. The adhesive bond to the concrete substrate shall exceed the tensile strength of the host concrete.

Instructions for use

Preparation

Clean all surfaces and remove any dust, unsound material, plaster, oil, paint, grease, corrosion deposits or algae. Roughen the surfaces, remove any laitance and expose the aggregate by light scabbling or grit-blasting.

Oil and grease deposits should be removed by steam cleaning, detergent scrubbing or the use of a proprietary degreaser. The effectiveness of decontamination and soundness of the substrate should then be assessed by a pull-off test.

Mixing

Any steel reinforcement and formwork should be prepared, cut to size and shape, and made ready for assembly before mixing commences.

Care should be taken to ensure that Nitobond EP is thoroughly mixed. The 'hardener' and 'base' components should be stirred separately before mixing to disperse any settlement. The entire contents of the 'hardener' tin should then be poured into the 'base' tin and the two materials thoroughly mixed using a suitable slow-speed drill and mixing paddle for 2 minutes until a fully uniform colour is obtained. The sides of the tin should then be scraped and mixing should continue for a further 2 minutes.

To facilitate mixing and application at temperatures below 20°C, the separate components should be stored in an environment heated to 20°C and only removed immediately before use.

Application

Nitobond EP should be applied as soon as the mixing process has been completed. It should be brush or spray-applied to the prepared surfaces.

The new concrete or screed may be applied to the coated substrate up to 24 hours after application at 20°C, 12 hours at 35°C and 5 hours at 45°C. However, the coated substrate should be left for one hour before the new concrete or screed is placed.

Where Nitobond EP is to be used as part of a repair system to form a substrate/repair barrier, care should be taken to achieve an unbroken coating. One coat should be applied and allowed to gel. A second coat should be applied and used as the bonding coat. In some situations (e.g. sprayed concrete repairs) it may be advantageous to scatter dust-free sharp sand over this coat and leave to harden.

As soon as the Nitobond EP has been applied, any required steel reinforcement and/or formwork should be erected and fixed securely in place.

Low temperature working

The minimum application temperature is 5°C. In temperatures below 15°C, the separate components should be heated in warm water (up to 25°C) or stored in heated environment for 12 hours before use. These measures will facilitate mixing and application. Normal precautions for winter working with cementitious materials should be adopted.

High temperature working

At ambient temperatures above 30°C, the material should be stored in the shade or in an air-conditioned environment for 12 hours before use.

Cleaning

Nitobond EP should be removed from tools, equipment and mixers with Fosroc Solvent 102 immediately after use. Hardened material can only be removed mechanically.

Limitations

Nitobond EP is formulated for application to clean, sound concrete.

Nitobond EP should not be applied over existing coatings.

Application should not be undertaken if the temperature is below 5°C or falling.

Although Nitobond EP may be applied to damp concrete, there must be no standing or running water.

Estimating

Supply

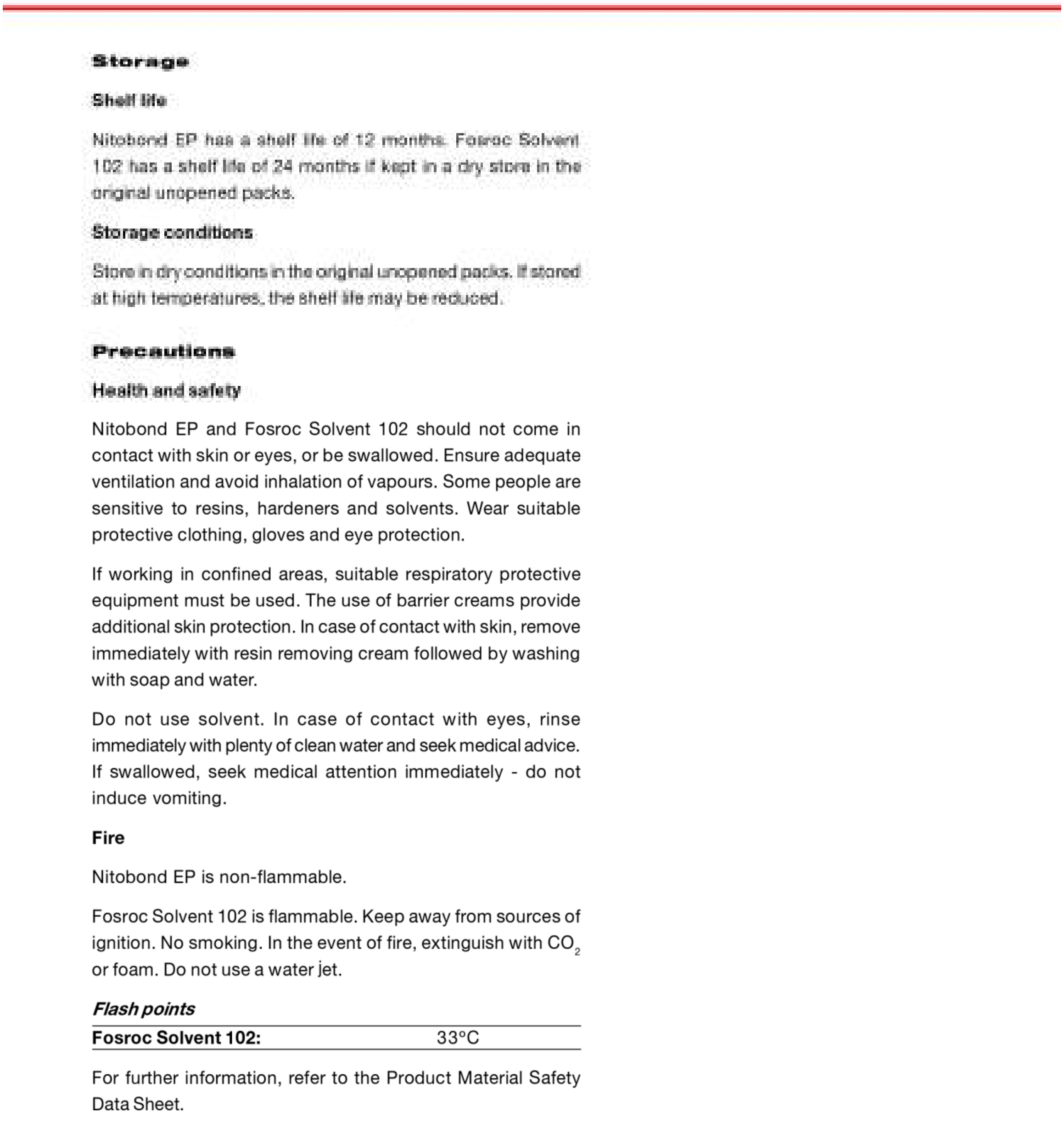
Nitobond EP slow set:	1 and 5 kg packs
Fosroc Solvent 102:	5 litre cans

Coverage

Nitobond EP:	3 to 3.5 m²/kg

Note: The coverage figures for Nitobond EP products are theoretical - due to wastage factors and the variety and nature of possible substrates, practical coverage figures will be reduced.

Nitobond EP 'standard ' can be made available when specifically requested.



Storage

Shelf life

Nitobond EP has a shelf life of 12 months. Fosroc Solvent 102 has a shelf life of 24 months if kept in a dry store in the original unopened packs.

Storage conditions

Store in dry conditions in the original unopened packs. If stored at high temperatures, the shelf life may be reduced.

Precautions

Health and safety

Nitobond EP and Fosroc Solvent 102 should not come in contact with skin or eyes, or be swallowed. Ensure adequate ventilation and avoid inhalation of vapours. Some people are sensitive to resins, hardeners and solvents. Wear suitable protective clothing, gloves and eye protection.

If working in confined areas, suitable respiratory protective equipment must be used. The use of barrier creams provide additional skin protection. In case of contact with skin, remove immediately with resin removing cream followed by washing with soap and water.

Do not use solvent. In case of contact with eyes, rinse immediately with plenty of clean water and seek medical advice. If swallowed, seek medical attention immediately - do not induce vomiting.

Fire

Nitobond EP is non-flammable.

Fosroc Solvent 102 is flammable. Keep away from sources of ignition. No smoking. In the event of fire, extinguish with CO_2 or foam. Do not use a water jet.

__Flash points__

Fosroc Solvent 102:	33°C

For further information, refer to the Product Material Safety Data Sheet.

Additional Information

Fosroc manufactures a wide range of complementary products which include :

- waterproofing membranes & waterstops
- joint sealants & filler boards
- cementitious & epoxy grouts
- specialised flooring materials

Fosroc additionally offers a comprehensive package of products specifically designed for the repair and refurbishment of damaged concrete. Fosroc's 'Systematic Approach' to concrete repair features the following :

- hand-placed repair mortars
- spray grade repair mortars
- fluid micro-concretes
- chemically resistant epoxy mortars
- anti-carbonation/anti-chloride protective coatings
- chemical and abrasion resistant coatings

For further information on any of the above, please consult your local Fosroc office.

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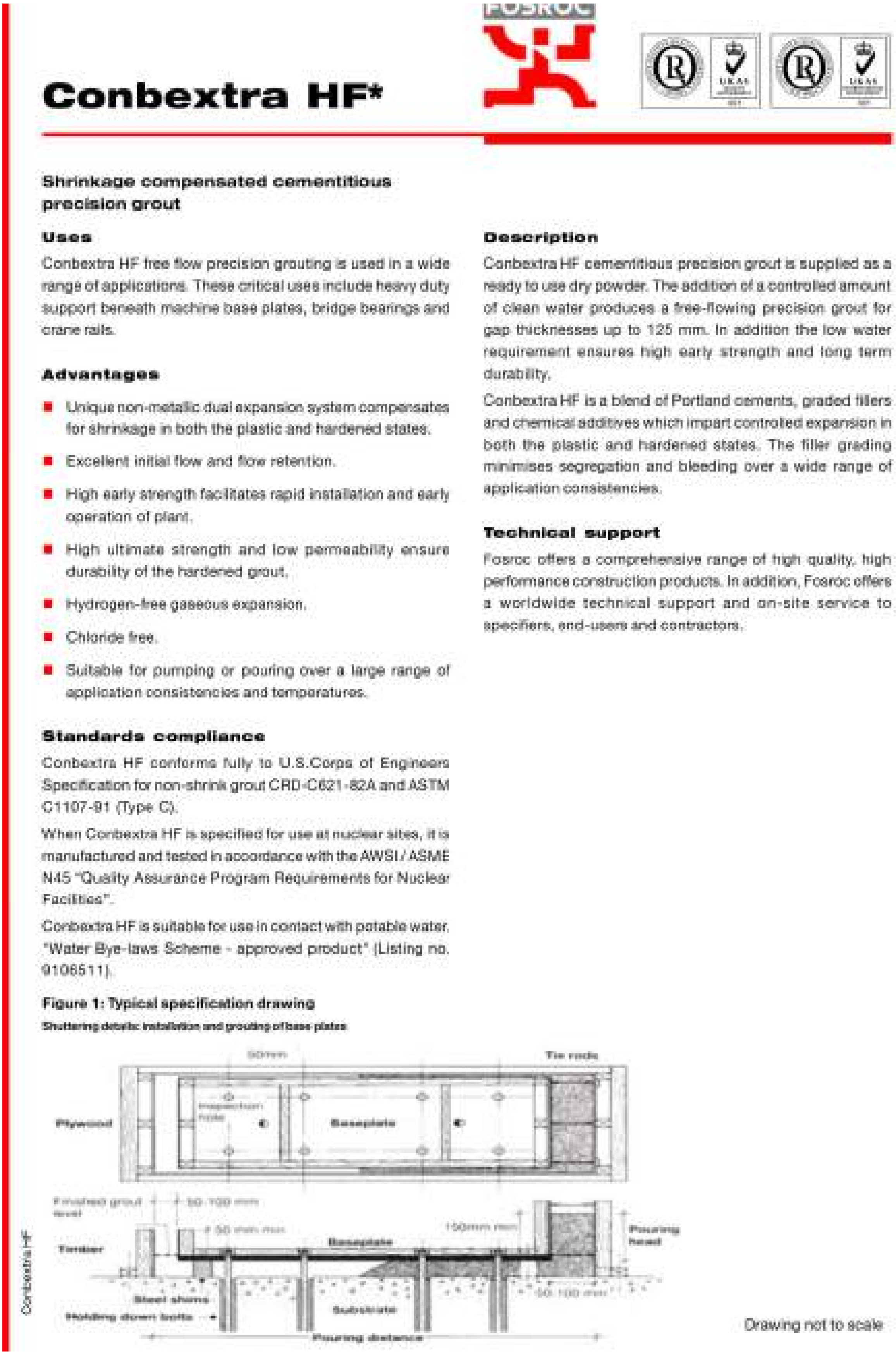
Important note

Fosroc products are guaranteed against defective materials and manufacture and are sold subject to its standard terms and conditions of sale, copies of which may be obtained on request. Whilst Fosroc endeavours to ensure that any advice, recommendation, specification or information it may give is accurate and correct, it cannot, because it has no direct or continuous control over where or how its products are applied, accept any liability either directly or indirectly arising from the use of its products, whether or not in accordance with any advice, specification, recommendation or information given by it.



Conbextra HF*

Shrinkage compensated cementitious precision grout

Uses

Conbextra HF free flow precision grouting is used in a wide range of applications. These critical uses include heavy duty support beneath machine base plates, bridge bearings and crane rails.

Advantages

- Unique non-metallic dual expansion system compensates for shrinkage in both the plastic and hardened states.
- Excellent initial flow and flow retention.
- High early strength facilitates rapid installation and early operation of plant.
- High ultimate strength and low permeability ensure durability of the hardened grout.
- Hydrogen-free gaseous expansion.
- Chloride free.
- Suitable for pumping or pouring over a large range of application consistencies and temperatures.

Standards compliance

Conbextra HF conforms fully to U.S.Corps of Engineers Specification for non-shrink grout CRD-C621-82A and ASTM C1107-91 (Type C).

When Conbextra HF is specified for use at nuclear sites, it is manufactured and tested in accordance with the AWSI / ASME N45 "Quality Assurance Program Requirements for Nuclear Facilities".

Conbextra HF is suitable for use in contact with potable water. "Water Bye-laws Scheme - approved product" (Listing no. 9106511).

Figure 1: Typical specification drawing

Shuttering details: installation and grouting of base plates

Drawing not to scale

Description

Conbextra HF cementitious precision grout is supplied as a ready to use dry powder. The addition of a controlled amount of clean water produces a free-flowing precision grout for gap thicknesses up to 125 mm. In addition the low water requirement ensures high early strength and long term durability.

Conbextra HF is a blend of Portland cements, graded fillers and chemical additives which impart controlled expansion in both the plastic and hardened states. The filler grading minimises segregation and bleeding over a wide range of application consistencies.

Technical support

Fosroc offers a comprehensive range of high quality, high performance construction products. In addition, Fosroc offers a worldwide technical support and on-site service to specifiers, end-users and contractors.

Properties

The following results were obtained at a water:powder ratio of 0.19 and a temperature of 25°C.

Test method for Typical result

Compressive strength		
BS 1881: part 116 1983:	16 N/mm^2 @ 1 day	
	40 N/mm^2 @ 7 days	
	52 N/mm^2 @ 14 days	
	58 N/mm^2 @ 28 days	
	76 N/mm^2 @ 180 days	
Flexural strength		
BS 4551 1980:	2.5 N/mm^2 @ 1 day	
	8.0 N/mm^2 @ 7 days	
	9.5 N/mm^2 @ 14 days	
	10.0 N/mm^2 @ 28 days	
	11.0 N/mm^2 @ 180 days	
Flow characteristics		
(Efflux time) CRD-C Cone:	19 - 25 seconds	
Setting time		
BS 4550 part 3 1978:		
Initial set:	5.5 hours	
Final set:	7.5 hours	
Time for expansion		
Plastic state:	**Start**	15 minutes
	Finish	initial set
Hardened state:	**Start**	initial set
	Finish	up to 28 days
Fresh wet density:	Approximately 2200 kg/m^3 depending on actual consistency used.	
Young's modulus		
ASTM C-469-83:	29 kN/mm^2	
Expansion characteristics:	An expansion of up to 2% when measured according to ASTM C 827 overcomes plastic settlement in the unset material. Longer term expansion in the hardened state is designed to comply with the requirements of ASTM C 1107-91 to compensate for drying shrinkage.	

Specification clauses

Performance specification

All precision grouting (specify details and areas of application) must be carried out with a pre-packaged cement based product, which is non-metallic and chloride-free.

It shall be mixed with clean water to the required consistency and not exhibit bleed or segregation.

A volumetric expansion of up to 2% shall occur while the grout is in a plastic state by means of a gaseous, hydrogen-free system. The grout must also be compensated for shrinkage in the hardened state.

The compressive strength of the grout must exceed 38 N/mm^2 at 7 days and 56 N/mm^2 at 28 days.

The grout shall fully conform to the requirements of US Army Corps of Engineers Specification for non-shrink grout CRD-C621-82A or ASTM C 1107-91.

The storage, handling and placement of the grout must be in strict accordance with the manufacturer's instructions.

Supplier specification

All precision grouting (specify details and areas of application) must be carried out using Conbextra HF manufactured by Fosroc and used in accordance with the manufacturer's data sheet.

Application instructions

Preparation

Foundation surface

The substrate surface must be free from oil, grease or any loosely adherent material. If the concrete surface is defective or has laitance, it must be cut back to a sound base. Bolt holes or fixing pockets must be blown clean of any dirt or debris.

Pre-soaking

Several hours prior to grouting, the area of cleaned foundation should be flooded with fresh water. Immediately before grouting takes place, any free water should be removed. Particular care should be taken to blow out all bolt holes and pockets.

Base plate

It is essential that this is clean and free from oil, grease or scale. Air pressure relief holes should be provided to allow venting of any isolated high spots.

Levelling shims

If these are to be removed after the grout has hardened, they should be treated with a thin layer of grease.

Formwork

The formwork should be constructed to be leakproof as Conbextra HF is a free flowing grout. This can be achieved by using foam rubber strip or mastic sealant beneath the constructed formwork and between joints.

In some cases it is practical to use a sacrificial semi-dry sand and cement formwork. The formwork should include outlets for the pre-soaking water.

The unrestrained surface area of the grout must be kept to a minimum. Generally the gap width between the perimeter formwork and the plate edge should not exceed 150 mm on the pouring side and 50 mm on the opposite side. There should be no gap at the flank sides.

Mixing

For best results a mechanically powered grout mixer should be used. For quantities up to 50 kg a slow speed drill fitted with a high shear paddle is suitable. Larger quantities will require a high shear vane mixer. Do not use a colloidal impeller mixer.

It is essential that machine mixing capacity and labour availability is adequate to enable the grouting operation to be carried out continuously. This may require the use of a holding tank with provision for gentle agitation to maintain fluidity.

The selected water content should be accurately measured into the mixer. Slowly add the total contents of the Conbextra HF bag, mix continuously for 5 minutes, ensuring a smooth, even consistency is obtained.

Consistency of mixed grout

To achieve the consistencies which are defined in CRD-C621-82A, the amount of clean water that is added to a 25 kg bag at 25°C is:

Flowable:	4.5 litres
Fluid:	4.8 litres

Maximum flow distance at 20°C

	Max. flow distance in mm		
Grout consistency	**Gap depth mm**	**100 mm head**	**250 mm head**
Flowable:	10	360	1200
	20	950	2600
	30	500	3000
	40	2200	3000+
	50	3000	3000+
Fluid:	10	900	2500
	20	1900	3000
	30	3000	3000+
	40	3000+	3000+

Placing

Place the grout within 15 minutes of mixing to gain the full benefit of the expansion process.

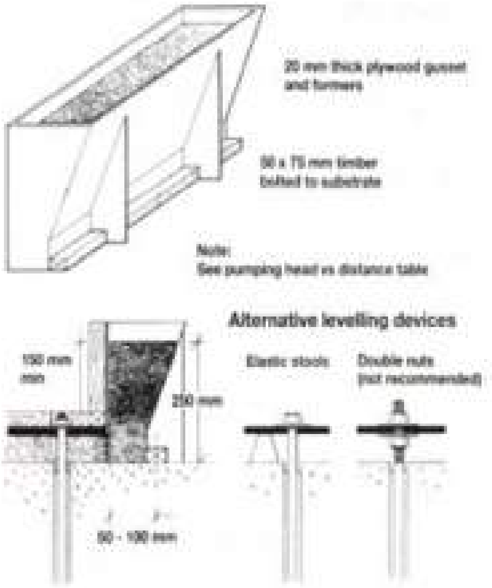
Conbextra HF can be placed in thicknesses up to 125 mm in a single pour. For thicknesses greater than 125 mm, please consult your nearest local Fosroc office.

Any bolt pockets must be grouted prior to grouting between the substrate and the base plate.

Continuous grout flow is essential.

Figure 2: Typical hopper system

Removable hopper: For larger pours the grout may be hand placed or pumped into a removable hopper (trough).



Sufficient grout must be available prior to starting and the time taken to pour a batch must be regulated to the time taken to prepare the next one.

The mixed grout should be poured only from one side of the void to eliminate the entrapment of air or surplus pre-soaking water. This is best achieved by pouring the grout across the shortest distance of travel. The grout head must be maintained at all times so that a continuous grout front is achieved.

Where large volumes have to be placed Conbextra HF may be pumped. A heavy duty diaphragm pump is recommended for this purpose. Screw feed and piston pumps may also be suitable.

Curing

On completion of the grouting operation, exposed areas should be thoroughly cured. This should be done by the use of Concure curing membrane, continuous application of water and/or wet hessian.

Cleaning

Conbextra HF should be removed from tools and equipment with clean water immediately after use. Cured material can be removed mechanically, or with Fosroc Acid Etch.

Limitations

Low temperature working

When the air or contact surface temperatures are 5°C or below on a falling thermometer, warm water (30-40°C) is recommended to accelerate strength development.

For ambient temperatures below 10°C the grout consistency should be flowable and the formwork should be maintained in place for at least 36 hours.

Normal precautions for winter working with cementitious materials should then be adopted.

High temperature working

At ambient temperatures above 35°C the mixed grout should be stored in the shade. Cool water (below 20°C) should be used for mixing the grout.

Estimating

Supply

Conbextra HF is supplied in 25 kg bags.

Yield

Allowance should be made for wastage when estimating quantities required. The approximate yield per 25 kg bag for different consistencies is:

Consistency	Flowable	Fluid
Yield (litres)	13.25	13.5

Storage

Conbextra HF has a shelf life of 12 months if kept in a dry store in sealed bags. If stored in high temperature and high humidity locations the shelf life may be reduced.

Precautions

Health and safety

Conbextra HF is alkaline and should not come into contact with skin and eyes. Avoid inhalation of dust during mixing. Gloves, goggles and dust mask should be worn.

If contact with skin occurs, wash with water. Splashes to eyes should be washed immediately with plenty of clean water and medical advice sought.

Fire

Conbextra HF is non-flammable.

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