**JIMMA UNIVERSITY**

**SCHOOL OF GRADUATE STUDIES**

**JIMMA INSTITUTE OF TECHNOLOGY**

**FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING**

**STRUCTURAL ENGINEERING STREAM**

# Experimental Investigation on the Effect of Reinforcement Bar Corrosion on Steel Fiber Reinforced Concrete Beam

A Research Thesis Submitted to School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Structural Engineering

By

Tsegaye Figa

January, 2020

Jimma, Ethiopia

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Main Advisor: Engr. Elmer C. Agon (Asso. Prof.)

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**EXPERIMENTAL INVESTIGATION ON THE EFFECT OF REINFORCEMENT BAR CORROSION ON STEEL FIVER REINFORCED CONCRETE BEAM**

**TSEGAYE FIGA**

**APPROVED BY BOARD OF EXAMINERS**

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# DECLARATION

I hereby declare that this research work is the original work of mine and has not been presented elsewhere yet. Additionally, all materials which used in this thesis work as a reference and primary sources was duly acknowledged.

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# *ABSTRACT*

*Reinforcement bar corrosion is one of the problems that affect the strength of RC structure. When the reinforcement bar is corroded, it will lose its strength and then the overall design strength of RC structure will be reduced. Currently, this is a major construction industries problem throughout the world. The same is true in Ethiopia; the country lost about 15million ETB for the maintenance of corrosion damaged RC bridges per year. Therefore, it is necessary to investigate the effect of reinforcement bar corrosion in RC structures and the influence of steel fiber addition.*

*The aim of this study is to investigate the effect of reinforcement bar corrosion on SFRC beam. The concrete with an average compressive strength of 25MPa and the reinforcement bar with steel grade S-300 are used. The total of eighteen RC beam specimens with/without steel fiber have taken with 5% and 10% bar corrosion, six cubic specimens for compressive strength test such as 3 plain concrete and 3 SFRC and six cylindrical specimens for splitting tensile test such as 3 plain concrete and 3 SFRC. Data was collected by carefully observing the experimental results. The experimental results shows that, the corrosion of the reinforcement bar which embedded into concrete affects not only the strength of the embedded steel bar but also it affects the strength of concrete which the bar embedded in and the mutual strength that the steel and the concrete which have in common. The experimental result analysis of this study revealed that SFRC beams carry more flexural loads before failure than plain RC beam with similar exposure of corrosion environments. Steel fiber reinforcement which used in this study has an obvious impact on the load capacity, which increased by approximately 14% for the reference beams and, even for those subjected to chloride exposure; all fiber reinforced beams exhibited a greater load at yielding than the reference beams of the plain series such that 35% for 5% corrosion and 15% for 10% corrosion than the reference (plain RC) beams. Therefore, the addition of the steel fiber extracted from used tire into concrete matrix improves the residual flexural strength of the corroded RC beam.*

*In this study only the effect of corrosion of the reinforcement bar on flexural strength of RC beam was considered, but still the corrosion of steel bar has effects on the shear strength of the RC beam, therefore it is recommended to study the effects of corrosion on the shear strength of the RC beam by including shear reinforcement/stirrups in addition to the longitudinal bar.*

***Key words;*** Corrosion, steel fiber, flexural strength, accelerated corrosion.

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# Table of Contents

Contents

# LIST OF TABLES

# LIST OF FIGURES

# ACRONYMS

| | |
|---|---|
| ACI | American Concrete Institute |
| ACT | Accelerated Corrosion Test |
| ASTM | American Standard Testing Method |
| BS | British standard |
| EBCS-EN | Ethiopian Building Codes of Standards- Euro Norm |
| ERA | Ethiopian Roads Authority |
| ES | European Standards |
| IS | Indian Standards |
| IUPAC | International Union of Pure and Applied Chemistry |
| JSCE | Japan Society of Civil Engineers |
| mA | Milli-amperes |
| MPa | Mega Pascal |
| mV | Millivolt |
| pH | Power of Hydrogen |
| RC | Reinforced Concrete |
| SFRC | Steel Fiber Reinforced Concrete |

# CHAPTER ONE

# INTRODUCTION

## 1.1. Background of the Study

Nowadays reinforced concrete is widely applicable composite material as load bearing and non-load bearing members in civil engineering projects/construction industries throughout the world. This is also true in the case of Ethiopia. Most of structures in Ethiopia like buildings, bridges, railways and others are made from reinforced concrete. It is suitable for Ethiopia's weather which is mostly tropical, comparatively less skilled man power requirement, concrete ingredient availability and others requirements.

Many reasons are for wider applicability of reinforced concrete material; among those reasons are availability, comparative cost effectiveness, flexibility for desired shape, comparatively less skilled man power requirement, fire and corrosion resistance, environmental friendly and others. Though, reinforced concrete is less susceptible for corrosion the involvement of steel reinforcement bar expose the reinforced concrete structure for corrosion. Due to reinforcement bar corrosion the overall strength of reinforced concrete structure will be affected.

Reinforced concrete has been used for decades as the construction material of choice. It is economical, versatile and can be molded to a variety of shapes and finishes. Concrete is generally resistant to harsh environmental conditions, and hence concrete structures are designed to have a long service life (1), (2), (3). It is usually durable and strong, performing well during its service life. Reinforced concrete is a composite construction material based on the principle that concrete is an ideal environment for reinforcing steel.

However, corrosion of steel reinforcement has become a major problem for the durability of RC structures in view of the economic consequences with regard to assessment, maintenance and repair, especially for the increasing amount of ageing infrastructure. It has been recognized as a serious problem throughout the world (4), and is a topic of continued international interest particularly for existing, perhaps already partly affected, structures. In the majority of cases, reinforcement corrosion is caused by either carbonation or chloride ingress to the embedded steel level. Concrete structures such as bridges, buildings, sanitary and water facilities can be severely damaged due to corrosion of the reinforcing steel.

Corrosion of reinforcement bar in concrete may cause damage and early failure of RC structures. It may also cause both the safety and performance of concrete structures to significantly deteriorate. Reinforcement corrosion may have severe implications for the mechanical properties of RC (5) including:

- ✓ Loss in load-carrying capacity due to a reduced cross sectional area of reinforcing steel,
- ✓ Loss in load-carrying capacity due to a loss in bond at the concrete-steel interface,
- ✓ Loss in ductility due to an uneven distribution of cross-sectional area along the length of the reinforcing bar and stress concentrations associated with the abrupt changes in geometry, and
- ✓ Loss in load carrying capacity of RC structure due to concrete cracks which results from tensile stress of cumulated corrosion products.

## 1.2. Statement of the problem

It is well-known that a lot of projects are not fully-landed as per the design due to several reasons; such that; lack of detailed and continuous supervision during construction, workmanship, lack of knowledge for understanding of the design details, carelessness, and others. Therefore, the structure is unable to sustain the coming load during its service. The structure already lost its design strength due to construction errors or unconsidered effect during design stage.

In Ethiopia, in most construction sites, the reinforcement bars are exposed to corrosion problem. The reinforcement bars used for construction are stocked very earlier to the commencement of construction; it is exposed for corrosion due to the way of handling and no care is taken about the problem. In addition to this, during the construction stage different parts of reinforcement bars exposed to corrosion environment due to progress delay of construction. For instance, beam/column/slab reinforcement is laid and it leaves for days, weeks, months even for years without casting concrete. Therefore, the reinforcements are exposed to harsh environment which results to corrosion. Even if the construction is started after long delay and already reinforcement is corroded, no one consider corrosion effect which may result in the failure/collapse of overall structure. In the present day another widely observed problems regarding with reinforcement bar corrosion is improper use of concrete cover; most of the buildings in Ethiopia was constructed without using concrete cover or by using concrete cover thickness improperly or other materials like timber, stone or other unspecified materials other than using the spacer made from cement and fine aggregate with specified thickness/dimension and approved quality. In addition to this most of construction participants from labor to high level engineers in Ethiopia have no in depth understanding of corrosion effects and its ways of propagation. In Ethiopia the use of fiber for construction is not widely observed, but there are many materials suitable for fiber which mixed with concrete are available; the used tire is one of such materials.

Corrosion has a huge economic and environmental impact on all facets of national infrastructure; from highways, bridges, buildings, oil and gas, chemical processing, water and waste water treatment and virtually on all metallic objects in use. In the US, total direct cost of corrosion is estimated at about 300 billion dollars per year; which is about 3.2% of domestic product (6).

Though there is no well-registered and recorded data concerning the corrosion of reinforcement bar in Ethiopia; an effort was made to get few data from Ethiopian Roads Authority (ERA) concerning reinforcement bar corrosion in reinforced concrete bridges. According to ERA the country lost more than 15 million ETB annually for the maintenance of reinforcement bar corrosion of RC bridges. Other than material loss, corrosion interferes with human safety, disrupts industrial operations and poses danger to environment. Awareness to corrosion and adaptation of timely and appropriate control measures hold the key in the abatement of corrosion failures.

It is obvious that reinforcement bar corrosion is one of the problems that affect the strength of reinforced concrete structure. When the reinforcement bar is corroded, it will lose its strength and then the overall strength of the reinforced concrete structure will be reduced. Currently, this is a major construction industries problem throughout the world. Therefore, it is necessary to investigate the effect of reinforcement bar corrosion in reinforced concrete structures and the influence of steel fiber addition.

## 1.3. Objectives of the Study

### 1.3.1. General Objective

The general objective of this research was investigating the effect of reinforcement bar corrosion on steel fiber reinforced concrete beam and the role of steel fiber in the improvement of the residual strength of corroded RC beam.

### 1.3.2. Specific Objectives

- To describe the effect of reinforcement bar corrosion on reinforced concrete beam.
- To verify the effect of steel fiber addition on the strength of corroded RC beam.
- To determine the difference between the actual weight loss of the corroded reinforcement bar and the results obtained from Faradays' law during accelerated corrosion test.

## 1.4. Significance of the Study

Currently, a lot of structures like bridge, buildings, offshore structures, and others failed due to reinforcement bar corrosion which is embedded into concrete in order to improve the strength of the structure. Thus, it is necessary to investigate the problem of corrosion and mitigation measures to improve the strength of RC structures against corrosion. Bearing this in mind the findings of this research work is useful in different aspects and levels;

Firstly, the output of this research expected to fill the gap of knowledge in design improvements of reinforced concrete beam with regards to the effect of corrosion of reinforcement bar in the reinforced concrete structures and the influence of steel fiber addition which is extracted from used tire in improving the strength reduction of the structure due to reinforcement bar corrosion and to carry the tensile stress which comes from the corrosion products and to serve as crack arrestors.

Secondly, the findings of this study might be used as a guideline or as a turning point for construction stakeholders i.e. construction design and consulting companies, contractors & governmental construction bureau in Ethiopia.

At last but not least, this research work plays its role in structural engineering as an input and adds values for Jimma University in order to attain goals of being a center of technological excellence in engineering and promoting community based problem solving through researches.

## 1.5. Research Questions

This research work answers the following questions.

- The corrosion of the reinforcement bar which embedded into concrete can affects the strength of the RC beam?
- The addition of steel fiber which extracted from the used tire into the concrete matrix can improve the residual strength of the corroded RC beam?
- The actual weight loss of corroded reinforcement bar agrees with the result obtained from the Faradays' law during accelerated corrosion test?

## 1.6. Scope of the Study

It is difficult to consider every parameter, effects, cases, problems, and other under one research study. Also, it is not mandatory a research to be earth-shaking but it is about adding something new. To this end this research includes and bounded to the following;

- Simply supported reinforced concrete beam was considered during the flexural test of the beams,
- The corrosion of only tensile reinforcement bar and its effect on the flexural strength of simply supported reinforced concrete beam for both plain and SFRC, and
- The effects of steel fiber which is extracted from used tire in improving the residual strength of corroded RC beam that lost its strength due to reinforcement bar corrosion.

## 1.7. Limitations of the Study

- ✓ Availability of data and literatures related with corrosion is scarce in Ethiopia,
- ✓ Lack of advanced Testing equipment/material and accessories.

# CHAPTER TWO

# RELATED LITERATURE REVIEW

## 2.1. General

Nowadays, many researches have done across the world on the reinforcement bar corrosions in reinforced concrete. Those research are includes what the corrosion is, the causes for corrosions, effects of reinforcement corrosion on reinforced concrete structures, the measures should be taken to prevent the reinforcement bar corrosion, remedial measures for structures which its reinforcement bar was corroded and many others related literatures have been written. However, it is difficult to get the literatures which are written in Ethiopia with regards to the reinforcement bar corrosion. Notwithstanding, effort was made to review few researches which was done in Ethiopia in order to assess the remedial measures for the improvement of strength that lost due to reinforcement bar corrosion like a measure to increase the tensile strength of concrete by adding fibers.

Furthermore, literatures about steel fiber are also included in the literatures review section. During literature review of steel fiber, different research work was studied to point out the effect of fiber inclusion in the reinforcement bar corrosion in reinforced concrete structures.

The degree to which performance of reinforced concrete is damaged as a result of reinforcement corrosion is a matter of great concern to those responsible for assessing and maintaining the corroded RC structures. While considerable research effort has been dedicated to the mechanisms and causes of reinforcement corrosion and to researching the durability of repair materials, considerably lower attention has been dedicated to the problem of assessing the residual strength of the corroded structure. A detailed guidance on assessment of residual strength of corrosion-damaged RC structures will be of a great importance to number of practicing and practitioners. Therefore, comprehensive knowledge (that understands and quantifies the effect of reinforcement corrosion on structural behavior) on the effect of corrosion on structural capacity and integrity is essential for the development of effective tools for the prediction of residual service life and for the development of cost effective repair strategies. This chapter will discuss the available information on the factors that cause and control corrosion of steel in concrete, as several metals will corrode under certain conditions when embedded in concrete. Factors influencing the electrochemical process are also discussed.

## 2.2. Strength of Concrete

Statistics and observations show that, concrete is a widely used construction material and has been for many centuries. While concrete has relatively outstanding compressive strength, it lacks in tensile strength. For this reason, reinforcement steel bars are normally used in concrete structures to compensate the tensile strength. However, the steel bars are susceptible to corrosion that can undermine the structural integrity of concrete. Once corrosion occurs, rust forms on the surface of the steel. The rust takes up more volume than the original steel, adding tensile stress to the concrete, causing concrete to eventually crack or spall as it has relatively low tensile strength (7). It is important to understand the mechanism of corrosion to effectively prevent corrosion from comprising reinforced concrete structures.

## 2.3. Steel Fiber Reinforced Concrete

Reinforced concrete is the most widely used man-made construction material and is used for various structural projects ranging from small concrete beams to major bridge constructions. Traditional reinforcement bars have been preferred as reinforcement system to compensate for the low tensile capacity of concrete, for more than a century. Over the past decades other types of reinforcement have gained foothold within the construction industry. One of the predominant alternatives to traditional reinforcement is steel fibers. Steel fibers are mixed-in during batching of concrete, i.e. the fibers are discretely dispersed throughout the concrete volume adding reinforcement in all directions. (7)

Steel fibers have been known as an alternative to traditional reinforcement bars for special applications of structural concrete for decades and the use of steel fiber reinforced concrete (SFRC) has gradually increased in recent years. Steel fibers lead to reduced crack widths in concrete formed, among other reasons, due to shrinkage and/or mechanical loading. Steel fibers are nowadays also used in combination with traditional reinforcement for structural concrete, where the role of the fibers is to minimize the crack widths whereas the traditional reinforcement bars are used for structural purpose. Although, such so-called combined reinforcement systems are gaining impact within the construction industry, they are only marginally covered by existing guidelines for structural design and the literature concerning their mechanical and, in particular their durability aspects, is sparse. (7)

Hertlein point out that the main favored mechanical, material property of concrete is its compressive strength. The compressive strength of concrete is mainly determined by the water-to cement ratio, the maturity and the curing conditions of the concrete. The addition of fibers has a beneficial effect on the mechanical performance of concrete subjected to compression (7). Examples of stress-strain curves for concrete (with and without fibers) subjected to compressive loading are given in Figure 2.1.

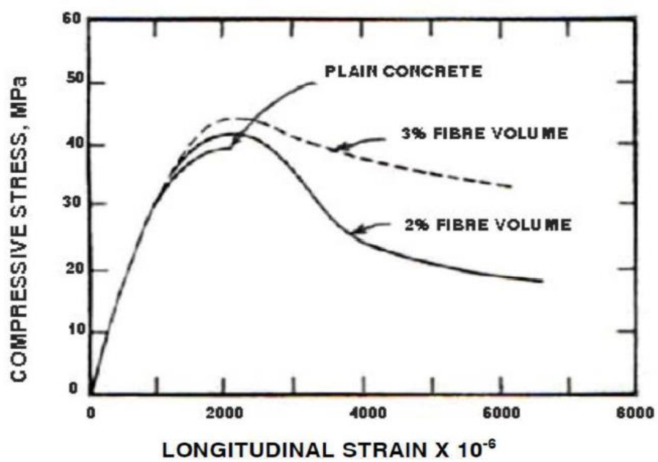



Figure 2. 1: Stress-strain curve for concrete (with/without fibers) subjected to compression (8).

As illustrated in Figure 2.1 the compressive strength of concrete is only moderately affected by the addition of up to 3 percentage volume of steel fibers, but the failure of the concrete specimen is changed from a quasi-brittle failure mechanism to a more ductile failure when fibers are added to the concrete. The addition of fibers allow for a redistribution of stresses as the fibers bridge the internal micro-cracks of the concrete formed during mechanical loading and thus transfer stresses across cracks, (9). Finally it is seen from Figure 2.1 that the Young's modulus of concrete with fibers is similar to that of concrete without fibers, which is plain concrete.

Another author suggests that the main beneficial effect on the mechanical properties of concrete caused by the addition of (steel) fibers is their impact on the formation of cracks caused by tensile stresses. Cracks in concrete are formed, when the tensile capacity is exceeded, but stresses can still be transferred across the crack in the so-called fracture process zone. This fracture process zone consists of micro-cracks, aggregate bridging and other toughening mechanisms (10). Van Mier confirms that in SFRC the fracture process zone furthermore contains fiber bridging. (11)

According to Maidl's statement the increased ductility of SFRC compared to that of plain concrete, is caused by redistribution of stresses by the fibers bridging the cracks. The mechanical response of SFRC in the post-peak (after cracks) state depends on, among other factors, the amount (percentage volume) and geometry (for example length and diameter or an aspect ratio) of the steel fibers and the bond between the concrete and the steel fibers. (9)

The researches depict that the compressive strength as well as the tensile strength of SFRC and plain concrete is similar, whereas the post-peak response is altered by the presence of fibers in SFRC. Hence, in general, SFRC cannot serve as a substitution for traditional reinforced concrete for structures where a high tensile capacity of the construction material is required. However, for structures mainly subjected to compressive load, SFRC is a competitive alternative to traditional reinforced concrete. (7)

To this end it is possible to deduce that steel fibers and traditional reinforcement have different benefits, i.e. the ability to limit crack widths and increase the tensile capacity, respectively, and using them simultaneously as one, so-called, combined reinforcement system has gained foothold within the past years. The phenomenon of utilizing such combined reinforcement systems is not new; Structures utilizing such combined reinforcement have been constructed worldwide in particular within the past decades. (7)

## 2.4. Corrosion

Corrosion is the deterioration or destruction of metals and alloys in the presence of an environment by chemical or electrochemical means. In simple terminology, corrosion processes involve reaction of metals with environmental species. (6)

As per IUPAC, "Corrosion is an irreversible interfacial reaction of a material (metal, ceramic, and polymer) with its environment which results in its consumption or dissolution into the material of a component of the environment. Often, but not necessarily, corrosion results in effects detrimental to the usage of the material considered. Exclusively physical or mechanical processes such as melting and evaporation, abrasion or mechanical fracture are not included in the term corrosion”.

According to (12), corrosion is a natural process that converts a refined metal into a more chemically stable form such as oxide, hydroxide, or sulfide. It is the gradual destruction of materials (usually metals) by chemical and/or electrochemical reaction with their environment.

Corrosion is the destructive attack of on metal by chemical or electrochemical reaction with its environment. In the case of RC structures, the most known degradation process is corrosion of reinforcement. RC members have to fulfill the conditions given in Euro-code (1, 2). (13)

Corrosion is the process of the transformation of a metal to its "native" form, which is the natural ore state, often as oxides, chlorides or sulphates. This transformation occurs because the compounds such as the oxides "involve" less energy than pure metals, and hence they are more stable thermodynamically. The corrosion process does not take place directly but rather as a series of electrochemical reactions with the passage of an electric current. Corrosion also depends on the type and nature of the metal, the immediate environment, temperature and other related factors. The corrosion may be defined as the destructive attack of a metal by chemical or electrochemical reaction with its environment. (14)

Steel in concrete is normally immune from corrosion because of the high alkalinity of the concrete; the pH of the pore water can be greater than 12.5, which protects the embedded steel against corrosion. This alkalinity of concrete causes passivation of the embedded reinforcing bars. A microscopic oxide layer, which is the “passive” film, forms on the steel surface due to the high pH, which prevents the dissolution of iron. Furthermore, the concretes made using low water-cement ratios and good curing practices have a low permeability, which minimizes the penetration of the corrosion inducing ingredients. In addition, low permeability is believed to increase the electrical resistivity of the concrete to some degree which helps in reducing the rate of corrosion by retarding the flow of electrical currents within the concrete that accompany the electrochemical corrosion. Consequently, corrosion of the embedded steel requires the breakdown of its passivity. However, as the global warming becomes worse along with the increase of $CO_2$ content in air, carbonation may break down the passive layer. Those structures in the tidal zone, or roads and bridge decks suffering from de-icing salt can also have the passive layer broken down due to the chloride attack. Without the passive layer, the steel is subjected to water and air and so corrosion happens. There are two steps of the corrosion process. The first step of the steel corrosion is shown in Figure 2.2, and the chemical reactions were given by (15) as:

The anodic reaction: $\mathrm{Fe} \rightarrow \mathrm{Fe}^{2+} + 2\mathrm{e}^{-} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. \mathrm{Eqn.\ 2.1}$

The cathodic reaction: $2\mathrm{e}^{-} + \mathrm{H_2O} + {}^{1}\!/_{2}\,\mathrm{O_2} \rightarrow 2\mathrm{OH}^{-} \ldots\ldots\ldots..\ldots\ldots\ldots\ldots\ldots.. \ \mathrm{Eqn.\ 2.2}$

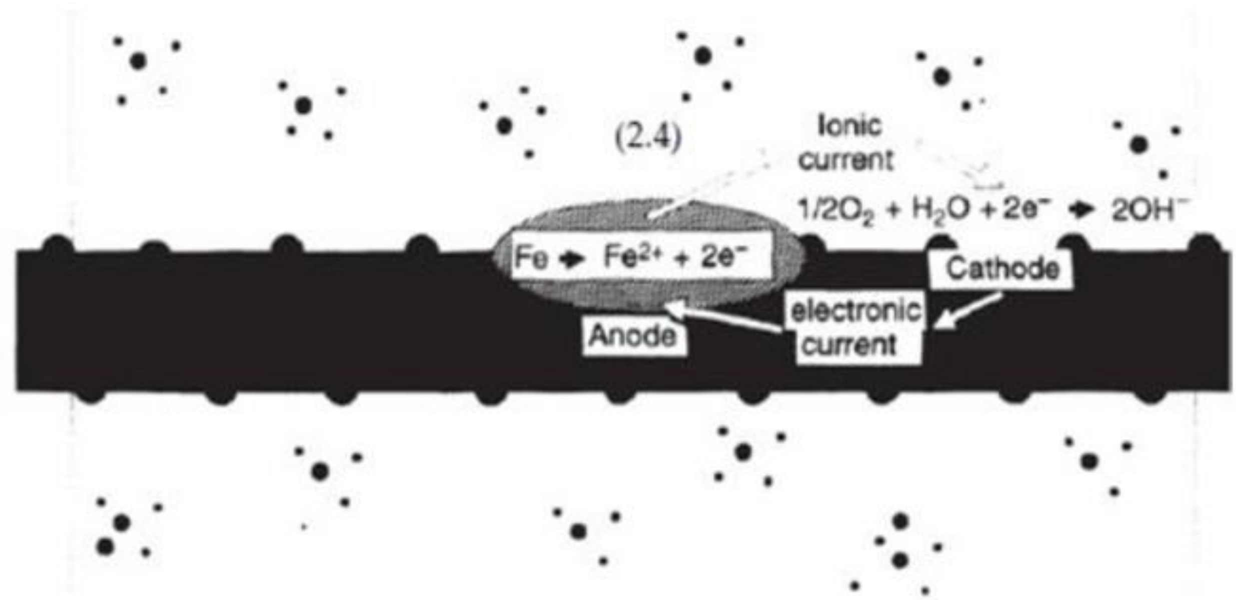



Figure 2. 2: The anodic and cathodic reactions (15)

The second step of steel corrosion is shown in Figure 2.3. The chemical reactions were given by (15) as:

$$\mathrm{Fe^{2+}} + 2\mathrm{OH^{-}} \rightarrow \mathrm{Fe(OH)_2} \ \ldots\ldots\ldots..\ldots\ldots\ldots\ldots\ldots. \mathrm{Eqn.\ 2.3}$$

$$4\mathrm{Fe(OH)_2} + 2\mathrm{H_2O} + \mathrm{O_2} \rightarrow 4\mathrm{Fe(OH)_3} \ \ldots\ldots\ldots.. \mathrm{Eqn.\ 2.4}$$

$$\mathrm{Fe(OH)_3} \rightarrow \mathrm{Fe_2O_3.H_2O} + 2\mathrm{H_2O} \ldots\ldots\ldots..\ldots\ldots \ \mathrm{Eqn.\ 2.5}$$

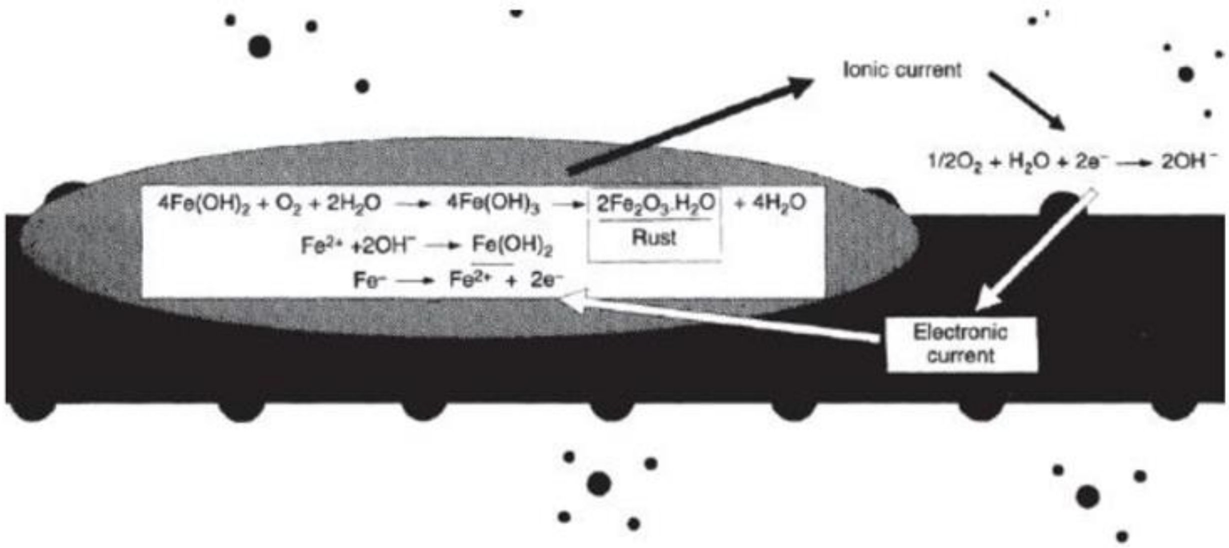



Figure 2. 3: The corrosion reactions on steel (15)

The un-hydrated ferric oxide $Fe_2O_3$ has a volume of about twice as that of the steel it replaces and will have higher volume up to 10 times when it becomes hydrates (15). In concrete structures, the volume of corrosion products is normally twice of un-corroded steel. The volume expansions of the corrosion products cause the cracking and spalling of concrete, decreasing the load carrying capacity of RC structures.

### 2.4.1. Types of Corrosion

Even though the fundamental mechanism of corrosion involves creation or existence of corrosion cells, there are several types or forms of corrosion that can occur. It should however be borne in mind that for corrosion to occur, there is no need for discrete (physically independent) anodes and cathodes. Innumerable micro level anodic and cathodic areas can be generated at the same (single) surface on which anodic (corrosion) and cathodic (reduction) reactions occur. Each form of corrosion has a specific arrangement of anodes and cathodes and specific patterns and locations depending on the type can exist. The most important types are; (6)

**Uniform corrosion** is a very common form found in ferrous metals and alloys that are not protected by surface coating or inhibitors. A uniform layer of ‘rust’ on the surface is formed when exposed to corrosive environments. Atmospheric corrosion is a typical example of this type.

**Pitting corrosion** is a localized phenomenon confined to smaller areas. Formation of micro-pits can be very damaging. Pitting factor (ratio of deepest pit to average penetration) can be used to evaluate severity of pitting corrosion which is usually observed in passive metals and alloys. Concentration cells involving oxygen gradients or ion gradients can initiate pitting through generation of anodic and cathodic areas. Chloride ions are damaging to the passive films and can make pit formation auto-catalytic. Pitting tendency can be predicted through measurement of pitting potentials. Similarly critical pitting temperature is also a useful parameter.

**Pitting Corrosion of Steel in Chloride Solution**

Pitting corrosion is a form of localized corrosion associated with the breakdown of the film and frequently occurs on a completely flat surface. Pits take place where the passive oxide film on the metal surface breakdown. In practice, metals or their alloys such as iron and steel exhibit passivity and are covered with oxide layers with high corrosion resistance, and if any breaks occur in the surface, it is likely to become location of intensive localized corrosion. This is a more insidious form of corrosion often undetected and with a small material loss leading to failure. Pits are also quite difficult to inspect, because of their small size and due to corrosion product which is covered by the metal surface (16).

It’s well known that when mild steel first corrodes, anodic and cathodic areas develop over the corroded surface. Conventionally, these are assumed to change in shape and to move across the surface, resulting in early corrosion that is approximately uniform. However, for saline solutions and marine condition usually this is not observed, but pits have been observed on the metal surface. When a high corrosion resistance is required stainless steel is recommended. Type 316L stainless steel is austenitic and has been used in chemical and petrochemical industries and offshore structures for many decades. The excellent corrosion resistance of this stainless steel is attributed to the formation of a stable passive oxide layer, but nevertheless stainless steel is susceptible to localized corrosion by chloride ions. In addition to choosing the right stainless steel grade for good corrosion resistance, it is equally important to specify the right surface condition of the materials used in many applications. (16)

### 2.4.2. Principles of Corrosion

Corrosion involves both oxidation and reduction reaction. Oxidation reaction occurs at anode and reduction at cathode. At the anode site where the loss of electrons from the metal occurs, the metal goes into solution forming positively charged ions within the electrolyte. The hypothetical metal *M* that has *n* valence electrons is shown in Equation 2.6 with an oxidation reaction:

$$\mathrm{M} \rightarrow \mathrm{M}^{n+} + \mathrm{ne}^{-} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. \mathrm{Eqn.}\ 2.6$$

The oxidation reaction is also referred to as an anodic reaction.

During reduction reaction at the cathode there is gaining of electrons from each oxidized/dissolved metal atom. This reaction is also known as a cathodic reaction. The reduction of dissolved oxygen and the liberation of hydrogen gas by reduction of hydrogen ions are most common reactions occurring during aqueous corrosion of metals. These reactions can be shown in Equations 2.7 and 2.8 as follows:

$$2\mathrm{H}^{+} + 2\mathrm{e}^{-} \rightarrow \mathrm{H}_2 \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ \ldots\ldots\ldots\ldots\ldots \mathrm{Eqn.}\ 2.7$$

$$\mathrm{O}_2 + 2\mathrm{H}_2\mathrm{O} + 4\mathrm{e}^{-} \rightarrow 4(\mathrm{OH})^{-} \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ \mathrm{Eqn.}\ 2.8$$

There are other possible cathodic reactions, depending on the nature of solution to which the metal is exposed. Also the electrolyte must allow for movement of cations from the anodic sites to the cathodic ones and anions in the opposite direction. Finally, the anode and cathode must be electrically connected to allow the flow of current. The product from reaction Equation 2.7 is the hydrogen gas, which can often cause problems such as hydrogen embrittlerment, while the product from Equation 2.8 refers to the dissolved oxygen present in the water; the complete corrosion equation is obtained by combining the equation of the oxidation of metal *M* with one of the Equations 2.6, 2.7 and 2.8. (16) In an acid environment, the complete reaction becomes:

$$\mathrm{M} + 2\mathrm{H}^{+} \rightarrow \mathrm{M}^{2+} + \mathrm{H}_2\ (\mathrm{g}) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ \ldots\ldots\ldots\ldots\ldots \mathrm{Eqn.}\ 2.9$$

$${}^{1}\!/_{2}\,\mathrm{O}_2 + \mathrm{H}_2\mathrm{O} + \mathrm{M} \rightarrow \mathrm{M}^{2+} + 2\mathrm{OH}^{-} \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. \mathrm{Eqn.}\ 2.10$$

Note that Equation 2.10 is a combination of Equations 2.6 and 2.8 and it represents the reactions in aerated water. Furthermore, the products of foregoing reactions often combine to form a precipitate:

$$\mathrm{M}^{2+} + 2\mathrm{OH}^- \rightarrow \mathrm{M(OH)}_2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ \ldots\ldots\ldots\ldots\ldots \text{Eqn. } 2.11$$

If the metal is iron (Fe), the $Fe\ (OH)_2$, or rust is precipitated when oxygen is involved.

## 2.5. The Effects of Reinforcement Bar Corrosion on RC Structures

The basic problem associated with the deterioration of reinforced concrete is corrosion products, i.e. rust. The formation of rust involves a substantial volume increase (a factor of about 4) which causes cracking, spalling and staining of concrete, and reduces the effective cross-sectional area of reinforcing bars and weakens the bond between reinforcement and concrete, seriously affecting the durability, and the service-life of structures. (16)

The bond between steel rebar and concrete is dependent on the cohesion and adhesion at the steel-concrete interface and the mechanical interlocking between the ribs or deformation of steel bars and surrounding concrete. When steel corrodes the initial corrosion product formed can slightly improve the bond, but increasing the level of corrosion will often result in cracking and decrease in bond at the steel-concrete interface. (16)

Corrosion of reinforcing steel causes a decrease in the diameter of steel rebar which reduces such mechanical properties as yield strength, tensile strength and ductility. In addition when reinforcing steel corrodes, the corrosion products occupy a much larger volume than the original steel, and will eventually exert a large tensile force on the surrounding concrete which causes cracking and spalling of the concrete cover and loss in adhesion between steel and concrete interface. (16)

As discussed earlier, there are two classifications of the corrosion of reinforcement namely: general or local corrosion. General corrosion may occur due to either chloride contamination or due to carbonation of the RC structure. The consequences of the steel corrosion are manifested as a decrease in the bar diameter, deterioration of the mechanical properties of the reinforcing steel (e.g., the change from the normal ductile response of low carbon steel bars to a relatively brittle response in bars damaged by pitting corrosion), cracking and spalling of the concrete by the expansive iron oxides and hydroxides, and a noticeable decrease in the bond at the steel-concrete interface. The oxides and hydroxides occupy a greater volume than the parent metal, and the expansion of the diameter of the bar as it corrodes generally leads to cracking and eventually spalling of concrete cover before an appreciable proportion of the cross sectional area is lost. (14)

Local or pitting corrosion is regularly associated with chloride contamination and not with carbonation. The corrosion products due to local or pitting corrosion do not exhibit the same degree of volumetric expansion as that of the general corrosion. Consequently, the tendency of a corroding bar to split the concrete cover is less with local or pitting corrosion, and extreme loss of bar section may occur without signs of deterioration on the surface of the member. (14)

In general, the residual strength of concrete structures may be affected by local (due to pitting) or general loss of reinforcement cross sectional area, through changes in the concrete cross section as a result of cracking or spalling, or through loss of bond between steel and concrete. It should be noted that the two types of corrosion, pitting and general corrosion, might occur together, and that the presence of general corrosion should not be taken to indicate an absence of local corrosion. It should also be noted that the loss of strength and ductility of reinforcement are of greatest concern when assessing structures affected by local corrosion. (14)

### 2.5.1. Influence of Bar Corrosion on Flexural Capacity of RC Members

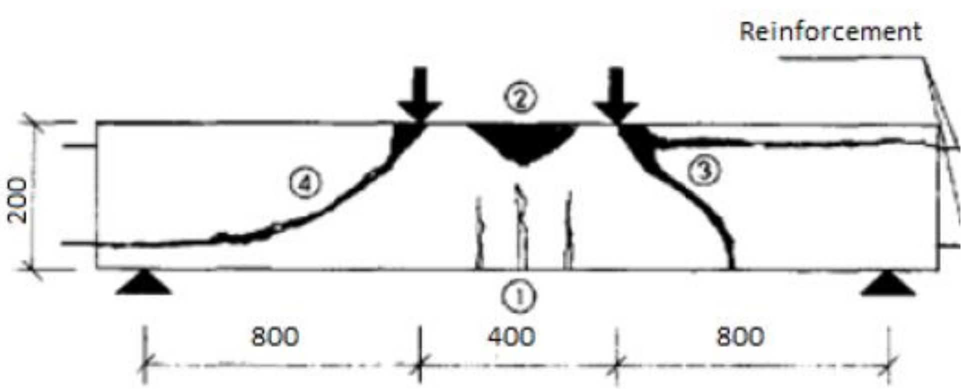
The majority of structural members used in practice are subjected to flexural stresses caused by bending moments, such as beams and slabs. This section summarizes the current understanding of the influence of corrosion on flexural capacity of a reinforced concrete element. Few researchers have investigated on how the corrosion of reinforcement influences the flexural capacity of reinforced concrete members, such as strength, deflection and steel and concrete strains in reinforced concrete beams.

The authors studied the flexural behavior of corroded RC beams under combined effect of corrosion and sustained loads. An accelerated corrosion process using a 5% solution of NaCl and a constant impressed current induced corrosion on tensile steel bars; they tested four RC beams, each with a width of 153 mm, a depth of 254 mm and a length of 3000 mm. Beams were tested under self-weight, under 10% of the ultimate load and under 33% of the ultimate load. Longitudinal tensile strains and longitudinal compressive strains were monitored during the corrosion process. Measured strains were used to determine the depth of the neutral axis, the curvature and the moment of inertia of beams. They concluded that depth of the neutral axis is independent of the level of corrosion for beams free from flexural cracks and beams free from corrosion but significantly reduces with an increase in degree of corrosion for corroded beams with flexural cracks. In addition, the longitudinal strains, depth of the neutral axis and curvature depend on both the level of corrosion and the applied load while the moment of inertia only depends on the level of corrosion (17).

Another researchers are studied the flexural behavior of corroded RC beams under combined effect of corrosion and sustained loads. Test results showed that the presence of a sustained load and associated flexural cracks during corrosion exposure significantly reduced the time to corrosion cracking and slightly increased the corrosion crack width. For example they found that crack width would propagate 22% faster in loaded conditions. They observed that with 8.9% and 22.2% mass loss, strength losses of 6.4% and 20.0%, respectively. It was also observed that the presence of flexural cracks during corrosion exposure initially increased the steel mass loss rate and, consequently, the reduction in the beam strength. They concluded that at low corrosion levels, the effect of bond loss can be ignored and that the ultimate load carrying capacity of the beam is affected only by the loss on steel reinforcement. (18)

Furthermore, the authors investigated the mechanical behavior of RC beams with corroded reinforcement. They conducted two separate experimental studies; the first one consisted of four beams, which were naturally corroded in a salt fog environment for 14 years in an attempt to mimic actual field conditions, with dimensions of 150 x 280 x 3000 mm. Beam ultimate strength were determined by using three-point loading tests. The average reported degree of corrosion was 10% and the reduction in flexural strength was 20% with a 70% reduction in ductility. They concluded that the decrease in stiffness was due to the reduction of both the steel cross -sectional area and bond strength. This was attributed to the fact that the average maximum cross -section loss near the center of the beam was 20%, which would theoretically result in a stiffness decrease of 15%. However, the total stiffness loss for one of the beams tested was 35%; hence, there was a 20% difference in loss that was unaccounted for, which the researchers suspected to be the result of bond deterioration (19). They also said that the concrete cracks resulting from compression reinforcement corrosion have an insignificant effect on the global behavior of RC beams.

Additionally, the researchers studied the effect of varying degrees of corrosion on concrete beams. The beam specimens were 200mm by 150mm with a clear span of 2000mm. Beams had both tensile, compressive as well as shear reinforcement that was corroded using accelerated corrosion techniques by immersing the specimens in a solution made of 3% calcium chlorides by weight to the mixing water, over a period of 101-190 days under a constant current density of 100 $\mu A/cm^2$. The results showed that corrosion increases deflections and crack widths at the service load, decreases strength at the ultimate load, and causes an increase in both the spacing and width of transverse cracking due to bond deterioration. The different types of failure within the beam specimens were observed as shown in Figure: 2.4. They reported that type 1 failure occurred in both corroded and un-corroded beams with a low tensile reinforcement ratio; type 2 failure was observed in beams with un-corroded tensile reinforcement of a high ratio and most corroded beams with a high ratio of shear reinforcement; type 3 failure occurred in nearly all beams having a high ratio of corroded tensile bars and large stirrup spacing; while type 4 failure was found in corroded and un-corroded beams with curtailed tensile reinforcement (20) .




1) Failure by bending (yielding of tensile reinforcement)
2) Failure by bending (crushing of concrete)
3) Failure by shear
4) Failure by both shear and bond splitting

Figure 2. 4: Illustration of types of failures of beams with corroded reinforcement (20)

It was determined that not only will there be a loss of strength in both shear and bending due to reinforcement corrosion, but also that corrosion can change the mode of failure as well. Failure in some beams shifted from bending to shear upon corrosion of reinforcement. This was prominent in beams with stirrup spacing close to the effective depth of the concrete sections, when combined with a high relative corrosion of the tensile bars. This change in failure mechanism was attributed to the reduction of the stirrups cross-section (shear reinforcement) due to pitting and the reduction of the concrete section due to spalling (20). In addition, they concluded that by using the reduced sections of steel and concrete with conventional RC models, conservative values of the ultimate moment and shear force could be calculated for RC members damaged by corrosion. However, this method of calculating the strength of damaged members is deficient in that it fails to consider the reduction of bond.

In addition another authors studied the behavior of RC beams simply supported with exposed reinforcement. In the tests performed, the concrete-steel interface was assumed to have zero-bond over various lengths of the beam and the capacity of the beam was observed to reduce with smaller bond lengths. They found that even with the use of a critical un-bonded length, the beams failed by the concrete crushing, regardless of steel ratio. Their results have demonstrated that beams may possess considerable strength despite bond being entirely eliminated over part of the span, provided ends of bars remain adequately anchored (21). They proposed an algebraic method for predicting the ultimate strength of corroded RC beams. However, this method assumes that concrete acts as a linear-elastic material and thus does not properly reflect the stress-strain behavior of concrete, and presumes a total loss of bond, which does not reflect actual (18).

Further study was done on the effects of corrosion on beams of 160 x 125 x 1000 mm, reinforced with 2-10 mm diameter bars at the top, 2-12 mm diameter bars at the bottom and 8 mm diameter stirrups. They also used accelerated corrosion techniques to corrode the tensile reinforcement by applying an unknown current density. They reported bond reduction of 35% and a maximum reduction in the steel cross-sectional area was 9%, which resulted in a 20% reduction in the ultimate bending moment and a 40% increase in deflection at the service load. They also developed expressions for the relationship between crack width and corrosion, based on different parameters. Predictions of the two sets of expressions are broadly consistent despite significant differences in conditioning. Others report crack widths in excess of 0.6mm without spalling. Given the importance of confinement from cover and links to bond, it is clear that bond will be severely depleted prior to spalling (22).

Other investigators conducted two series of tests, one to study the influence of corrosion on the behavior of beams failing in bond, the other to evaluate the behavior of beams designed to fail in flexure. These beam tests were designed to simulate relatively uniform corrosion, while pullout tests were used to simulate severe local corrosion. These beams and pullout specimens were subjected to different levels of corrosion. In the test program on the study of corrosion-bond behavior for beams designed to fail in bond, beam specimens150x150 mm in cross section and 1000mm in length, with one 12mm bottom bar having an embedment length of 144mm, were used. In the series aimed at the study of corrosion-bond behavior for beams designed to fail in flexure, the same specimen were used except that the embedment length was increased to 300mm. All beam specimens were cast from a concrete with a water/cement ratio of 0.45 and an average compressive strength of 40 MPa (23).

Sufficient development length and shear stirrups were provided to prevent bond or shear failures for the beams designed to fail in flexure. They found that the average bond stress over the embedded length was found to be well below the ultimate bond stress observed in pullout tests for bars with similar corrosion, indicating that the failure of the beams was due to the yielding of steel and was not a bond failure (23). Furthermore, they concluded that the reduction in beam capacity was not due to a decrease in bond stress but rather, it could be attributed primarily to the reduced area of the reinforcing steel.

Additionally the authors carried out a series of tests on beam specimens in which loss of cover to and full de-bonding of tension reinforcement due to corrosion damage was simulated. Beam section, which contained 0.95% reinforcement, and span were tested with a varying length of bar exposed. All beams were tested under a single point load offset from mid-span, and the load was applied at a section where reinforcement remained bonded to concrete. They noted a marked difference in the pattern of crack formation was noted in specimens with bars disbanded. They also noted that the beam strength was not adversely affected by the absence of cover. This structural action was described as changing from flexural to tied arch behavior with secondary effects. Exposure of reinforcement of about 20% of the span of the beam resulted in a slight loss of load carrying capacity; however, exposure of over 60% of the span of the beam resulted in a strength loss of around 20% (24).

## 2.5.2. The Effect of Bar Corrosion on the Bond Strength of RC Structures

### 2.5.2.1. Fundamentals of Bond Strength

In concrete structures, bond stress is the name assigned to the shear stress at the steel bar-concrete interface which modifies the steel stress along the length of the bar, by transferring load between the bar and the surrounding concrete. Bond stress is calculated as the nominal shear force per unit surface area of the reinforcing bar.

The external load is very seldom applied directly to the reinforcing steel, which receives its share of the load through the surrounding concrete. Thus an effective reinforced concrete member must have a positive interaction between the steel bar and the surrounding concrete in order to achieve a force transfer between two materials (i.e. steel bar and concrete). Figure 2.5 illustrates the load sharing between the reinforcement and the concrete.

The bond between concrete and steel reinforcement is of paramount importance to the strength of reinforced concrete. Bond strength between steel and concrete can be divided into two parts: adhesion, and mechanical (25). Adhesion itself comes from three different sources. First is the chemical adhesion between concrete and the steel. The second is the frictional force between the concrete and steel. The last is the confining pressure exerted by the concrete on the reinforcing steel. The mechanical aspect comes from the ribs on the rebar. These ribs interlock with the surrounding concrete and resist any translational motion. In deformed bars, the mechanical action of the ribs is the dominant contributing factor. In smooth bars, there is very little mechanical action due to the absence of ribs. Therefore it is adhesion that provides the majority of bond strength. (26)

### 2.5.2.2. Bond Mechanism for Reinforced Concrete with Steel Bars

The bond mechanism is the interaction between the steel reinforcement and the surrounding concrete which allows forces to be transferred from the reinforcement to the surrounding concrete. The bond of reinforcing steel bars depends primarily on mechanical interlocking, adhesion and friction (27). The effect of adhesion is small and friction forces don't develop until adhesion has failed and relative displacement between reinforcement bar and concrete occurs. Both mechanisms are important in the case of plain bars. For deformed steel bars, the mechanical interlock of the ribs of the bars embedded in concrete governs the bond stress-deformation behavior. (28)

**Adhesion**

Adhesion is a chemical bond, which occurs at the interface between the reinforcing bar and concrete. The adhesion at steel/concrete interface depends on concrete composition, steel bar surface roughness and the reinforced concrete specimen age. For relatively small loads, the basic resisting mechanism is the chemical adhesion; however, as the load increases the chemical adhesion along the bar surface is lost rapidly. It has been generally assumed that adhesion can breakdown because of the action of the service loads, or due to shrinkage of concrete (29).

The ACI Committee 408 (1991) suggested that the bond strength due to adhesion is between 0.48 to 1.03MPa. The authors have examined the adhesion mechanism with both uncoated and epoxy coated steel rebar, and found that the uncoated bars did adhere to the concrete while there was no evidence of adhesion between the epoxy coated and concrete (30). Furthermore, other investigators reported the epoxy coating of reinforcing bar resulted in a significant loss in bond strength of the order of 20% compared to uncoated reinforcing steel; however there is no adverse effect on bond strength with the use of galvanized reinforcing steel (31).

**Friction**

Friction is similar to the mechanical interlock; the frictional bond is greatly dependent on the surface characteristics of the reinforcing steel bar, aggregate size and shape (29). After the chemical adhesion is destroyed, some friction slip occurs before the full bearing capacity at steel bar ribs is mobilized. According to the work of (30), and the ACI committee 408 (1991) suggested that friction can contribute up to 35% of the ultimate strength controlled by splitting of concrete cover.

**Mechanical Interlock**

The surface profile of a steel bar dictates the amount of mechanical bond that can be generated between steel bar and concrete. The mechanical interlocking of deformed steel bars is enhanced by the geometry of the ribs along the length of the steel bar. For deformed steel bars, bearing against the lugs is considered to be the most significant transfer mechanism at higher load levels. The force transfer mechanism is due to the mechanical interlocking between the ribs and concrete. As the ultimate bond strength is reached, shear cracks start to form in concrete between ribs as the interlocking forces induce large bearing stresses around the ribs, and slip occurs. Therefore, bar ribs restrain the slip movement by bearing against concrete. The slip of deformed bar may occur in two ways, either through pushing the concrete away from the bar by ribs, i.e. wedging action, or through crushing of concrete by the ribs (27), (32).

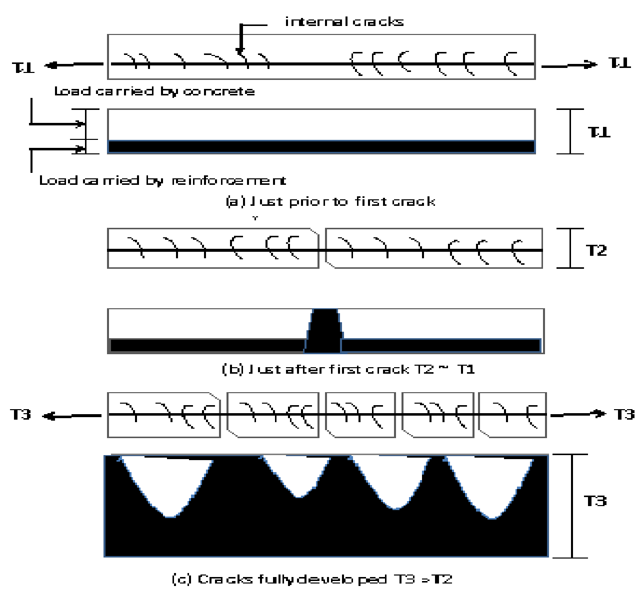



Figure 2. 5: Load sharing between concrete and reinforcement (33)

Bond failure at the steel/concrete interface does not enable the tensile force to be developed in steel reinforcing bars, thus affecting the resistance of the structure. Bond failure can result generally in a catastrophic failure of structure, which could cause serious problems such as fatalities, injuries, and loss of the structure. Therefore, it is the responsibility of designers to ensure the bond performance is satisfactory (34).

Corrosion of steel reinforcement in concrete reduces the durability of concrete structures. Furthermore, it is even more serious for the deterioration of bond between steel and concrete interface which may cause reduction in the load carrying capacity of steel bars due to decrease in the cross-sectional area (20), (35).

#### 2.5.2.3. The Effect of Corrosion on Bond Strength

In the case of general corrosion, bond is more likely to affect structural capacity than is loss of tensile strength of reinforcement. Experiment results indicated that the level of reinforcement corrosion does not influence the tensile strength of steel bars (calculated on the actual area of cross-section), but reinforcing steel bars with more than 12% corrosion indicates a brittle failure. It’s concluded that the strength ratio and elastic modulus of reinforcement are not significantly affected by corrosion and consequently the strength and modulus of elasticity of non-corroded bars can be adopted in practice (35).

The corrosion products have higher volume than the original steel. With the increase of corrosion level, the volume expansion of excessive rust products on the surface of steel induces radial compression pressure and hoop tensile pressure on the surrounding concrete after corrosion products fill the pores in concrete. When the hoop tensile stress exceeds the tensile strength of concrete, the concrete will crack. Hence, properties of cracked concrete should be considered to demonstrate the behavior the RC member after concrete cracks. (14)

The transfer of the load between the steel and the concrete is affected by the phenomenon of bond at the steel-concrete interface, which ensures secure gripping of the reinforcement, and the working of the reinforcing steel in conjunction with the concrete, to form a reliable structural element, capable of withstanding both tension and compression forces. By simplifying the real behavior, bond stress may be considered as a shear stress over the surface of a reinforcing bar. Bond strength initially comes from weak chemical bonds between steel and hardened cement, but this resistance is broken at a very low stress. Once slip occurs, friction contributes to bond. In plain reinforcing steel bars, friction is the major component of strength. With deformed (ribbed) reinforcing steel bars, and under increasing slip bond depend principally on the bearing, or mechanical interlock, between ribs rolled on the surface of the bar and the surrounding concrete. In this stage, the reinforcing bar generates bursting forces which tend to split the surrounding concrete. The failure load may be limited by the resistance provided to these bursting forces by concrete cover and confining reinforcement (36).

Corrosion affects bond strength in several ways. The bond strength between steel and concrete is reduced with the increase of reinforcement corrosion. One reason is that the cracking of concrete implies loss of confinement and thus reduces the bond. Another reason is that the weak layer of corrosion products reduces the friction force. Furthermore, the ribs are also deteriorated and the interlocking force decreases. Corrosion may reduce the height of the ribs of a deformed bar, this is unlikely to be significant except at advanced stages of corrosion, however, disengagement of ribs and concrete occurs, and the layer of corrosion products formed by oxidation of the steel may force the concrete away from the bar and reduce the effective bearing area of the ribs. Therefore, the bond strength is significantly reduced due to the corrosion of reinforcement. However, it should be noted that within certain level of corrosion (2% to 4% mass loss of steel), the bond strength is observed to increase in many researches (37), (23), (38), (39) and others. The reason is that with the slight formation of corrosion products, increases in the diameter of a corroding bar at first, which increases radial stresses between bar and concrete and hence increases the frictional component of bond. However, further corrosion will lead to more corrosion products, development of longitudinal cracking and a reduction in the resistance to the bursting forces generated by bond action. Corrosion products at the bar-concrete interface will affect friction at the interface.

### 2.5.3. Mechanism of Reinforcement Bar Corrosion in Concrete

The mechanism of corrosion of steel in concrete is two-fold: either by chloride attack or by carbonation of concrete. These two mechanisms usually do not attack the integrity of concrete but they attack steel bars. However, other ions such as sulfates attack the integrity of concrete before attacking the steel. The presence of chloride in sufficient concentration at steel concrete interface causes damage to reinforcement by attacking the passive layer. The de-passivation mechanism for chloride attack differs from carbonation (16).

The chloride ions act as a catalyst for the broken passive film. In the absence of chloride, the passive film dissolves slowly as ferric ions. The FeOOH is a hydrated passive film with iron in the ferric oxidation state. Chlorides catalyze the dissolution of this ferric hydroxide as shown in reactions Equations 2.12 and 2.13:

$$\mathrm{FeOOH} + 2\mathrm{Cl}^- \rightarrow \mathrm{FeOCl} + \mathrm{OH}^- \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..\mathrm{Eqn. 2.12}$$

$$\mathrm{FeOCl} + \mathrm{H_2O} \rightarrow \mathrm{Fe}^{3+} + \mathrm{Cl}^- + 2\mathrm{OH}^- \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..\mathrm{Eqn. 2.13}$$

The reactions in Equations 2.16 and 2.17 destroy the passive film at the steel surface, anodic dissolution of *Fe* at the bare site follows, and then ferrous ions are produced and react with chlorides to form ferrous chloride as shown in the Equation 2.14.

$$\mathrm{Fe}^{2+} + 2\mathrm{Cl}^- \rightarrow \ \mathrm{FeCl_2} \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \ \ldots\ldots\ldots\ldots\ldots\mathrm{Eqn. 2.14}$$

The ferrous chloride reacts with water and the hydroxyl ions in the pore water to form ferrous hydroxide, a greenish black product at the anodic sites. Chloride ions are simultaneously released into pore water and then the chloride ions further react with ferrous ions to continue the corrosion process in a cycle as indicated in the chemical reaction of Equation 2.15:

$$\mathrm{FeCl_2} + 2\mathrm{H_2O} \rightarrow \mathrm{Fe(OH)_2} + 2\mathrm{H}^+ + 2\mathrm{Cl}^- \ \ldots\ldots\ldots\ldots\ldots\ldots.\mathrm{Eqn. 2.15}$$

The ferrous hydroxide reacts with oxygen and pore water to form ferric hydroxide which is dissociated into ferric oxide, and hence red-brown rust product is formed as given in Equations 2.16 and 2.17:

$$\mathrm{Fe(OH)_2} + {}^{1}/_{2}\,\mathrm{O_2} + \mathrm{H_2O} \rightarrow 2\mathrm{Fe(OH)_3} \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.\mathrm{Eqn. 2.2}$$

$$2\mathrm{Fe(OH)_3} \rightarrow \mathrm{FeO_3} \ + 3\mathrm{H_2O} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..\mathrm{Eqn. 2.3}$$

Carbonation attack is the result of the interaction of carbon dioxide in the atmosphere with alkalis in concrete. First, carbon dioxide from the atmosphere diffuses through the concrete forming carbonic acid with the pore water in concrete according to the reaction in the Equation 2.18:

$$\mathrm{CO_2} + \mathrm{H_2O} \rightarrow \mathrm{H_2CO_3} \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..\mathrm{Eqn. 2.4}$$

Then the carbonic acid reacts with the calcium hydroxide in concrete to form calcium carbonate and waters as shown in the Equation 2.19:

$$H_2CO_3 + Ca(OH)_2 \rightarrow CaCO_3 + 2H_2O \; \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{Eqn. } 2.19$$

The transformation of calcium hydroxide to calcium carbonate referred to as carbonation, lowers the pH of the pore water to less than 9.0 in a fully carbonated concrete. This phenomenon leads to de-passivation and catastrophic reinforcement corrosion. This type of attack is common for old, badly built structures where low cement content concrete is used and where structures are built with a low concrete cover. Carbonation can be prevented by high cement to water ratio, good curing and enough cover depth of concrete (16).

**2.5.3.1. Passivity of Steel in Concrete**

Passivity term predates the modern concern of the protective crystalline structure in solids. Passivity is provided by an insoluble layer formed on the metal surface which protects the metal against corrosion. It has been proved that steel in concrete passivates in a similar way to steel in alkaline solution in the absence of chloride. (16)

The high alkaline environment of concrete offers satisfactory corrosion protection to the embedded steel. This protection is largely electrochemical in nature and is due to the passive film formed on the steel surface. However, a decline in durability of reinforced concrete structures due to steel corrosion is common. The alkaline nature of pore solution and the presence of oxygen and water result in the formation of insoluble protective layer consisting of ferric oxides. This layer reduces the dissolution of ferrous ions from anodic sites and therefore the corrosion rate to a very low level (40).

A relative high pH and presence of oxygen and moisture are essential for maintenance of the passivity of the reinforcing steel embedded in the concrete. The penetration of chloride ions or carbon dioxide into the concrete causes a reduction in the pH which may lead to active corrosion. The elimination of moisture inhibits the corrosion process in the absence of oxygen in pore solution at steel-concrete interface and if the pH is greater than 9 then corrosion process will continue. However, this will result in the evolution of hydrogen instead of the reduction of oxygen at the cathode as shown in the Equation 2.20 (16).

$$2H_2O + 2e^- \rightarrow H_2 + 2OH^- \; \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{Eqn. } 2.20$$

### 2.5.4. Chloride Induced Reinforcement Corrosion

#### 2.5.4.1. Chloride Penetration

Chlorides have been known to be introduced in concrete through several sources. Chlorides can be cast into concrete using accelerator agents containing chloride ions, use seawater for concrete mixing and aggregates containing chlorides. Chloride ingress from the environment can be due to seawater entering the concrete structure, ground water with high chloride concentration and deicing salts. (41), (42)

The most important source of chloride ions in concrete is deicing salts which are used in cold climate countries during winter time. The salt mixture penetrates concrete by different mechanisms. These include diffusion which is movement of substance due to a concentration gradient, permeation which is the flow of liquid in concrete due to pressure, capillary absorption which is the transport of liquid into porous non-saturated concrete due to surface tension forces, and migration which is the transport of ions in an electrolyte due to an electrical potential gradient (43), (44).

Diffusion is considered to be the principal form of chloride transport through concrete in aqueous condition. Different factors influence the rate of diffusion of chloride through concrete such as the water/cement ratio, type of cementations material used pore size and distribution, temperature and time (45).

The main factors considered when dealing with diffusion in concrete are pore sizes and distribution in concrete, since the pores filled with water, considered to be the medium which the ions travel through. It has been found that concrete samples with high water/cement ratios have a higher diffusion rate than samples with lower water/cement ratios. This has been attributed to the higher volume of macrospores and un-segmented capillary pores present in concrete with high water/cement ratios. Low water/cement ratio can resist chlorides penetration into reinforcing steel, also provides a barrier against the entry of oxygen and therefore, provides better concrete corrosion resistance. The permeability of concrete is a key factor in determining the durability of reinforced concrete structure (46), (47).

It has been estimated that the typical diffusion rates in fully saturated hydrated cement paste to be about $10^{-12}$ meter square per second, which is so small that it would require several months for the chloride ions to penetrate a 10mm thick hydrated cement paste layer, showing the importance of concrete cover thickness and quality. (48)

Chemical reactions occur between the different phases at the inter-phase surface while the transport processes transmit the reactions to the surface and withdraw the reaction products. The transport process and ingress of moisture or aggressive agents and air which result in the chemical reactions and consequent concrete deterioration are controlled the permeability of concrete. The author defined the permeability of a medium which characterizes the ease with which a fluid will pass through the medium under the action of a pressure differential and therefore, it represents the relative ease with which concrete can become saturated with water (48).

It is generally believed that some chloride ions can react chemically with a calcium aluminate mineral in the cement gel and therefore, tricalcium aluminates ($C_3A$) amount of the cement has a strong influence on the mount of chlorides remaining in the hydrated cement paste pore solution. It was reported that the effect of different percentages (from 2-10%) of $C_3A$ on the corrosion of embedded steel bars in presence of 5% NaCl reduced the chloride diffusion and improved the corrosion resistance. (49)

The permeability of blended cement concrete to chloride ions has been found to be lower than for Ordinary Portland Cement (OPC) concrete of the same composition. Blend agents (slag, pozzolans and fillers) can influence the permeability and therefore the rate of penetration of chloride ions. Blending cement with blast furnace slag has been found to reduce the diffusion rate of chloride ions. Also it was reported that the uses of silica fume in concrete reduce concrete permeability, improve durability and lower the penetration rate of chloride. The effect of fly ash (Up to 50%) on concrete samples under immersion in chloride has been studied. It was found that fly ash reduces chloride diffusion and decreases the corrosion rate (50).

#### 2.5.4.2. Mechanism of Chloride Attack

Chloride ions cause damage to reinforcing steel by attacking the passive film and corrosion starts at certain locations along the steel bar where the loss in passive layer occurs, which shown in the Figure 2.6. This mechanism leads to iron dissolution and hydrolysis produces $FeOH^{+}$ and $H^{+}$. This produces an acidic environment that attracts anions (e.g. $Cl^{-}$ and $OH^{-}$). If the level of chlorides is high compared with available hydroxyl ions the affected area becomes more acidic and chloride rich leading to further breakdown in the passive layer and a decrease in corrosion potential with increasing anodic activity in the form of iron dissolution (51). In addition, hydroxyl ions are formed due to cathodic reactions leading to increase in the cathodic area. The small anode to cathode area further promotes corrosion, which result in an increase in the corrosion current density. These processes encourage the formation of corrosion pits and substantial local loss of cross-sectional area of steel bar, and hence increase aggravation of the corrosion process. This, however, can be reversed by the available hydroxyl ions that have the ability to neutralize the acidity, stabilize and repair the damaged parts of the passive film through further iron oxide deposition. (52)

The risk of corrosion increases as the chloride content increases and when the chloride content at the surface of the steel exceeds a certain limit, called the threshold value, corrosion will occur if the water and oxygen are also available (51).

Other results have been reported on mild steel embedded in concrete with large variations in the threshold level of chloride. The chloride threshold level is significantly higher than values obtained in alkaline solutions, where the increased resistance of steel in concrete against chloride-induced corrosion was attributed to buffering effect of hydrated cement paste at the steel concrete interface. The author assumed the threshold concentration of chloride in concrete to be in the range of 0.2% to 0.4% chloride ions by weight of cement, this concentration of chloride is the minimum concentration of chloride ions which causes a significant level of steel rebar corrosion (2). However, other authors reported the chloride content less than 0.2% to 0.6% by weight of cement is acceptable in most concrete structure specifications (40). Furthermore, another author reported a level of 0.7 to 0.89 Kg/m$^3$ is usually considered a threshold level for steel bar in reinforced concrete bridge check (53). Given the inherent complexity and heterogeneity of concrete as a corrosion medium, there exists large uncertainty in the chloride threshold value.

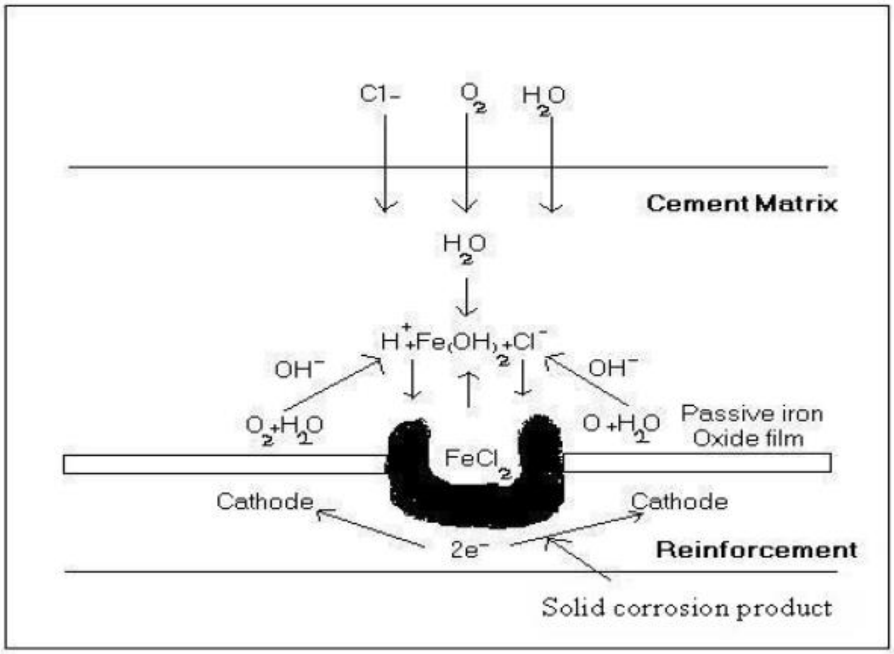



Figure 2. 6: Pitting corrosion due to chloride contamination (54)

## 2.6. Stages of Reinforcement Bar Corrosion

It is well known that the strength of reinforced concrete structure is affected by many factors. Reinforcement corrosion is one of the main durability-threatening mechanisms in reinforced concrete structure. Much research has focused on increasing the basic understanding of both the transport mechanisms of aggressive ions (chloride ions) through concrete and the subsequent corrosion of reinforcing steel (55).

Based on the authors' description the degradation of reinforced concrete structures mainly involves three stages as shown in Figure 2.7, (56):

i) An initiation period before corrosion activation, during which little deterioration occurs.

ii) A propagation period after corrosion activation that generates expansive corrosion products causing cracking of the cover concrete.

iii) An acceleration period where corrosion increases due to easy access of oxygen, water and further aggressive agents through cracks and spalls.

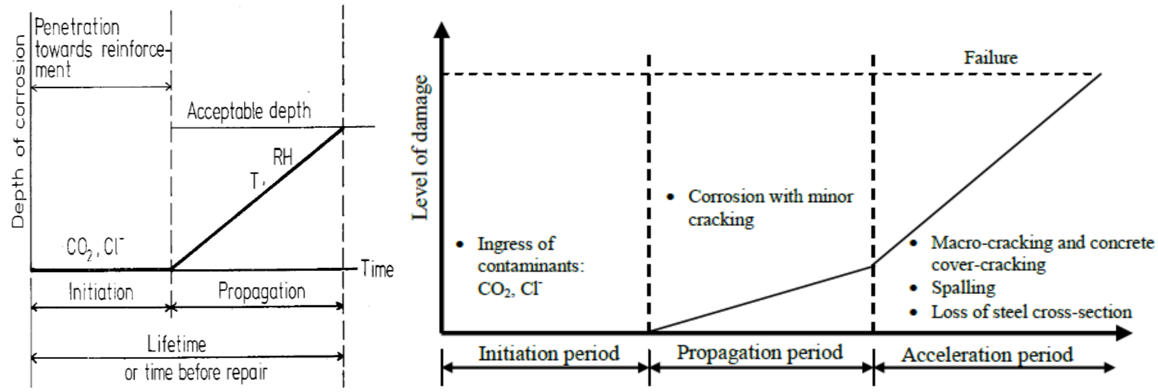



Figure 2. 7: Tuutti corrosion damage model and three-stage corrosion damage model (56).

Corrosion initiation period defines the time it takes for chlorides to penetrate from the external environment through the cover concrete and accumulate at the level of the steel in sufficient concentration to break down the passive protective layer on the steel surface and thereby cause active corrosion. If no cracks are present, the length of the initiation phase is a function of the penetrability of the concrete, the concrete cover, the binder type used, and the corrosion resistance of the bars (57). However, when cracks are present, then corrosion resistance of the bars may be the only factor that has any practical influence on the corrosion initiation phase.

The propagation stage is the period during which the corrosion rate and the accumulative amount of corrosion products gradually increase until an unacceptable level of damage has occurred. The length of this period therefore depends on the definition of unacceptable corrosion damage which directly depends on the rate of the corrosion process.

The acceleration period depicts a time when visible corrosion damage is clearly evident and the penetrability of concrete cover may be insignificant in controlling corrosion rate due to extensive macro-cracking and spalling. The boundary between the propagation and acceleration stages may be dependent on factors such as tensile strength of the concrete (55). From this it is possible to deduce that it is necessary to increase the tensile strength of the concrete. It is well known that concrete is good in compression but weak in tension. It is not a solution to increase the compressive strength of concrete; for example from 25MPa to 30MPa because it increases the compressive strength of the structure not the tensile strength. To this end it may be good option to include steel fiber in to the concrete matrix to increase the tensile strength, since it have better tensile strength when compared to plain concrete.

## 2.7. Beneficial Effects of Fiber Addition to RC with Respect to Corrosion

Fiber reinforced concrete defined as a composite material made with Portland cement, aggregates, and incorporating discrete discontinuous fibers. The reason to add such fibers to concrete is that; plain or unreinforced concrete is a brittle material, with a low tensile strength and a low strain capacity. The role of randomly distributed discontinuous fibers is to bridge across the cracks that develop provides some post-cracking “ductility”. If the fibers are sufficiently strong, sufficiently bonded to material, and permit the FRC to carry significant stresses over a relatively large strain capacity in the post-cracking stage.

Steel fiber reinforcement cannot therefore be regarded as a direct replacement of longitudinal reinforcement in reinforced and pre-stressed structural members. However, because of the inherent material properties of fiber concrete, the presence of fibers in the body of the concrete or the provision of a tensile skin of fiber concrete can be expected to improve the resistance of conventionally reinforced structural members to cracking, deflection and other serviceability conditions.

It is now well established that one of the important properties of steel fiber reinforced concrete is its superior resistance to cracking and crack propagation. As a result of this ability to arrest cracks, fire composites possess increased extensibility and tensile strength, both at first crack and at ultimate, particular under flexural loading; and the fibers are able to hold the matrix together even after extensive cracking. The net result of all these is to impart to the fiber composite pronounced post – cracking ductility which is unheard of in ordinary concrete. The transformation from a brittle to a ductile type of material would increase substantially the energy absorption characteristics of the fiber composite and its ability to withstand repeatedly applied, shock or impact loading.

Corrosion damage to reinforced concrete affects the strength of the reinforced concrete, as well as its serviceability. There are additional safety concerns, even if the reinforced concrete remains structurally sound. Advanced corrosion damage can cause spalling which can lead to additional property damage, or injury to persons. The addition of fiber to the mix design of reinforced concrete can help to mitigate the harmful effects of corrosion (26).

To understand how the addition of fiber can mitigate these effects, the method by which corrosion damages concrete should first be presented. After the initiation of corrosion at the surface of the steel reinforcement, corrosion products begin to form. These corrosion products occupy a larger volume than the original iron atoms. As these corrosion products continue to form, an internal pressure develops, as they attempt to expand away from the concrete. At first, they can fill up some of the pores or existing cracks within the concrete. Eventually, a critical pressure value is reached, and the surrounding concrete fails in tension. This causes one or more cracks to develop in the concrete at the steel-concrete interface. This process continues as the corrosion products continue to form, causing these cracks to propagate through the concrete. Eventually, one of these cracks will reach the exterior surface of the concrete. This damage to the concrete can become extensive throughout the reinforced concrete member or structure (26).

Furthermore, the addition of fiber to the concrete mix can help to limit this damage. Each fiber can act as a small tension element within the concrete, increasing its resistance to cracking. Once the concrete has begun to crack, the fiber engages and carries the tensile forces caused by the expansion of corrosion products. The process is similar to how steel is used in reinforced concrete (26).

In un-cracked SFRC, the ingress rate of chlorides is not adversely influenced by the presence of fibers compared to plain concrete. Hence the susceptibility of chloride induced corrosion of traditional bar reinforcement embedded in un-cracked concrete is similar when considering plain concrete and SFRC, assuming similar concrete/steel interface-conditions. With regards to the risk of corrosion of the steel fibers, a higher chloride threshold has been reported for steel fibers compared to traditional reinforcement bars, (58). Possible explanations for this increased chloride threshold of steel fibers compared to traditional bar reinforcement are:

**Size-effect**

- ✓ Due to the small dimensions of a single fiber, the potential differences and the polarization of one fiber, which is required for the formation of a corrosion cell (anode and cathode) on that fiber, are limited (58).
- ✓ The probability of chloride-induced (pitting) corrosion is stochastically distributed along the total steel surface (59), i.e. a small steel surface, as found on a fiber, corresponds to a small probability of pitting corrosion. Hence the probability of pitting corrosion of steel fibers is much smaller than for traditional reinforcement due to their reduced size.

**Casting conditions**

- ✓ In SFRC, the steel fibers are mixed-in during batching as opposed to casting of traditional bar reinforced concrete where the reinforcement is fixed during casting. The procedure used for batching and casting of SFRC leads to a reduction of the voids at the concrete/steel fiber interface compared to the procedure used for traditionally reinforced concrete which can result in bleeding-channels at the circumference of the reinforcement. A reduction of the number of voids at the concrete/Steel interface is known to reduce the chloride threshold. Additionally, the casting procedure of SFRC leads to a dense and well-defined concrete/steel fiber interface which results in the formation of a more even passive layer (compared to traditional reinforcement bars) (58).

Based on this information, i.e. the ingress rate of chlorides in un-cracked SFRC and the increased chloride threshold of steel fibers compared to traditional bar reinforcement, it is concluded that the susceptibility of chloride induced corrosion initiation of steel fibers in un-cracked concrete is considerably less than for traditional bar reinforcement in un-cracked concrete, while the probability of corrosion initiation of traditional reinforcement bars in SFRC is the same as for reinforcement bars in plain concrete.

## 2.8. Corrosion Rate Measurements

The rate of corrosion provides information on local corrosive conditions and on the best remedial action to achieve the most effective corrosion prevention. Corrosion measurements can provide early warning of damage in process that result in corrosion induced failure. Determining corrosion rate by measuring weight loss of samples is still in use because it is simple and effective in some situations. However, weight loss only gives an average corrosion of an entire metal sample over the entire test period. The less corrosive medium the longer it would take to get a meaningful test result. Linear polarization resistance (LRP) and potentiodynamic polarization curve measurements are the main electrochemical techniques used to evaluate corrosion rates (16).

Another technique to measure the percentages or degree of corrosion is by immersing the corroded bar into the rust removing chemicals like; phosphoric acid, baking soda, kerosene, and others. In this study the accelerated corrosion test (ACT) technique was used to corrode the reinforcement bar and measuring the percentage of corrosion through the current that flows for the certain period of time.

### 2.8.1. Accelerated Corrosion Test technique

The phenomenon of corrosion of reinforcement bar in concrete is a time dependent process. Under severe environmental conditions also, it takes years for the steel reinforcement to be corroded and to cause deterioration of reinforced concrete structures. However when it becomes imperative to evaluate the relative performance of different types of steel and binder in a short time, the accelerated corrosion test (ACT) can be adopted (60).

Particularly lacking in the testing of reinforced concrete for its corrosion protective properties has been an accelerated test or test procedure. An exemption to this is the technique developed by Tremper, which involves exposure of a freely corroding, partially submerged reinforced specimen. The principle of this technique is that the region near or below the water line becomes anodic due to $Cl^-$ penetration; and corrosion of this area is driven by the cathodic portion of the reinforcing steel, which is in the air. A disadvantage to this test procedure is the length of the time required for data to be evolved. This is in spite of relatively small concrete cover the reinforcing metal (Approximately 0.75 inches 0r 1.9 cm). For greater cover even longer times should be required. One of the famous test that is being adopted these days is accelerated corrosion test or Impressed voltage test. The test has gained popularity because of its simplicity and more user friendly behavior (60).

Corrosion tanks were used for the process of accelerating the corrosion of the steel rebar. These corrosion tanks were set up using the schematic shown in Figure 2.8. The tank itself was made of plastic. The counter electrode used was stainless steel. 3.5% NaCl solution was used as the electrolyte for the corrosion cell. Insulated copper wire was used to connect the positive terminal of the power supply to the reinforcing steel, while a stainless steel wire was used to connect the negative terminal to the counter electrode. An image of the corrosion tank can be found in Figure 2.8 (28).

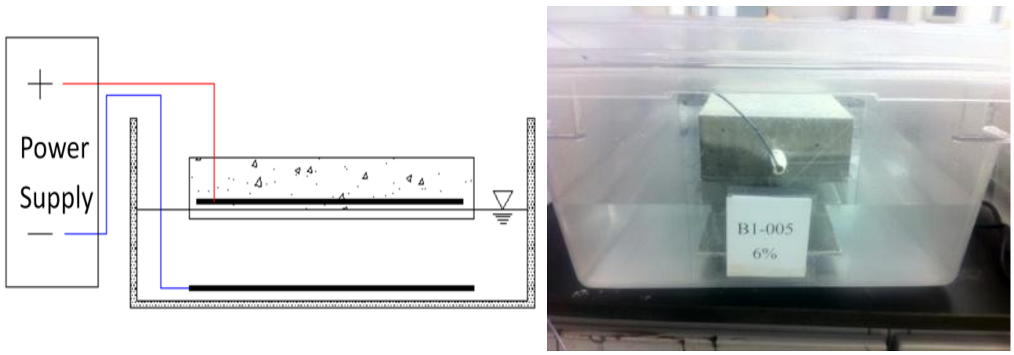



Figure 2. 8: Schematic of Corrosion Tank and Image of Corrosion Tank (28).

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 (61).

Furthermore, another author suggest that in order to accelerate the corrosion process, an electrochemical system depends on the concept of Faraday's second law is 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 consist electric circuit and force steel ions to translate from anode to cathode (62).

Figure 2. 9: Electrical Connection of and Rust on beams at the end of corrosion process (62)

### 2.8.2. Rust Removing Chemicals

Phosphoric acid and kerosene can be used to remove the rust from metals in its industrial application. It is commonly used for ferrous corrosion removal. It can be applied to corroded metals in order to convert the corrosion product to a water-soluble phosphate compound. When phosphoric acid/kerosene added to the corroded iron, iron oxide will be converted to a black iron phosphate coating. Then this iron phosphate coating can be removed easily by using dry compress. Therefore, phosphoric acid or kerosene can be used to remove corroded layer of the reinforcement steel bars. To do this the corroded reinforcement bar is immersed into the acid/kerosene until its total layer is covered by the acid/kerosene and stayed for certain minutes. By doing this the phosphoric acid/kerosene reacts with the corrosion product and decomposes it. Then after minutes removing the reinforcement bars from the acid/kerosene and cleaning by dry compress.

Finally, the corroded layer removed and the remaining of the bar is measured and the difference will be recorded and the depth of the corroded layer can be measured through mathematical computation.

# CHAPTER THREE

# RESEARCH METHODOLOGY AND MATERIALS

## 3.1. Methodology

### 3.1.1. Study Area

This research was conducted at Jimma and Addis Ababa city. Due to unavailability of testing machine the laboratory was done in Jimma and Addis Ababa, Lideta sub-city (Latitude: 9°1'4.44" and Longitude: 38°44'48.84") at Ethiopian institute of architecture, building construction and city development (EiABC) materials and research testing center, Addis Ababa University. Addis Ababa is a political and business capital city of Ethiopia. Most of huge infrastructures and structures of the country also amassed there. Therefore, it is a place where a lot of technological achievements and construction industries are emerged for the country. For this reason, it is comparatively a better place to conduct a research which needs latest technology, most of practical observations regarding the problems and the research topic and experimental equipment. The geographical location is presented in the Figure 3.1.

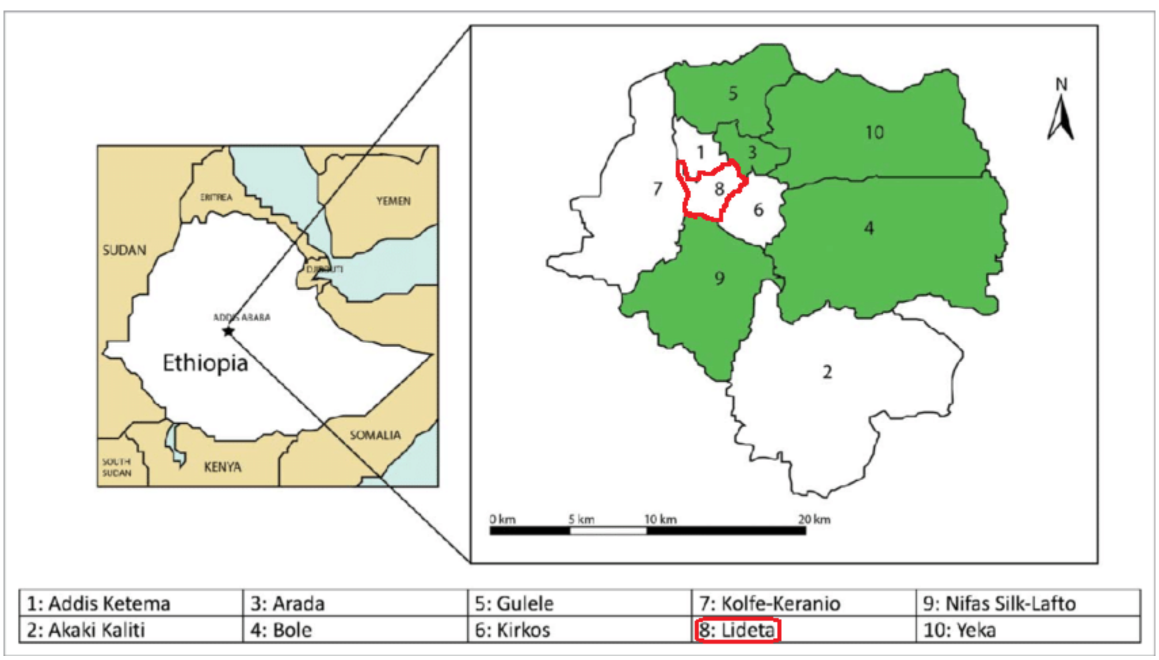



Figure 3. 1: Geographical location of Addis Ababa, Lideta sub-city.

### 3.1.2. Study Period

This research has been started on May 2019 and it was ended on December 2019.

### 3.1.3. Study Design

Basically, this research is focused on experimental investigations and tried to find out the effect of different independent variables on the dependent variable. To this end, experimental and explanatory/comparative/analytical research strategies were used in the study. Effort was made to gather data from experimental results, software analysis (like Microsoft Excel, Graph Graber) and related population to analyze the effect of the corrosion of reinforcement bar in steel fiber reinforced concrete beams. It compares controlled (both in terms of fiber and corrosion) and uncontrolled (both in terms of fiber and corrosion) experimental results. Generally, there are two comparing mechanisms in this laboratory work. The first is that plain (without fiber) reinforced concrete beam, cube test (for compressive strength test of the plain concrete) and cylinder test (for tensile split test of plain concrete) is compared with fiber reinforced concrete RC beam, cube test (for compressive strength test of fiber reinforced concrete) and cylinder test (for tensile split test of fiber reinforced concrete). And the second one is that regards with corrosion; the flexural strength of corroded reinforcement bar RC beam with different percentage of corrosion is compared with the flexural strength of un-corroded (control) reinforcement bar RC beam for both plain and fibered RC beams.

### 3.1.4. Study Population, Sample Size and Sampling Procedures

The experiments were done on both plain and fiber reinforced concrete which have characteristics average compressive strength of 25MPa. The reinforcement bar which used for reinforced concrete beam specimens have a diameter of 12mm deformed bar which have a steel grade of S-300.

This research used quota sampling techniques which is the combination of judgment sampling (non-probability sampling) and probability sampling (systematic sampling in this case). The procedure started first by deciding the sample size from the study population by arranging the sampling population in a systematic way. This involves the extraction of the steel fiber from used tires of car manually instead of burning the whole tire by fire to remove the plastic (simple way to remove the plastic and to extract the steel fiber within short period of time) which covers the steel inside it in order to maintain its normal strength and durability. ASTM A 820 establishes a minimum tensile strength and bending requirements for steel fibers as well as tolerances for length, diameter (or equivalent diameter), and aspect ratio. The minimum tensile yield strength required by ASTM A 820 is 345 MPa. The selection of percentage volume of the fiber, length of the fiber and percentages of rebar corrosion based on the previous literatures' findings and recommendations.

During the preparation of specimens eighteen sample of reinforced concrete beam was prepared. Among the samples; three control beams without fibers, three control beams with fiber, six corroded beam samples without fiber (three beams with 5% corrosion and three beams with 10% corrosion) and another six beams are corroded beam with fiber inclusion with 5% and 10% reinforcement bar corrosion. The amount of the steel fiber that used was 1% by volume of concrete with average length of 40mm based on the previous literature (63) and the diameter of the fiber is 0.89mm. Therefore, the aspect ratio (the ratio of fiber length to its diameter) of the fiber will be 45. The sample size have summarized in the Table 3.1.

The attainment of percentage corrosion is controlled by accelerated corrosion circuit. Accelerated corrosion circuit is an electrochemical circuit which used to corrode the reinforcement bar by using DC (Direct Current) power supply. The amount of corrosion of the reinforcement bar depends on the current which flows through the circuit. The required amount of current for the desired percentage corrosion is computed from Faradays' law.

The time required to corrode the reinforcement bars, using accelerated corrosion technique can be calculated based on Faraday's law;

$$ml = \frac{MIT}{zF} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. eqn. (3.1)$$

Where,

- *ml=weight of steel consumed (g)*
- *M = the atomic weight of the steel (56g for iron, Fe)*
- *I = current (Amperes)*

- *T = time (seconds)*
- *z = ionic charge* ($Fe \rightarrow Fe^{2+} + 2e^{-}$) *= 2*
- *F = Faraday's constant = 96500 Amperes. Second*
- *For practical purposes, the current density, i is used instead of current, I:*
    - $ml = \frac{M.i.S_{a}.T}{zF} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. eqn. (3.2)$
        - *Where $S_a$ is the surface area of the corroded steel and i is the current density level.*

In the accelerate corrosion process the reinforcement bar acted as an anode and the stainless steel acted as a cathode in this artificial corrosion cell. A schematic diagram showing the details of the connection between the reinforcement bars, the stainless steel tube and the power supply is shown in Figure 3.2.

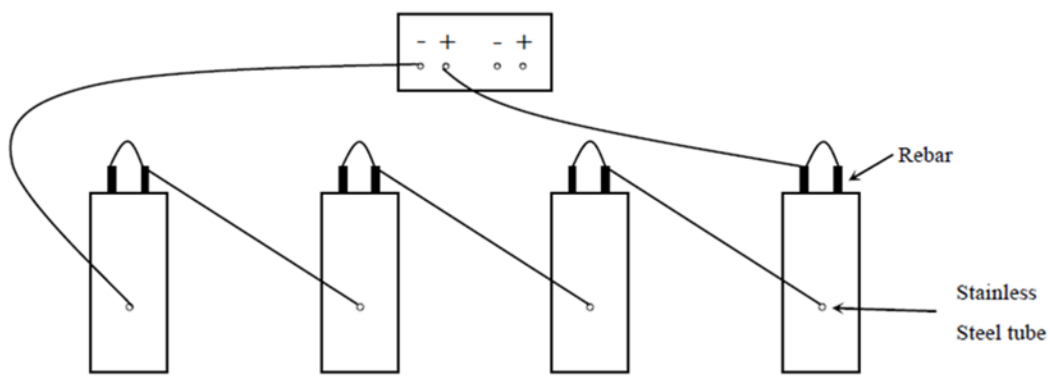



Figure 3. 2: Schematic diagram of accelerated corrosion circuit

Table 3. 1: Sample Size Description

<table>
<tr><td rowspan="5"></td><td colspan="6">Conditions</td><td colspan="2">Cubic specimens</td><td colspan="2">Cylindrical specimens</td></tr>
<tr><td colspan="2">Controlled</td><td colspan="4">Corroded beam Specimens</td><td rowspan="4">Plain</td><td rowspan="4">SFRC</td><td rowspan="4">Plain</td><td rowspan="4">SFRC</td></tr>
<tr><td rowspan="3">Without fibre</td><td rowspan="3">With fibre</td><td colspan="2">Without fibre</td><td colspan="2">With fibre</td></tr>
<tr><td colspan="2">Percentage corrosion</td><td colspan="2">Percentage corrosion</td></tr>
<tr><td>5%</td><td>10%</td><td>5%</td><td>10%</td></tr>
<tr><td>Number of Specimens</td><td>3</td><td>3</td><td>3</td><td>3</td><td>3</td><td>3</td><td>3</td><td>3</td><td>3</td><td>3</td></tr>
</table>

### 3.1.5. Study Variable

#### 3.1.5.1. Dependent variable;

- ✓ Flexural Strength of RC beam

#### 3.1.5.2. Independent variables;

- ✓ Reinforcement bar corrosion
- ✓ Steel fiber
- ✓ Concrete strength

### 3.1.6. Data Collection Process

This research have been conducted first by reviewing literatures which are related with the effect of reinforcement bar corrosion and the influence of steel fiber in the reinforced concrete beam which is the reinforcement bar was corroded and the effort have made to assess the problem in the case of Ethiopia specifically in Jimma town and Addis Ababa city. After literature review and desk study the checklists have been prepared in order to focus on the main important points during experimentation, data observation and recordings, result analysis and discussion.

### 3.1.7. Data Sources

The data for this research collected from both primary and secondary data sources in order to get precise and accurate information that makes the final findings more reliable.

#### 3.1.7.1. Primary Data Sources

The primary data for this research is results which obtained from experimental investigation. These experimental results include compressive strength of both plain and fiber reinforced concrete, cylindrical tensile split strength of both plain and fiber reinforced concrete, and different corrosion percentage of reinforcement bar and flexural strength of reinforced concrete beam (both with/without fiber and control/corroded). This study also try to asses some practical observations in constructions sites especially in Jimma town and Addis Ababa city regarding to reinforcement bar corrosion and the cause that made the reinforcement bar to be corroded during construction and how it was stocked in the market of building construction materials.

#### 3.1.7.2. Secondary Data Sources

The secondary data for this research was obtained from the literatures which are related with corrosion of reinforcement bars, fiber reinforced concrete, authorized offices and other related books and standard documents in order to support the research with accepted theories, standards and facts.

### 3.1.8. Data Processing, Analysis and Presentation

It is necessary to sort data into different group in order to make suitable for the comparison of results. All specimens are coded before starting experimentation and quality control check is mandatory for completeness and consistency of the data.

Qualitative data analysis: in order to analyze qualitative data thematic and comparative method of data analysis have used. By using these methods data from experimentation of different specimens was compared & contrasted and then theme have emerged from such data.

Quantitative data analysis**:** statistical software like, Microsoft Excel and Graph Grabber have used in order to analyze numerical data. The numerical data which analyzed by Microsoft excel and Graph Grabber was obtained from the universal testing machine which gives the results of compressive strength of concrete, cylindrical tensile split test, and flexural strength of the reinforced concrete beam specimens.

The results were presented in the form of;

- Graphs: - graph have used for the presentations' which shows the relationships between variables or parameters. For instance, the relationships between flexural strength of reinforced concrete beam versus corrosion percentage of reinforcement bar, load versus deflection of the beam and others.
- Tables:- tables are to be used for the presentation of numerical results obtained from experiments like slump test results of concrete (either plain or fiber reinforced), compressive strength of concrete (either plain or fiber reinforced), flexural strength of reinforced concrete beam (both with and without fiber and both control and corroded beams).
- Words: - the meanings or descriptions of tables, graphs, and other numerical data are presented in words. For example, the clarification of load versus deflection curve of the beam was presented in words.

## 3.2. Experimental Programs

Concrete is widely used construction materials throughout the word. There are many reasons for wider applicability of concrete as a construction material; its strength, molded into a desired shape, fire resistance and environmental friendly are the main reasons. However, if the ingredients of concrete don't attain the required specifications it is impossible to get the aforementioned advantages. Therefore, it is must to check and take measures to improve their quality, strength, quantity and other required parameters of each concrete ingredient (coarse aggregate, fine aggregate or sand, cement and water).

Since this research work was mainly focused on experimentation and also the study is experimental in nature. Hence, a lot of experiments have done during this research studies. In general this research has three experimental phases;

- Phase-1:- tests related with the construction materials;
  - ✓ Physical tests of concrete ingredients i.e. coarse aggregate, fine aggregate (sand), cement and water.
  - ✓ Extraction of the steel fiber from the tire and adjusting into the desired length of the steel fiber with appropriate aspect ratio.
  - ✓ Casting and placing of plain (without fiber) and fiber reinforced concrete.
  - ✓ Curing of concrete in order to attain the required strength.
  - ✓ Conducting compressive strength and tensile split test for both plain concrete and SFRC.
- Phase-2 :- electrochemical tests
- Phase-3 :- flexural strength tests

Therefore, this chapter deals with those three phases of the experimentation that mentioned above.

### 3.2.1. Steel Fiber

Concrete made from Portland cement, is relatively strong in compression but weak in tension and tends to be brittle. The weakness in tension can be overcome by the use of conventional steel bars reinforcement and to some extent by the mixing of a sufficient volume of certain fibers. The use of fibers also recalibrates the behavior of the fiber-matrix composite after it has cracked through improving its toughness.

The steel fibers, which are uniformly distributed in the cementations mix; this mix have various volume fractions, geometries, orientations and material properties. The types of steel fibers defined by ASTM A302, 2012 are:

- Type I: cold-drawn wire
- Type II; cut sheet
- Type III: melt-extracted
- Type IV: mill cut
- Type V: modified cold-drawn wire

Generally, SFRC is very ductile and particularly well suited for structures which are required to exhibit:

- ✓ High fatigue strength resistance to impact, blast and shock loads
- ✓ Shrinkage control of concrete
- ✓ Tensile strength, very high flexural, shear
- ✓ Erosion and abrasion resistance to splitting
- ✓ Temperature resistance, high thermal
- ✓ Earth quake resistance.

The degree of improvement gained in any specific property exhibited by SFRC is dependent on a number of factors that include: -

- ➢ Concrete mix and its age,
- ➢ Steel fiber content, volume fraction

- Fiber geometry, its aspect ratio (length to diameter ratio) and bond characteristics volume fraction.

#### 3.2.1.1. Type of Steel Fiber

The fiber type which used in this research work is the steel fiber that extracted from used tire of car. The selection of fiber type considers not only structural performance of the fiber but also its impact on the environment and economic aspects through reuse of wastages. It is well-known that in Ethiopia when the tire is complete its design life for the purpose it is mainly intended to be used as a roller for vehicles, people use them for different purposes. Most of the people used them as support for cloth washing pool or it may be used as traditional sandals with low price which made from the rubber part of the tire. But the steel fiber inside the tire have no that much application for technical purposes in a scientific way in Ethiopia, therefore those are the reasons to make this type of steel fiber preferable than other fibers in addition to subjective benefits (structural use).

## 1. Fiber Extraction and Cutting

During this study except the method of fiber extraction the data with regards to the steel fiber was taken from the work of N. Bedewi (63).

**Fiber extraction**

The steel fiber which inside the car tire is complicated to extract from the plastic rubber cover of the tire, however, it is simple to extract the steel from the plastic fiber by burning the whole part of the fiber and finally the steel alone lefts. But the fiber extracted through this method lost its normal strength due to the burning process. Hence, to keep the normal strength of this fiber; it is better to use other method of extraction.

During this research work the effort was made to extract the steel fiber without altering its strength through mechanical way of extracting the steel from the tire through the manual process of extraction. The method of extraction is discussed as follows;

- Firstly, the type of the tire is to be checked whether it is the product of identical company to minimize the difference in their production details. The whole tire which used here is the product of Addis Tyer Company.
- Then, the internal part of the tire which contains the steel was removed from the tire by using blade like material which was prepared by the traditionally skilled labor (this needs the labor force who was familiar with this and related activities).
- After isolating the part of the tire which contain the band of the steel fiber, there also few plastic rubber which binds this steel fiber together; to extract the clean steel fiber which is free from any plastic rubber it needs to slice the plastic cover with blade like material until the small part of a single steel fiber is pulled out from the band of steel fibers.
- If the small part of a single steel fiber is pulled out from the band of steel fibers, then it is easy to pulling out the remaining part of the steel fiber till the fiber is out of the plastic rubber cover, since the band of one tire steel fiber is interconnected and have one continuous prismatic dimension which was bind together for better functioning of the intended use. To make the extracting process simpler it is better to cut the part of the extracted fiber with the length suitable for further extraction.
- Finally, when the extraction of the whole steel fiber was completed, it should be adjusted by straightening the curved fiber to measure the length suitable for mixing of concrete matrix.

**Tensile strength of the steel fiber**

Since the diameter and tensile strength of steel fibers in a tire vary from one factory product to another, the tires used for the extraction of the steel fibers are all from the same source (the product of Addis Tire Factory). The steel fiber used in this research was obtained by extracting the steel wire carefully from the rubber without altering its original properties. They are clean and free from rubber as shown in the Figure 3.4.

From the work of N. Bedewi the tensile strength test was conducted on 10 randomly chosen samples by Testometric machine (M350-5KN) in Matador Addis Tire Laboratory. From the test an average tensile strength of 1892.6 MPa were found for the steel fiber which is extracted from used tire of Addis tire product. The obtained test results is in good agreement with ASTM and meet the tensile requirement specified in ASTM A820 stated as “The average tensile strength shall not be less than 345 MPa and the tensile strength of any one of the ten test specimens shall not be less than 310 MPa." The result of the test was presented in the Annex-I.


Figure 3. 3: Testometric machine for tensile strength test of steel fiber. (63)

**Fiber cutting**

Structural performance of steel fiber reinforced concrete members depends on different parameters such as aspect ratio of fiber, the geometrical shape of fiber and placement/orientation of fiber in concrete.

According to ACI Committee 544.3 R Report, fiber length varies from 12.7 to 63.5 mm. The most common fiber diameters are in the range of 0.45 to 1.0 mm. Modern steel fibers have shapes which include round, oval, rectangular, and crescent cross sections, depending on the manufacturing process and raw material used.

**Aspect Ratio of the Steel Fiber**

A numerical parameter describing a fiber is its aspect ratio; which is defined as the fiber length, l divided by an equivalent fiber diameter, d (l/d). Among the factors that have a major effect on workability is the aspect ratio. The workability decreases with increasing aspect ratio. Typical aspect ratio (l/d) range from 30 to 150 for length dimensions of 0.1 to 7.62 cm typical fiber diameters are 0.25 to 0.76 mm for steel and 0.02 to 0.5 mm for plastic. In practice it is very difficult to achieve a uniform mix if the aspect ratio is greater than about 100. The steel fiber (extracted from used tire) which used in this research is according to the standard ASTM A302, 2012.

During this reach work there are two methods of steel fiber cutting was used to cut the fiber in to the desired length for mixing. The first method is by using the grinder machine and the second is the manual method by using the instrument known as *gutet* (the local name). The latter is used due to the malfunctioning of the grinder machine during the operation.

The length of the fiber which used in this research was taken from the work of N. Bedewi (63). According to his work averagely by considering different parameters a better result (especially in terms of workability,) was obtained by using the steel fiber with the dimension of 40mm and it is also the average length among he have used during his work (it is doesn't mean that the fiber with this length was yield a better results in all parameters he have compared and contrast). The diameter of the fiber was found to be 0.89mm. Therefore, the aspect ratio (the ratio of the fiber length to fiber diameter) of the fiber which used in this study is 45.

Figure 3. 4: Extraction and Cutting of the steel fiber

**Percentage of fiber**

The usual amount of steel fibers ranges from 0.25% by volume, i.e., 20 kg/m$^3$, to 2% by volume, i.e., 157 kg/m$^3$. The low end of the range applies to lightly loaded ground slabs, some precast applications, and composite steel deck toppings. The upper end of the range is common for security applications such as safes, vaults, and others. Recent investigations have also given rise to highly reinforced SFRC containing up to 20% volume of steel fibers. The recent developments are due to the introduction of new generation of additives such as super-plasticizers and micro-silicas, which allow the use of high volume of steel fibers and high-strength concrete. From the work of N. Bedewi among three different percentages of steel fiber (0.5%, 1% and 1.5%), the better results obtained with 1% i.e. 78.5Kg/m$^3$ volume of concrete; hence, the amount of the steel fiber which used here is 1% by volume of concrete.

## 3. 2.2. Aggregate

An aggregate for concrete making is any hard, inert material composed of fragments in a wide gradations range of sizes, which is mixed with a cementing material and water to form concrete. Aggregates should be clean sound, tough, durable and uniform in quality. Aggregates makes up 65 to 75 % of the volume of concrete. Therefore the quality of concrete produced is very much influenced by the property of its aggregates.

**3. 2.2.1. Classification of aggregates**

Aggregates are generally classified based on their sources (natural or artificial), their chemical composition (argillaceous, siliceous or calcareous), their weight (heavy aggregates with specific gravity more than 4.0, normal weight aggregates with specific gravity between 2.4 and 3.0 and lightweight aggregate which are used to make lightweight concrete), their size or the mode of preparation.

It is well-known that the method of aggregate classification widely used in concrete works is that based on aggregate size. Aggregates bigger than about 4.75mm in diameter is classified as coarse aggregate (type CA) and the one smaller as fine aggregate (type FA).

In most parts of Ethiopia fine aggregates (sand) is obtained from river beds while coarse aggregate is prepared from crushed rock and sold as crushed stone aggregate. The preparation of crushed rock or crushed stone aggregate at the quarries requires several steps which include the blasting of the rock which is conveyed by belt conveyor to the crushing plant. The crusher which could be different type is generally adjusted so as to give a range of aggregate sizes. The output is made to pass over a set of screens and the different fraction sizes collected for sale. In the south western regions of Ethiopia (Jimma, Illubabor, Kaffa, Gamo-Gofa) where river sand is scarce; fine aggregate (sand) is prepared from stone as crushed sand. But, for this research the fine aggregate is taken from Werabe.

**3. 2.2.2. Grading of Aggregates**

In order to calculate the proportions of different ingredients and produce concrete of desired properties, it is important and indeed required to determine the characteristics of the aggregate which include among things its gradation.

The grading or particle size distribution of aggregate is determined by a sieve analysis. Any sieve down the list has half the clear opening of the one above.

A sample of aggregate for sieve analysis is first surface dried and then sieved through the series, starting with the largest. The weight retained on each sieve is recorded and the percentage computed. The summation of the cumulative percentage of the material retained on the sieves (not including the intermediate sieves) divided by 100 is called the fineness modulus. It is used as an index to the fineness or coarseness and uniformity of aggregate supplied, but it is not an indication of grading since there could be an infinite number of grading which will produce a given fineness modulus.

## 3. 2.3. Coarse Aggregate

The maximum content of all concrete ingredients is covered by the coarse aggregate. It is about 50% of the volume of concrete. Therefore, it is necessary to take care during the selection of coarse aggregate to have the required quality of concrete during the mix design. It should be free from dust, inert and other unnecessary particles that negatively affect the concrete. The coarse aggregate should also have optimum moisture content which is suitable for the required type of concrete.

During the laboratory work of this study for concrete, the coarse aggregate is washed to remove the dust and inert particles on the surface of concrete, and also the sieve analysis also have done in order to get the required grain size distribution of coarse aggregate according to ASTM 136-01.

### 3. 2.3.1. Sieve Analysis of Coarse Aggregate

In this study, mechanical shaker was used to determine the particle grain size distribution of the coarse aggregate. The maximum size of aggregate that used during this experiment is 19mm in diameter.

## 3. 2.4. Fine aggregate

Coarse aggregate is blended with finer aggregates such as sand to fill in the spaces left between the large pieces and to lock the larger pieces together. This reduces the amount of cement paste required and decreases the amount of shrinkage that could occur. In order to fulfill those requirements the tests related with fine aggregates have done during this experimental work. Sieve analysis and silt content test of the sand are done. The fine aggregate was washed to make free from any impurities like dust and silt.

#### 3. 2.4.1. Sieve Analysis of Fine Aggregate

Similarly as coarse aggregate, the fine aggregate also sieved by using mechanical sieve shaker by using the maximum sieve size of 9.5mm at the top.

Figure 3. 5: Mechanical sieve shaker

### 3. 2.5. Silt Content Test of Sand (FA)

This test helps to know the percentage of silt content in sand (fine aggregates). The amount of silt content affects the strength and the quality of concrete. The major Effects of silt fines on concrete properties are;

- ✓ The increase in sand fines content decrease the compressive and tensile strength of concrete,
- ✓ Inadequate sand fine content also has adverse effect on the strength of concrete due to poor adhesion,
- ✓ The decrease in slump value as a result of increase in fines percentage in sand can be attributed to the fact that finer particles have larger surface area which absorbs more water in concrete mix. As a result the increases in water/cement ratio result to a decrease in the compressive strength of concrete.

The Apparatus that used in the silt content test of sand was the measuring cylinder which has the volume of 250ml, the sand specimen and pure water. After preparing the necessary materials that related with the test the following procedures are followed in order to get the amount of silt in the sand;

- The measuring cylinder should be free from moisture and dry,
- Fill the water up to 50ml of the measuring cylinder,

- Then the sand was added to measuring cylinder until it reaches 100ml,
- Close the mouse of the measuring cylinder and shake well then keep the cylinder on a flat surface,
- The silt content shall be settled on the sand layer after 2 hours
- Note down the silt layer alone volume as $V_1$ ml (settled over the sand)
- Then note down the sand volume (below the silt) as $V_2$ ml

Therefore, repeating the above procedure for three samples (in order to minimize the error during sampling and experimentation) finally the result to be the average of the three samples.

**Computations of silt content percentage**

Consider $V_1$ = volume of silt layer (silt settled over the sand layer) and $V_2$ = volume of sand layer (which is settled under the silt) the percentage of silt content can be calculated as;

$$percentage\ of\ silt = \left(\frac{V1}{V2}\right) * 100 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots . (3.3)$$

The permissible silt content percentage in sand is only up to 6% (ASTM) and 10% (IS).

a) Shaking b) keeping to settling down

Figure 3. 6: Silt test for fine aggregate (sand)

### 3.2.6. Cement

Cement is a finely ground inorganic material which has cohesive and adhesive properties; able to bind two or more materials together into a solid mass. Cohesion is the tendency of a material to maintain its integrity without separating or rupturing within itself when subjected to external forces while adhesion is the tendency of a material to bond to another material.

Cement when mixed with water form a paste which sets and harden by means of hydration reactions, and which after hardening retain its strength and stability even under water. Generally cementing materials are of two types: non-hydraulic cements: are cements which are either not able to set and harden in water (e.g. Non-hydraulic lime) or which are not stable in water (e.g. gypsum plasters) and hydraulic cements: are cements which are able to set and harden in water, and give a solid mass which does not disintegrate, i.e. remain stable in water (e.g. Portland cement). The type of the cementing material which used in this study was hydraulic Portland cement.

Portland cement is a cementing material which is obtained by thoroughly mixing together calcareous or other lime bearing material with, if required, argillaceous and/or other silica, alumina or iron oxide bearing materials burning them at a clinkering temperature and grinding the resulting clinker.

#### 3.2.6.1. Compound Composition of Portland Cement

The raw materials used for the manufacture of cement consist mainly of lime, silica, alumina and iron oxide. These oxides interact with one another in the kiln at high temperature to form more complex compounds. The maximum amount of alumina and iron oxide is determined by the need to control the rapidity of the setting of cement. The silicate phases form about 70% of the weight of an ordinary Portland cement. Despite their small percentages, the minor compounds can have strong influence on the properties of fresh and hardened cement paste.

Table 3. 2: Chemical composition of cement with the symbolic representation

| **Chemical compound** | **Oxide composition** | **Abbreviation** |
|---|---|---|
| **Major compounds** | | |
| Tricalcium silicate | $3CaO.SiO_2$ | $C_3S$ |
| dicalcium silicate | $2CaO.SiO_2$ | $C_2S$ |
| Tricalcium aluminate | $3CaO.Al_2O_3$ | $C_3A$ |
| Tetracalcium alumino ferrite | $4CaO.Al_2O_3.Fe_2O_3$ | $C_4AF$ |
| **Minor compounds** | | |
| Gepsum | $CaSO_4.2H_2O$ | $CSH_2$ |
| Free lime | CaO | C |
| Magnesia | MgO | M |
| ***Alkali oxides*** | | |
| • Soda | $Na_2O$ | N |
| • Potassa | $K_2O$ | K |
| Manganese oxide | $Mn_2O_3$ | |
| Titanium oxide | $Ti_2O$ | |
| Phosphorous Pentoxide | $P_2O_5$ | |

### 3. 2.6.2. Types of Portland cement

In many parts of the world, different types of Portland cement are manufactured for different uses. The main types are listed in the Table 3.3 shown below:

Table 3. 3: Percentage composition of cement type

| **European Description** | **ASTM Description** | **Potential composition, %** | | | |
|---|---|---|---|---|---|
| | | $C_3S$ | $C_2S$ | $C_3A$ | $C_4AF$ |
| Ordinary Portland cement | Type-I | 49 | 25 | 12 | 8 |
| Modified cement | Type-II | 46 | 29 | 6 | 12 |
| Rapid hardening Portland cement | Type-III | 56 | 15 | 12 | 8 |
| Low heat Portland cement | Type-IV | 30 | 46 | 5 | 13 |
| Sulphate resisting Portland cement | Type-V | 43 | 36 | 4 | 12 |

The type of cement which used in this study is the product of Dangote cement factory ordinary Portland cement (OPC) or Type-I.

#### 3. 2.6.3. Properties of ordinary Portland cement

It is well-known that the properties of cement significantly affect the strength of concrete structures. Therefore, it is necessary to primarily focus on the properties before preceding any related studies with concrete. The major properties of the ordinary Portland cement are;

- ✓ Ordinary,
- ✓ 70 % of total cement consumption,
- ✓ Moderate rate of hardening,
- ✓ Minimum fineness i.e. $225m^2/kg$.

### 3. 2.7. Water

Water fit for drinking is generally suitable for making concrete. Substances in water which, if present in large amounts, may be harmful are; salt, oil, industrial wastes, alkalies, sulphates, organic matter, silt, sewage, etc. tests by the sense of smell, sight or taste should reveal such impurities, however water of doubtful quality should be submitted for laboratory analysis and tests.

Water used in concrete mixes has two functions, the first is to react chemically with the cement which will finally set and harden, and the second function is to lubricate all other materials and make the concrete workable.

The cement used in the concrete mix needs less than 30 percent by weight of water for hydration. However because of the dual function of water, concrete containing such small amount of water would be very dry and very difficult to fully compact. The quantity of water used in a concrete mix has, therefore, to be sufficient to fully satisfy both functions, namely the hydration of the cement and lubrication of the dry materials. For these reasons water used in concrete mixes is usually much greater than the 30% of the cement weight. On the other hand this extra water, the one needed for lubrication, will not be needed once the concrete is placed and compacted. Since all of it will evaporate when the concrete dries, leaving voids which would make the concrete porous and consequently weak, it is important that this portion of the water should be kept to a minimum.

#### 3. 2.7.1. Water- Cement Ratio

Water for a concrete mix is normally added during mixing. It is quite possible, however, that the aggregates are wet and release their surface water when they are mixed. On the other hand aggregates could be very dry and readily absorb the free water. The relationship between the total free water and the cement is given by what is known as the water-cement ratio of the concrete mix. For a given type of cement and aggregate, the strength and porosity of the paste structure (cement stone) are dependent almost entirely upon the water-cement ratio. For a given consistency, the lower the water-cement ratio in the fresh concrete, the less the voids (for reasons given above) more strength, less drying shrinkage and more durability, meaning, all in all a better resulting concrete.

### *3. 2.8.* Concrete Mix Design

The proportioning of concrete mixtures, more commonly referred to as mix design, is a process that consists of two interrelated steps: the first is selection of the suitable ingredients (cement, aggregate, water and admixtures) of concrete and the second one is determining their relative quantities, which is known as proportioning to produce, as economically as possible, concrete of the appropriate workability, strength and durability.

Since cement is much more expensive than aggregate, it is clear that minimizing the cement content is the most important single factor in reducing concrete costs. In general, this can be done by;-

- Using the lowest slump that will permit adequate placement,
- Using the largest practical maximum size of aggregate,
- Using the optimum ratio of coarse to fine aggregates,
- When necessary, by using appropriate admixtures.

There are a lot of mix design methods. Some of the prevalent concrete mix design methods are:

- *ACI*; American Concrete Institute Mix Design Method,
- *DOE*; Department of Environment Mix design practice (British),
- *DIN;* Mix design Method (German)
- *IS*; Indian Standard Mix Design Method

This study uses *ACI*; American Concrete Institute Mix Design Method. According to this mix design method the following information for available materials are useful:

- Sieve analyses of fine and coarse aggregates.
- Unit weight of coarse aggregate.
- Bulk specific gravities and absorptions of aggregates.
- Specific gravities of Portland cement and other cementitious materials, if used.
- Optimum combination of coarse aggregates to meet the maximum density grading.

According to ACI 211.1 concrete mix design method the following procedures have been followed during the mix design of this study's experimental session.

Collected data from physical tests of concrete ingredients for mix design:-

- Fineness modulus of selected fine aggregate = 3.1
- Unit weight of dry rodded coarse aggregates =1692kg/$m^3$
- Specific gravity of coarse and fine aggregate in saturated surface dry condition is 2.75 and 2.6 respectively
- Absorption characteristic of both coarse and fine aggregate is 0.88% and 3.1% respectively
- Specific gravity of ordinary Portland cement = 3.15
- Free surface moisture in sand = 1.01%

**Step-1 Choice of slump**

The slump recommended for beams and a reinforced wall is ranges between 100mm and 25mm (the maximum and minimum slump respectively).

**Step-2: Choice of maximum size of aggregate**

Large nominal maximum sizes of well graded aggregates have fewer voids than smaller sizes. Hence, concretes with the larger-sized aggregates require less mortar per unit volume of concrete. Generally, the nominal maximum size of aggregate should be the largest that is economically available and consistent with dimensions of the structure. For this study the maximum size of aggregate that used was 19mm.

**Step-3: Estimation of mixing water and air content**

The quantity of water per unit volume of concrete required to produce a given slump is dependent on: the nominal maximum size, particle shape, and grading of the aggregates, the concrete temperature, the amount of entrained air, and use of chemical admixtures. According to ACI code for the slump of 25mm and 19mm maximum size of aggregate and for non-air entrained concrete, the maximum water content is 185kg/m$^3$.

**Step-4: Selection of water-cement ratio**

Approximate and relatively conservative values for concrete containing Type-I Portland cement can be taken from ACI table. For 25MPa compressive strength of concrete at 28 days and for non-air entrained concrete water-cement ratio by mass is 0.5.

**Step-5: Calculation of cement content**

The amount of cement per unit volume of concrete is fixed by the determinations made in Steps 3 and 4 above. The required cement is equal to the estimated mixing-water content (Step 3) divided by the water-cement ratio (Step 4). If, however, the specification includes a separate minimum limit on cement in addition to requirements for strength and durability, the mixture must be based on whichever criterion leads to the larger amount of cement. Therefore, the amount of cement for this experiment to be found;

$$\frac{water}{cemnet} = 0.61 \rightarrow amount\ of\ cement = \frac{185}{0.5} = 370Kg/m3$$

**Step-6: Estimation of coarse aggregate content**

Appropriate values for this aggregate volume are given in ACI table 11.4. From this table for maximum size of 19mm coarse aggregate and fineness modulus of sand 3.1, the dry rodded bulk volume of coarse aggregate is 0.60 per unit volume of concrete (volume of dry rodded coarse aggregate is 0.6).

Hence, the coarse aggregate content for unit weight of dry rodded coarse aggregates 1692kg/m$^3$ to be;

The weight of coarse aggregate = 0.6*1692Kg/m$^3$= 1015.2kg/m$^3$

**Step-7: Estimation of fine aggregate content**

At completion of Step 6, all ingredients of the concrete have been estimated except the fine aggregate. Its quantity is determined by difference. Either of two procedures may be employed:

- The weight method or
- The absolute volume method.

**The weight method**

If the weight of the concrete per unit volume is assumed or can be estimated from experience, the required weight of fine aggregate is simply the difference between the weight of fresh concrete and the total weight of the other ingredients. First estimate of weight of fresh concrete can be determined from ACI Table. From ACI table 11.9, the first estimate of density of fresh concrete for 19mm maximum size of aggregate and non-air entrained concrete is 2355kg/m$^3$.

Therefore, the weight of fine Aggregate =2355-(185+370+1015.2) = 784.8 Kg/m$^3$.

Table 3. 4: Summary of the amount of concrete ingredients

| Ingredient | Weight (Kg/m$^3$) | Absolute volume (cm$^3$) |
|---|---|---|
| Cement | 370 | 128*10$^3$ |
| Water | 185 | 185*10$^3$ |
| Gravel | 1015.2 | 369*10$^3$ |
| Air | | 20*10$^3$ |
| Total | | *702*10$^3$* |

Therefore, absolute volume of fine aggregate = (1000-702)*10$^3$= 298*10$^3$

Weight of fine aggregate = 298*2.6= 774.8 Kg/m$^3$

**Proportions**

Table 3. 5: The proportion of concrete ingredients

| Ingredients | Cement | Sand (FA) | Gravel (CA) | Water |
|---|---|---|---|---|
| Quantity (Kg/m$^3$) | 370 | 774.8 | 1015.2 | 185 |
| Ratio | 1 | 2.09 | 2.74 | 0.5 |
| One bag cement (Kg) | 50 | 104.5 | 137 | 25 |

**Step-8: Adjustment for moisture in the aggregate**

The aggregate quantities actually to be weighed out for the concrete must allow for moisture in the aggregates. Generally, the aggregates will be moist and their dry weights should be increased by the percentage of water they contain, both absorbed and surface. The mixing water added to the batch must be reduced by an amount equal to the free moisture contributed by the aggregate which is total moisture minus absorption.

- ✓ Fine aggregate has surface moisture of 1.01%.
- ✓ Weight of fine aggregates =(774.8+0.02*774.8)Kg/m$^3$= 790.30 Kg/m$^3$
- ✓ Coarse aggregate absorbs 0.88% of water.
- ✓ Weight of coarse aggregate=(1015.2+0.0088*1015.2)Kg/m$^3$= 1024.13 Kg/m$^3$

**Adjust the amount of water based on moisture content**

The required mixing water =185-790.30(0.02-0.031)-1024.13(-0.0088) = 202.72 Kg/m$^3$

**Step-9: Final design proportion**

Table 3. 6: Final concrete ingredients proportion for mix design

| Ingredients | Cement | Sand (FA) | Gravel (CA) | Water |
|---|---|---|---|---|
| Quantity (Kg/m$^3$) | 370 | 790.30 | 1024.13 | 202.72 |
| Ratio | 1 | 2.14 | 2.77 | 0.55 |
| One bag cement (Kg) | 50 | 107 | 138.5 | 27.5 |

## Mixing of the steel fiber in concrete

Mixing of fiber reinforced concrete needs careful conditions to avoid balling of fibers, segregation, and in general the difficulty of mixing the materials uniformly. Increase in the aspect ratio, volume percentage and size and quantity of coarse aggregate intensify the difficulties and balling tendencies. Steel fiber content in excess of 2 % by volume and an aspect ratio of more than 100 are difficult to mix.

It is important that the fibers are dispersed uniformly throughout the mix. This can be done by the addition of fibers before the water is added. During the mixing of the steel fiber with concrete, firstly, the coarse and fine aggregate was mixed thoroughly before adding the fiber into the concrete mixer; this is done to avoid balling of fibers before mixing with other ingredients of the concrete matrix. The formation of balling of fibers during the mixing process may result in non-uniform steel fiber concrete mix.

### Fiber orientation

One of the differences between conventional reinforcement and fiber reinforcement is that in conventional, bars are oriented in the direction desired while fibers are randomly oriented. According to the experimental work of N. Bedewi the effects of randomness, mortar specimens reinforced with 0.5 percent volume of fibers were tested. In one set specimens, fibers were aligned in the direction of the load, in another along the direction perpendicular to that of the load, and in the third randomly distributed.

From his work it was observed that the fibers aligned in the direction of the applied load offered more tensile strength and toughness than randomly distributed or perpendicular fibers and randomly distributed fibers yield a better result than perpendicularly oriented fibers. In this study for the purpose of ease and to be practical fiber was oriented randomly during the casting of SFRC.

Figure 3. 7: Steel fiber extracted from used tire.

Figure 3. 8: Fiber reinforced fresh concrete (SFRC)

### 3. 2.9. Fabrications of Beams, Cubes and Cylindrical Specimens

During construction materials laboratory session of this study the concrete specimens are fabricated by using the steel molds for casting beams, cubes and cylindrical specimens. The dimension of the steel mold for reinforced concrete specimen is 500mm long, with the depth and the width of 100mm (500mm*100mm*100mm). The cubic mold for compressive strength test of concrete have a dimension of equal length, width and depth which is 150mm*150mm*150mm.

And also the cylindrical steel mold with a dimension of 200mm in length and 100mm in diameter was used for tensile splitting test of the concrete.

The reinforcement bar used for the flexural strength test is with the diameter of 12mm. for the ease of electrochemical testing and accurate results which the study intended to i.e. the residual flexural strength of corroded reinforced concrete beam; only one longitudinal bar was used by keeping its center to minimize the eccentricity and other related effects. The spacer which made from concrete with 25mm thickness is tied with the longitudinal reinforcement bar as a concrete cover.

**3. 2.9. 1. Tests on fresh concrete**

Unfortunately, no direct test methods measure the workability and consistency of fresh concrete. However, within a range there are approximate methods; among them slump test is used widely. In this study also the method of slump have used for the workability test of the fresh concrete.

**3. 2.9. 2. Slump test**

It is the most widely used method of checking the consistence of concrete at the construction sites. The method as devised in the late twenties in America was a simple frustum of cone. There are some slight differences in the details of both the dimensions of the apparatus and the details of the procedures used in different countries. According to ASTM 143 the frustum of a cone should be 305mm high, 203 and 102mm diameter at the bottom and top respectively. After being moistened, it is placed on a smooth surface with the smaller opening at the top, and filled with the concrete sample in three layers, each approximately one third of the volume of the cone. Each layer is tamped 25 times with the standard straight tamping rod 16mm in diameter while the mold is held firmly against its base. Immediately after filling, the cone is slowly lifted leaving the unsupported concrete to slump. The consistence is measured in terms of the amount it has slumped in centimeters. Three types of results could be obtained;

i) The sample could slump evenly all around in which case it is said to be a true slump,
ii) Part of the top cones might shear off and slide down an inclined plane giving a shear slump, and
iii) The cone could be completely collapse.

The first type of slump indicates a well-proportioned concrete whereas the second, or shear slump occurs usually with harsh mixes with lack of cohesion. Mixes of stiff consistence have a zero slump.

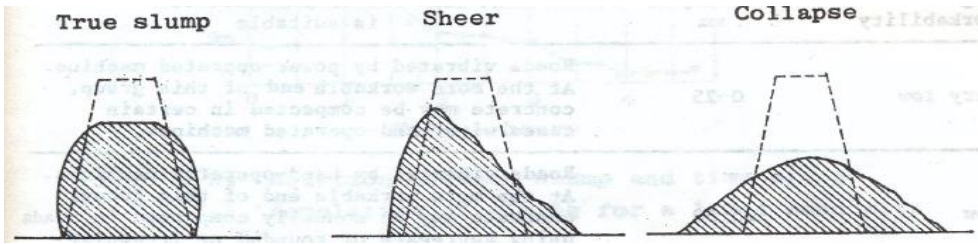



Figure 3. 9: Types of slump. (64)

The slump test measures the resulting behavior of a compacted inverted cone of concrete under the action of gravity. It indicates the consistency or wetness of the concrete.

Alternatively, the test can be done by using a metal mold in the shape of a conical frustrum known as a slump cone or Abrams cone that is open at both ends and has an attached handle. The tool typically has an internal diameter of 4 in (100 mm) at the top and of 8 in (200 mm) at the bottom with a height of 1 ft. (300 mm).The cone is placed on a hard non-absorbent surface. This cone is filled with fresh concrete in three stages. Each time, each layer is tamped 25 times with a 2 ft (600 mm)-long bullet-nosed metal rod measuring 58 in (16 mm) in diameter. At the end of the third stage, the concrete is struck off flush with the top of the mold. The mold is carefully lifted vertically upwards with twisting motion, so as not to disturb the concrete cone. The concrete then subsides. This subsidence is termed as slump, and is measured to the nearest 5 mm if the slump is <100 mm and to the nearest 10 mm if the slump is >100 mm.

### 3.2.9.2. Interpretation of slump results

As discussed before the slumped concrete takes various shapes, and according to the profile of slumped concrete, the slump is termed as true slump, shear slump or collapse slump. If a shear or collapse slump is achieved, a fresh sample should be taken and the test repeated. A collapse slump is an indication that the mix is too wet. Only a true slump is of any use in the test. A collapse slump will generally mean that the mix is too wet or that it is a high workability mix, for which the slump test is not appropriate. As per ACI 211.1 Very dry mixes; having slump 0 – 25 mm are used in road making, low workability mixes; having slump 10 – 40 mm are used for foundations with light reinforcement, medium workability mixes; 50 - 90 for normal reinforced concrete placed with vibration, high workability concrete; > 100 mm.

The slump test is referred to in several testing and building codes, with minor differences in the details of performing the test.

The American standards explicitly state that the slump cone should have a height of 12-in, a bottom diameter of 8-in and an upper diameter of 4-in. The ASTM standards also state in the procedure that when the cone is removed, it should be lifted up vertically, without any rotational movement at all.

In the United Kingdom, the standards specify a slump cone height of 300 mm, a bottom diameter of 200 mm and a top diameter of 100 mm. The British Standards do not explicitly specify that the cone should only be lifted vertically. The test should be carried out by filling the slump cone in three equal layers with the mixture being tamped down 25 times for each layer.

Frustum of cone SFRC Slump (true slump)

Figure 3. 10: The frustum of cone and the slump for SFRC

Figure 3. 11: Plain concrete slump (true slump)

According to European Standard EN 206-1:2000 the slump test is suitable for slumps of medium to high workability, slump in the range of 5 – 260 mm, the test fails to determine the difference in workability in stiff mixes which have zero slump, or for wet mixes that give a collapse slump. It is limited to concrete formed of aggregates of less than 38 mm (1.5 inch). In this study the slump for normal concrete is 30.5mm and for that of steel fiber reinforced concrete is 12mm which is that both are the true slumps with maximum aggregate size of 19mm (0.8 inch). Therefore, the results attained the requirement of both EN 206-1:2000 and ACI 211.1.

### 3. 2.10. Compressive Strength of Concrete

Concrete mixtures can be designed to provide a wide range of mechanical and durability properties to meet the design requirements of a structure. The compressive strength of concrete is the most common performance measure used by the engineer in designing buildings and other structures. The compressive strength of concrete is measured by breaking cylindrical or cubic concrete specimens in a compression-testing machine. The compressive strength is calculated from the failure load divided by the cross-sectional area resisting the load and reported in units of pound-force per square inch (psi) in US customary units or mega-Pascal (MPa) in SI units. Concrete compressive strength requirements can vary from 2500psi (17 MPa) for residential concrete to 4000psi (28 MPa) and higher in commercial structures. Higher strengths up to and exceeding 10000 psi (70 MPa) are specified for certain applications. A test result is the average of at least *two standard-cured* strength specimens made from the same concrete sample and tested at the same age. In most cases strength requirements for concrete are at an age of 28 days.

Strength of concrete is generally tested after 28 days as concrete cube strength or concrete cylinder strength. The reason for testing concrete strength after 28 days is that concrete gains strength with time after casting. It takes much time for concrete to gain 100% strength and the time for same is still unknown. The rate of gain of concrete compressive strength is higher during the first 28 days of casting and then it slows down. Many of the previous studies shows that concrete gains 16% strength in one day, 40% in 3 days, 65% in 7 days, 90% in 14 days and 99% strength in 28 days. Thus, it is clear that concrete gains its strength rapidly in the initial days after casting i.e. 90% in only 14 days. When, its strength have reached 99% in 28 days, still concrete continues to gain strength after that period, but that rate of gain in compressive strength is very less compared to that in 28 days.

After 14 days of casting concrete, concrete gains only 9% in next 14 days. So, the rate of gain of strength decreases. Still there is no clear idea up to when the concrete gains the strength, 1 year or 2 year, but it is assumed that concrete may gain its final strength after 1 year of casting. So since, the concrete strength is 99% at 28 days, it is almost close to its final strength, thus it is possible to rely upon the results of compressive strength test after 28 days and use this strength as the base for design and evaluation.

The CIP-35 (Concrete in Practice) specifies that cylindrical specimens for acceptance testing should be 6x12 inch (150x300mm) size or 4x8 inch (100x200mm) when specified. The smaller specimens tend to be easier to make and handle in the field and laboratory. The diameter of the cylinder used should be at least three times the nominal maximum size of the coarse aggregate used in the concrete.

Figure 3. 12: Curing of beams, cubes and cylindrical specimens in the curing tank

Figure 3. 13: RC beams after 28 days of curing

### 3. 2.10.1. Interpretation of Compressive strength Test Results

According to IS-456 the compressive strength test of concrete specimens can be interpreted as follows; (65)

- The test results of the sample shall be the average of strength of three specimens,
- The individual variation should not more than 15% of the average strength,
- Cube weight should be verified in order to ensure compacted concrete density.
  - For concrete density of 2400Kg/m$^3$ the corresponding weight of cube in Kg will be;

*Weight of concrete specimen = volume of specimen x concrete density*

Hence, for 150mm x 150mm x 150mm cubic specimen of concrete the theoretical weight of the specimen will be;

$$w = 2400 * (0.150 * 0.150 * 0.150) = 8.1kg$$

Therefore, the minimum cube weight for each specimen shall be 8.1kg. Low cube weight (less than 8.1kg) could be an indication of poor compaction.

For 200mm x 100mm cylindrical specimen of concrete the theoretical minimum weight of the specimen should be;

$$w = \gamma c(\pi r^2)l = (2400 * \pi * 0.05^2 * 0.2) = 3.77kg$$

Therefore, the minimum cylindrical specimen weight for each specimen shall be 3.77kg. Low cylindrical specimen weight (less than 3.77kg) could be an indication of poor compaction.

**3. 2.10.2. The Compressive Strength Test for plain concrete**

For plain concrete three cubic specimens was prepared to test the compressive strength. The specimens given identification code as a subscript to the capital letter S as $S_{10,}$ $S_{20}$, $S_{30}$ which described as the compressive strength of plain concrete specimen 1 and 0 designate for zero percentage of fiber ($S_{10}$) and in similar manners for specimen 2 and 3 as shown in the Figure 3.14 (the letter "C" which shown in specimens figure was replaced by "S" to minimize confusion with the popular destination of the number which describes the amount of compressive strength of the concrete specimen).


Figure 3. 14: Cubic specimens for compressive strength test of plain concrete


Figure 3. 15: Mass of the specimens (cubic and cylindrical)

Figure 3. 16: SFRC Cubic specimen in compressive strength testing machine

**3. 2.10. 3. The Compressive Strength Test for SFRC**

Similarly as plain concrete cubic specimen, for steel fiber reinforced concrete three cubic specimens was also prepared to test the compressive strength of SFRC. The specimens given identification code as a subscript to the capital letter C as $C_{1F,}$ $C_{2F}$, $C_{3F}$ which described as the compressive strength of fiber reinforced concrete specimen 1 ($C_{1F}$) and in similar manners for specimen 2 and 3 as shown Figure 3.17.


Figure 3. 17: *SFRC Cubic Specimen*


Figure 3. 18: SFRC Cubic specimen in compressive strength testing machine

### 3. 2.10.4. Splitting Tensile Strength Test of Concrete

Tensile strength for concrete specimen is defined as the tensile stresses developed due to application of the compressive load at which the concrete may crack. Splitting tensile strength is used to evaluate the shear resistance provided by concrete in reinforced concrete members. (ASTM: C496-96). Split tensile strength for concrete relates to its tension strength. This is obtained by performing split tensile test on concrete specimen. The concrete specimen in this test is taken as cylindrical in shape.

According to ASTM the results of two properly conducted tests on the same material, should not differ by more than 14% of their average for splitting tensile strength.


Figure 3. 19: Cylindrical specimens for splitting tensile strength test of plain concrete

### 3. 2.10.5. Cylindrical Tensile Splitting Strength Test for SFRC

Since, concrete is good in compression the main aim of adding the fibers into the concrete matrix is not to increase the compressive strength of concrete. But it is a matter of increasing the tensile strength of the concrete, because concrete is poor in tension.

Therefore, in this study three cylindrical steel fibered concrete specimens are prepared, in order to test the tensile strength. These sample specimens are coded with subscript after the letter "T" as $T_{1F}$, $T_{2F}$ and $T_{3F}$ in which the subscript describes the tensile splitting strength of cylindrical fibered concrete specimen 1 ($T_{1F}$) and similarly for specimens 2 and 3 as shown in the Figure 3.20.


Figure 3. 20: SFRC cylindrical specimens

Figure 3. 21: Plain and SFRC cylindrical specimen in tensile splitting testing machine

## 3. 2.11. Electrochemical Tests

For electrochemical experiments there are different components are participated to accelerate the corrosion process. These components which act together were known as corrosion cell. Generally this corrosion cell has four Parts. They are;

- ❖ Anode (location where corrosion takes place)
  - Oxidation Half-Reaction
- ❖ Cathode (no corrosion)
  - Reduction Half-Reaction
- ❖ Electrolyte (Soil, Water, Moisture, etc.)
- ❖ Electrical Connection between anode and cathode (wire, metal wall, etc.)

Electrochemical corrosion can be stopped by eliminating any one of the above four components.

In order to study effect of the reinforcement bar corrosion which is embedded in concrete, it is must to corrode the steel reinforcement bar in reinforced concrete beam. To do that the aforementioned four components of corrosion cell must be fulfilled. Therefore, the following materials are used in this experimental program.

### 3. 2.11.1. Anode

Anode is one component of corrosion cell in which corrosion (Oxidation Half-Reaction) takes place. In this experiment the reinforcement bar that embedded in concrete serve as an anode. In practical case the reinforcement bar which embedded into concrete are fully covered by the concrete, due to this reason it is difficult to connect the reinforcement bar with positive terminal of DC (Direct Current) power supply in order to precede the anodic reaction during electrochemical process. It is possible to attach the copper wire with the reinforcement bar which will be plugged into the positive terminal of DC power supply before casting the concrete, but it is difficult to manage the safety of the copper wire which is embedded in the concrete (it may be broken or detached from the steel bar after casting the concrete) during or before the process of electrochemical reaction is takes place. Therefore, it is a better way to extend the reinforcement bar from the concrete with optimum length which is enough for connecting the copper wire. However, the extended bar for the purpose of wire connection may have a disadvantage due to localized short circuit effect at the edge of the extended wire just around the extension point. In order to minimize such effect (end effect) the part of the reinforcement bar where the bar has direct contact with concrete was coated with epoxy.


Figure 3. 22: Steel epoxy and epoxy coated reinforcement bar

**3. 2.11.2. Cathode**

It is the corrosion cell component in which reduction half-reaction where takes place. In this component no corrosion should takes place. Therefore the material that used as a cathode should be stainless steel.

There are a lot of stainless steels, among them aluminum, graphite, copper are might be used as a cathodic component. In this experiment the graphite rod with the diameter about 12mm and about 150mm length used as a stainless steel and to complete the reduction-half reaction.

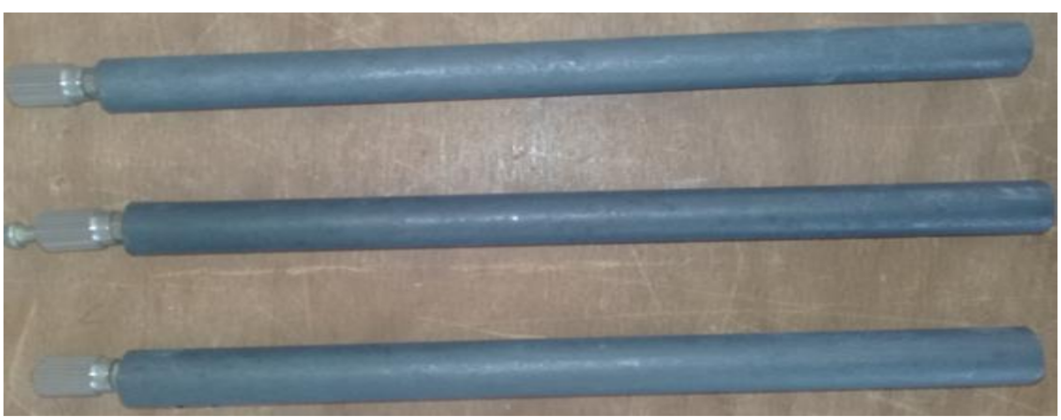


Figure 3. 23: Graphite rod

### 3. 2.11.3. Electrolyte

Water is serving as an electrolyte, and to facilitate the corrosion process certain amount of sodium chloride powder was added. In this experiment the amount of sodium chloride added to the solution is 3% by mass of water in the solution.

Figure 3. 24: Sodium Chloride (NaCl)

### 3. 2.11.4. Power Source

In order to facilitate the electrochemical reaction in the corrosion cell the electric current is necessary. Therefore, for that matter direct current (DC) power supply is used as a power source. In order to make the full-circuit the positive terminal of the DC power supply connected with the reinforcement bar which was embedded into concrete through the copper wire and this make the anodic reaction to proceed. Another terminal which is the negative one was connected to the graphite rod to complete the cathodic reaction in the corrosion cell.

During electrochemical reaction the current supplied to the corrosion cell should be constant current as much as possible. According to the experimental work of (66) accelerated corrosion using constant current is recommended over the constant voltage. This is due to the reason; accelerated corrosion using constant current will produces greater structural damage to the concrete structure than using constant voltage with the same other parameters.

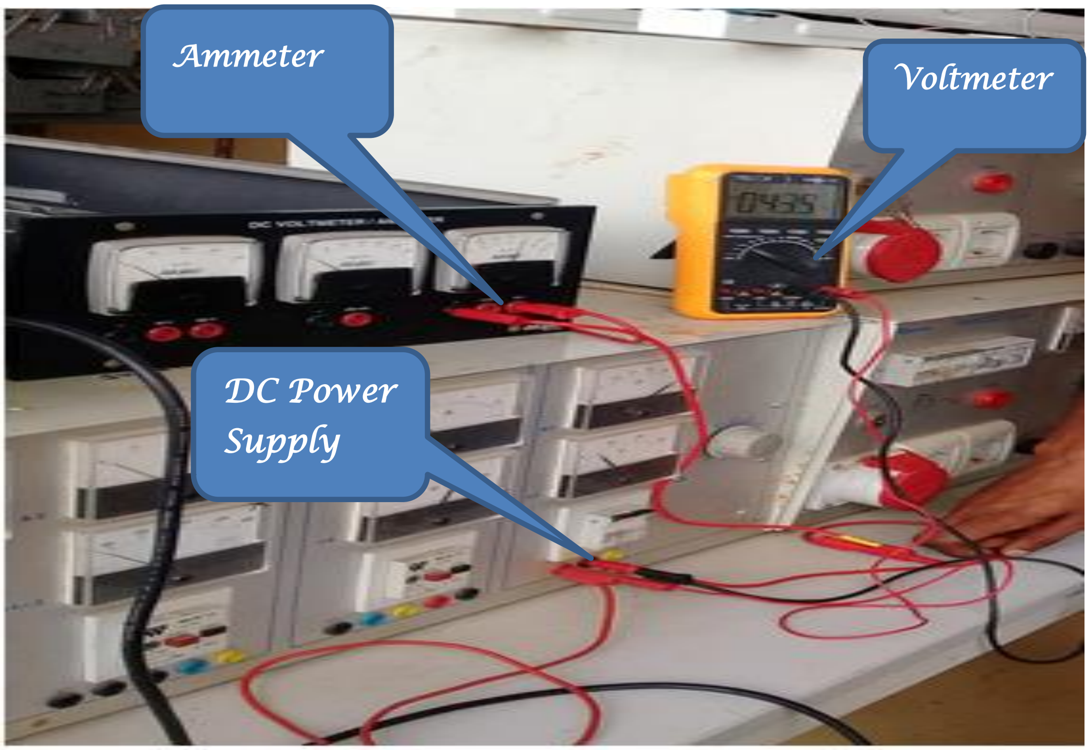



Figure 3. 25: DC power supply with ammeter and voltmeter

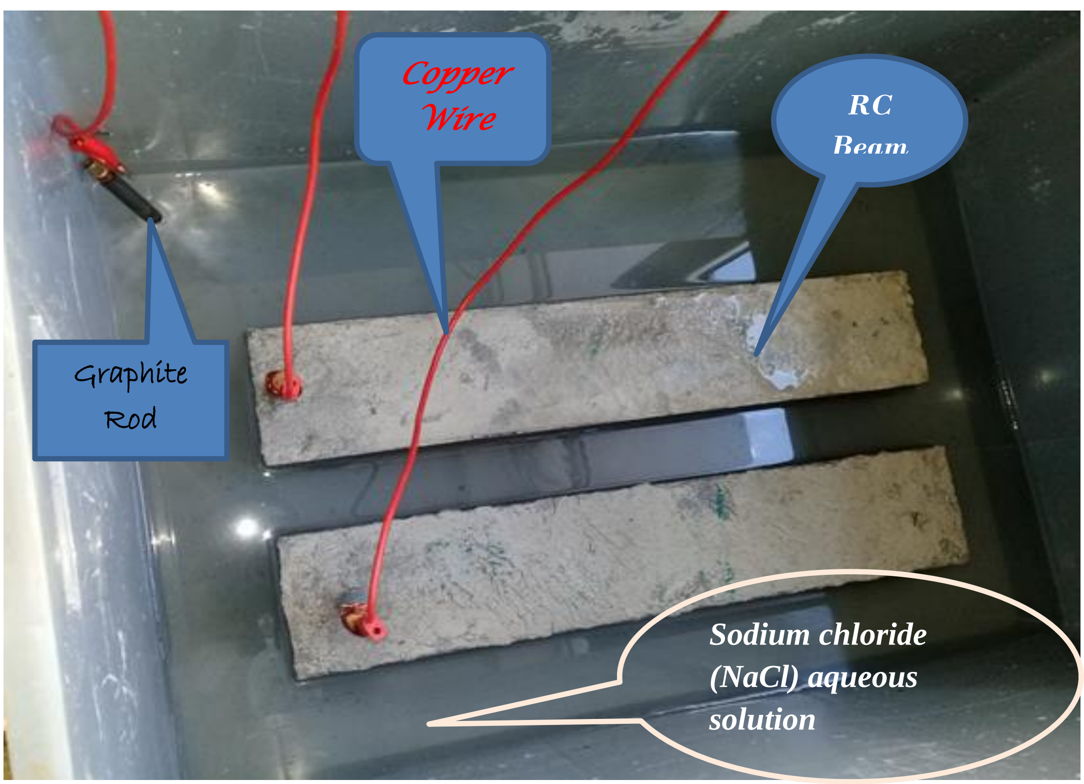



Figure 3. 26: Corrosion cell

### 3.2.11.5. Accelerated Corrosion Test (ACT)

Corrosion in nature is time consuming process, because the electrochemical reaction takes place through process. The time required to corrode the reinforcement bar depends on the properties/type and amount of the components which participate in electrochemical reaction. Especially if the electrical resistivity of the corrosion cell is high, it may take a long time for corrosion to occur.

Practically, the reinforcement bar which considered in this study is embedded in concrete (which have high electrical resistivity), it need a long period of time to corrode the steel reinforcement bar that embedded in the concrete due to high electrical resistivity of the concrete unless a favorable conditions which make the bar exposed for corrosion environment is satisfied. This make difficult to study the reinforcement bar corrosion in reinforced concrete structure in short period of time. In order to overcome such difficulty the method of accelerated corrosion helps to study the effect of reinforcement bar corrosion on reinforced concrete structures that would happen in decades or century within short period of time (within two or three weeks).

Accelerated corrosion is a way of corroding the steel within short period of time to study about corrosion of metals. It might be simple to corrode metals especially, those metals which have no any protective layer on it and freely exposed for environment. But it is difficult and complex to corrode the metals which have corrosion protective layer with high electrical resistivity like concrete. Another difficulty in corrosion science is to know how much it was corroded. To address such difficulties supplying electric current through direct current (DC) power supply and using the sodium chloride aqueous solution as an electrolyte to facilitate the electrochemical reaction in the corrosion cell. During this electrochemical process electrons are migrated from steel reinforcement bar (which is iron, Fe) to graphite rod (which is the stainless steel act as a cathode) by using the sodium chloride aqueous solution (an electrolyte) as a medium of electron flow.

As an electrochemical process continues hydroxyl ion ($OH^-$) from aqueous solution is react with an iron atom which disintegrated due to oxidation reaction at anodic half-cell reaction. After the reaction of this ions i.e. $Fe^{2+}$ and $OH^-$ the solid accumulation of ferrous hydroxide ($Fe(OH)_2$) is produced and cumulated at steel concrete interface inside reinforced concrete beam. This accumulation is known as corrosion product or rust. This product finally results in the corrosion of reinforcement bar.

The level of the effects of reinforcement bar corrosion on concrete structures is depends on different factors; among them the main parameters are as follows;

- Percentage by mas of reinforcement bar corrosion,
- Duration in which the reinforcement bar exposed for corrosion environment,
- Percentage content of the aqueous solution (the amount of sodium chloride in this case) which acts as an electrolyte,
- The type of accelerated corrosion techniques (either constant current or constant voltage) that used during accelerated corrosion test and others.

In this study there are two percentages by mass of reinforcement bar was used during the experimentation. In order to take the average results, three reinforced concrete beam with 5% by mass of the controlled (un-corroded) bar and another three reinforced concrete beam with 10% corrosion is tested for the residual flexural strength of corroded reinforcement bar RC beam without steel fiber (plain) and another six beams with similar amount and parameters was used for steel fiber reinforced concrete (SFRC) beam.

Therefore, totally twelve beams are subjected for accelerated corrosion test. Among them six beams are with plain concrete (without fiber) and the rest six are with steel fiber that extracted from used tire.

**3. 2.11.6. Corrosion Percentage Calculation (Δw)**

The amount of steel loss, $\Delta w$ during accelerated corrosion test is calculated based on Faraday's law by using the equation;

$$\Delta w = \frac{MIT}{zF} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (3.5)$$

Where,

$\Delta w$ = *mass of steel consumed due to corrosion (in grams, g)*

*I = the current that flow during ACT (in amperes, A)*

*T = the time duration for which corrosion to occur (in seconds, sec.)*

*F = the Faraday's constant (F = 96,500 amperes. second)*

*z = the ionic charge (2 for iron, Fe)*

*Since from anodic reaction of the corrosion cell,* $Fe \rightarrow Fe^{+2} + 2e^{-}$

*M = the atomic weight of iron (Fe) metal, M = 56g (from the periodic table)*

**Computation of the amount of current for 5% corrosion**

First calculate $\Delta w$, in grams, since $\Delta w = 5\%$, and the original mass of the reinforcement bar obtained from the average weight of three 60cm in length is Wo = 523.33g (the mass before reinforcement bar corrosion), and 14 days of corrosion duration was taken, hence

$$\Delta w = 0.05 * 523.33 = 26.17g$$

Now, from Equation (3.3) the required amount of current to obtain 5% corrosion can be calculated as;

$$I = \frac{\Delta wzF}{MT} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots. (3.6)$$

$$I = \frac{26.17\text{g} * 2 * 96500A.sec.}{56g * (14 * 24 * 60 * 60sec.)} = 0.075A = 75mA = 75000\mu A$$

Therefore, the amount of current to be used in order to get 5% corrosion within 14 days of duration is 75mA.

**Computation of the amount of current for 10% corrosion**

In order to keep the current flow for both cases similar and to minimize the effect of current variation, it is recommended to vary only the duration. Therefore, by using the value of current, I = 75mA we can calculate the value of T, the duration.

$$\Delta w = 0.10 * 523.33 = 52.33g$$

Hence, from Equation (3.3) the required duration to obtain 10% corrosion can be calculated as;

$$T = \frac{\Delta wzF}{MI} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots.. (3.7)$$

$$T = \frac{52.33\text{g} * 2 * 96500A.sec.}{56g * 0.075A} = 2.4192 * 10(6)sec. = 28\ days$$

Therefore, based on Faradays' law 5% and 10% weight loss of reinforcement bar which embedded in the concrete was studied.

### 3.2.11.7. Duration for Corrosion

As discussed before, corrosion is time dependent electrochemical process; it is necessary to consider the effect of duration in corrosion cell. When only weight loss of the bar is considered during corrosion study, it might be possible to corrode the reinforcement bar within short period of time (may be within hours) with required percentage of corrosion. This is done by supplying large amount of current to accelerate the corrosion process. But it is not the representative of real situation for corrosion of reinforcement bar in reinforced concrete structure. Because the actual effect of reinforced bar corrosion is not only weight loss.

In the case of corroding the bar within short period of time it is not possible to get the effect of corrosion other than weight loss, which is not the practical case. In practical case when the bar is corroded, the corrosion products accumulated at steel-concrete interface and the accumulation of these corrosion products is time consuming phenomenon to exert the tensile stress and de-bonding between concrete and steel. When more corrosion products cumulated through time it starts to exert the tensile stress on the surrounding concrete, then this stress results in the cracking of concrete, since as discussed before concrete is poor in tension.

To this end it is better to corrode the bar within appropriate time to account and observe such effect which comes through time. Therefore, time is the most influential factor during accelerated corrosion test.

During this experimental research the electrochemical process is carried out within two weeks (14days) for 5% corrosion and four weeks (28days) for 10% corrosion. To facilitate the electrochemical reaction the aqueous solution of sodium chloride with 3% by mass of water was used as an electrolyte.

Before soaking the RC beam in the solution, the powder of sodium chloride have thoroughly mixed with water.

It is difficult to keep both electric current and voltage simultaneously constant during electrochemical reaction. Therefore it must to maintain one of them constant. It is not the process to be done randomly, due to their difference in damage effect with similar electric current flow and other components which participate in corrosion cell.

From this study trial experiment and other researches experimental result (as discussed in the literature review section), keeping the current constant will result in more corrosion damage than constant voltage. Therefore, this experimental work also has done by keeping the electric current constant.

### 3. 2.11.8. The Mechanism of Reinforcement Bar Corrosion in RC Beam

As discussed earlier in the foregoing discussions, the mechanism of corrosion of steel reinforcement bar in concrete is two-fold; either by *chloride attack* or by *carbonation* of concrete. Chloride attack was considered during the accelerated corrosion experimental program of this study.

The consideration of chloride attack during this experimental program is based on the simulation of the actual case in which chloride ions are introduced into the concrete structure. The sources of chloride which introduced into the concrete are;

- Chloride may be cast into concrete using accelerator agents containing chloride ions,
- Using sea water for concrete mixing, ground water with high chloride concentration and deicing salts and,
- Aggregates containing chlorides.

### 3. 2.11.9. How the Reinforcement Bar is corroded during ACT

The process of corrosion is electrochemical, i.e. a chemical reaction involving the transfer of electrons from one specimen to another. It involves a redox (oxidation-reduction) reaction.

Redox reaction is a type of chemical reaction that involves a transfer of electrons between two species. Therefore, in corrosion cell during ACT the following reaction takes place;

**Oxidation (Anodic) Reaction**

Oxidation is the loss of electrons during a reaction by a molecule, atom or ion. When iron reacts with oxygen it forms a chemical called rust because it has been oxidized i.e. the iron has lost some electrons.

$$Fe \rightarrow Fe^{+2} + 2e^-$$

Where;

*Fe* – is the iron metal (the reinforcement bar)

$2e^-$ – Valence electrons (the number of electrons in the outer most shell during electron configuration of iron atom)

**Reduction (Cathodic) Reaction**

A reduction reaction is one in which a reactant in a chemical reaction gains one or more electrons. Reduction reaction always occurs in conjunction with oxidation reactions, in which a reactant loses one or more electrons. A reduction reaction is only one half of a redox reaction. The other half is the oxidation reaction.

$$2H^+ + 2e^- \rightarrow H_2$$

$$O_2 + 2H_2O + 4e^- \rightarrow 4(OH^-)$$

This reaction is takes place due to the water which added in aqueous solution.

**The Complete Reaction**

$$Fe + 2H^+ \rightarrow Fe^{+2} + H_{2\,(g)}$$

$$\frac{1}{2} O_2 + H_2O + Fe \rightarrow Fe^{+2} + 2OH^-$$

The products of this reaction often combine to form precipitate;

$$Fe^{+2} + 2OH^- \rightarrow Fe(OH)_2$$

Where, $Fe(OH)_2$ is the corrosion product which is known as rust.

### 3. 2.11.10. Mechanism of Chloride Attack in RC beam during ACT

When reinforcement bar is embedded into concrete the passivity is provided by an insoluble layer formed on the steel reinforcement bar surface which protects the steel bar against corrosion. But, chloride ions cause damage to reinforcing steel by attacking the passive film and corrosion starts at certain locations along the steel bar where the loss in passive layer occurs. This mechanism leads to iron (the reinforcement bar) dissolution and hydrolysis produces $FeOH^+$ and $H^+$. Then this produces an acidic environment that attacks anions (e.g. $Cl^-$ and $OH^-$). Therefore, the presence of chloride ions in concrete leads to variation in the anodic behavior of steel. As chloride content increased the breakdown potential decreased and the passive range reduced.

## 3. 2.12. Flexural Strength Test

Flexural strength (or the modulus of rupture) is the amount of force an object can take without breaking or permanently deforming. It is a measure of the tensile strength of concrete, (a measure of a resistance against failure in bending). Flexural strength really tells the maximum amount of stress the material can take, and it’s quoted as a force per unit area. The flexural strength of the specimen is measured by using a device which known as flexural strength test machine. A flexural test machine is a device that can apply increasing amounts of force and precisely record the amount of force at the point of breaking. There are two methods of testing flexural strength; a three-point bending test and four-point bending test but they’re very similar.

- For a **three-point bending** test, the continually increasing load is applied in the center of the sample until there is a break or permanent bend in the material. The schematic representation of three-point bending test (center-point bending) is shown in the Figure 3.27.

- A **four-point bending** test, is very similar, except the load is applied at two points simultaneously again towards the center of the sample, which is that the two loads or forces are applied so they split the specimen into thirds.

For this study the three-point bending test was used due to the reason of availability. During the flexural test of this research the ASTM C-293 test standard procedures was followed.

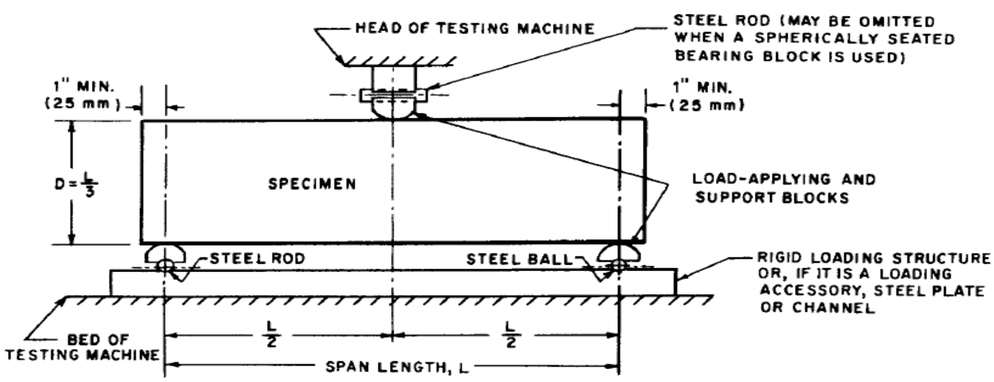



Figure 3. 27: Schematic representation of three-point bending (ASTM C-293)

For a three-point test, the flexural strength σ can be calculated as;

$$\sigma = \frac{3Pmax\ \mathrm{L}}{2\mathrm{bd}^2} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (3.8)$$

Where,

σ = the flexural strength, in Pascal (Pa) or newton per meter squared ($N/m^2$)

$P_{max}$ = the maximum load applied (peak failure load), in Newton (N)

L = the length of the specimen, in meter (m)

b = the width of the specimen, in meter (m)

d = the depth of the specimen, in meter (m)

Figure 3. 28: Three-point bending machine

During the flexural test the necessary data are collected for each specimen. The peak failure load, deflection, failure mechanisms and other related data are presented in the following sections.

**Size of concrete specimen for flexural test**

- Indian standard (IS) states that a size of 100mm width, 100mm depth, and span of 500mm can be used if the maximum aggregate size used is not greater than 19mm.
- British standard (BS) specifies square specimen cross-section with 100mm or 150mm dimension and the span ranges from four to five times specimen depth.

In the current study the size of the specimen used for flexural test have the dimensions of 100mm width, 100mm depth, and span of 500mm since the maximum aggregate size that used during the concrete mix design is 19mm.


a) Adjustment for flexural test b) the specimen failed in flexure

Figure 3. 29: Flexural strength test

The length between the centers of the support to the external edge of the RC specimen in both sides used in this study is 25mm, which is that the minimum value that recommended by ASTM E290. In this case, the span between rollers is 450mm.

All of eighteen RC beam specimens (both plain/SFRC and un-corroded/corroded) are tested with similar conditions in the three-point bending machine. At the end of the test the testing machine gives different results for each beam and also additional parameters like, peak failure load, maximum deflections and load-deflection curves. However, the flexural strength of the specimen cannot be obtained directly from the testing machine instead it can be obtained indirectly by using equation (3.8).

# CHAPTER FOUR

# RESULTS AND DISCUSSION

In this chapter the results of different experimental tests and their discussion was presented. The test results were presented in the form of tables, charts, graphs and figures and each have their own respective discussions.

## 4.1. Sieve Analysis Result

The test results obtained from the sieve analysis was presented in the form of table and their compliance with the standard depicted in the form of graph.

Table 4. 1: The results of Sieve Analysis of Coarse Aggregate

| Sieve Size (mm) | Wt. of Sieve (g) | Wt. of Sieve + retained (g) | Retained (g) | Retained % | Cumulative retained % | Cumulative Passing% | Mini limit, % | Max. limit , % |
|---|---|---|---|---|---|---|---|---|
| 25 | 1186 | 1186 | 0.00 | 0.00 | 0.00 | 100.00 | 100.00 | |
| 19 | 1414 | 1477 | 63.00 | 3.13 | 3.13 | 96.87 | 90.00 | 100.00 |
| 12.5 | 1164 | 1954 | 790.0 | 39.30 | 42.43 | 57.57 | 40.00 | 80.00 |
| 9.5 | 1170 | 1558 | 388.0 | 19.30 | 61.73 | 38.27 | 20.00 | 55.00 |
| 4.75 | 1176 | 1915 | 739.0 | 36.77 | 98.50 | 1.50 | 0.00 | 10.00 |
| Pan | 1059 | 1089 | 30.00 | 1.50 | 100 | 0.0 | 0.00 | 5.00 |
| Summation of Columns | | | 2010 | 100 | 205.79 | | | |

Retained= (wt. of sieve + retained) - (wt. of sieve)

$$fineness\ moduless = \frac{\sum cumulative\ retaind}{100} = \frac{205.79}{100} = 2.06$$

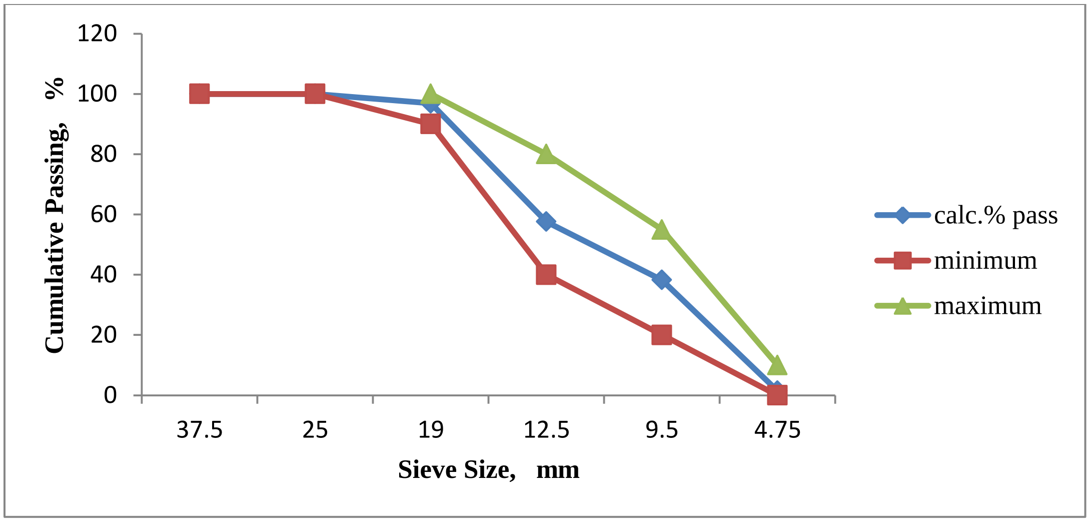



Figure 4. 1: Graph of cumulative passing and sieve size for coarse aggregate

As shown from the above graph the coarse aggregate is in the range of the fineness modulus ( 2.06 < 4) and also the graph lie between maximum and minimum limit.

Table 4. 2: Sieve Analysis of Fine aggregate

| Sieve Size (mm) | Wt. of Sieve (g) | Wt. of Sieve + retained (g) | Retained % | Cumulative Coarser% | Cumulative Passing% | Minimum limit % | Maximum limit % |
|---|---|---|---|---|---|---|---|
| 9.5 | 585 | 590 | 0.64 | 0.64 | 99.37 | 100.00 | |
| 4.75 | 431 | 463 | 4.04 | 4.68 | 95.30 | 95.00 | 100.00 |
| 2.36 | 402 | 495 | 11.73 | 16.41 | 83.57 | 80.00 | 100.00 |
| 1.18 | 531 | 685 | 19.42 | 35.83 | 64.13 | 50.00 | 85.00 |
| 0.6 | 506 | 725 | 27.62 | 63.45 | 36.53 | 25.00 | 60.00 |
| 0.3 | 287 | 498 | 26.61 | 90.06 | 9.90 | 10.00 | 30.00 |
| 0.15 | 461 | 523 | 7.82 | 97.88 | 2.00 | 2.00 | 10.00 |
| Pan | 423 | 439 | 2.02 | 100 | 0 | | |
| | | | $\sum$ 100 | | | | |

% Retained=$\frac{\text{(Wt.sieve + retaind) – (Wt.Sieve)}}{\text{total weight of fine agg.}}$

*Cumulative Passing=100 – Cumulative Coarser*

$$fineness\ moduless = \frac{\sum cumulative\ coarser}{100} = \frac{308.95}{100} = 3.1$$

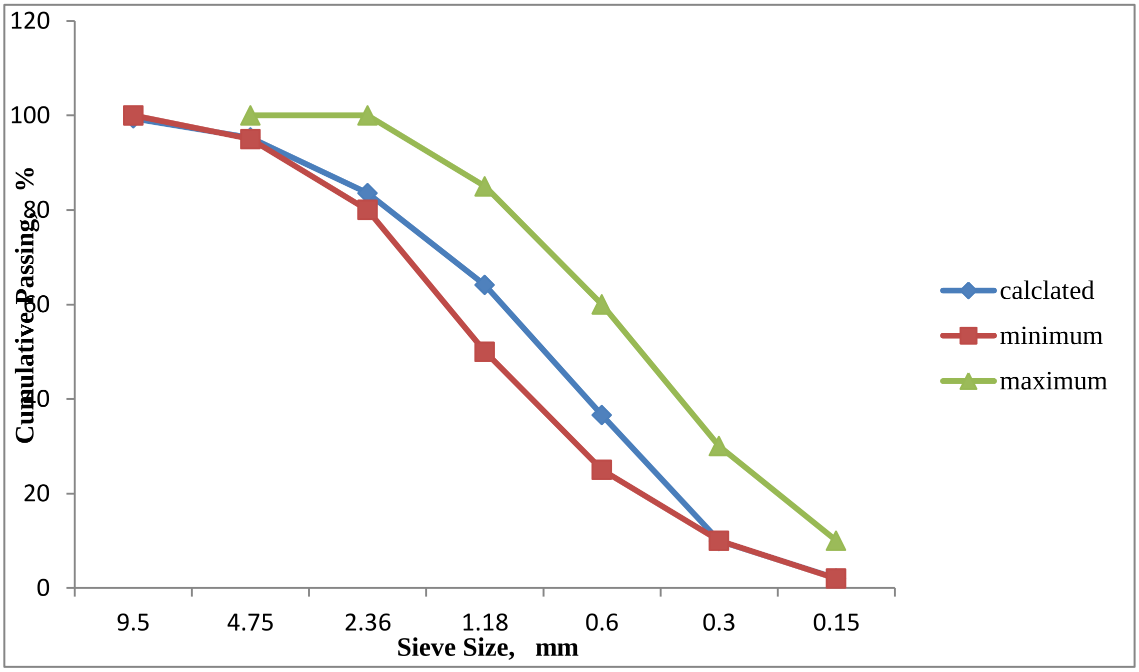



Figure 4. 2: Graph of cumulative passing and sieve size with the limit for fine aggregate

According to ASTM standard for sieve analysis for aggregate, if 2.3 < *fine Modulus = 3.1 <3.5*……It is in a good range, therefore, sieve analysis result for fine aggregates is good agreement with the standard.

## 4.2. Silt Content of Sand

Table 4. 3: The result of sand silt content analysis for unwashed sand specimen

| S. No. | Description | Sample number | | | Remark |
|---|---|---|---|---|---|
| | | Sample-1 | Sample-2 | Sample-3 | |
| 1 | Volume of sample $(V_1)ml$ | 50 | 50 | 50 | |
| 2 | Volume of silt $(V_2)ml$ | 1.0 | 0.6 | 0.75 | |
| 3 | Percentage of silt $(V_2/V_1)*100$ | 2% | 1.2% | 1.5% | |
| | *Average* | *1.6%* | | | *OK!* |

As shown in the Table 4.3 above, the silt content of sand before washing with water is with the permissible standard range; which less than 6% of ASTM or 10% of IS standard.

Table 4. 4: The result of sand silt content analysis for washed sand specimen

| S. No. | Description | Sample number | | | Remark |
|---|---|---|---|---|---|
| | | Sample-1 | Sample-2 | Sample-3 | |
| 1 | Volume of sample ($V_1$)*ml* | 50 | 50 | 50 | |
| 2 | Volume of silt ($V_2$)*ml* | 0.5 | 0.3 | 0.4 | |
| 3 | Percentage of silt ($V_2/V_1$)*100 | 1% | 0.6% | 0.8% | |
| | *Average* | *0.8%* | | | *OK!* |

As depicted in the Table 4.4, the silt content of the sand after washing was reduced to half. Therefore, it is better to have a god concrete quality with regards to the silt content of the fine aggregate.

## 4.3. Compressive and Tensile Splitting Test Results of Concrete

After curing both plain and fiber reinforced concrete specimens for 28 days the test which regards to cubic compressive strength and tensile splitting cylindrical test have done and their results was presented in the tables 4.5 and 4.6.

Table 4. 5: Cubic compressive strength test results for plain concrete

| Designation | Cube weight (kg) | Failure load (KN) | Compressive strength (MPa) | Average Compressive strength, $C_{avg}$ (MPa) | Difference in specimens strength (MPa) | Difference with average strength (%) | Remark |
|---|---|---|---|---|---|---|---|
| $S_{10}$ | 8.357 | 655.72 | 29.15 | 33.59 | $S_{10} - S_{avg} = 4.44$ | 13.22 % | Ok |
| $S_{20}$ | 8.755 | 749.50 | 33.32 | | $S_{20} - S_{avg} = 0.27$ | 0.804 % | Ok |
| $S_{30}$ | 8.679 | 861.59 | 38.30 | | $S_{30} - S_{avg} = 4.71$ | 14.02 % | Ok |
| $m_{Cavg} = 8.597$ | | | | | | | |

From compressive strength test the average compressive strength of the plain concrete is 33.58MPa. It also attains the percentage deviation from the average which that all are below 15%.

Table 4. 6: Cubic compressive strength test results for SFRC

| Designation | Cube weight (kg) | Failure load (KN) | Compressive strength (MPa) | Average Compressive strength (MPa), $C_{Favg}$ | Difference in specimens strength (MPa) | Difference with average strength (%) | Remark |
|---|---|---|---|---|---|---|---|
| $C_{1F}$ | 8.949 | 856.12 | 38.05 | *35.52* | $C_{1F} - C_{Favg} = 2.53$ | 7.12 % | Ok |
| $C_{2F}$ | 8.634 | 840.63 | 37.37 | | $C_{2F} - C_{Favg} = 1.85$ | 5.21 % | Ok |
| $C_{3F}$ | 8.614 | 700.85 | 31.15 | | $C_{3F} - C_{Favg} = 4.37$ | 12.30 % | Ok |
| $m_{CFavg} = 8.732$ | | | | | | | |

Figure 4. 3: Steel fiber reinforced concrete crushed cubic specimen

As discussed in the Table 4.6 the compressive strength of steel fiber reinforced concrete the average cubic compressive strength of SFRC is 35.52MPa.

When it is compared with the control specimen (plain concrete) it has more compressive strength. The difference between plain and steel fiber reinforced concrete compressive strength is about 1.93 MPa. Therefore, the fiber used in this study provides about 5.75% more compressive strength than the plain concrete.

Table 4. 7: Cylindrical tensile splitting strength test results for plain concrete

| Designation | Cylindrical Specimen weight (kg) | Failure load (KN) | Split tensile strength (MPa) | Average Split tensile strength, $T_{avg}$ (MPa) | Difference in specimens strength (MPa) | Difference with average strength (%) | Remark |
|---|---|---|---|---|---|---|---|
| $T_{10}$ | 3.828 | 153.63 | 4.94 | *4.58* | $T_{10}$ - $T_{avg}$ = 0.36 | 7.86 % | Ok |
| $T_{20}$ | 3.847 | 136.66 | 4.35 | | $T_{20}$ - $T_{avg}$ = 0.23 | 5.02 % | Ok |
| $T_{30}$ | 3.789 | 137.64 | 4.45 | | $T_{30}$ - $T_{avg}$ = 0.13 | 2.84 % | Ok |
| $m_{Tavg}$ = 3.822 | | | | | | | |

From splitting tensile strength test the average tensile strength of the concrete is 4.58MPa which is one-seventh of its average compressive strength almost 14% of the average compressive strength.

Table 4. 8: Cylindrical tensile splitting strength test results for SFRC

| Designation | Cylindrical Specimen (kg) | Failure load (KN) | Split tensile strength (MPa) | Average Split tensile strength (MPa) $T_{Favg}$ | Difference in specimens strength (MPa) | Difference with average strength (%) | Remark |
|---|---|---|---|---|---|---|---|
| $T_{1F}$ | 3.928 | 169.43 | 5.39 | *5.26* | $T_{1F}$ - $T_{Favg}$ = 0.13 | 2.47 % | OK |
| $T_{2F}$ | 3.930 | 159.95 | 5.12 | | $T_{2F}$ - $T_{Favg}$ = 0.14 | 2.66 % | OK |
| $T_{3F}$ | 3.904 | 165.29 | 5.27 | | $T_{3F}$ - $T_{Favg}$ = 0.01 | 0.19 % | OK |
| $m_{TFavg}$ = 3.921 | | | | | | | |

From splitting tensile strength test of steel fiber reinforced concrete the average split tensile strength of the concrete is 5.26MPa which is one-seventh of its average compressive strength almost 14.81% of the average compressive strength of steel fiber reinforced concrete. The result implies almost the same percentage difference with that of plain concrete compressive and split tensile strength.

When the split tensile strength of the plain concrete is compared with the split tensile strength of steel fiber reinforced concrete the experimental result of this study shows that the average split tensile strength of 1% by the volume of concrete steel fiber boost up the tensile strength about 15% that of the plain concrete split tensile strength.

Figure 4. 4: Crushed SFRC cylindrical specimens

## 4.4. Accelerated Corrosion Test Results

After accelerated corrosion test for reinforced concrete beam the followings are observed;

- The color of the aqueous solution was completely changed to brown color due to corrosion precipitate was observed during the electrochemical reaction,
- The crack of concrete cover was observed along the reinforcement bar after the occurrence of the reinforcement bar corrosion,
- The electrical resistivity of the concrete was decreased,
- The cross-section of the reinforcement bar was reduced and the color of the concrete beam was changed to brown.

The aforementioned results of the accelerated corrosion test are more elaborated in the Figures 4.5 and 4.6.


Figure 4. 5: Comparison of the RC beam before and after reinforcement bar corrosion

Figure 4. 6: The RC beam after ACT

## 4.5. Results of Flexural Strength Test

During the flexural strength test, reduction in flexural capacity of reinforced concrete beams is observed due to the corrosion of reinforcement which embedded into concrete. Reduction in cross section and continuity of the surface are the consequences of steel corrosion which results in the reduction of the tensile strength of steel and decrease the bond between steel and concrete. In this study the effects of steel reinforcement bar corrosion on flexural capacity of RC beam are investigated and results shows that reinforcement corrosion reduces the flexural capacity of RC beams. The results of flexural strength test which obtained from the three-point bending test are presented in the Table 4.9 and Table 4.10.

Table 4. 9: Flexural strength test results for plain corroded and un-corroded beam

| Steel loss (corrosion) percentage | Beam ID | Beam weight (Kg) | Peak load (KN) | Average peak load (KN) | Mid span deflection at peak load (mm) | Average Mid span deflection at peak load (mm) |
|---|---|---|---|---|---|---|
| Control (0%) | $B_{100}$ | 13.0 | 47.0 | 46.98 | 4.22 | 4.2 |
| | $B_{200}$ | 13.0 | 46.95 | | 4.17 | |
| | $B_{300}$ | 12.5 | 46.99 | | 4.21 | |
| 5% | $B_{102}$ | 13.0 | 35.0 | 36.66 | 3.44 | 3.6 |
| | $B_{202}$ | 13.0 | 38.31 | | 3.75 | |
| | $B_{302}$ | 12.5 | 36.67 | | 3.61 | |
| 10% | $B_{103}$ | 13.0 | 30.35 | 31.53 | 1.52 | 1.84 |
| | $B_{203}$ | 12.5 | 32.7 | | 2.15 | |
| | $B_{303}$ | 13.0 | 31.54 | | 1.85 | |

Table 4. 10: Flexural strength test results for both corroded and un-corroded SFRC beam

| Steel loss (corrosion) percentage | Beam ID | Beam weight (Kg) | Peak load (KN) | Average peak load (KN) | Mid span deflection at peak load (mm) | Average Mid span deflection at peak load (mm) |
|---|---|---|---|---|---|---|
| Control (0%) | $B_{1F0}$ | 13.5 | 52.79 | 53.4 | 6.27 | 6.19 |
| | $B_{2F0}$ | 13.5 | 52.00 | | 6.10 | |
| | $B_{3F0}$ | 14.0 | 55.41 | | 6.20 | |
| 5% | $B_{1F2}$ | 14.5 | 52.00 | 49.70 | 5.88 | 5.47 |
| | $B_{2F2}$ | 13.5 | 47.40 | | 5.06 | |
| | $B_{3F2}$ | 13.5 | 49.7 | | 5.47 | |
| 10% | $B_{1F3}$ | 13.0 | 39.85 | 42.77 | 3.23 | 3.43 |
| | $B_{2F3}$ | 13.5 | 45.69 | | 3.62 | |
| | $B_{3F3}$ | 14.0 | 42.77 | | 3.44 | |

### 4.5.1. Load-Deflection Curve

Deflection refers to the movement of a beam or node from its original position due to the forces and loads being applied to the member. It is also known as displacement, can occur from externally applied loads or from the weight of the structure itself, and the force of gravity in which this applies. When a beam is loaded, any point on its longitudinal axis will be displaced from the unloaded position. This vertical displacement of a point from the unloaded position is known as deflection.

Load-deflection curve is a graph in which increasing flexural loads on a beam are plotted along the vertical axis, and deflections resulting from these loads are plotted along the horizontal axis. It helps to analyze the behavior of the beam subjected to unit loading.

From the experimental work of the current study different load-deflection curves was obtained for a beam with different conditions. The load-deflection curve for different beams (plain/SFRC and un-corroded/corroded) are presented as follows;

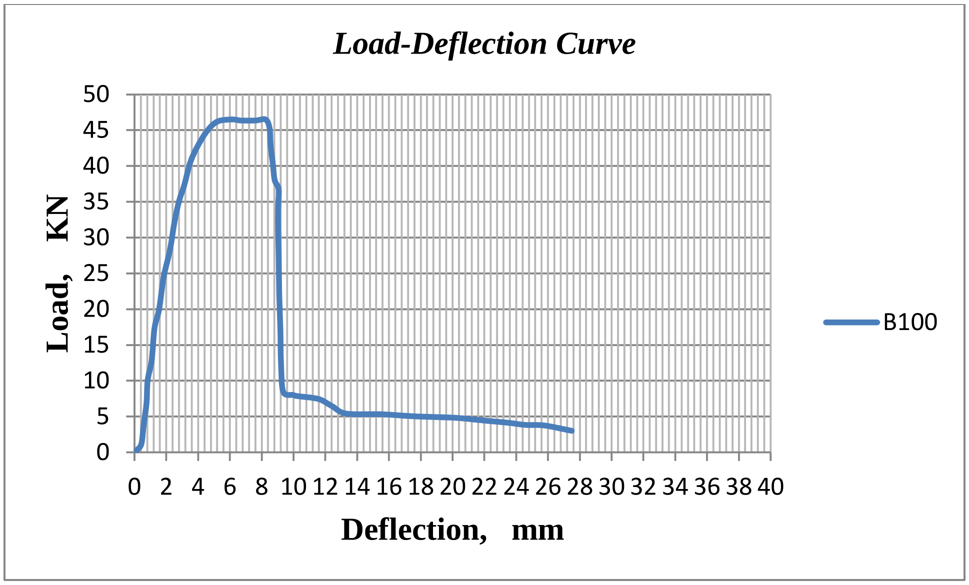



Figure 4. 7: Load-deflection curve for beam $B_{100}$

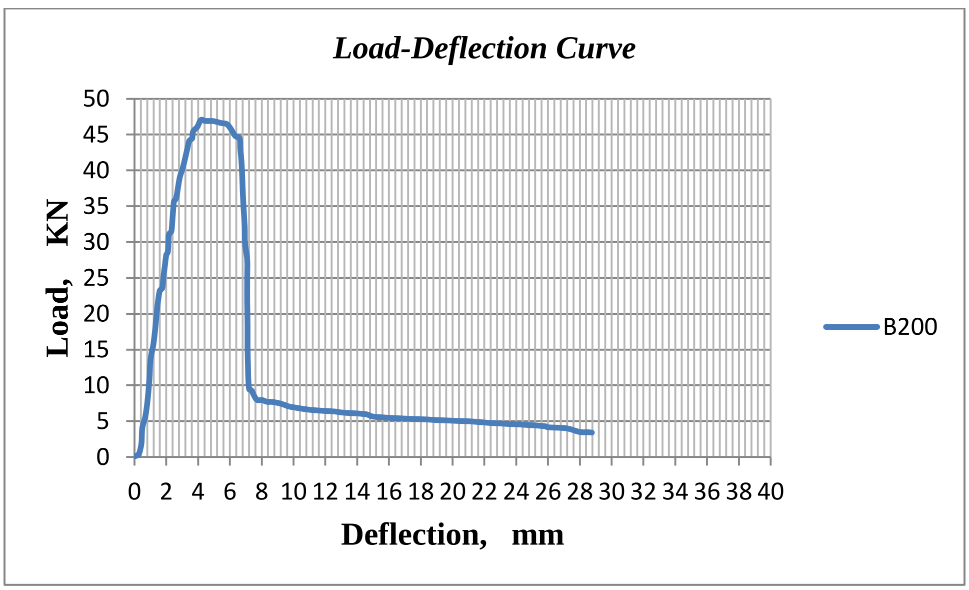



Figure 4. 8: Load-deflection curve for beam $B_{200}$

Graph 1 (Figure 4.8 and Figure 4.9): 0% fiber and 0% corrosion RC beam

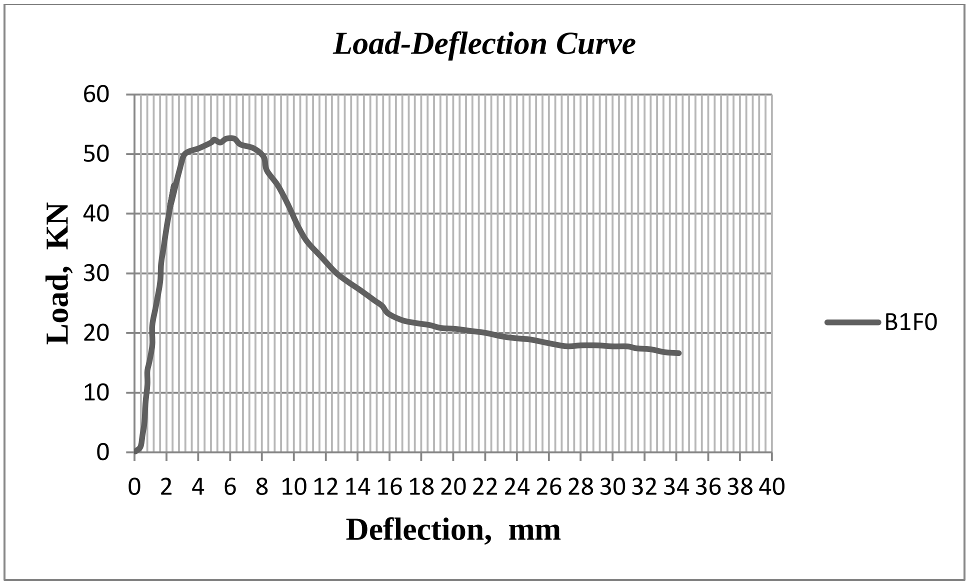



Figure 4. 9: Load-deflection curve for beam $B_{1F0}$

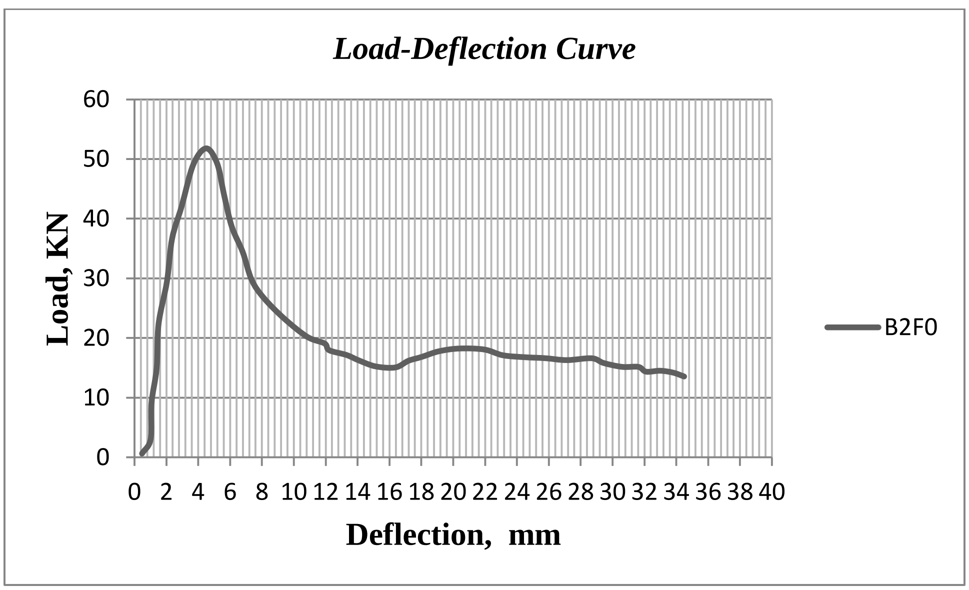



Figure 4. 10: Load-deflection curve for beam $B_{2F0}$

Graph 2 (Figure 4.10 and Figure 4.11): 1% fiber and 0% corrosion RC beam

As clearly observed in the above graphs the difference in ductility for plain RC beams ($B_{100}$ & $B_{200}$) and SFRC beams ($B_{1F0}$ & $B_{2F0}$) comes from the inclusion of the fiber. When SFRC beams yield, it can absorb more energy than the plain RC beams. The failure mechanism of SFRC beams is more ductile than the plain RC beam, since when SFRC beams cracked due to the applied load the steel fibers start to bridge the cracks and then limit more crack propagation by arresting the cracks and it can absorbs more energy before final failure.

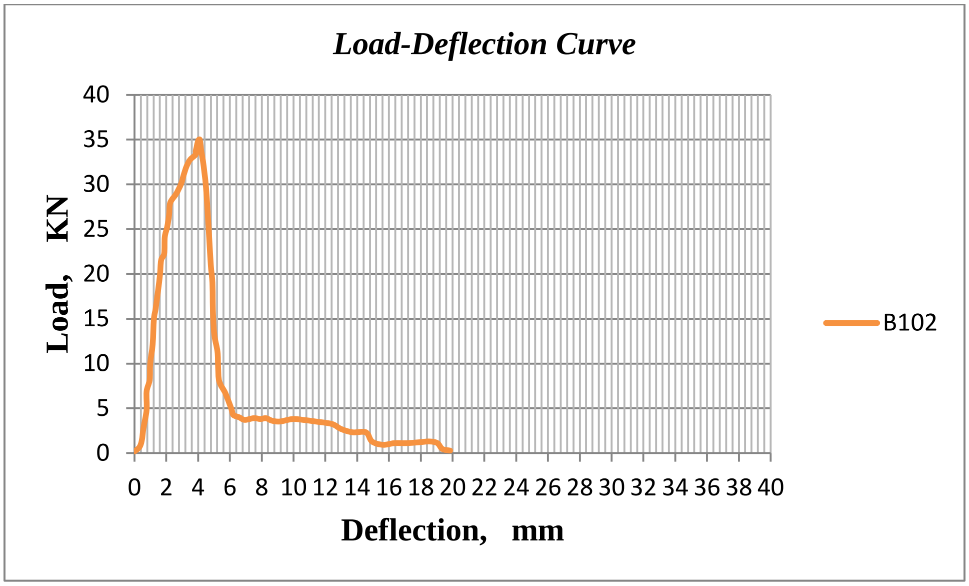



Figure 4. 11: Load-deflection curve for beam $B_{102}$

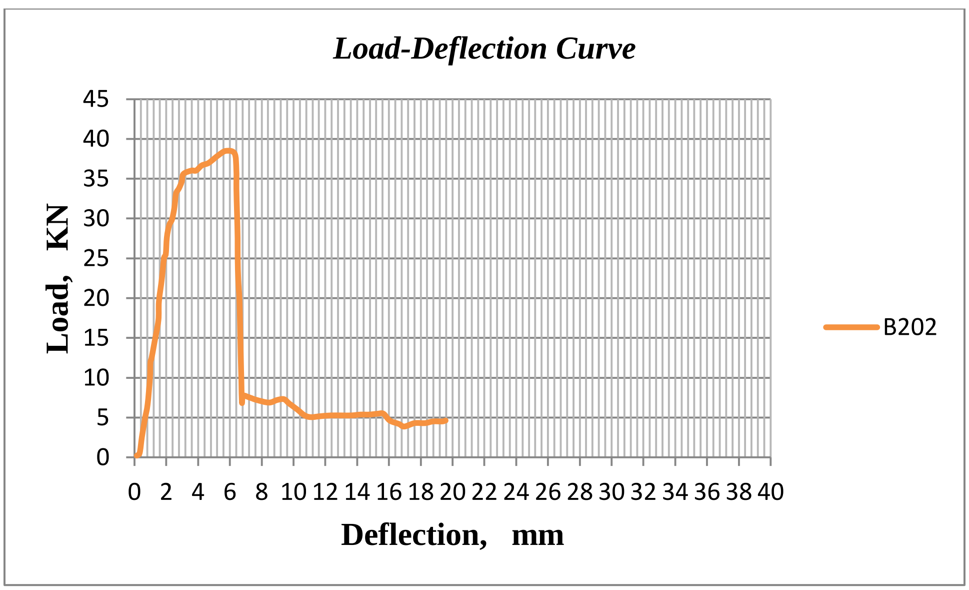



Figure 4. 12: Load-deflection curve for beam $B_{202}$

Graph 3 (Figure 4.12 and Figure 4. 13): 0% fiber and 5% corrosion RC beam

The load-deflection curve shown in the graph *$B_{102}$* and *$B_{202}$* are the results which obtained from the flexural test of 5% corroded reinforcement bar of plain/without fiber RC beams. As shown in the graph corrosion of the reinforcement bar which embedded into concrete affects the ductility of the structure and decreasing the energy absorption capacity of the overall member.

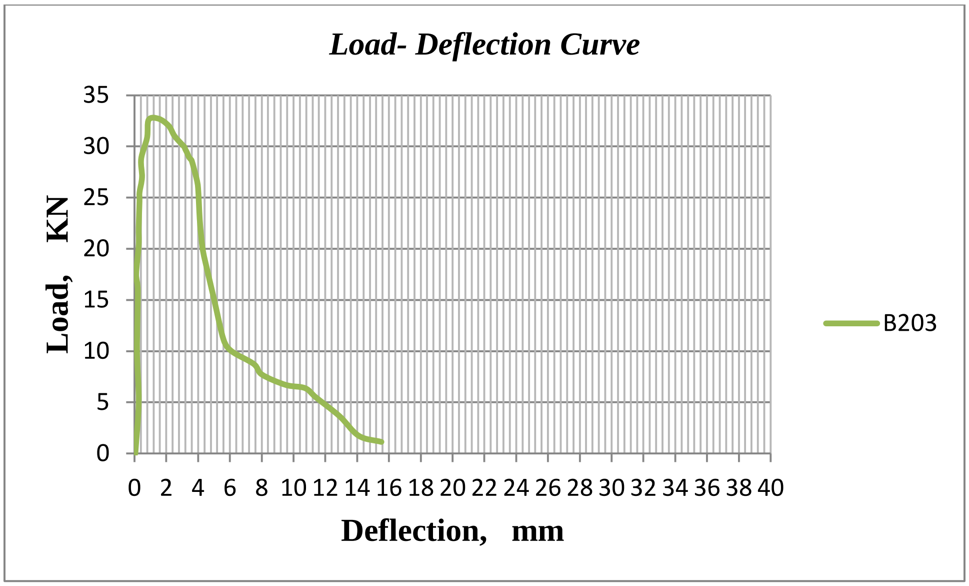



Figure 4. 13: Load-deflection curve for beam $B_{203}$

Graph 4 (Figure 4.14): 0% fiber and 10% corrosion RC beam

The load deflection curve which shown in the graph *$B_{203}$* is the result which obtained from the flexural test of 10% corroded reinforcement bar that embedded into the plain concrete. Due to relatively higher percentage of the reinforcement bar corrosion the beam specimen lost its strength significantly. As compared to 5% corrosion load-deflection curve, the energy absorption capacity of 10% corroded RC beam is lower and the failure mechanism is more brittle. The brittleness of the beam leads to the lower ductility. Hence, the result reveals that the higher percentage of reinforcement bar corrosion the faster load-deflection curve to decline.

Figure 4. 14: a) Brittle failure b) Ductile failure

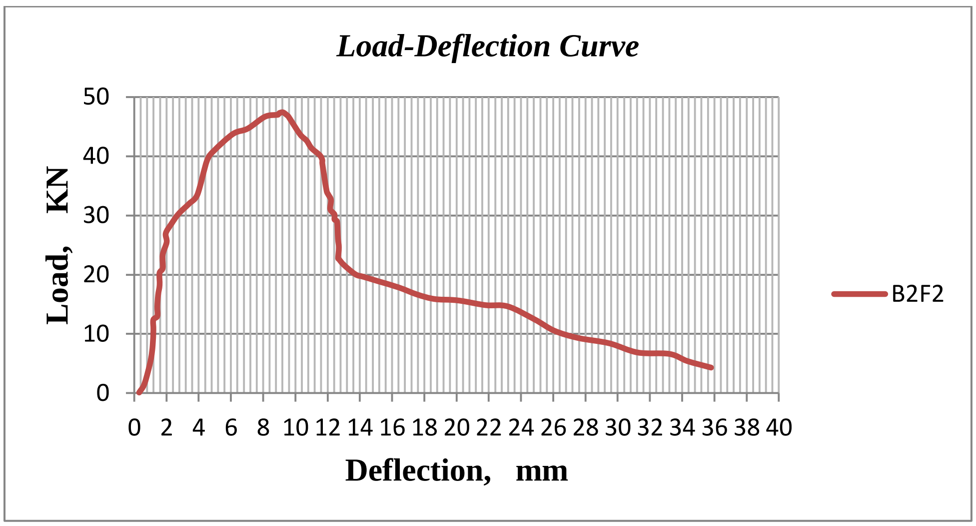



Figure 4. 15: Load-deflection curve for beam $B_{2F2}$

Graph 5 (Figure 4.16): 1% fiber and 5% corrosion RC beam

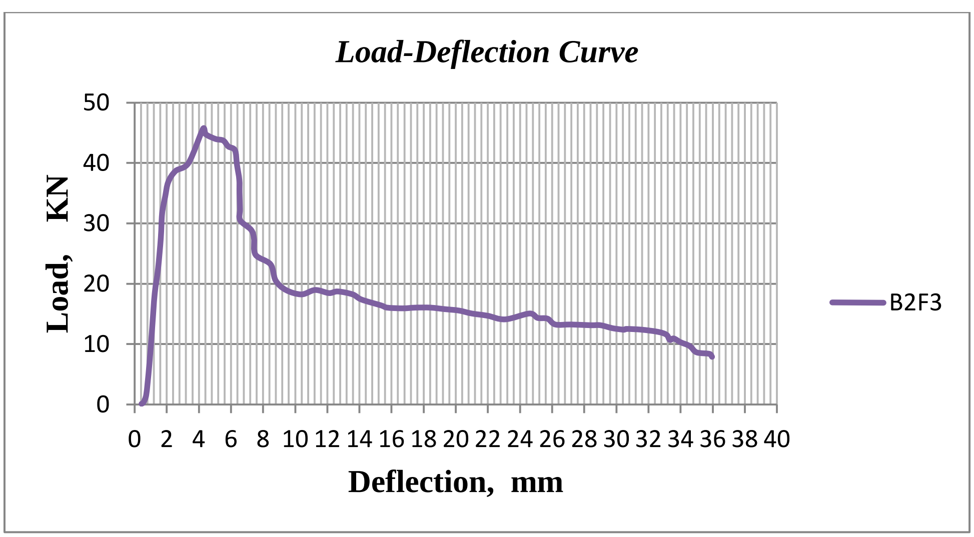


Figure 4. 16: Load-deflection curve for beam $B_{2F3}$

Graph 6 (Figure 4.17): 1% fiber and 10% corrosion RC beam

The graphs $B_{2F2}$ and $B_{2F3}$ shown in the Figure 4.15 and 4.16 respectively are the load deflection curves which gained from the flexural strength test of 5% and 10% corroded SFRC beam specimens respectively. As clearly depicted in the graph the inclusion of the steel fiber makes the beam specimen to have more ductility and flexural strength than plain RC specimen with similar corrosion environments.

The corroded SFRC beams have possess not only higher flexural strength and ductility than corroded plain RC beam but also it is less susceptible for corrosion than plain RC beam. This is due to the reason that the fiber prevents more crack propagation and arrest the initiated crack that allows the ingress of corrosion components into the beam from the external environments even though the specimen was exposed with similar corrosion environments. The gravimetric weight loss analysis after accelerated corrosion test confirms this phenomenon which discussed later.

## 4.6. The Effect of Reinforcement Bar Corrosion on RC Beam

The test results revealed that the corrosion of reinforcement bar induced by chloride ions is one of the main causes of early damage of reinforced concrete structures. A practical method was used for accelerated corrosion of steel bars embedded in concrete by immersing the reinforced concrete specimens in 3% NaCl solution and applying a current of 0.075A (75mA). This procedure helps to get the required levels of corrosion in short period time.

As depicted in the results of ACT in addition to the reinforcement bar deterioration the surrounding concrete was cracked along the reinforcement bar. From this experimental result there are no visible cracks across or other than longitudinal cracks in corroded beams. This is to the reason that the beam specimens have only longitudinal reinforcement bar and it have no cross or shear reinforcement, since the crack pattern follow the path of corroded reinforcement bar. Hence, the observed concrete cracks after accelerated corrosion test is a longitudinal crack which is parallel to the reinforcement bar. The reason is that, when the bar which is embedded in concrete was corroded cracks propagate along the corroded bar; since the stressed portion of the concrete due to the corrosion product is the concrete surrounding the reinforcement bar.

From experimental program, it was observed that the effect of reinforcement bar corrosion highly affect the reinforced concrete structures. In addition to weight (cross-section) and strength loss of the reinforcement bar; corrosion induces another effect on the concrete which is concrete cover crack. This additional effect of steel reinforcement bar corrosion expressed in the form of concrete cover crack shows that the reinforcement bar corrosion which embedded in concrete affects not only the reinforcement bar but also the concrete which it is embedded in. The mutual strength that the steel and the concrete which have in common is lost due to the bond failure between steel and concrete which results from the ribs (the strips in steel that hold the surrounding concrete with steel to act in common against the coming load) on the steel fails and the deformed bar changes into plain bar due to corrosion damage.


Figure 4. 17: Cracked RC beam due to corrosion

Form experimental results generally there are three primary indicators of corrosion damage was observed;

- ✓ **Steel loss**

  When the electrochemical reactions takes place in the corrosion cell during accelerated corrosion the iron atom was fractured due to the chloride attack of the reinforcement bar and this fracture of particles leads to the loss of steel cross-section.

- ✓ **Circumferential expansion**

  At the initiation stage of corrosion the volume of the reinforcement bar was much more increased from the actual, since the corrosion product occupies much more space than the steel bar during ingress of the corrosion components into the RC beam. This leads to circumferential expansion of the area initially occupied by the reinforcement bar due to the corrosion products.

- ✓ **Visual observation of cracks.**

The Circumferential expansion of the corrosion product than what was the space actually occupied by the reinforcement bar exerts the additional tensile stress on the surrounding concrete; and it is obvious that concrete is weak in resisting tensile stress, and then it was finally cracked.

## 4.7. The Effect of Steel Fiber on the Residual Strength of Corroded RC Beam

This study also investigates the effect of steel fiber which extracted from used car tire on the corroded reinforced concrete beam. The advantage gained from fiber inclusion is that the improvement of the tensile strength of concrete. When the reinforcement bar which embedded in the concrete was corroded volume of the bar is increased much more than the original volume of the reinforcement bar and corrosion product was accumulated at the interface of the concrete and the reinforcement bar; the increase in volume of the reinforcement bar and the accumulated corrosion products exert a pressure on the surrounding concrete. These pressures create the tensile stress on the concrete. Because of low capacity of concrete to carry the tensile stress it was cracked.

SFRC beam with similar conditions was exposed for corrosion environment. It has seen that there is an observable crack width difference between the plain and SFRC. In the same corrosion exposure conditions the RC beam without steel fiber have wider crack width than steel fibered RC beam. This is due to the ability of the steel fiber to arrest cracks in the concrete. This phenomenon was depicted in the Figure 4.18 shown below.


Figure 4. 18: Comparison between corroded RC (plain) beam and corroded SFRC beam

The steel fiber in the corroded beam has two major functions; the first is it carries the additional tensile stress which comes from the volume increment of corroded bar and the accumulated corrosion products. And the second function is to limit the propagation of cracks by arresting the initiated crack. The later role prevents further penetration of chloride ions in to the embedded reinforcement; these make the reinforcement to be less corroded than plain concrete beam.

The flexural test results for both corroded/un-corroded and plain/SFRC beams revealed that the more the corrosion of the reinforcement bar embedded into the concrete the lower flexural capacity of the RC beam. The presence of the steel fiber improves the residual flexural capacity of the corroded RC beam which was discussed later.

Fiber reinforcement has an obvious impact on the load carrying capacity, which increased by approximately 14% for the reference beams and, even for those subjected to chloride exposure; all fiber reinforced beams exhibited a greater load at yielding than the reference beams of the plain series such that 35% for 5% corrosion and 15% for 10% corrosion than the reference (plain RC) beams. These observations show the beneficial contribution of the fibers to the load capacity of conventionally RC elements even in prolonged exposures to highly corrosive environments, which could be attributed to a lower degree of rebar corrosion in SFRC and the enhanced corrosion resistance of the fibers. The visual inspection of steel fibers bridging bending cracks which showed no signs of corrosion.


Figure 4. 19: Corroded SFRC beam after flexural strength test.

Additionally, the load-deflection curves for corroded plain and SFRC obtained from flexural testing machine shows that the corrosion of the reinforcement bars significantly affects the ductility of the structure. This is due to the lower energy absorption capacity of the corroded RC beam. The load-deflection curve for corroded plain RC beam and corroded SFRC beam which obtained from the flexural test was depicted in the Figure 4.20 and 4.21 respectively.

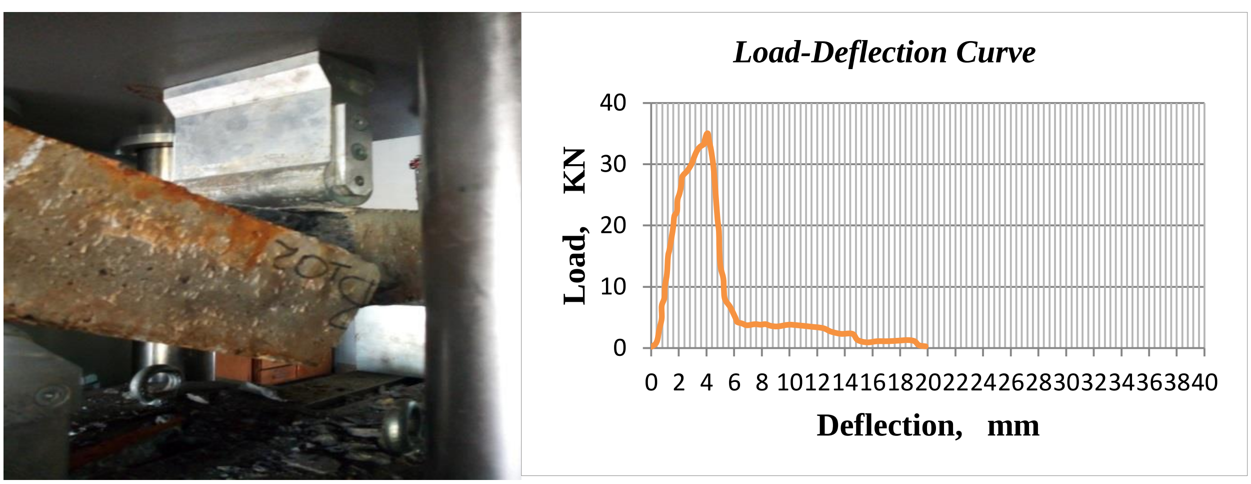



Figure 4. 20: Flexural test and load-deflection curve for corroded RC beam (plain)

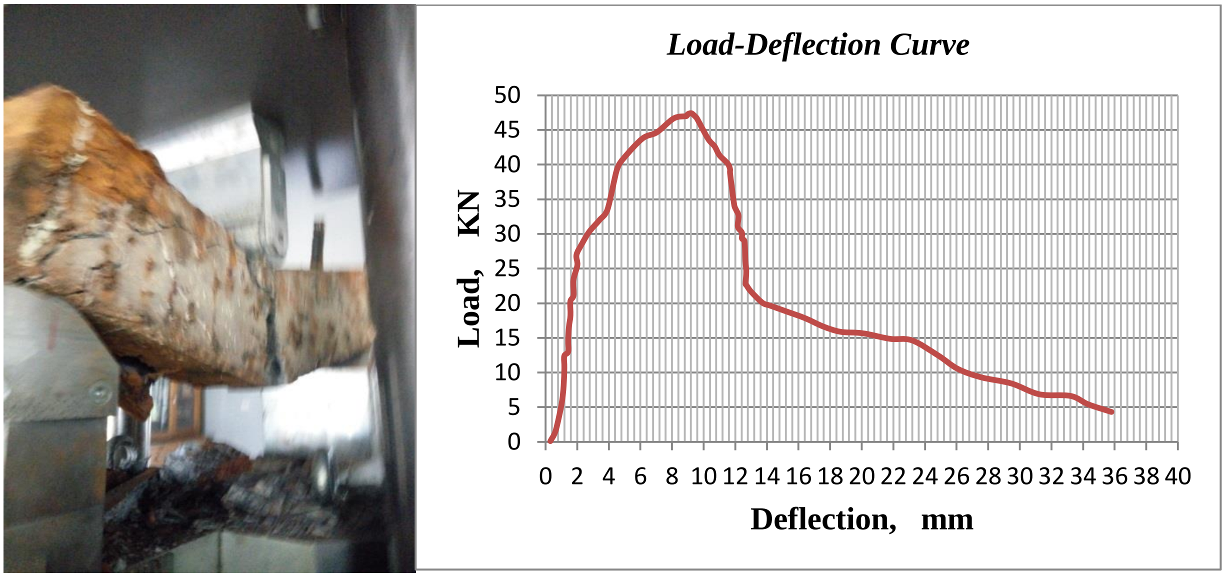



Figure 4. 21: Flexural test and load-deflection curve for corroded SFRC beam

During the flexural tests, there are three different failure modes were identified; these were concrete crushing, bond failure and a brittle failure as shown in the Figure 4.22. Fiber reinforcement proved to be effective in preventing the occurrence of such brittle failure while providing a more stable post-peak behavior. On the other hand, a change in failure mode from progressive concrete crushing to steel rupture was observed in un-corroded specimens with fibers. This was attributed to a reduced yield penetration caused by an improved steel-concrete bond in SFRC, which might result in a reduction of the deformation capacity compared to plain concrete.

Figure 4. 22: Types of corroded beam failures (plain RC beam)

Regarding the residual capacity of corroded beams, it was observed that fiber reinforced series consistently showed a greater load capacity than plain series and the failure of the corroded SFRC beams are pure bending which is that no crushing, bond failure or a brittle failure during the flexural strength test as shown in the Figure 4.23.

Figure 4. 23: Corroded SFRC beam failures.

### 4.7.1. Residual Flexural Strength Analysis

The residual strength of the corroded reinforced concrete beam which obtained from the flexural test reveals that; the corrosion of the reinforcement bar which embedded into concrete is greatly affect the strength of the overall reinforced concrete beam. The experimental result of this study shows that the corrosion of the bar does not affect only the strength of the steel bar but it also affect the strength of the concrete which it is embedded. The chart presented in the Figure 4.24 clearly depicts the difference in flexural load carrying capacity of plain and SFRC beam with different percentage of reinforcement bar corrosion.

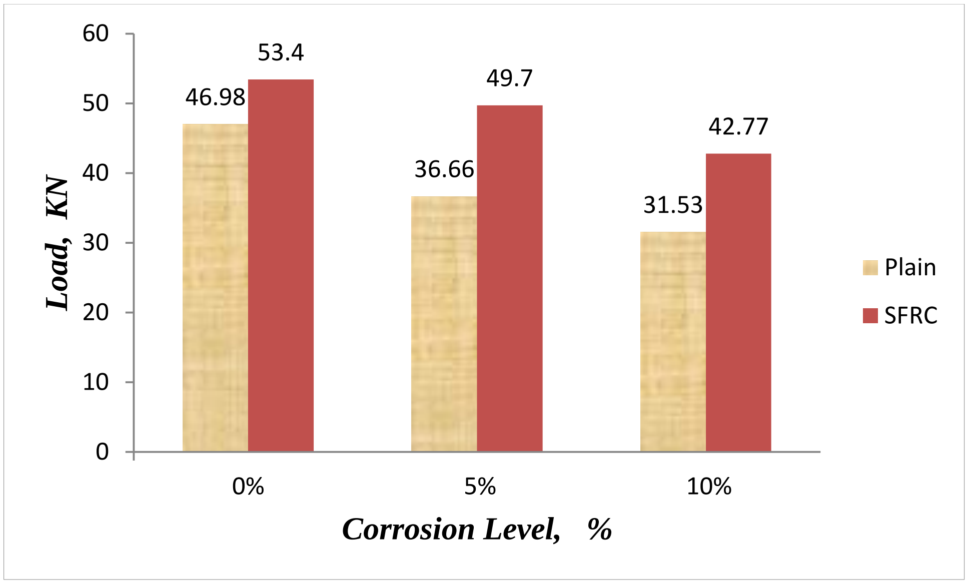



Figure 4. 24: Comparison of plain and SFRC beam flexural test with corrosion level.

From this chart it is possible to deduce that as the percentage of reinforcement bar corrosion increases, the flexural load carrying capacity of plain/SFRC decreases. It also shows that SFRC beams carry more flexural loads before failure than plain RC beam with similar exposure of corrosion environments.

For center-point loading or three-point bending flexural test, the flexural strength of the beam σ can be calculated by using Equation 3.8. Where, L = 0.5m, b = 0.1m, and d = 0.1m. The results of flexural strength of all specimens were presented in the Table 4.11.

Table 4. 11: Summary for flexural strength of the beams

| Beam ID | $P_{max}$, KN | $P_{max}$, N | Flexural Strength, σ $(N/m^2)*10^3$ | Average, σ $(N/m^2)*10^3$ |
|---|---|---|---|---|
| $B_{100}$ | 47.00 | 4700 | 3525.00 | 3523.50 |
| $B_{200}$ | 46.95 | 4695 | 3521.25 | |
| $B_{300}$ | 46.99 | 4699 | 3524.25 | |
| $B_{1F0}$ | 52.79 | 5279 | 3959.25 | 4005.00 |
| $B_{2F0}$ | 52.00 | 5200 | 3900.00 | |
| $B_{3F0}$ | 55.41 | 5541 | 4155.75 | |
| $B_{102}$ | 35.00 | 3500 | 2625.00 | 2749.50 |
| $B_{202}$ | 38.31 | 3831 | 2873.25 | |
| $B_{302}$ | 36.67 | 3667 | 2750.25 | |
| $B_{103}$ | 30.35 | 3035 | 2276.25 | 2364.75 |
| $B_{203}$ | 32.70 | 3270 | 2452.50 | |
| $B_{303}$ | 31.54 | 3154 | 2365.50 | |
| $B_{1F2}$ | 52.00 | 5200 | 3900.00 | 3601.50 |
| $B_{2F2}$ | 42.36 | 4236 | 3177.00 | |
| $B_{3F2}$ | 49.70 | 4970 | 3727.50 | |
| $B_{1F3}$ | 39.85 | 3985 | 2988.75 | 3207.75 |
| $B_{2F3}$ | 45.69 | 4569 | 3426.75 | |
| $B_{3F3}$ | 42.77 | 4277 | 3207.75 | |

The chart in the Figure 4.24 and Table 4.11 depict that inclusion of steel fiber in concrete during the mix design of concrete increases the flexural strength of concrete beam. From the experimental results, it was observed that 1% by volume of the steel fiber addition boosts the flexural strength of the un-corroded reinforced concrete beam by 14%.

In the case of corroded reinforcement bar 5% corrosion declines the flexural strength of the reinforced concrete beam (plain concrete/without fiber) by 22%. This indicates that the corrosion of the reinforcement bar which embedded into concrete highly affects the overall strength of the reinforced concrete beam.

When the steel fiber is added to the mix proportion of the concrete with the same other properties, the percentage reduction of the flexural strength of the beam is reduced to 7%, almost three times the plain concrete.

Furthermore, when the flexural strength of the un-corroded RC beam without fiber is compared with the corroded RC beam with steel fiber there is no difference in strength, even as depicted in the chart, the flexural strength of the corroded SFRC beam is slightly higher than the flexural strength of the un-corroded plain/without fiber RC beam with the corrosion level of 5%.

When the percentage corrosion is raised to higher value which was doubled than the former, the flexural strength of the corroded RC beam without fiber is reduced by 33% than the controlled one (un-corroded RC beam without fiber). 10% corroded SFRC beam lost its flexural strength by 23% than un-corroded plain/without fiber RC beam which shows 10% strength improvement gained from the addition of 1% by volume of concrete steel fiber.

When the comparison is done with the flexural strength of the corroded RC beam without fiber and the flexural strength of the corroded SFRC beam with the corrosion level of 10%, the corroded SFRC is higher than corroded RC beam without fiber by 15%. Therefore, the addition of the steel fiber extracted from used tire into concrete matrix improves the residual flexural strength of the corroded RC beam.

## 4.8. The Comparison between the Actual Weight Loss and Faradays' Law Results

### 4.8.1. Gravimetric Weight Loss Analysis

The weight loss of the reinforcement steel bar due to corrosion can be computed by using Faraday's law based on the amount of current which passes in to the corrosion cell for certain time period during the accelerated corrosion process. But, the result which obtained from this method deviate a little bit from the actual mass loss due to different reasons such as constant current flow, balanced chemicals distribution, and others. Therefore, it is necessary to compute the actual weight loss through other methods. In this study, the gravimetric weight loss analysis was used to get the actual weight loss of the corroded bars. In addition to computing the actual weight loss, the effort was made to compare the result from Faraday's law (theoretical) and gravimetric weight loss analysis (actual).

The actual weight loss can be calculated by using the Equation 3.2.;

$$Percentage\ of\ corrosion\ (weight\ loss) = \frac{Wo - Wc}{Wo} * 100$$

Where, *Wo and Wc* are weight of the reinforcement bar before and after corrosion respectively.

In order to minimize measurement error of the reinforcement bar (it may have some variation in length during bar cutting) the mass of three 60cm bar weight was taken before the corrosion of the bar and the average of the three have taken as the weight of the reinforcement bar before corrosion, Wo.

$m_1$ = 527g, $m_2$= 520g and $m_3$ =523g

$m_{avg} = (m_1 + m_2 + m_3)/3 = (527 + 520 + 523)/3 = 523.33$g

Therefore, weight of the reinforcement bar before corrosion, Wo = *523.33g*

Now, the percentage of weight loss can be calculated by using Equation (3.2) for each specimen's bar;

- For the bar in the specimen, $B_{102}$, the weight of the bar after corrosion, Wc = 497.60

$$\%Wl = \frac{Wo - Wc}{Wo} * 100$$

$$= \frac{523.33 - 497.60}{523.33} * 100 = 4.92\%$$

The rest are done in similar way and the results are summarized in the Table 4.12.

Table 4. 12: Percentage of mass loss by gravimetric weight loss analysis method

| Specimen ID | Weight of the bar after corrosion, Wc (g) | Average, Wc (g) | Relative weight loss (%) | Average weight loss (%) | % Difference |
|---|---|---|---|---|---|
| $B_{102}$ | 497.60 | 489.54 | 4.92 | 6.46 | 1.46 |
| $B_{202}$ | 482.48 | | 7.81 | | |
| $B_{302}$ | 488.54 | | 6.66 | | |
| $B_{103}$ | 452.60 | 456.58 | 13.51 | 12.75 | 2.75 |
| $B_{203}$ | 457.90 | | 12.51 | | |
| $B_{303}$ | 459.25 | | 12.24 | | |
| $B_{1F2}$ | 499.72 | 494.54 | 4.51 | 5.50 | 0.5 |
| $B_{2F2}$ | 491.20 | | 6.14 | | |
| $B_{3F2}$ | 492.7 | | 5.87 | | |
| $B_{1F3}$ | 460.83 | 462.58 | 11.94 | 11.61 | 1.61 |
| $B_{2F3}$ | 464.91 | | 11.16 | | |
| $B_{3F3}$ | 462.00 | | 11.72 | | |

From Table 4.12, the minimum residual weight of bar obtained from the corroded bar of $B_{103}$, which is the reinforcement of the corroded plain RC beam with the corrosion level of 10%. This result depict that the higher the corrosion level, the more loss of the reinforcement bar. The maximum residual weight of the corroded bar recorded from the corroded reinforcement bar of $B_{1F2}$; this tells that the weight loss of fibered RC beam lower than un-fibered RC beam with equal amount of corrosion exposure.

The scenario explained in the foregoing paragraph is among the reasons to have a better residual flexural strength of corroded SFRC beam than plain/without fiber RC beam.

Therefore, reasonably can deduce that, the result obtained from Faraday's law have a good agreement with the result obtained from the gravimetric weight loss analysis.

# CHAPTER FIVE

# CONCLUSION AND RECOMMENDATIONS

## 5.1. Conclusions

The test results revealed that the corrosion of reinforcement bar induced by chloride ions is one of the main causes of early damage of reinforced concrete structures. The corrosion of the reinforcement bar which embedded into concrete affects not only the strength of the embedded steel bar but also it affects the strength of concrete which the bar embedded in and the mutual strength that the steel and the concrete which have in common. The mutual strength is lost due to the bond failure between steel and concrete which results from the ribs (the strips in steel that hold the surrounding concrete with steel to act in common against the coming load) on the steel fails and the deformed bar changes into plain bar due to corrosion. Generally, the major indicators of the reinforcement bar corrosion effects on RC beam are, steel loss, circumferential expansion and visual observation of concrete cracks.

The experimental result analysis of this study revealed that SFRC beams carry more flexural loads before failure than plain RC beam with similar exposure of corrosion environments. Steel fiber reinforcement which used in this study has an obvious impact on the load capacity, which increased by approximately 14% for the reference beams and, even for those subjected to chloride exposure; all fiber reinforced beams exhibited a greater load at yielding than the reference beams of the plain series such that 35% for 5% corrosion and 15% for 10% corrosion than the reference (plain RC) beams. These observations show the beneficial contribution of the fibers to the load capacity of conventionally RC elements even in prolonged exposures to highly corrosive environments, which could be attributed to a lower degree of rebar corrosion in SFRC and the enhanced corrosion resistance of the fibers. Therefore, the addition of the steel fiber extracted from used tire into concrete matrix improves the residual flexural strength of the corroded RC beam.

The result of corroded reinforcement bar weight loss obtained from Faraday's law during accelerated corrosion test have a good agreement with the result obtained from the gravimetric weight loss analysis which is an actual weight loss weighted after the corrosion of the reinforcement bar. The weight loss percentage difference at 14 days for plain RC beam is 1.46% and 0.5% for SFRC beam, and at 28 days for plain RC beam is 2.75% and 1.61% for SFRC beam. Hence, the percentage difference of weight loss between the actual weight loss of the corroded reinforcement bar and the results obtained from Faradays' law during accelerated corrosion test have closely related values.

## 5.2. Recommendations for Future Work

In this study only the effect of corrosion of the reinforcement bar on flexural strength of RC beam was considered, but still the corrosion of steel bar has effects on the shear strength of the RC beam, therefore it is recommended to study the effects of corrosion on the shear strength of the RC beam by including shear reinforcement/stirrups in addition to the longitudinal bar.

It also recommended to include in future work, the influence of different percentage of steel fibers in the corrosion of reinforcement bar. Here, only 1% by volume of steel fiber extracted from used tire was studied, in the future it is better to include the percentage of fiber below and above than it have used in this research. In addition to this the study also recommend to do further investigation on strength improvements of fibers other than steel fibers and comparing their results.

Another recommendation for future work is that using AC power supply to compare and contrast the effect of current source in the overall output of accelerated corrosion test. And also it is another assignment for future works to do the comparative study on the accelerated corrosion test (laboratory) and actual corrosion (field condition) comparison to verify whether or not ACT is the representation of the field conditions which is the actual weight loss and failure mechanism.

## REFERENCES

1. Hertlein, B. H. Assessing the role of steel corrosion in the deterioration of concrete in the national infrastructure : a review of the causes of corrosion and current diagnostic techniques, Corrosion Forms and Control for Infrastructure. San Diego : ASTM Publication, 1992.

2. Broomfield, J. P. Corrosion of Steel in Concrete Corrosion of Steel in Concrete. London : E & FN Spon, 1997.

3. Phenomenological modelling of reinforcement corrosion in marine environments. Melchers , R. E. and Li, C. Q. ACI Materials Journal, 2006, Vol. 103(1).

4. Corrosion of reinforcement in concrete structure under various conditions. Swarup, J. and Trikha, D. N. Transactions of the Metals Finishers' Association of India, 1996, Vol. 5(1).

5. Mechanical response of corroded steel reinforcement of abandoned concrete bridge. Palsson , R. and Mirza, M. S. ACI Structural Journal, 2002, Vol. 99(2).

6. Natrajan, K.A. Advances in corrosion engineering. [book auth.] K.A Natrajan. Advances in corrosion engineering. IISc Bangalore : s.n.

7. Solgaard, Anders Ole Stubbe. Corrosion of reinforcement bars in steel fibre reinforced concrete structures. s.l. : Department of Civil Engineering Technical University of Denmark, 2013.

8. Steel fibre reinforced mortar and concrete. Johnston, C.D. A review of mechanical properties Fiber reinforced concrete ACI – SP44, 1974.

9. Maidl, B.R. and Dietrich, J. Steel Fibre Reinforced Concrete. Berlin : Ernst & Sohn, 1995.

10. Shah, S.P., Swartz, S.E and Ouyang, C. Fracture Mechanics of Concrete. s.l. : John Wiley & Sons, Inc, 1995.

11. Fracture Processes in Concrete Assessment of Material Parameters for Fracture Models. van Mier, J.G.M. s.l. : CRC Press, 1997.

12. Wikipedia. Wikipedia. [Online] [Cited: November 19, 2019.] hhttps://en.m.wikipedia.org.

13. researchgate. researchgate. [Online] [Cited: November 19, 2019.] http://www.researchgate.com.

14. Wu, Hao. BOND DEGRADATION AND RESIDUAL FLEXURAL CAPACITY OF CORRODED RC BEAMS. Toronto, Ontario, Canada, : Ryerson University, 2012. 1-1-2012.

15. Mietz, J., Polder, R. and Elsener , B. Corrosion of Reinforcement in Concrete. Aachen : Maney Publishing, 2000.

16. ABOSRRA, L. R. CORROSION OF STEEL REINFORCEMENT IN CONCRETE. Bradford, UK : University of Bradford, 2010.

17. Structural behaviour of beams under simultaneous load and steel corrosion. Proceedings Of The 2nd International Conference On Concrete Repair, Rehabilitation And Retrofitting (ICCRRR). Malumbela , G., Moyo, P. and Alexander, M. Cape Town, South Africa, : s.n., November 24–26, pp. 645-650. 2009.

18. Analytical Model to Predict Nonlinear Flexural Behavior of Corroded Reinforced Concrete Beams. Maaddawy, T. E., Soudki, K. and Topper, T. 4, s.l. : ACI Structural Journal,, 2005, Vols. 102, .

19. Mechanical Behaviour of Corroded RC Beams - Part 1: Experimental Study of Corroded Beams. Castel, A., Francois, R. and Arliguie, G. 233, 2000, Vol. 33.

20. Load Carrying Capacity of Concrete Structures with Corroded Reinforcement. Rodriguez, J., Ortega, L. M. and Casal, J. 4, 1997, Vol. 11.

21. Strength Assessment of Corrosion Damaged Reinforced Concrete Slabs and Beams. Proceedings Institution of Civil Engineers, Structures & Buildings. Eyre, J. R. and Nokhasteh, M. A. 1992.

22. The Effect of Reinforcement Corrosion on the Strength of the Steel/Concrete Interface. International Conference on Bond in Concrete. Cabrera, J. G. and Ghoddoussi, P. Riga, Latvia. : s.n., 1992.

23. Influence of Corrosion and Cracking on Bond Behaviour and Strength of Reinforced Concrete Members. ACI Structural Journal. Al-Sulaimani, G. J., Basunbul, M. K. and Rasheeduzzafar, I. A. 2, 1990, Vol. 87.

24. Minkarah, I. and Ringo, B. C. Behaviour and Repair of Deteriorated Reinforced Concrete Beams. s.l. : Transportation Research Record, 1981.

25. A. Nilson, D. Darwin and C. Dolan. Design of Concrete Structures. 2010.

26. Lewis, Jeremy . THE EFFECTS OF CORROSION ON REINFORCED CONCRETE WITH FIBER ADDITION. s.l. : University of Akron, 2012.

27. Wang, X. and Liu, X. A strain-softening model for steel-concrete bond. Cement and Concrete Research. 2003, Vol. 33.

28. Elbusaefi, Adel A. THE EFFECT OF STEEL BAR CORROSION ON THE BOND STRENGTH OF CONCRETE MANUFACTURED WITH CEMENT REPLACEMENT MATERIALS. Cardiff : Cardiff University, 2014.

29. Momayez, A., et al., et al. Comparison of methods for evaluating bond strength between concrete structure and repair materials. Cement and Concrete Research. 2005, Vol. 35.

30. Bond strength of epoxy coated reinforcing bars. Treece, R. A. and Jirsa, J. O. s.l. : ACI Material Journal, Vol. 86.

31. Kayali, O. and Yeomans, S. R. Bond of ribbed galvanized reinforcing steel in concrete. Cement and Concrete Composites. 2000, Vol. 22.

32. Martin, B. and Pantazoprulou, S. J. Effect of bond, aggregate interlock and dowel action on the shear strength degradation of reinforced concrete. Engineering. 2000, Vol. 23.

33. Collins, M. P. and Mitchell, D. Prestressed concrete structures. Englewood Cliffs, N.J. : Prentice Hall, 1991.

34. Broomfield, J. G. Corrosion of steel in concrete: understanding, repair and. E and FN Spon, UK. : s.n., 1997.

35. Almusallam, A. Effect of degree of corrosion on the properties of reinforcing steel bars. Construction and Building Materials. 2001, Vol. 15.

36. Amleh, L. and Mirza, S. Corrosion Influence on Bond between Steel and Concrete. ACI Structural Journal. 1999, Vol. 96, 3.

37. Fu, X. and Chung, D. D. Effect of Corrosion on the Bond Between Concrete and Steel Rebar. Cement and Concrete Research. 1997, Vol. 27, 12.

38. Almusallam, A. A., et al., et al. Effect of Reinforcement Corrosion on Bond Strength. Construction and Building Materials. 1996, Vol. 10, 2.

39. Chung, L., et al., et al. Correction Factor Suggestion for ACI Eevelopment Length Provisions Based on Flexural Testing of RC Slabs with Various Levels of Corroded Reinforcing Bars . Engineering Structures. 2004, Vol. 26, 8.

40. Probabilistic study chloride-induced corrosion of carbon steel in concrete structures. Lounis, Z., Zhang, J. and Daigle, L. Albuquerque, New Mexico : 9th ASCE Joint Specialty Conference on Probablistic Mechanisms and Structural Reliability, 2004.

41. Morris, W., Vico, A. and Vazquez, M. Chloride induced corrosion of reinforcing steel evaluated by resistivity measurements. s.l. : Electrochimica Acta, 2004.

42. Ann, K. Y. and Song, H. W. Chloride threshold level for corrosion of steel in concrete. s.l. : Corrosion Science,, 2007.

43. Mangat, P. S. and Limbachiya, M. C. Effect of initial curing on chloride diffusion in concrete repair materials. Cement and Concrete Research. 1999, Vol. 29.

44. Erdogdu, S., Kondrtova, I. L. and Bremner, T. W. Determination of chloride diffusion coefficient of concrete using open-circuit potential measurements. Cement and Concrete Research. 2004, Vol. 34.

45. Song, H. W., Lee, C. H. and Ann, K. Y. Factors influencing chloride transport in concrete structures exposed to marine environments. Cement and Concrete Composites. 2008, Vol. 30.

46. Goto, S. and Roy, D. M. The effect of water/cement ratio and curing temperature on the permeability of hardened paste. Cement and Concrete Research. 1891, Vol. 11.

47. Guneyisi, E, Qzturan, T. and Gesoglu, M. A study on reinforcement corrosion and related properties of plain and blended cement concretes under different curing conditions. Cement and Concrete Composites. 2004, Vol. 27.

48. Neville, A.M. Properties of Concrete. Pitman, UK : s.n., 1981.

49. Suryavanshi, A. K., Scantlebury, J. D. and Lyon, S. B. Corrosion of reinforcement steel embedded in high water-cement ratio concrete contaminated with chloride. Cement and Concrete Composites. 1998, Vol. 20.

50. Saraswathy, V. and Song, H. Electrochemical studies on the corrosion performance of steel embedded in activated fly ash blended concrete. Electrochimica Acta. 2006, Vol. 51.

51. Ann, K. YY. and Song, H. W. Chloride threshold level for corrosion of steel in concrete. Corrosion Science. 2007, Vol. 49.

52. Fraczeck, J. A review of electrochemical principles as applied to corrosion of steel in a concrete or grout environment. ACI Material Journal. 1987, Vol. 102.

53. Funahashi, M. Predicting corrosion-free service life of a concrete structure in a chloride environment. ACI Materials Journal. 1990, Vol. 87.

54. Treadaway, K.W.J. Corrosion of steel reinforcement in concrete. Magazine of Concrete Research. 1988, Vol. 40.

55. Otieno, Mike. CORROSION PROPAGATION IN CRACKED AND UNCRACKED CONCRETE. s.l. : University of Cape Town, 2008.

56. Heckroodt, R. O. Guide to deterioration and failure of building materials. London : Thomas Telford Ltd, 2002.

57. Cracks in concrete bridge decks and their contribution to corrosion of reinforcing steel and prestressed cables. Ryell, J. and Richardson, B. S. Ontario : s.n., 1972.

58. Dauberschmidt, C. Corrosion Mechanisms of Steel Fibres in Chloride Containing Concrete. Proceedings of the 4th International PhD Symposium in Civil Engineering. 2002.

59. Angst, U., et al., et al. Critical chloride content in reinforced concrete – A review. Cement and Concrete Research. 2009, Vol. 39.

60. Accelerated Short-Term Techniques to Evaluate Corrosion in Reinforced Concrete Structures. Deb, Sonjoy. s.l. : The Masterbuilder, July 2012.

61. Corrosion of Concrete Reinforcement and Electrochemical Factors in Concrete Patch Repair. Canadian Journal of Civil Engineering, Canada : s.n., 2006, Vol. 33.

62. El-Ebweini, Mohamed. Structural Performance of Repaired Corroded Reinforced Concrete Beams. s.l. : The Islamic University of Gaza, 2009.

63. BEDEWI, NASIR. STEEL FIBER REINFORCED CONCRETE MADE WITH FIBERS EXTRACTED FROM USED TYRES. ADDIS ABABA : ADDIS ABABA UNIVERSITY, 2009.

64. Belayneh, M. Construction Materials. Addis Ababa, Ethiopia : Addis Ababa University Press, 1987.

65. Compressive Strength of Concrete. G.Sivakumar. s.l. : Concrete Initiatives and Innovations.

66. Laboratory Simulation of Corrosion Damage in Reinforced Concrete. S. Altoubat, M. Maalej and F. U. A. Shaikh. International Journal of Concrete Structures and Materials, 2016, Vol. 10. 3.

67. Types of fiber. Qasim, Ola Adel. s.l. : Al-Mansour University , 2018.

68. The influence of Aspect Ratio of Steel Fibers on the Mechanical Properties of Concrete. Arbab, Amirhossein and Tavousi, Shahriar . Rome, Italy : s.n., 2016.

69. STEEL FIBER REINFORCED CONCRETE. CHANH, Nguyen Van. s.l. : Ho Chi Minh City University.

70. Concrete slump test - Wikipedia. Wikipedia. [Online] [Cited: December 15, 2019.] https://en.wikipedia.org/wiki/Concrete_slump_test.

71. Testing Compressive Strength of Concrete. (CIP-35), Concrete In Practice. s.l. : technichal information prepared by National Ready Mixed Concrete Association (NRMCA), 2003.

72. What is Corrosion. Shaw, Barbara A. and Kelly, Robert G. 2006.

73. Basic Mechanisms of Corrosion and Corrosion Control. E.C.Bell, Graham.

# ANNEXES

## ANNEXES-I: Specification of Virgin Steel Fiber (Bead Wire) (63)

| SUPPLIER: | BEAD WIRE SPECIFICATION: ROUND BEAD WIRE ø 0, 89 mm | CUSTOMER: MAT-ATC CODE:25.1 |
|---|---|---|

**Construction**: Round bead wire diameter 0,89 mm

| Coating | Bronze |
|---|---|

| SPECIFICATION | UNIT | VALUE TARGET | TOLERANCE | TEST METHOD | NOT |
|---|---|---|---|---|---|
| Nominal diameter | [mm] | 0,89 | +/- 0,025 | MP 066 | |
| Nominal weight per length | [g/m] | 4,884 | - | MP 061 | |
| PHYSICAL MECHANICAL PROPERTIES | | | | | |
| ✓ Tensile Strength min. | [MPa] | 1900 | - | MP 068 | |
| ✓ Nominal breaking force min. | [N] | 1182 | | MP 068 | * |
| ✓ Elongation at break | [%] | 6-9 | - | MP 068 | |
| Stiffness-number of bends | - | 13 | - | MP 065 | |
| Stiffness-number of torsions | - | 25 | - | MP 060 | |
| Mass of Bronze coating | [g/kg] | 0,30-0,95 | - | MP 046 | |
| Copper content | [%] | 97-99,5 | - | MP 046 | |
| Straightness | [cm/3m] | 50 | - | MP 072 | |
| Weight of reelless coil or metal reel | [kg] | 350-450 | - | - | |
| Overall width of reelless coil | [mm] | 280-295 | - | - | |
| Overall width of metal reel | [mm] | 343 | - | - | |
| Adhesion /stand. comp./ min. | [N/25mm] | 300 | - | MP 053 | |

**CONDITION OF STOCKING:**

Temperature: *Recommended 5-35°C*

Humidity: *Protection against moisture and UV radiation is trough packing, which can not be destructed.*

**PERIOD OF GUARANTEE:**

*The six months from date of expedition, in case that are keeping condition of stocking and packing and material is not destructed.*

**NOTICE:**

*Testing is according laboratory guidebook or according separate agreement with supplier.*

**- critical parameter for tyre safety.*

**Issue date:**

SUPPLIER

CUSTOMER

Prepared by : Research and development department MATC

Date :19/02/07

**Tensile strength, breaking load and elongation at break of virgin steel fiber (VSF) and reused steel fiber (RSF)** (63)

| Sample No. | Breaking Load (N) | | Tensile Strength (MPa) | | Elongation at Break (%) | |
|---|---|---|---|---|---|---|
| | RSF | VSF | RSF | VSF | RSF | VSF |
| 1 | 424.6 | 1156.1 | 682.5 | 1858.3 | 5.4 | 12.2 |
| 2 | 574.9 | 1148.5 | 924.1 | 1846.1 | 6.8 | 13.0 |
| 3 | 476.1 | 1146.9 | 765.2 | 1843.6 | 4.4 | 9.9 |
| 4 | 571.9 | 1146.9 | 919.3 | 1842.3 | 4.9 | 9.7 |
| 5 | 432.0 | 1154.5 | 694.5 | 1855.8 | 7.2 | 10.9 |
| 6 | 443.4 | 1134.2 | 712.8 | 1823.1 | 3.8 | 8.9 |
| 7 | 642.8 | 1261.6 | 1033.3 | 2027.9 | 4.6 | 9.4 |
| 8 | 1021.1 | 1252.9 | 1641.3 | 2013.9 | 6.3 | 12.2 |
| 9 | 862.7 | 1241.8 | 1386.7 | 1996.1 | 6.5 | 10.5 |
| 10 | 585.9 | 1131.6 | 941.8 | 1819.0 | 11.4 | 10.9 |
| Min. | 424.6 | 1131.6 | 682.5 | 1819.0 | 3.8 | 8.9 |
| **Mean** | **603.5** | **1177.4** | **970.2** | **1892.6** | **6.1** | **10.8** |
| Max | 1021.1 | 1261.6 | 1641.3 | 2027.9 | 11.4 | 13.0 |
| S.D. | 196.79 | 52.25 | 316.3 | 84.07 | 2.17 | 1.35 |

**Geometry and tensile strength of RSF, VSF and ISF (industrial steel fiber)** (63)

| Fiber Type | Length (mm) | Diameter (mm) | Tensile strength (MPa) |
|---|---|---|---|
| RSF | 20/40/60 | 0.89 | 970.2 |
| VSF | 20/40/60 | 0.89 | 1892.6 |
| XOREX | 25/35/50/63 | 1.0 | 828 |
| MATRIX-CS 1.0 | 25 | 0.33-0.6 | 414-825 |
| NOVOCON XR | 38/50 | 1.14 | 966-1242 |