

INVESTIGATION OF REBAR CORROSION IN PLAIN AND STEEL FIBER
REINFORCED CONCRETE UNDER STATIC AND DYNAMIC LOADING

by

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ABSTRACT

INVESTIGATION OF REBAR CORROSION IN PLAIN AND STEEL FIBER REINFORCED CONCRETE UNDER STATIC AND DYNAMIC LOADING

Reinforcement corrosion is one of the most important causes of deterioration for concrete structures. In codes and standards, there are recommendations for protection against reinforcement corrosion. In the European and Turkish standards (TS EN206 and TS ENV 1992), these recommendations are based on the protection of the steel reinforcement by limiting the access of aggressive agents. Concrete cover thickness and quality are increased in more aggressive environmental conditions to assure the required lifetime for the structure. However, the cracking of the concrete is not taken into consideration in lifetime estimations. Concrete is a brittle material that can be easily cracked due to several factors. The role of cracks in reinforcement corrosion is controversial. According to the ACI 318-08 and ACI 224R-01, more important parameters are concrete cover and concrete quality as the corrosion may localize despite an earlier onset. In ACI 224R-01, the importance of cyclic loading is emphasized as it can cause an increase in crack width and thus in corrosion. In the present study, the role of cracks on reinforcement corrosion was investigated. Concrete cover and quality recommended by standards were tested for cracked concrete. Cracks obtained under specified flexural loads that were applied on the prismatic specimens were compared for plain and steel fiber reinforced concretes. Obtained results were used for the evaluation of the limits recommended by the standards. Results showed the importance of water-to-cement ratio even in the presence of the cracks. Corrosion rate was found to increase by increasing crack width which is in corresponding order, specimens cracked under dynamic load, specimens cracked under static load, uncracked specimens. Steel fiber use decreased corrosion rate results when water-to-cement ratio was low.

ÖZET

LİFLİ VE LİFSİZ BETONARME YAPI ELEMANLARINDA STATİK VE DİNAMİK YÜK ALTINDA DONATI KOROZYONUNUN İNCELENMESİ

Donatı korozyonu betonarme yapılardaki tahribat sebeplerinin en önemlilerindendir. Yönetmelik ve standartlarda, donatı korozyonuna karşı çeşitli önlemler önerilmiştir. Avrupa ve Türk Standartlarındaki (TS EN 206 and TS ENV 1992), bu önlemler ve öneriler, zararlı etkilerin betona girişini kısıtlayarak donatı çeliğinin korunmasına dayalıdır. İstenen yapı ömrünü sağlamak için, beton örtüsü ve kalitesi çevre koşullarının şiddetiyle birlikte arttırılmaktadır. Ancak, betonun çatlaması, ömür tahminlerinde dikkate alınmamaktadır. Halbuki, beton gevrek bir malzemedir ve çeşitli etkiler altında kolayca çatlayabilir. Çatlakların donatı korozyonundaki rolü ise tartışmalıdır. ACI 318-08 (Building Code Requirements for Structural Concrete) ve ACI 224R-01 (Control of Cracking in Concrete Structures)'e göre, en önemli parametreler beton örtüsü ve kalitesidir çünkü korozyon erken başlasa da sadece çatlak bölgesinde kalabilir. ACI 224R-01'de tekrarlı yüklerin önemi vurgulanmıştır. Bu yükler çatlak genişliğinde artışa neden olabilir, ve böylece korozyonu arttırabilirler. Bu çalışmada, çatlakların donatı korozyonundaki rolü araştırılacaktır. Standartlarda farklı çevre koşulları için önerilen beton örtü kalitesi ve yeterliliği çatlaklı beton için test edilecektir. Deney numunelerinde çatlaklar belirli eğilme yükleri altında elde edilecek, bu çatlak genişlikleri yalın ve lifli betonlar için karşılaştırılacaktır. Korozyon ölçümleri çatlaklı ve çatlaksız beton için yapılmıştır. Sonuçlar, standartlar tarafından önerilen sınır değerlerin değerlendirilmesi için kullanılacaktır. Korozyon hızı ise çatlak genişliği arttıkça artmaktadır ve büyükten küçüğe sırasıyla dinamik yük altında çatlamış numuneler, statik yük altında çatlamış numuneler, çatlaksız numuneler olarak bulunmuştur. Çelik lif kullanımı korozyon hızını su/çimento oranı azaldıkça azalır.

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LIST OF SYMBOLS

i_{corr}	corrosion current
R_p	polarization resistance
C_{dl}	double layer capacitance
E	corrosion potential
R_Ω	ohmic resistance
T	temperature
V_t	polarized potential of reinforcement
V_{max}	final steady potential
β_a	the Tafel slope of the anodic reaction
β_c	the Tafel slope of the cathodic reaction
ΔE	imposed potential change
Δi	current density change caused by ΔE

LIST OF ACRONYMS/ABBREVIATIONS

<i>L0</i>	non-fibered reinforced concrete
<i>L1</i>	steel fiber reinforced concrete
<i>RH</i>	relative humidity

1. INTRODUCTION

Concrete is the most widely used construction material in the world and it is used in many structures such as residential, commercial and industrial buildings. Because of the low tensile strength, concrete is usually strengthened by steel reinforcement. This combination of concrete and steel provides a durable material that has become widely used in construction.

Durability of concrete can be defined as the resistance of concrete to physical and chemical attack while maintaining its engineering properties. The deterioration of reinforced concrete due to loss of durability can be divided into three main groups;

- (i) physical deterioration (due to cracking, fire, abrasion),
- (ii) chemical deterioration (due to sulphate and acid attack, leaching and alkali-aggregate reaction),
- (iii) reinforcement corrosion (due to chloride attack and carbonation) (Isgor, 2001).

The major cause of durability deterioration is the chloride-induced corrosion of the steel reinforcement that can lead to structural problems and occasional catastrophes on a large scale.

Reinforcement corrosion is one of the most important causes of deterioration for concrete structures. Corrosion of the reinforcement is caused by either carbonation of the concrete cover or ingress of a sufficient amount of chloride ions towards the reinforcing steel. In general, in order for the corrosion, which is basically an electrochemical reaction, to proceed, the presence of oxygen, water and chloride ions is essential. The environment may weaken the passive layer around the steel reinforcement. Some chemicals (such as chlorides) have the potential to react with the passive layer and diminish its thickness (depasivation). As a result, some parts (pitting corrosion) or the entire length of the steel reinforcement (general corrosion) may destroy the passive layer, and therefore, the corrosion of steel reinforcement will commence. The chemical composition of the concrete pore solution changes after corrosion starts.

This affects the electrical resistance of the concrete which plays a major role in the rate of ion diffusion necessary to complete the corrosion reaction. As corrosion proceeds, the amount of corrosion products can rise to such levels that it can cause cracks in the concrete. The process ends with the failure of the structure due to one of the possible reasons such as loss of carrying capacity, excessive deflection, unacceptable amount of cracking and spalling, bond failure, etc. (Isgor, 2001). One of the objectives of this study is to examine accelerated electrochemical corrosion process of steel reinforcement in concrete.

In codes and standards, there are recommendations for protection against reinforcement corrosion. In the European and Turkish standards (TS EN 206 and TS ENV 1992), these recommendations are based on the protection of the steel reinforcement by limiting the access of aggressive agents. According to the above standards, the corrosion of the reinforcement can be prevented either by increasing the increasing the concrete cover so that the time needed for reaching of chloride ions to the steel reinforcement increases or increasing the concrete quality (decreasing water-to-cement ratio) so that the penetrability of the concrete decreases, which result in low corrosion rates. However, the cracking of the concrete is not taken into consideration in lifetime estimations. Concrete is a brittle material that can be easily cracked due to several factors. Therefore, how structure is influenced in the existence of the cracks is a big and important question that has to be solved and taken into consideration in the standards.

Many studies have been carried out on the onset of steel corrosion in reinforced concrete containing flexural cracking. These cracks, as a result of mechanical loading (static and dynamic loading), represents a preferential route for penetration of chloride and reduce the initiation period of the corrosion process (ACI Committee 222). However, Schiessl and Raupach showed that, if corrosion firstly appears on the steel located at the flexural crack tip, a self-healing phenomenon due to crack filling by the corrosion products slows down the migration of chlorides and reduces the corrosion activity (Schiessl and Raupach, 1997). Therefore, the influence of the flexural cracking on corrosion is still a contentious subject (Vidal *et al.*, 2007). According to ACI 224-01, as the crack width on the concrete surface increases, the corrosion risk increases.

On the other hand, ACI 318-08 stated that crack width values under 0.4 mm on concrete surface are not important for corrosion of reinforcing steel, but this statement was removed later since it is hard to set certain critical opening values for a material which can be produced with a wide variety of materials and properties as concrete. In the present study, the role of cracks on reinforcement corrosion will be investigated. The concrete cover and quality recommended by standards will be tested for cracked concrete. Moreover, the effects of the cracks (crack widths changes according to cover depths and loading conditions of the specimens), which were formed on the concrete surface under static and dynamic loading to the reinforcement corrosion, are examined. In ACI 224R-01, the importance of cyclic (dynamic) loading is emphasized as it can cause an increase in crack width and thus in corrosion. In addition to this, how steel fiber use in concrete affects the opening of the cracks under static and dynamic loading and its effects on the corrosion of reinforcing steel will be investigated. The corrosion resistance performance of specimens with cracks obtained under specified flexural loads will be compared for the plain and steel fiber reinforced concretes.

The main objective of this study was to determine the importance and influence of the crack size, depth of concrete cover and water-to-cement ratio, under different loading conditions (static and dynamic) and with and without the use of steel fibers on the chloride-induced corrosion activity of steel reinforcement by using galvanostatic pulse technique. Corrosion measurements were carried out for cracked and uncracked concrete. The results were used for the evaluation of the limits recommended by the standards.

2. LITERATURE REVIEW

2.1. Introduction

Concrete is one of the major building materials. Structures forming the modern civil infrastructure are built by using reinforced concrete. Although reinforced concrete, as a composite material, is a durable system, it may deteriorate, particularly, due to the corrosion of steel reinforcements in concrete. That is, corrosion of reinforcing steel is the most dominant factor for the premature deterioration of concrete structures. Corrosion of steel reinforcement may decrease its cross-sectional area and deteriorate the bond between the steel reinforcement and concrete, which directly affects the serviceability and ultimate strength of concrete elements within a structure. The permeability of concrete is regarded as a basic indicator of its durability. The penetration of agents such as chloride ions, oxygen, and water, into concrete causes deterioration. The lack of durability can very often be attributed to factors, such as poor design, inadequate cover depth, and the presence of cracks and aggressive agents that cause corrosion of the steel rebar.

2.2. Mechanism of Corrosion in Concrete

2.2.1. Process and Mechanism

Despite the generalized belief that the concrete is so compact and dense that the reinforcement is protected by alkaline environment, the reinforced concrete has not an indefinite service life as it can deteriorate for different reasons: mechanical, structural, physical, chemical (including reinforcement corrosion). The most important cause of damage is steel corrosion, that causes the formation of rust, with a six times bigger volume than iron from which it had been created; rust is therefore able to cause spalling reduction of bars cross section.

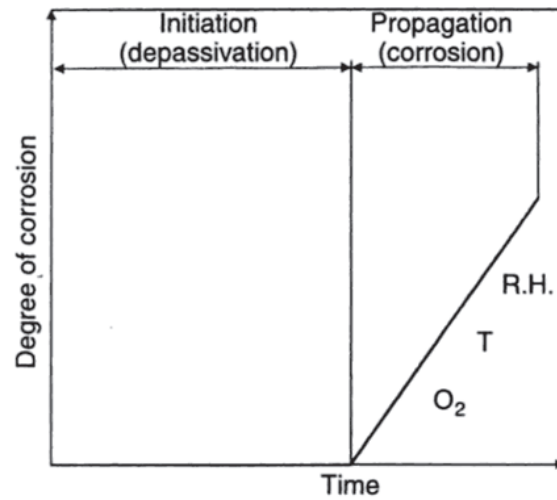


Figure 2.1. Corrosion model for steel in concrete (Tuutti, 1982).

In 1982, Tuutti idealized the problem of reinforcement corrosion in two distinct stages. The first stage, initiation, can be simply defined as the period during which corrosive agents, such as chloride ions or CO_2 move toward the reinforcement from the surface of the concrete but steel remains passive. The length of this period is a function of concrete quality, cover depth, exposure conditions (chloride level at the surface, temperature) and threshold chloride concentration required to initiate corrosion. The loss of passivity shows the beginning of the second stage, propagation, which involves active corrosion of the reinforcement.

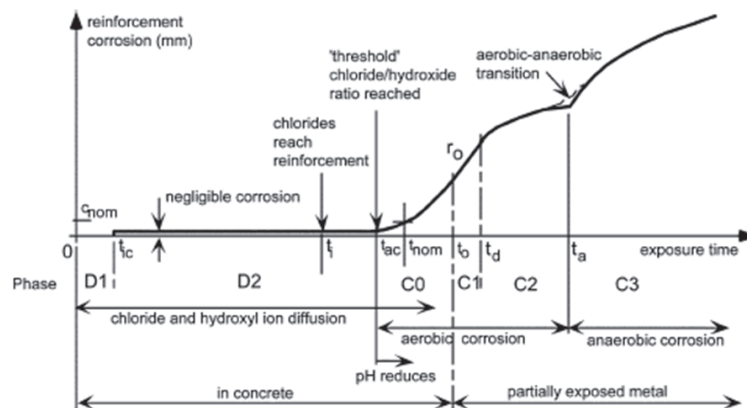


Figure 2.2. Phenomenological model for steel corrosion in concrete (Melchers and Li, 2006).

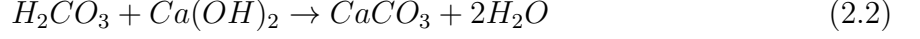
In addition to Tuutti model, Melchers and Li improved a phenomenological model for the corrosion of reinforcing steel bars in concrete as a function of time (Figure 2.2) (Melchers and Li, 2006). In this model, corrosion of cracked concrete is represented more details. The model consists of two stages with six detailed phases. Initiation and propagation stages of model are analogous to those of (Tuutti, 1982). However, the detailed phases are derived from the mechanics of corrosion (Melchers, 2003). As shown in Figure 2.2 phase D1 is the diffusion of chlorides into the concrete and the start of infiltration of hydroxyl ions out of concrete. If there are cracks present in the member, the local time to initiation t_{ic} is governed by the emergence time of the local cracks (Li, 2002). Otherwise, t_{ic} tends to be the initiation time t_i , which is governed by diffusion rate and concrete permeability (Melchers and Li, 2006). At t_i , although chlorides reaches the bars, active corrosion may not actually start (which is denoted as t_c) due to the imbalance between the concentrations of the Cl^- and $(OH)^-$ ions and the pH. During Phase C0, the pH will generally lessen because of the infiltration of hydroxyl ions out of concrete, which result in an increase on corrosion rate. In phase C1, oxygen rate, water supplies and corroding steel surface conditions control the propagation of corrosion. As corrosion progresses, there will be an increasing build-up of corrosion products on the corroding surfaces. This gives rise to longitudinal cracking and concrete spalling. Apart from this, these corrosion products lead to an increasing resistance to oxygen diffusion. Phase C2 represents the period when this controls the corrosion rate. Finally, diffusion rate of oxygen to the corroding bars through the rust layer will become so low that anaerobic corrosion activity will emerge (Melchers, 2003). This is denoted as Phase C3 in Figure 2.2.

2.2.1.1. Initiation Period. Reinforcing steel in concrete is susceptible to corrosion when it is exposed to the atmosphere. However, reinforcing steel embedded in concrete is usually protected from corrosion by both physical and chemical barriers (Aligizaki, 1999). Concrete includes a highly alkaline solution (pH 12 ~ 13) that envelopes the bars and the aggregate particles. The high alkalinity results from the presence of sodium, calcium and potassium hydroxides, derived from reaction between the cement particles and the mixing water. In this alkaline environment, corrosion is prevented

by a protective iron oxide film. This protected state of steel in concrete is known as passivity. The pH of the concrete pore solution is required to be between 11.5 and 13.8 to sustain the passivity of the steel. If the pH is decreased, the film becomes unstable and oxygen is able to react with the steel, causing corrosion. The passive film can be disrupted by carbonation, because of the penetration of CO_2 into the concrete, which decreases the pH of the pore solution, or by the presence of aggressive ions, especially Cl^- , which react with the passive layer (Gong, 2006).

(i) *Carbonation*: Carbonation of concrete occurs when atmospheric carbon dioxide penetrates the concrete. If atmospheric carbon dioxide diffuses into concrete continuously, the pH of the pore solution will be decreased because dissolution of CO_2 in water produces a weak acid that will react with hydroxyl ions in concrete pore solution. It occurs in concrete when carbon dioxide (CO_2) reacts with water, calcium, alkaline hydroxides so as to form calcium carbonate ($CaCO_3$), resulting in decrease of pore solution pH to values between 8.5 and 9. This gives rise to depassivation of the steel in contact with the carbonated zones. As concrete is a porous material, the pore structure of the cementitious phase and the humidity inside concrete determine the diffusion of CO_2 into concrete. Carbonation does not occur, when concrete pores are dry, and also when the concrete pores filled totally with water. In the latter, CO_2 cannot readily soluble in water (diffusion rate of CO_2 is about 10^4 times lower in water than in air). Carbonation becomes more important, if the pores are partly filled with water, which is normally the case close to the concrete surface. Optimum conditions for increased carbonation rates include temperatures near $20^\circ C$, increased carbon dioxide concentrations, relative humidity in the range of 50 – 70%, and water-to-cement ratios at or above 0.6. The reactions involved in the carbonation of calcium hydroxide are: (Pereira, 2003)





(ii) *Chloride Attack*: When chloride ions are introduced in concrete during mixing or after exposure to the environment, they may depassivate the steel by locally breaking down the protective layer. Chlorides act as a catalyst to corrosion when they are present in sufficient concentrations at the steel surface. They are not consumed in the process but aid to break down the passive layer and allow corrosion to proceed faster. This is illustrated in Figure 2.3.

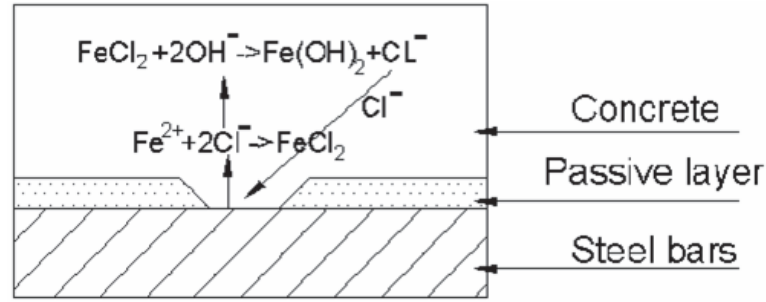


Figure 2.3. Chloride attack and chloride recycle (Bi, 2008).

Chloride ions surrounding the reinforcement react at anodic sites to form hydrochloric acid, which destroys the passive protective film on the steel. The surface of the steel then gets activated locally to form the anode, with the passive surface forming the cathode: the resulting corrosion in the form of localized pitting (pit-corrosion). The chloride ions react with Fe^{2+} to form chloride or oxychloride compounds (Melchers and Li, 2006). The exact mechanism of breakdown of passivity by chloride ions is not clearly known. There have been a number of suggestions by different researchers as to how the chloride ions depassivate steel under stable conditions (Foley, 1970; Alvarez and Galvele, 1974; Uhlig and Revie, 1985). These are summarized as follows

- Complex formation of steel halides on the surface. The overall process occurs with a chemico-adsorption reaction between a surface atom of the metal and

the chloride. The complex can pass into solution after a certain potential is reached. Once in solution the halide complex will dissociate and other more stable hydroxides will be formed (Foley, 1970).

- Chloride ions are preferentially adsorbed in competition with oxygen and may displace passivating species, so allowing for increased metal corrosion (Foley, 1970).
- Chloride ions bring about local acidification thus destabilizing the anodic film at low pitting potentials. Further anion adsorption is required as local acidification is not sufficient to cause pitting at higher pitting potentials (Alvarez and Galvele, 1984).
- Chloride ions penetrate the oxide layer through concrete pores easier than other ions (Uhlig and Revie, 1985).
- Cl^- ions colloidal disperse the oxide layer and raise its permeability (Uhlig and Revie, 1985).
- Cl^- ions raise the exchange current and lessen the overpotential for anodic dissolution (Uhlig and Revie, 1985).

However, passivity should not be viewed as a complete protection of steel as it limits the value of corrosion. The presence of Cl^- ions will help the breakdown of the passive film while other anions such as OH^- are responsible for its stability and have inhibiting properties. If the concentration of aggressive ions overcomes the inhibiting ions, then corrosion will start. This point is known as pitting potential, below which passivity is maintained (Bockris *et al.*, 1981; Otieno *et al.*, 2010).

2.2.1.2. Propagation Period. When corrosion agents (O_2 , CO_2 , H_2O and Cl^-) continue to be available at the corrosion sites (anode and cathode) after corrosion onset, corrosion may progress into the propagation period. Active corrosion indicates corrosion propagation and this gives rise to generation of crack, delamination, and spalling of concrete cover because of expansive corrosion products (Yoon *et al.*, 2000). Ordinarily, corrosion rate of $0.1 \mu A/cm^2$ is accepted as transition point from passive to active corrosion (Alonso *et al.*, 2002).

2.2.2. Electrochemical Nature of Corrosion

The corrosion processes include chemical reactions and electronic charge transfer between the metal and its environment. Such reactions are known as electrochemical reactions. A summary of the different reactions is provided in the next sections.

2.2.2.1. Half Cell Reaction. A potential difference emerges at the liquid/solid interface when a piece of metal is immersed in a solution (Ramachandran and Beaudoin, 2001). This results from the uneven charge distribution in both solid and liquid phases. It is not possible to determine the absolute value of the potential difference at single interface, so a second electrode is essential to complete an electrical circuit. The sum of two half-cell potentials constitutes the cell potential. Considering iron dissolution in an acid. The cell reaction is written as:



The above reaction indicates summation of the following two half-cell reactions. Cathodic reaction:



Anodic reaction:



A fixed potential difference can be obtained with reference to a so-called reference electrode that has a half-cell potential defined as zero. The half-cell potential of a system in which the reaction is not at unit activity can be easily calculated from the

Nernst equation as follows: For reaction:



$$E = E_0 + \frac{RT}{nF} \ln\left(\frac{C_O}{C_R}\right) \quad (2.7)$$

where E is the half-cell potential, E_0 is the standart half-cell potential, n is the transferred electrodes number, F is Faraday's constant, R is gas constant, T is absolute temperature, and C_0 , C_R are the concentrations of species O and R (Ramachandran and Beaudoin, 2001). Cathodic current emerges when reduction takes place at the working electrode. If the working electrode potential is driven negative from equilibrium potential, a cathodic current will be created. The polarity of the cathodic current is usually considered to be negative (Bi, 2008).

2.2.2.2. Butler-Volmer Equation. A knowledge of the rate at which a metal will corrode, especially under different solution conditions and pH is essential when trying to design systems that will prevent or slow down corrosion processes. Since the kinetics of corrosion essentially the kinetics of electron transfer, the equations used to understand electrode kinetics can be used. The most important one is Butler-Volmer Equation as this relates current (rate of interfacial reaction) with both applied voltage and the E_0 for a specific system. The corrosion form of the Butler-Volmer Equations:

$$i = i_o \left[\exp\left(\frac{(1 - \beta)\eta f}{RT}\right) - \exp\left(\frac{-\beta\eta f}{RT}\right) \right] \quad (2.8)$$

The first term describes the anodic reaction and the second one the cathodic reaction. The $1 - \beta$ and $-\beta$ are the energy fractions of the reaction in the anodic

and cathodic direction, respectively. Overpotential, measures how far the electrode potential is shifted away from the equilibrium potential is termed as η . A corroding system has both anodic and cathodic processes taking place, with the electrons being liberated by the oxidation process being used up in the reduction process. Therefore, one would expect the value of i_{corr} in both the anodic and cathodic reactions to be identical (Bi, 2008).

2.2.2.3. Mixed Potential Theory. According to mixed potential theory, any electrochemical can be divided into separate oxidation and reduction reactions with no net accumulation of electrical charge. In the absence of an externally applied potential, the oxidation of the metal and the reduction of some species in solution take place simultaneously at the metal/electrolyte interface. Under these circumstances the net measurable current is zero and the corroding metal is charge neutral, with no net accumulation of charge.

The mixed potential theory is shown in Figure 2.4 for iron corrosion in acid, which contains two reversible reactions, the Fe/Fe^{++} and the H_2/H^+ . Both reversible reactions are connected to reversible equilibrium potentials $E_{Fe/Fe^{++}}$ and E_{H_2/H^+} . The intersection between the iron dissolution curve (the anodic branch of the Fe/Fe^{++} reversible reaction) and the hydrogen evolution curve (the cathodic branch of the H_2/H^+ reversible reaction) represents the condition at which the anodic (oxidation) and cathodic (reduction) currents are equal and no net external current flows. The current value at such a point is defined as the corrosion current i_{corr} and the corresponding potential is the mixed-potential or corrosion potential, i_{corr} (Bi, 2008).

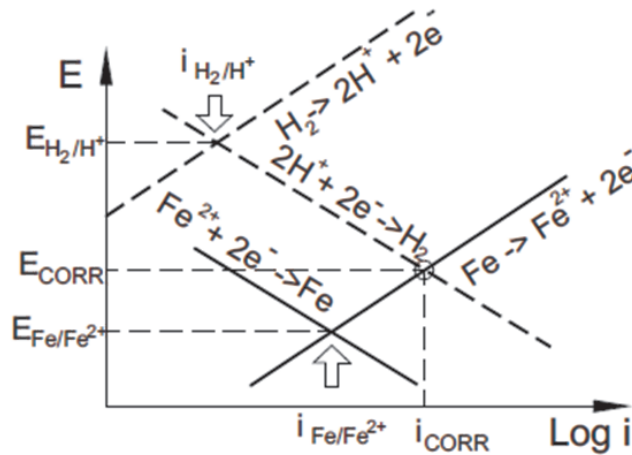


Figure 2.4. Description of mixed potential theory (Bi, 2008).

2.2.2.4. Corrosion of Steel in Concrete. Corrosion of steel includes electrochemical reactions. Such reactions are based on the principle of donating or receiving electrons. Generally these are called oxidation (emission of electrons) and reduction (consumption of electrons) reactions. Oxygen is one of many oxidizing agents and is very important when dealing with corrosion of steel in concrete. The basic mechanism for corrosion is illustrated in Figure 2.5. The figure shows not only cathodic and anodic processes but also the stable products, which are thermodynamically feasible. Environmental conditions such as temperature and moisture content affect corrosion process by influencing the pH and corrosion rate. The figure also includes chloride and carbonation attacks as environmental conditions impacting the process (Küter, 2009).

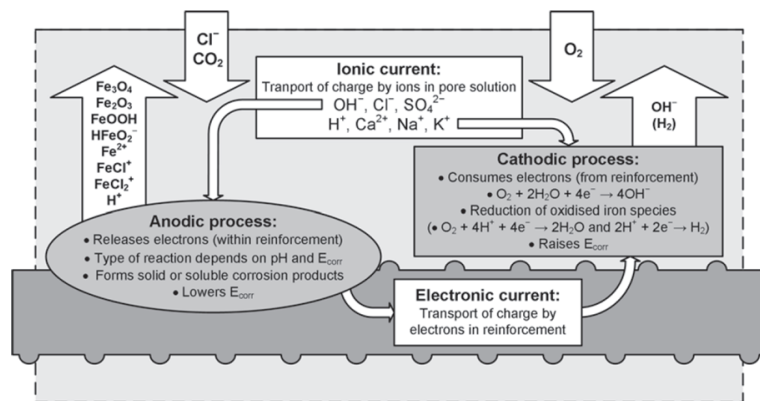


Figure 2.5. Fundamental mechanism for corrosion (Küter, 2009).

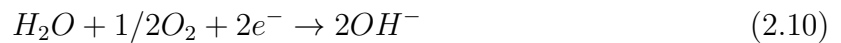
Corrosion process involves an anodic (oxidation) and a cathodic (reduction) reaction, which take place on the steel surface. Initially the anodic process occurs and the emitted electrons react at the cathodic area.

Anodic reaction:



There is therefore a difference between the electrodes potential. To achieve equilibrium of neutrality, the two electrons must be consumed. Therefore, the electron will be transported over to cathode electrode through the solution (electrolyte). Transportation of the electrons gives rise to an electrical current to flow in the electrolyte. Over at the cathodic region, a cathodic reaction takes place:

Cathodic reaction:

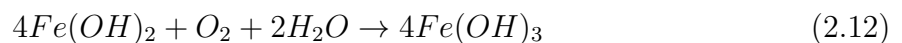


Both the anodic and cathodic reactions are only the first step in the process of creating rust. The hydroxyl ions (OH^{-}) created in the cathode would then be transported back to anode and form a reaction to create ferrous hydroxide.

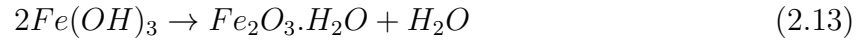
Anode:



With the presence of oxygen (O_2) and water (H_2O), ferrous hydroxide would generate a reaction of:



to produce ferric hydroxide. Finally, the ferric hydroxide would react:



to form hydrated ferric hydroxide, a rust production. The iron oxides formed and their corresponding volumes compared to the volume of iron are shown in Figure 2.6 (Bi, 2008).

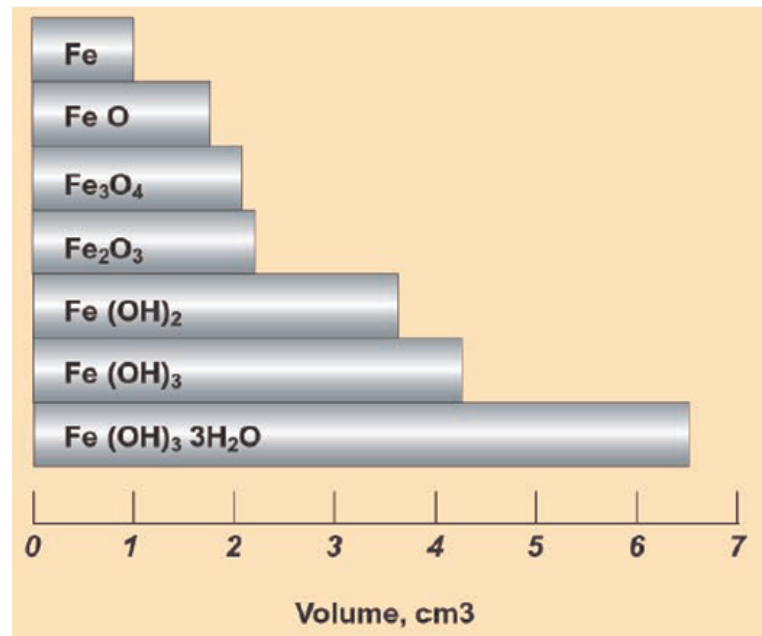


Figure 2.6. Relative volume for various oxides of iron. The volumes of all oxides are expressed per iron-atom (Bi, 2008).

2.2.3. Two Mechanism: Localized Corrosion and Distributing (Uniform) Corrosion

Microcell and macrocell are two terms commonly used when discussing reinforcement steel corrosion. These terms are related to describe the relationship between anode and cathode. Microcell is used when the anode and cathode are localized and located closely side by side, leading to uniform iron dissolution over the whole surface,

whereas macrocell indicates when corrosion is more generalized and the anode and the cathode are a finite distance apart. In microcell corrosion, the anode and cathode may take place on the same bar or different bars with electrical continuity. In the other, due to the large cathode/anode area ratio, the local dissolution (reduction in cross-section of the rebar) may be greatly accelerated. This rapid corrosion attack may give rise to structural safety problems.

Generally, reinforcing steel in concrete will corrode under the effect of a combination of many microcell and macrocell interactions (Sharp, 2005).

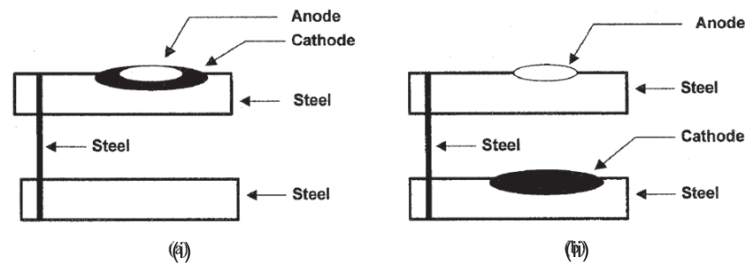


Figure 2.7. Illustration of (i) microcell versus (ii) macrocell corrosion (Sharp, 2005).

2.3. Methods to Detect Corrosion

Many electrochemical techniques are available for monitoring of steel in reinforced concrete structures. This section provides an overview of these techniques.

2.3.1. Half Cell Potential Measurement

This is an electrochemical method (also referred to as open circuit or corrosion potential) and is one of the most widely used methods for the detection of the potential for steel reinforcement corrosion in concrete. The principle of this method is based on the measurement of the potential difference between the reinforcement and a reference electrode in the form of a half-cell (Rodriguez *et al.*, 1994, Broomfield *et al.*, 2002). This technique is particularly useful because it can be used to evaluate the probability of corrosion before damage is evident at the surface of a reinforced concrete structure. The

half-cell is a simple device consisting of an electrode and an electrolyte. The electrolyte is made from a soluble salt of the electrode metal i.e. the reference electrode is a piece of metal in a solution of its own ions, such as copper/copper sulphate ($Cu/CuSO_4$), Calomel (Hg/Hg_2Cl_2) and silver/silver chloride ($Ag/AgCl$).

Half-cell potential (E_{corr}) measurement is based on the electrical and electrolytic continuity between the embedded steel, the reference electrode on the concrete surface and the voltmeter. A high impedance voltmeter is utilized to collect the data, connecting the reference electrode and the reinforcement as shown in Figure 2.8. When there is active corrosion, current flow (ion migration) through the concrete between anode and cathode is accompanied by an electrical potential field surrounding the corroding bar. The equipotential lines intersect the concrete surface and the potential at any point can be measured using the half-cell potential method. This potential value depends on the type of reference electrode and the corrosion condition of the steel in the concrete (Otieno *et al.*, 2010).

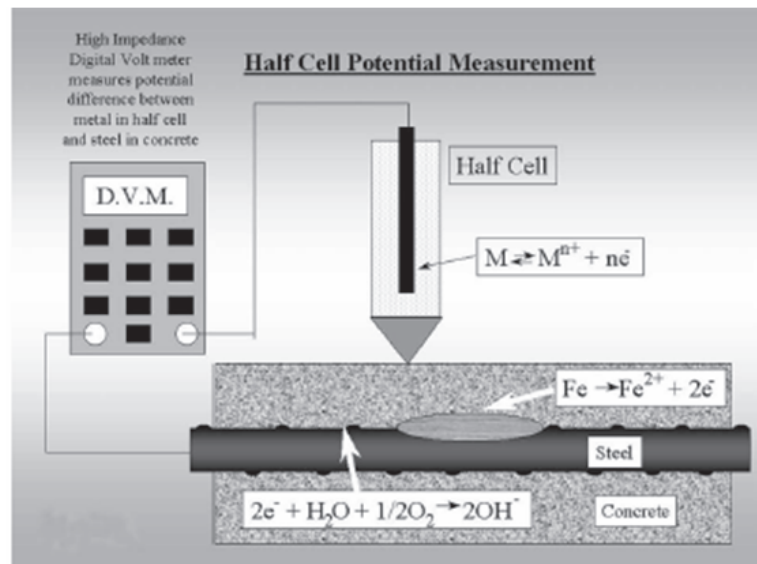


Figure 2.8. Illustration of half-cell potential measurement (Broomfield *et al.*, 2002).

The basic test procedure for half-cell potential is given in ASTM C876 (Bi, 2008). The half-cell potential readings only give an indication of corrosion risk of steel and are linked to the probability of corrosion. This standard provides guideline for interpreting

the half-cell potential readings as shown in Table 2.1.

Table 2.1. Interpretation of corrosion potential measurements.

Cu/CuSO ₄	Calomel (SCE)	Ag/AgCl	Interpretation
$E > -200\text{mV}$	$E > -126\text{mV}$	$E > 119\text{mV}$	Greater than 90% probability that no corrosion is occurring
$-200\text{mV} < E < -350\text{mV}$	$-126\text{mV} < E < -276\text{mV}$	$-119\text{mV} < E < -269\text{mV}$	Corrosion activity is uncertain
$E < -350\text{mV}$	$E < -276\text{mV}$	$E < -269\text{mV}$	Greater than 90% probability that corrosion is occurring

Some factors such as concrete moisture content, chloride content, oxygen content, temperature, concrete resistivity, cover thickness and cell placement affect half-cell potential measurements. Especially a decrease in oxygen concentration at the surface of steel will result in a more negative corrosion potential reading. This kind of negative potential reading may not be associated with a high probability of steel corrosion. The main disadvantage of the half-cell potential is that it does not give any information about corrosion rate. Apart from this, data analysis and interpretation of half-cell potential measurement is complicated. For example, a variation of 100 mV (from -150 mV to -250 mV) in half-cell potential reading may show more active rebar corrosion at a given section with a 30 mV variation (from -250 mV to -280 mV). Therefore, half-cell potential measurements should not be taken into consideration alone when evaluating corrosion of reinforcing steel (Bi, 2008).

2.3.2. Linear Polarization Resistance (LPR)

Linear polarization resistance (LPR) provides a method for determining the total corrosion rate (macrocell and microcell) of a metal by measuring its response to an applied voltage (polarization). If there is no externally applied voltage, a metal will corrode with a current density i_{corr} and a potential E_{corr} . Forcing the potential to shift by an amount ΔE will give rise to the current by an amount Δi . The change in voltage

(ΔE) and applied current per unit area of electrode (Δi) is given as

$$R_p = \frac{\Delta E}{\Delta i} = \frac{\Delta E}{\Delta I / AREA} \quad (2.14)$$

where R_p is the polarization resistance, ΔE is the imposed potential change, Δi is the current density change caused by ΔE .

For small changes in potential, the polarization curve is linear. In this region, the polarization resistance is inversely proportional to the current density.

$$i_{corr} = \frac{B}{R_p} \quad (2.15)$$

$$B = \frac{\beta_a \beta_c}{2.303(\beta_a + \beta_c)} \quad (2.16)$$

where i_{corr} is the corrosion current density in $\mu A/cm^2$, B is a constant in V (26 mV for passive steel and 52 mV for active steel), R_p is the polarization resistance in $\Omega \cdot cm^2$, β_a is the Tafel slope of the anodic reaction, β_c is the Tafel slope of the cathodic reaction.

A typical setup, which is known as the 3LP system, for obtaining linear polarization measurement from steel in concrete is shown in Figure 2.9. The first electrode, the working electrode, is the steel that is embedded in the concrete, the second is the counter/auxiliary electrode that applies current or potential to the working electrode, the last electrode, or the reference electrode, enables the corrosion potential and over-potential of the steel to be measured between the steel reinforcement bar and an auxiliary electrode placed on the surface of the concrete.

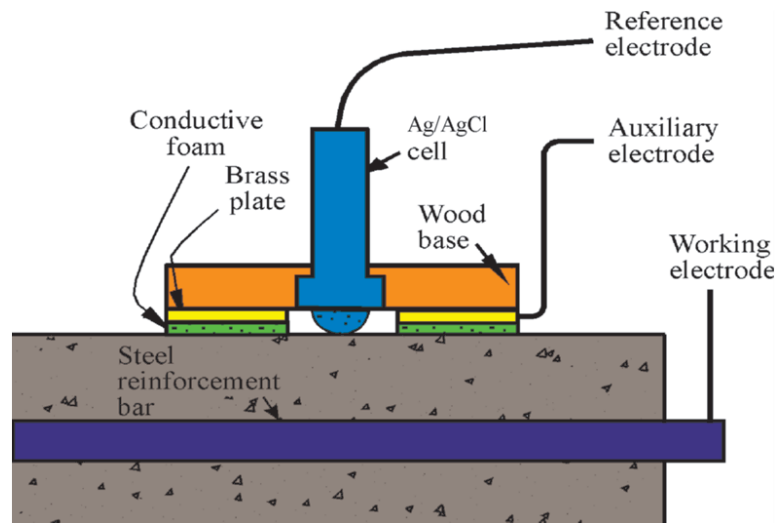


Figure 2.9. Linear polarization resistance measurement (Millard and Sadowski, 2009).

In a typical measurement, R_p can be found from the slope of potential-current density curve. Broomfield gave a guideline for estimation of corrosion extent using I_{corr} . Metal loss caused by corrosion can be found by using Faraday's law on condition that the assumption that is corrosion uniform along the metal surface is accepted (Broomfield, 1994).

Table 2.2. Guideline for estimation of corrosion extent using I_{corr} (Broomfield, 1994).

R_p (Ohms*cm ²)	Corrosion current density, ($\mu A/cm^2$)	Extent of corrosion
$R_p > 260,000$	$I_{corr} < 0.1$	P: passive condition
$10000 < R_p < 260,000$	$0.1 < I_{corr} < 0.5$	L: low to moderate corrosion
$5000 < R_p < 260,000$	$0.5 < I_{corr} < 1$	M: moderate to high corrosion
$R_p < 5000$	$I_{corr} > 1$	H: high corrosion

The major limitation of LPR technique is the difficulty in knowing the actual surface area of steel that is polarized by the applied potential, or the area that is actively corroding. This difficulty is solved by using a guard ring around the counter/auxiliary electrode to confine the applied potential from the counter electrode (Bi, 2008).

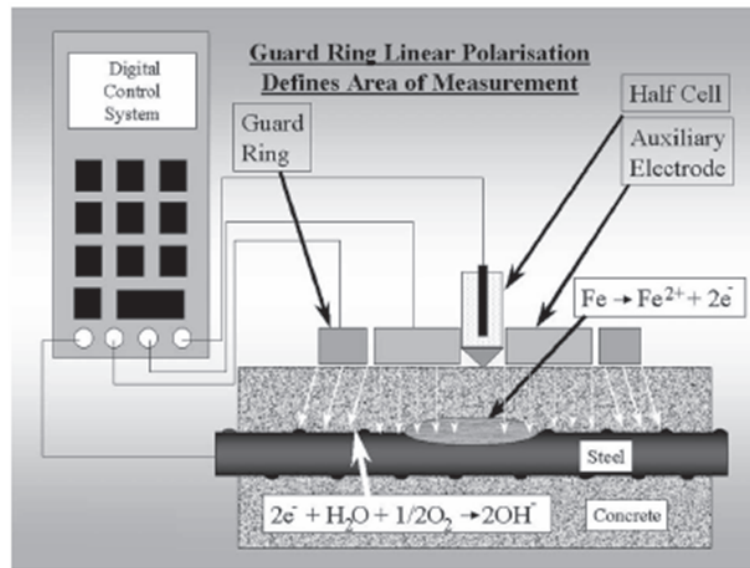


Figure 2.10. Guard ring linear polarization defines area of measurement (Broomfield *et al.*, 2002).

The disadvantages of using the linear polarization method for determining the corrosion of steel in concrete are the high resistance of electrolytes and the existence of macrocell action. The presence of macrocell action states that the corroding reinforced steel is polarized away from its normal free-corrosion potential.

2.3.3. Electrochemical Impedance Spectroscopy (EIS)

The Electrochemical Impedance Spectroscopy (EIS) technique is a non-destructive model and it is widely used for solid materials, both fundamental and applied electrochemical studies, and has been used commonly in determining the corrosion rate of reinforcing steel in concrete.

This technique includes the application of an alternate voltage of about 10 to 20 mV to the rebar and the resultant current and phase angle are measured for various frequencies. The impedance of the system is the ratio of the A.C. voltage to the current that is easily evaluated. An equivalent electrical circuit can be determined which would give the same response as the corrosion system by studying the variation of the impedance with frequency.

EIS technique can measure the corrosion rate of the rebar and it provides corrosion kinetics information. However, interpretation of an impedance spectrum is difficult because of the complexity of cement chemistry and concrete microstructures. Also, due to cost of equipment and sophistication of the measurement, this technique is less frequently used in the field (Miller, 2010).

2.3.4. Concrete Resistivity Measurement

Electrical resistivity (or its inverse conductivity) is a measure of the ability of concrete to resist the passage of electric current. It is basically related to the permeability of fluid diffusivity of ions through concrete. The electrical resistivity of concrete is an important parameter about determination of intensity of the initiated corrosion process; a high resistivity of concrete will reduce corrosion currents and slow the rate of corrosion.

Corrosion activity on steel reinforcement can be evaluated by measuring the electrical potential of the reinforcement against a reference half-cell. Alternating Current (AC) and Direct Current (DC) measurements are used for determination of electrical resistivity of concrete. In DC measurements the current and voltage do not vary with time whereas in AC measurements the current and voltage changes with time. The AC measurements can be conducted both by means of two and four-pin methods. The most common surface mounted probe is known as the Wenner array. An alternating current passes between two outer probes and the potential difference is measured across two inner probes so that resistivity is calculated (Figure 2.11). Resistivity is strongly affected by concrete qualities such as cement content, water/cement ratio, admixtures, curing, and so on. The electrical resistivity of concrete also depends on the capillary pore size, porosity of concrete, and moisture content. For example, moist concrete has a resistivity of order of 10^4 ohm-cm, value in the range of semiconductors. Oven-dried concrete has a resistivity of the order of 10^{11} ohm-cm, which is a good insulator. Electrical resistivity of concrete rises with the age and it decreases with increase in salinity of mixing water (David, 2007).

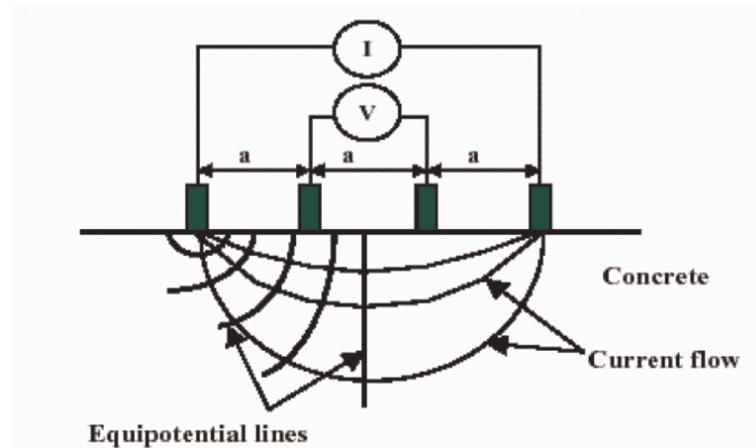


Figure 2.11. Four Probe Resistivity Meter or Wenner Probe (Ramniceanu, 2004).

2.3.5. Galvanostatic Pulse Technique

The galvanostatic pulse technique is a rapid non-destructive polarization method that was investigated to evaluate the corrosion rate of steel reinforcement in concrete structures. This method is a transient polarization technique working in the time domain. The method set-up is shown in Figure 2.12.

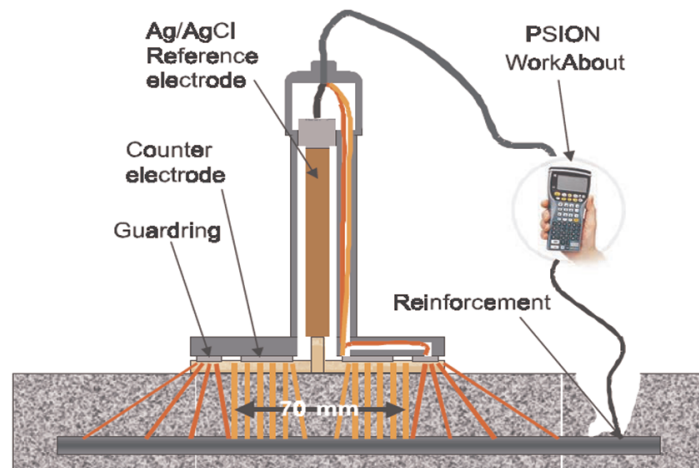


Figure 2.12. Set-up of the galvanostatic pulse technique (Baessler *et al.*, 2003).

A short time anodic current pulse is imposed into the reinforcement galvanostatically from a counter electrode placed on the concrete surface together with a reference electrode, such as Ag/AgCl. The applied current is generally in the range of 5 to 400

μA and the typical pulse duration is up to 10 seconds. The small anodic current gives rise to a change of the reinforcement potential, recorded as a function of time. The steel reinforcement is polarized in the anodic direction compared to its free corrosion potential. Ag/AgCl reference electrode records the change of the reinforcement potential as a function of polarization time. A typical polarization response is shown in Figure 2.13.

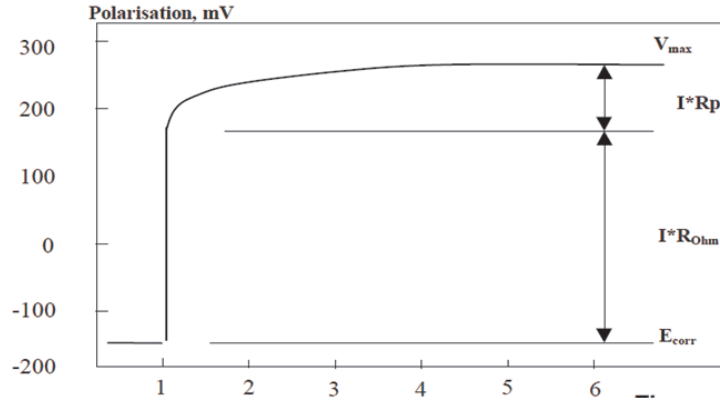


Figure 2.13. Typical potential time curve as response to a galvanostatic pulse (Klinghoffer *et al.*, 2000).

When the constant current I_{app} is applied to the system, an immediate ohmic potential jump and a slight polarization of the rebar emerge. The polarized potential of reinforcement V_t at a given time t , can be expressed as:

$$V_t(t) = I_{app}[R_p[1 - \exp(-t/R_p C_{dl})] + R_\Omega] \quad (2.17)$$

where R_p is the polarization resistance, C_{dl} is the double layer capacitance, R_Ω is the ohmic resistance.

So as to get the values of R_p and C_{dl} separately from the ohmic resistance R_Ω , the above equation changes to linear form:

where V_{max} is the final steady potential value reached after long polarization. Extrapolation of this straight line to $t = 0$, using least squares linear regression analysis, gives an intercept corresponding to in $(I_{app}R_p)$ and a slope of $1/(R_pC_{dl})$.

$$\ln(V_{max} - V_t(t)) = \ln(I_{app}R_p) - t/(R_pC_{dl}) \quad (2.18)$$

The remaining overpotential corresponds to $I_{app}R_\Omega$, the ohmic voltage drop. After polarization resistance (R_p) is determined via this analysis, the corrosion current I_{corr} can be calculated by using Stern Geary equation:

$$I_{corr} = B/R_p \quad (2.19)$$

where B is empirical constant, 25 mV for actively corroding steel and 50 mV for passive steel. And Faraday's law of electrochemical equivalence, the corrosion rate is estimated as:

Corrosion Rate:

$$(\mu m/year) = 11.6 * I_{corr}/A \quad (2.20)$$

where A is the confined area (in cm^2) of the reinforcement below the central counter electrode. The factor 11.6 is for black steel.

The value of R_o , the electrical resistance of the concrete between the counter electrode and the reinforcement, is also determined.

Clear made his model on the combination of laboratory, outdoor exposure, and field studies. He suggested the use of the following relationships between the corrosion rate and remaining service life (Clear, 1989):

- I_{corr} less than $0.5 \mu\text{A}/\text{cm}^2$ ($6 \mu\text{m}/\text{year}$) - no corrosion damage expected.
- $0.5 < I_{corr} < 2.7 \mu\text{A}/\text{cm}^2$ (6 and $30 \mu\text{m}/\text{year}$) - corrosion damage possible in the range of 10 to 15 years.
- $2.7 < I_{corr} < 27 \mu\text{A}/\text{cm}^2$ (30 and $300 \mu\text{m}/\text{year}$) - corrosion damage expected in 2 to 10 years.
- I_{corr} more than $27 \mu\text{A}/\text{cm}^2$ ($300 \mu\text{m}/\text{year}$) - corrosion damage expected in 2 years or less.

Frolund measured the corrosion activity with Galvapulse instrument. The following threshold values to categorize the risk of the corrosion rate (I_{corr}) (Frolund, 2002):

- $< 0.5 \mu\text{A}/\text{cm}^2$ - passive areas ($< 0.006 \text{ mm}/\text{year}$)
- $0.5 - 2 \mu\text{A}/\text{cm}^2$ - negligible corrosion activity ($0.006 - 0.023 \text{ mm}/\text{year}$)
- $2 - 5 \mu\text{A}/\text{cm}^2$ - low corrosion activity ($0.023 - 0.058 \text{ mm}/\text{year}$)
- $5 - 15 \mu\text{A}/\text{cm}^2$ - moderate corrosion activity ($0.058 - 0.174 \text{ mm}/\text{year}$)
- $> 15 \mu\text{A}/\text{cm}^2$ - high corrosion activity ($> 0.174 \text{ mm}/\text{year}$)

The corrosion rate values are very dependent on the environmental conditions during measurements and the local area of the corroding rebar. Varying temperature and humidity have a great influence on the measured corrosion rate values. Due to the influence of the environmental conditions the linearity of the change of corrosion rate with time cannot be expected (Frolund *et al.*, 2000).

2.4. Factors Affecting Corrosion of Steel in Concrete

2.4.1. Effect of Concrete Cover on Corrosion of Steel in Concrete

The concrete cover to steel reinforcement is a very important design parameter affecting corrosion deterioration. Concrete cover thickness influences mainly the ingress of chlorides and oxygen into concrete. The function of concrete cover is to protect the reinforcement from exposure to corrosive environment. Increased clear cover provides improved protection for the steel, especially if low permeability concrete is used. Mathew found out that an increased cover will increase the time needed for chlorides to reach steel reinforcement (Mathew, 2006). Moreover, Austin *et al.*, stated that the thickness of the concrete cover had a greater impact than the concrete strength on the time needed for chlorides to permeate through the cover (Austin *et al.*, 2004). Apart from this, an increased concrete cover depth can reduce the number of cracks when concrete subjected to external loading. Arya and Ofori-Darko studied the effect of crack frequency on corrosion of steel and they found out that thicker concrete cover gives rise to a reduction of the crack frequency and eventually the rate of corrosion (Arya and Ofori-Darko, 1996).

2.4.2. Effect of Water-to-Cement Ratio on Corrosion of Steel in Concrete

The water-to-cement ratio of the concrete influences the permeability and strength of the concrete. A higher water-to-cement ratio allows the formation of a more porous framework and an easier access to the steel reinforcement by the environmental aggressors. Petterson and Sandberg stated that faster diffusion of chloride ions to the steel rebar may result in easier oxygen penetration and lower electrical resistivity of concrete. With a decrease in water-to-cement ratio, the quantity of oxygen decreases because of decreased concrete permeability. This gives rise to a reduction in the corrosion rate (Petterson and Sandberg, 1997). Mohammed *et al.*, studied the effects of crack width on corrosion of steel in concrete and they found out that the relationship between water-to-cement ratio and corrosion rate of steel bars in concrete is more significant than the relationship between crack width and corrosion rate (Mohammed *et*

al., 2001). Otieno *et al.*, found out that concrete quality was the major factor even in the presence of cracks in reinforcement corrosion. In addition, concrete resistivity increases with a decrease in water-to-cement ratio (Otieno *et al.*, 2010). Therefore, the water-cement ratio is very important factor that affects the durability of the overall reinforced concrete structures and must be taken into consideration at the design stage if the RC structure will be subjected to aggressive environments during its service life.

2.4.3. Effect of Cracks on Corrosion of Steel in Concrete

Concrete is waterproof in an uncracked state when it is properly placed, cured and sufficiently cured. With the formation of cracks, reinforced concrete may deteriorate due to the ingress of aggressive ions, such as chloride ions, which result in corrosion of steel reinforcement. The effects of cracks in concrete cover vary not only with their effective depth, but also with their width and frequency.

2.4.3.1. Effect of Cracking on Chloride Ion Penetration. Cracking of concrete structures due to bending or tension has great significance on structural behavior. Structural cracks affect not only serviceability but also durability of structures. From the serviceability point of view, the reduction of stiffness and increase of deformations, the aesthetical concerns and water leakage through cracks can be mentioned. On the other hand, the possible attack of steel corrosion and the reduced service life of structures can be important parameters of durability stage. Cracks can affect the durability of concrete structures in marine environments by accelerating the ingress of chlorides into the concrete, which gives rise to high corrosion rate. Sagues and Kranc studied the corrosion distribution of reinforcing steel in cracked concrete and they found out that when cracks occurred in reinforced concrete, mass transfer of corrosion species, such as chloride ions, increased (Sagues and Kranc, 1998). Cracks can have adverse impacts on significant parameters such as permeability and hence the rate of chloride ingress, which affects reinforcement corrosion process (Jacobsen *et al.*, 1998). Francois and Maso studied the effects of different cracks width ranging from 0.05 to 0.5 mm on permeability of concrete and they found that chlorides penetrate rapidly through

cracks (Francois and Maso, 1988). A study of Kansas bridge decks has shown that, the chloride content at the level of reinforcement remained in uncracked concrete below the critical chloride corrosion threshold after 12 years of service for most decks, whereas in cracked concrete the critical chloride threshold for reinforcing steel was exceeded within two years, and often within the first year (Lindquist *et al.*, 2006). These studies have shown that cracks have a significant impact on chloride ion penetration, which increases corrosion of reinforcing steel.

2.4.3.2. Effect of Crack Frequency on Corrosion of Steel in Concrete. The number of cracks present on the concrete surface per given length is known as crack frequency. Crack frequency (crack density) may be more important than the crack width, since as the crack frequency increases, the permeability of concrete will increase, which ease of chloride ion ingress into concrete. Arya and Ofori-Darko studied the influence of crack frequency on reinforcement corrosion in concrete and that concluded that the corrosion rate can be reduced significantly by limiting the crack frequency rather than the surface crack widths (Arya and Ofori-Darko, 1996).

2.4.3.3. Impact of Crack Width on Corrosion of Steel in Concrete. A debate among the researchers on crack width and corrosion rate relationships (greater crack widths result in greater corrosion rate) has been continuing for a long time. Several researches found little relationships between crack width and corrosion rates (Beeby, 1983; Arya and Wood, 1995; Arya and Ofori-Darko, 1996). Others support that corrosion rate is influenced by crack width. For two reinforced concrete members with different crack widths, the time required to initiate corrosion is shorter for one with the wider crack (Bentur *et al.*, 1997). Pettersson studied the influence of crack width on corrosion process and found out that when the crack widths remained small (< 1 mm), they had little effect on corrosion process; whereas, larger cracks (> 1 mm) increased the corrosion rate (Pettersson, 1996). Otieno *et al.*, also found out that corrosion rate increased with the increasing crack width in the following orders: uncracked $<$ incipient-cracked, 0.4 mm, 0.7 mm (Otieno *et al.*, 2010). Based on these studies, several design codes limit the crack widths so as to control the rate of corrosion (American Concrete

Institute (ACI), 1980; British Standards Institution (BSI), 1992; Japan Society of Civil Engineers (JSCE), 1986). On the other hand, European design code recommended that the limitation of crack widths is no means to avoid the attack to the reinforcement in the case of severe chloride attack to horizontal concrete surfaces (Comité Européen International du Béton (CEB), 1989). This may be due to the ponding effect of the chlorides on the concrete surface, and therefore faster ingress of the chlorides into the concrete.

From the viewpoint of durability, maximum crack width in an aggressive environment should be small. Results of exposure tests and site inspections confirm that within a common range of crack widths (up to 0.4 mm), the effect of the transverse crack on the corrosion rate of the steel reinforcement is relatively small (ACI 318, 2008). Therefore, most codes adopt a crack width of 0.4 mm as a threshold value. However, Otieno *et al.*, 2010 assessed the corrosion in cracked concrete and they concluded that it is not possible such a universal limiting/threshold crack width for all concrete types and the major factor affecting the corrosion of reinforcing bars was concrete quality and therefore must be taken into consideration. They also noted that concrete cover depth was also an important factor. Scott (2004) concluded that at a constant crack width, the decrease in the concrete cover result in a significant increase in corrosion rate. Gowripalan and his friends (2000) studied the chloride diffusivity on concrete cracked in flexure. They concluded that crack width/cover ratio can be suitable parameter to estimate the durability of a cracked reinforced concrete. It can be depicted that if surface crack width increases for a constant cover, the corrosion rate will also increase, since at a constant cover, an increase in the surface crack widths may cause a greater steel surface being exposed. Therefore, disregarding the concrete cover effect may give rise to mislead the corrosion rate result.

2.4.4. Effect of Steel Fibers on Corrosion of Steel in Concrete

Fiber addition into the concrete brings a better control of its cracking and enhances its mechanical properties. Especially, steel fiber use decreases its fragile character and gives it ductility and so that the steel fibers distribute the stress evenly

throughout the material, causing multiple small cracks instead of a bigger crack. The main function of the fibers in members subjected to loading is that of resisting the opening of the cracks due to the micro cracking, increasing the ability of the composite to withstand loads, and to allow larger strains. The considerable increase in ductility in fiber composites implies that the fibers perform an important confining action inside a loaded structural member (Kwak *et al.*, 1991). Recent studies have shown that the fiber reinforcement concrete reduces permeability. More steel fiber use reduces permeability, but this turns into the disadvantage that the fiber reinforced concrete undergoes more inelastic (unrecoverable) deformation (Rapoport *et al.*, 2001). Steel fibers may cause surface corrosion in reinforced concrete due to the lack of concrete cover, but this type of corrosion remains at the surface and it causes aesthetic problem. Kim and his friends studied the effect fiber types on steel reinforcement corrosion and they found out that the addition of steel fibers having the greater resistance to mass transport of aggressive materials resulted in the earliest corrosion initiation time, but the localized corrosion of steel fibers as a result of microcracking on the concrete surface because of spalling failure seems to accelerate chloride ions movement near the steel reinforcement (Kim *et al.*, 2011).

2.4.5. Effect of Cyclic Loading on Corrosion of Steel in Concrete

Many structures are generally subjected to cyclic loadings during their service life, such as machine vibration, wind action, sea waves, and automobile traffic. This repeated loading gives rise to a decrease in the stiffness of structure, which may result in fatigue failure. Fatigue is a process of progressive, permanent internal structural changes in a material subjected to repetitive stresses. In concrete, these changes cause the progressive growth of internal microcracks and that cracks may influence the corrosion response of the reinforcing steel. Ahn and Reddy stated that the growth rate of cracks is considerably faster and more and longer longitudinal cracks occur under cyclic loading than under static loading, and therefore, concrete deteriorates more rapidly (Ahn and Reddy, 2001).

Fatigue loading can be divided into two categories: low-cycle and high-cycle

loading. Low-cycle loading includes the application of a few load cycles at high stress levels. High-cycle loading involves a large number of cycles at lower stress levels. Hsu presents the different classes of fatigue loading as shown in Table 2.3 (Hsu, 1981).

Table 2.3. Different classes of fatigue loading.

Low-cycle fatigue	Hihg-cycle fatigue		Super-high-cycle fatigue	
1 10 ¹ 10 ²	10 ³ 10 ⁴	10 ⁵ 10 ⁶	10 ⁷ 10 ⁸	10 ⁹
Stuctures subjected to earthquake	Airport pavements and bridges	High and railway bridges, highway pavements	Mass rapid transit structures	Sea structures

In addition to above parameters, environmental factors influence corrosion of steel in concrete.

2.4.6. Effect of Temperature on Corrosion of Steel in Concrete

Temperature changes have important effects on concrete resistivity. A temperature increase causes a decrease of resistivity since temperature influences on ion mobility, ion-ion and ion-solid interactions. In addition, corrosion rate is influenced by temperature changes. Uhlig and Laque found out that corrosion rate is increased by high temperature and high humidity (Uhlig and Laque, 1983). So and Millard also showed that the variation of corrosion rate is approximately a factor of 5 to 6 with a seasonal temperature range from -1 to 38°C. They also found out that the corrosion rate is increased by a factor of approximately 1.8 per 10°C within the daily and seasonal fluctuations (So and Millard, 2007).

2.4.7. Effect of Relative Humidity on Corrosion of Steel in Concrete

Moisture content in concrete is an important factor for both corrosion initiation and propagation period. It mainly influences the availability of oxygen (which is required to support corrosion) depending on whether concrete is dry or saturated. In dry condition, there is no moisture to support corrosion process whereas in saturated condition oxygen does not dissolve in water. Therefore, corrosion cannot occur when it is too dry and too wet (saturated). Corrosion activity becomes significant at relative

humidity (RH) values above 80%. Richardson stated that chloride induced corrosion is at a maximum if the RH is around 90 – 95% in the concrete (Richardson, 2002).

2.4.8. Effect of Cyclic Wetting and Drying on Corrosion of Steel in Concrete

Cyclic wetting and drying increases the concentrations of ions such as chlorides, by evaporation of water. The drying of the concrete increases the availability of oxygen required for steel corrosion, since oxygen has a lower diffusion coefficient in saturated concrete. As the concrete dries and the pores becomes less saturated, oxygen will diffuse into the concrete and reach the level necessary to induce and sustain corrosion. However, when the concrete is fully submerged, less chloride can enter the concrete because diffusion through the pore solution is the dominant penetration mechanism. Moreover, the ingress of chloride ions into concrete is mostly affected by the sequence of wetting and drying, and on the duration. Especially, drying conditions and the degree of dryness is very important. Neville stated that drying to a greater depth allows subsequent wettings to carry the chlorides deeper into the concrete, thus speeding up the penetration of chloride ions. If the concrete remains wet, some salts may penetrate into the concrete by diffusion. However, if the wetting period is short, absorption governs the entry of salt water into the concrete and further concentrated during the next drying cycle (Neville, 1996).

Different loading cycles produce different failure mechanism within the concrete. For low-cycle motion results in the gradual bursting and detachment of the concrete cover, which finally leaves the reinforcing steel bars as the sole load-bearing elements, thus forcing them to carry both shear forces and bending moments, causing permanent plastic deformation of steel. On the other hand, high-cycle motion produces bond cracks in reinforced concrete structures.

The mechanism of fatigue failure in concrete or mortar can be divided into three stages (Gao and Hsu, 1998). The first stage is termed as flaw initiation that occurs in the weak region of concrete. The second stage is known as microcracking and this stage includes the slow and progressive growth of the inherent flaws into a critical size.

In the last stage, a continuous or macrocrack will developed due to a sufficient number of unstable cracks, eventually causing failure.

In general, parameters such as the number of cycles, load frequency, loading condition, stress level, matrix composition, and stress ratio will affect the fatigue performance of the concrete specimen. Saito and Imai found out that plain concrete subjected to repeated uniaxial tensile stresses exhibits no fatigue limit under 2×10^6 cycles. Therefore, there is no stress level below which the fatigue life of the plain concrete will be infinite. Fatigue failure under bending or tension load will depend on not only on the maximum load level, but also on the minimum load level. The effect of minimum load on the bending fatigue life of concrete describes the influence of minimum crack width on the rate of crack bridging degradation (Saito and Imai, 1983).

The effect of cyclic loading on corrosion of reinforcing steel may be greater than the effect of static loading, one possible explanation of this may be further microcracks occur during cyclic loading and may be greater degradation of concrete-steel interface. Ahn and Reddy studied the effect of cyclic loading on corrosion of reinforced concrete and they concluded that some specimens under fatigue loading had more positive half-cell potentials than those under static loading for the first month of the exposure, but with further exposure, the values for those under fatigue loading became more negative until the end of the exposure period (90th day of exposure) (Ahn and Reddy, 2001). Jaffer and Hansson studied the chloride-induced corrosion products of steel in cracked concrete subjected to different loading conditions and they found out that more corrosion products were observed at the aggregate-paste interface in dynamically loaded specimens than in statically loaded specimens (Jaffer and Hansson, 2009).

In this study, these factors, such as water/cement ratio, concrete cover depth, crack width variation, loading conditions (static and dynamic) and adding of steel fiber into reinforced concrete has taken into consideration so as to interpret the corrosion of reinforcing steel accurately by using galvanostatic pulse technique.

3. EXPERIMENTAL STUDIES

3.1. Materials and Mix Design

3.1.1. Materials

3.1.1.1. Cement. The cement used for concrete specimens was CEM I 42.5 R Portland cement, which was produced by Akçansa. Physical, mechanical and chemical properties of this cement are as following.

Table 3.1. Physical properties of cement.

Physical Properties	
Specific Gravity	3.14 (g/cm^3)
Initial Setting Time	111 (min.)
Final Setting Time	168 (min.)
Le Chatelier	1 (mm)
Specific Surface	3910 (cm^2/g)
Residue on 45 μ m sieve	14.4 (%)
Residue on 90 μ m sieve	2.2 (%)

Table 3.2. Mechanical properties of cement.

Mechanical Properties		
Mechanical properties/day	Standards	Test results
Early Strength (2 day)	≥ 20 Mpa	25.1
Early Strength (7 day)	-	39.5
Standard Strength (28 day)	≥ 42.5 Mpa ≤ 62.5 Mpa	53.7

Table 3.3. Chemical properties of cement.

Chemical Properties	
SiO_2	20.17 (%)
Insoluble Residue	0.69 (%)
Al_2O_3	4.91 (%)
Fe_2O_3	3.41 (%)
CaO	64.28 (%)
MgO	1.18 (%)
SO_3	2.84 (%)
Loss on Ignition	1.61 (%)
Cl^-	0.0371 (%)
$\text{Na}_2\text{O}/\text{K}_2\text{O}$	0.13 - 0.96 (%)
Unknown	0.47 (%)
S.CaO – Free Lime	1.65 (%)
Mineralogical Composition	55.65 C_3S 15.93 C_2S 7.25 C_3A 10.38 C_4AF
LSF	0.97

3.1.1.2. Aggregates. Four types of aggregates were used: Sand, Crushed Sand, Aggregate No I, and Aggregate No II. The specific gravity of these aggregates were 2.62, 2.64, 2.71, 2.74 g/cm³, respectively and the maximum grain size of the aggregates was less than or equal to 20 mm.

3.1.1.3. Reinforcing Steel. $\Phi 14$ ribbed bars with a characteristic yield strength of 420 MPa were used for the longitudinal reinforcement.

3.1.1.4. Steel Fibre. Dramix RC-65/35-BN steel fibre was added to concrete mix. This type, is a cold drawn wire fibre, with hooked ends, has a length of 35 mm, and

diameter of 55 mm. Aspect ratio of fibers is 65 as is seen in the code. The characteristic properties of the steel fibre are given in Table 3.4.

Table 3.4. Characteristic properties of RC-65/35-BN fiber.

Fibre Type	Length (mm)	Diameter (mm)	Aspect Ratio (L/D)	Min. Tensile Strength (MPa)	Fibers/ kg
RC-65/35-BN	35	0.55	65	1100	14500

3.1.1.5. Superplasticizer. A new generation of superplasticizer, called Rheobuild 1000, was used in the experiments. It is a strength enhancing, superplasticising, high range, water reducing admixture that contains sulphonated polymer.

3.1.2. Mix Design

Concrete mix was prepared in the Bogazici University Engineering Materials Laboratory. Two water-to-cement ratio (0.45-0.65), two different cover depths (25-45), and reinforced concrete with and without steel fiber were prepared in the laboratory so as to investigate corrosion activity on the reinforcing steel. To sum up, 4 types of mixtures were prepared and 8 different series were obtained as is seen in Table 3.5. All series were named accordingly. *L0*, representing reinforced concrete without steel fiber, *L1*, representing reinforced concrete with steel fiber, 65 and 45, representing the water/cement ratios and the last values 45 and 25, representing the cover depth. For example, *L1/65/45* represents reinforced concrete with steel fiber with a water-to-cement ratio of 0.65 and a cover depth of 45 mm as shown in Figure 3-1.

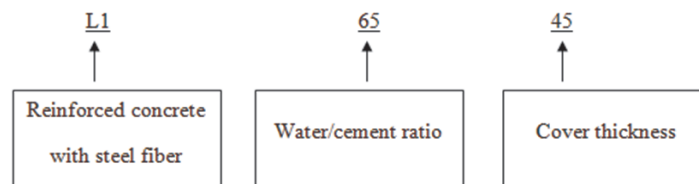


Figure 3.1. Representation of specimen codes.

The mix design values for the four types of concrete (*L0/65-L1/65-L0/45-L1/45*)

are shown in Table 3.5.

Table 3.5. The mix proportions used in the experiment (S* stands for superplasticizer).

Series	W/C	Cement kg/m ³	Water kg/m ³	Sand kg/m ³	Crushed Sand kg/m ³	Aggregate No I kg/m ³	Aggregate No II kg/m ³	S* kg/m ³	Steel Fiber kg/m ³
L0/65/25	0.65	310	201	560	237	581	473	2,22	0
L0/65/45	0.65	310	201	560	237	581	473	2,22	0
L0/45/25	0.45	400	180	554	235	575	468	6,3	0
L0/45/45	0.45	400	180	554	235	575	468	6,3	0
L1/65/25	0.65	310	201	560	237	581	473	3,48	39
L1/65/45	0.65	310	201	560	237	581	473	3,48	39
L1/45/25	0.45	400	180	554	235	575	468	6,85	39
L1/45/45	0.45	400	180	554	235	575	468	6,85	39

3.2. Casting and Curing of Specimens

3.2.1. Casting Procedure

Concrete mixtures were produced in a laboratory pan mixer, which has a maximum capacity of 0.04 m³. The workability of fresh concrete was adjusted so that fresh concrete showed 17 ± 2 cm slump by using the required amount of superplasticizer. It was observed during trial mixing that the fresh concrete behavior was clearly affected by the mixing sequence and timing. Therefore, a specific mixing procedure was adopted:

- All concrete mixtures are prepared in 40 L batches in the laboratory pan mixer.
- First, aggregates and cement was dry mixed for 2 minutes.
- Then, all water and half of the superplasticizer mixed in a water bucket and this mixture was poured to the mixer in 1 minute. At the same time fibers (if it was a fiber reinforced mix) were added and the mixture was mixed for another minute.
- In the end, rest of the superplasticizer was added in 30 seconds and then mixed for another 1.5 minutes. After 6 minutes, the concrete was ready for casting.

3.2.2. Specimens

Beam specimens and cylinder specimens were produced for each series. Table 3.6 represents the dimensions and number of specimens cast for various stages of the research.

Table 3.6. The dimensions and number of specimens.

Series	Beam specimens (100 * 100 * 500mm)			Cylinder specimens (100*200mm)
	Corrosion activity measurement			Compressive strength measurement
	Control (uncracked specimens)	Specimens cracked under static loading	Specimens cracked under dynamic loading	
L0/45/25	3	3	3	3
L0/45/45	3	3	3	
L0/65/25	3	3	3	3
L0/65/45	3	3	3	
L1/45/25	3	3	3	3
L1/45/45	3	3	3	
L1/65/25	3	3	3	3
L1/65/45	3	3	3	

100x100x500 mm prismatic beams were cast to investigate corrosion activity on the reinforcing steel. Each prismatic beam has a $\Phi 14$ mm longitudinal reinforcing rebar, which was placed at 25 mm or 45 mm from the bottom surface of the formwork to form two different cover depths as mentioned before. Figure 3.2 shows positioning of the reinforcements in the moulds before casting of concrete.



Figure 3.2. Replacement of the steel rebar into the steel mold.

The specimens were cast in two layers and compacted by the aid of vibration table, a steel trowel was used for finishing. Usually vibration has been applied until concrete surface becomes relatively smooth as shown in Figure 3.3.



Figure 3.3. The preparation of specimens.

The specimens are covered by using nylon sheets and kept in laboratory environment for 24 hours. After the removal of the moulds, they were labeled and steel reinforcement outside the concrete was covered with electrical band to prevent corrosion of the tips of the bars. Then specimens were stored in a humidity room at a temperature of 20°C and a relative humidity of 95% for 28 days as shown in Figure 3.4.



Figure 3.4. Storage of the specimens in the laboratory.

Three cylinder specimens with a diameter of 100 mm and a height of 200 mm were also cast for each series to determine compressive strength of the specimens and they were kept in a water tank at $20 \pm 2^\circ\text{C}$ for 28 days until the day of testing.

3.3. Tests and Measurements for Fresh Concrete

3.3.1. Slump Test

The slump test was performed on fresh concrete immediately after production according to ASTM C 143 (Standard Test Method for Slump of Hydraulic Cement Concrete). A slump cone was filled with concrete in three layers. Each layer was approximately one third the volume of the cone and was hit 25 strokes by a tamping rod. As soon as the cone filled with concrete, it was removed from the concrete by raising it in vertical direction. The slump was immediately measured by determining the vertical distance between the top of the cone and the displaced original center of the top surface of the concrete as shown in Figure 3.5.



Figure 3.5. Determination of the slump on fresh concrete.

3.3.2. Air Content and Density Test on Fresh Concrete

The density of fresh concrete was calculated by dividing the net mass of concrete by the volume of the measure. The air content of freshly mixed concrete was also determined in accordance with TS EN 12350-7 as shown in Figure 3.6.



Figure 3.6. Determination of air content on fresh concrete.

3.4. Tests for Determination of Mechanical Properties of Hardened Concretes

3.4.1. Compressive Strength

The compressive strength of the 100x200 mm cylinder specimens were measured in accordance with ASTM C 39 (Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens) at the age of 28 days. The specimens were capped using a capping compound to ensure parallel loading faces of the test specimens and constant height for all cylinders when tested. Capping of cylindrical concrete specimens was performed as recommended by ASTM C 617. The compressive strength of cylindrical concrete specimens was calculated by dividing the maximum load attained during the test by the cross sectional area of the specimen. This test method consisted of applying a compressive axial load to 100x200 mm cylinders at a rate of 2.4 kN/sec until failure occurred. The concrete on the compression test machine is shown in Figure 3.7.



Figure 3.7. Compression test machine.

3.4.2. Failure Strength

As mentioned in the introduction part, the main objective of this study is to investigate corrosion activity for cracked and uncracked concrete specimens by using beam specimens as shown in Table 3.7. Uncracked specimens (3 specimens for each series) are cast and cured for 28 days until the beginning of corrosion activity testing.

Table 3.7. Beam specimens for corrosion tests.

Beam specimens (100*100*500mm)		
Corrosion activity measurement		
Control (uncracked specimens)	Specimens cracked under static loading	Specimens cracked under dynamic loading

To see the corrosion activity on cracked specimens, cracks are formed on the specimens under static and dynamic loading. For this purpose an MTS Closed-Loop Displacement Controlled Dynamic Testing Machine with a 100 kN capacity was used. Details of the tests are given below.

3.4.3. Forming Cracks under Static Loading

Three 100x100x500 mm specimens for each series were statically loaded by using MTS loading machine. Loading was applied in displacement control at a rate of 0.02 mm/sec. Displacement of stroke was used for vertical displacement of specimens. For measuring crack opening displacement (COD), 2 LVDTs (Linear variable displacement transducer) were placed horizontally on the sides of specimens and average of these 2 values was used to draw Load vs. COD curves, so as to get more information about crack behavior of different types of beams. Loading set-up is given in Figure 3.8.

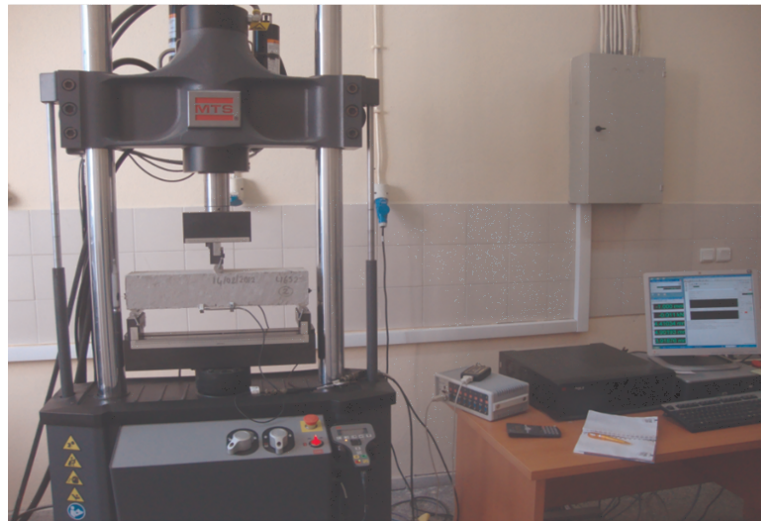


Figure 3.8. Three point bending set-up.

As mentioned above specimens with 2 different concrete cover thickness were produced (25 and 45 mm). Previous research shows that both crack width and concrete cover thickness are important parameters and affect the extent of corrosion damage that the concrete material undergoes (Gowripalan *et al.*, 2000). Gowripalan and his colleagues suggested using a parameter (crack width/cover thickness) to consider the effects of both crack width and cover thickness. A similar approach is adopted in the scope of this study to create a similar damage for the specimens with different cover thicknesses. First, a constant crack width/cover depth ratio was selected based on the definition given in ACI 318M-08. ACI 318 implies 0.4 mm as the critical crack width for corrosion. Crack width - cover depth ratio is found to be 0.0089 for the specimens

with a cover depth of 45 mm when critical crack width is considered to be 0.4 (0.4 mm/45 mm=0.0089). Based on this value (0.0089), critical crack width was calculated to be 0.22 mm for the specimens with a cover depth of 25 mm (0.0089*25 mm=0.22 mm). After, specimens were loaded as explained above to form required crack widths. Following the removal of load, crack width decreased as expected for all the specimens. Final crack width values were read from LVDTs and given in section 4.2.

3.4.4. Dynamic Loading Test

A short summary of dynamic loading procedure is given below (Figure 3.9). During the dynamic loading, the frequency was selected to be 3 Hz and 20000 cycles applied.

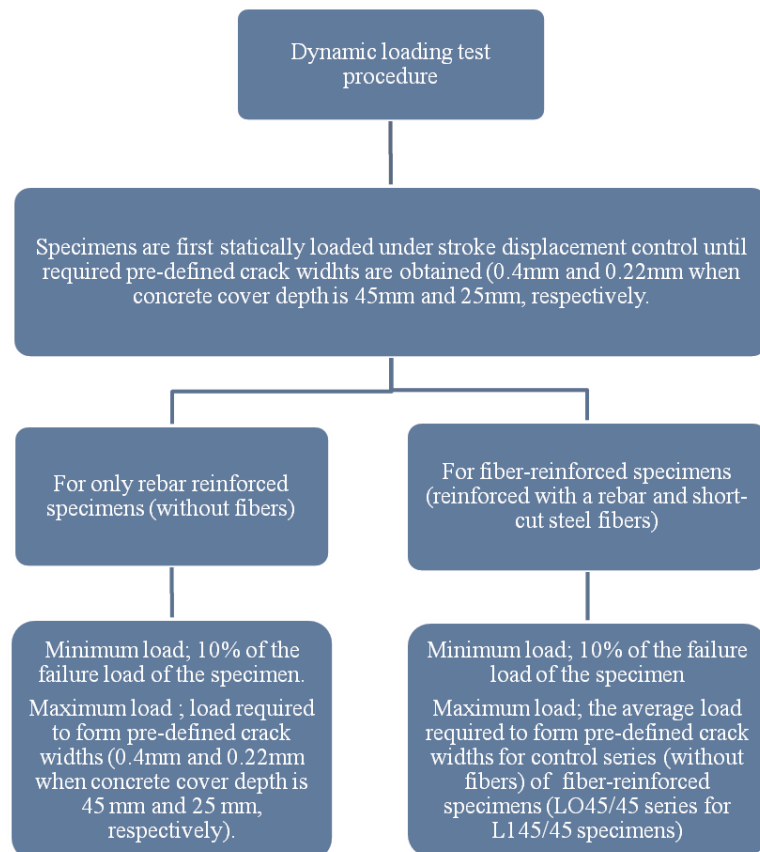


Figure 3.9. Dynamic loading test procedure.

Three 100x100x500 mm specimens for each series were dynamically loaded by

using MTS loading machine. First, the specimens were statically loaded until above mentioned crack widths are obtained (0.4 mm and 0.22 mm when concrete cover depth is 45 mm and 25 mm, respectively). Then dynamic load is applied to the specimens. Minimum and maximum limits for dynamic loading are selected as follows. For reinforced concrete specimens (reinforced with a rebar - without fibers), minimum load is 10% of the failure load of the specimen and maximum load is selected to be the load required to form pre-defined crack widths (0.4 mm and 0.22 mm when concrete cover depth is 45 mm and 25 mm, respectively). For fiber-reinforced specimens (reinforced with a rebar and short cut steel fibers), minimum load is 10% of the failure load of the specimen, on the other hand maximum load is selected as the average load required to form pre-defined crack widths (0.4 mm and 0.22 mm when concrete cover depth is 45 mm and 25 mm, respectively) for control series (without fibers) of the same specimens (for example; Average of the loads required to create 0.40 mm cracks for L045/45 specimens is used for L145/45 specimens).

3.5. Corrosion Tests

Following specimens are obtained as a result of the casting and loading procedures explained above; (i) control specimens with and without fibers (with no cracks), (ii) statically loaded specimens with and without fibers loaded under static stress (cracks are formed as a result of loading), (iii) dynamically loaded specimens with and without fibers (cracks are formed as a result of loading). These 3 different types of specimens are then exposed to wetting drying cycles and corrosion measurements are carried out on the specimens throughout the process. The details are given below.

3.5.1. Wetting and Drying Cycles

Wetting and drying cycles are applied by using 0.6M NaCl solution on each specimen. Small reservoirs with dimensions of 70x130x15 mm are formed by using a fast setting mortar on the concrete surface. In order to prevent any water leakage, the inner surface of the mortar was coated with antibacterial silicon as shown in Figure 3.10. This method was applied for all beams.



Figure 3.10. Reservoir on concrete surface.

Following preparation of all beams, the experiment started with a wetting cycle. On Monday mornings, all reservoirs were filled with 35 g/l NaCl solution. On Friday mornings, this solution was taken from the reservoirs with the aid of sponges and syringes. After 4 hour passed, the corrosion measurements were taken by both a Galvapulse machine and a copper/copper reference electrode. In the drying period, specimens were also kept in the laboratory conditions and this period last for 3 days from Friday morning to Monday morning. Therefore, specimens were kept 4 days under wetting period and 3 days under drying period. As a result, a cycle is completed within a week. Specimens are always kept under ambient temperature conditions in the lab. However, since durability experiments are long term experiments, temperature and humidity changes are expected (temperature and humidity of the room is noted every week during measurements to control and evaluate the results of the ambient temperature change Table 3.8 gives variation of temperature and humidity). This effect is seen when 0.6M NaCl solution evaporated before the end of wetting cycle. This effect was not seen during the first weeks). To prevent/delay evaporation of 0.6M NaCl solution, specimens were covered with nylons and as shown in Figure 3.11.

Table 3.8. Variation of Temperature and Humidity.

Weeks	6	7	8	9	10	11	12	13	14
T ($^{\circ}$ C)	24.6	26.6	28.2	28.4	29.1	25.9	27.2	25.6	25
RH(%)	67	60.4	66	66.7	66	71	68	64	81



Figure 3.11. Specimens in the laboratory environment.

3.6. Corrosion Tests

Corrosion tests are carried out by using two different corrosion measurement methods, half-cell potential and galvanostatic pulse technique. The details of the measurements are given below.

3.6.1. Half-Cell Potential Measurement

This test is undertaken to measure the possible active areas of the reinforcing steel. These measurements are taken with a copper/copper reference electrode in accordance with ASTM C 876 standard. Figure 3.12 shows half-cell measurement equipment with a copper/copper reference electrode.

Half-cell potential measurements were taken every week on Friday as mentioned above. The principle of half-cell potential measurement can be summarized as follows. The steel reinforcement in the specimen acts as an electrode, and the pore solution of the concrete acts as the electrolyte. A high impedance voltmeter is attached to the anode bar and the probe which rests on a moistened sponge completes the electric circuit as shown in Figure 3.12. Corrosion potentials are measured as a result and evaluated by using ASTM C 876.



Figure 3.12. Half-Cell potential measurement.

3.6.2. Galvanostatic Pulse Technique (Galvapulse)

Galvanostatic pulse method is a rapid non-destructive polarization technique which has been used for evaluation of steel reinforcement corrosion. A short time anodic current pulse is impressed galvanostatically to reinforcement from a counter electrode placed on concrete surface together with a reference electrode. Equipment with a trademark name of Galvapulse produced by Germann Instruments is used for this study to obtain measurement. For the tests under consideration, the applied current was $25 \mu\text{A}$ and the pulse duration was five seconds. By using this technique, corrosion potential, corrosion rate and the resistance of the concrete specimens were measured. These measurements were taken every week on Friday following the end of wetting period. All measurements were performed on specimen as shown in Figure 3.13. As mentioned before, specimens are left for drying for 4 hours after removing water from the reservoirs. Before the measurements the surface and the contact sponge were wetted. The sponge was cleaned (squeezed in fresh tap water) after measurements (maximum 9 readings). Potential values were measured with an Ag/AgCl electrode (inside the measurement head).



Figure 3.13. Testing with the Galvapulse device.

4. RESULTS AND DISCUSSION

4.1. Overview

The results from the experimental testing will be presented in the following sections. The results of the experimental program are separated into three sections. The first section offers the results of the tests on the fresh concrete and compressive strength of hardened concrete. The second section provides the results of the flexural test on hardened concrete. The remaining section provides corrosion results from control specimens, specimens cracked under static load and specimens cracked under dynamic load, respectively.

4.2. Material Properties

Table 4.1 represents the fresh concrete properties. Apart from the fiber addition, all other concrete components are of the same type and the casting and compacting methods are the same for fiber reinforced concrete and control concrete. The target range for the slump was 17 ± 2 cm and the obtained results were in the desired range.

Table 4.1. Fresh concrete properties.

TYPE	W/C	COVER DEPTH (mm)	Batch 1		Batch 2		AIR VOID (%)
			Slump (cm)	Density (g/cm ³)	Slump (cm)	Density (g/cm ³)	
L0/65/25	65	25	16,5	2.349	17.5	2.370	3.70%
L0/65/45	65	45	17	2.394	18	2.365	3.70%
L0/45/25	45	25	17	2.389	18.5	2.408	3.90%
L0/45/45	45	45	18,5	2.395	19	2.400	3.90%
L1/65/25	65	25	18,5	2.380	19	2.376	5.00%
L1/65/45	65	45	17	2.365	17	2.374	5.00%
L1/45/25	45	25	19	2.379	19	2.448	4.80%
L1/45/45	45	45	18	2.379	18.5	2.393	4.80%

The 28 day compressive strength test were performed on three companion cylinder specimens with dimensions of 100x200 mm of each type and the results were given in Table 4.2.

Table 4.2. Compressive strengths of hardened concretes.

TYPE	W/C	LOAD (kN)	STRESS (MPa)	AVG STRESS (MPa)
L0/65	65	251.6	32.03	29.85
L0/65	65	232.8	29.64	
L0/65	65	218.9	27.87	
L1/65	65	219.6	27.96	28.12
L1/65	65	232.9	29.66	
L1/65	65	209.9	26.73	
L0/45	45	425.2	54.14	49.05
L0/45	45	379.9	48.37	
L0/45	45	350.7	44.65	
L1/45	45	337.6	42.98	47.92
L1/45	45	374.8	47.72	
L1/45	45	416.7	53.05	

4.3. Failure Capacity of Beams

Flexural tests were carried out on beam specimens with dimensions of 100x100x500 mm under three point loading by using MTS loading machine and the results were given in Table 4.3 and Figure 4.1. Test procedure was given in Section 3.4.3. Three beams were tested until failure for each mixture, and maximum load and crack opening displacement were recorded. Finally, the average values of these results were calculated and they were compared to each other. According to these results both cover depth and water-to-cement ratio affects the failure capacity of beams. In addition to this, the positive contribution of steel fibers on the failure capacity of the specimens can be

seen.

Table 4.3. Failure capacity of specimens.

Beam Specimens (100 * 100 * 500mm)								
TYPE	Specimen 1		Specimen 2		Specimen 3		Average Values	
	LOAD (kN)	COD (mm)	LOAD (kN)	COD (mm)	LOAD (kN)	COD (mm)	AVG LOAD (kN)	AVG COD (kN)
L0/45/25	30.00	0.50	29.2	0.50	30.75	0.44	29.98	0.48
L1/45/25	32.00	0.45	34.05	0.52	29.4	0.37	31.82	0.45
L0/65/25	26.00	1.00	25.32	0.66	24.88	0.57	25.40	0.74
L1/65/25	27.00	0.25	28.34	1.24	27.97	0.37	27.77	0.62
L0/45/45	28.00	3.50	26.08	3.26	25.88	2.67	26.65	3.14
L1/45/45	26.50	1.40	30.15	2.76	30.31	2.05	28.99	2.07
L0/65/45	21.00	3.30	23.24	2.99	21.53	2.88	21.92	3.06
L1/65/45	22.00	3.20	23.33	1.91	21.38	2.61	22.24	2.57

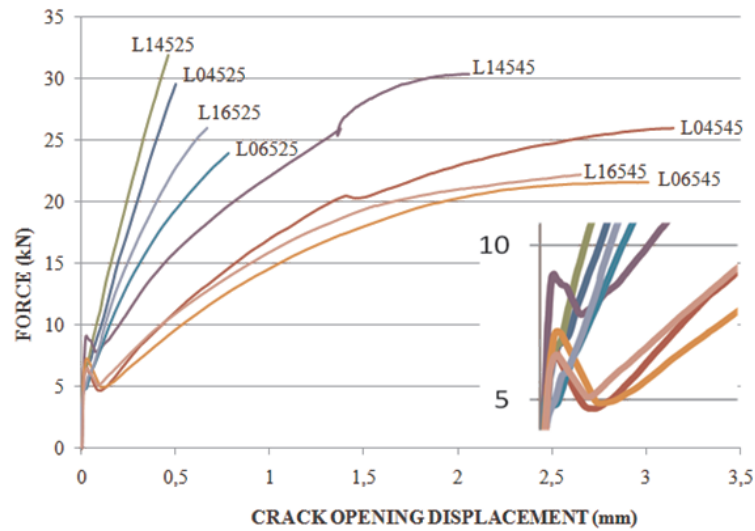


Figure 4.1. Three point bending test results.

The effect of water-to-cement ratio can be seen in Figure 4.1. As water-to-cement ratio increases, the flexural capacity of beams decreases. On the other hand,

as water-to-cement ratio increases, the crack opening displacement also increases (except L0/45/45). The impact of steel fibers can be also seen in Figure 4-1. The use of steel fibers increases the flexural capacity and decreases crack opening displacement of the beams as expected. Apart from this, the effect of steel fibers seems to increase as cover depth increases for the specimens with a water-to-cement ratio of 0.45. It is also observed that the crack opening displacements decreased more when the cover depth was 45 mm. For example, COD for L1/45/45 specimen decreased 34% when it is compared to COD for L0/45/45 specimen. Similarly, COD for L1/65/45 specimen decreased 16% when it is compared to COD for L0/65/45. On the other hand, COD for L1/65/25 declined 16% when it is compared to COD for L0/65/25. Likewise, COD for L1/45/25 declined 6% when it is compared to COD for L0/45/25. That is, average COD for specimens with 45 mm cover depths decreased 25%, whereas average COD for specimen with 25 mm cover depth decreased 11%.

4.4. Results for the Beams Cracked under Static Load

Beams were loaded statically under three point loading by using MTS loading machine and test results were given in Table 4.4. Three beams were tested until the required crack opening displacement was reached for each mixture. Crack width values had been selected 0.4 mm and 0.22 mm for specimens that have 45 mm and 25 mm cover depth, respectively, so that crack width-cover depth ratio is equal to 0.0089 for all specimens (detailed explanation was given in Section 3.4.3). Following the removal of load, crack width decreased as expected for all the specimens. Final crack width values were read from LVDTs and given in Table 4.4.

According to the results, cracks were decreased from 0.22 mm to approximately 0.08 mm on average for beams that have 25 mm cover depth, whereas cracks were decreased from 0.4 to approximately 0.18 mm on average for beams that have 45 mm cover depth after load removal. Decrease in crack width following the removal of load was found to be 10% less when the cover depth was 45 mm. It is hypothesized that as crack width increases, plastic deformation also increases, therefore, cracks of the specimens with 45 mm cover depth were closed approximately 10% less than the

cracks of the specimens with 25 mm cover depth.

As shown in Table 4.4, when crack widths were kept constant (0.22 and 0.4 mm), the required loads needed for cracking of beams were more for the fiber reinforced concrete specimens. Moreover, this required load decreases, as water-to-cement ratio and cover depth increases.

Table 4.4. Static loading of specimens.

Beam Specimens (100 * 100 * 500mm)											
	Specimen 1			Specimen 2			Specimen 3			Average Values	
MIXTURE	LOAD (kN)	COD (mm)	AFTER LOAD REMOVAL (mm)	LOAD (kN)	COD (mm)	AFTER LOAD REMOVAL (mm)	LOAD (kN)	COD (mm)	AFTER LOAD REMOVAL (mm)	AVG LOAD (kN)	AVG COD AFTER LOAD REMOVAL (mm)
L0/45/25	18.3	0.22	0.067	16	0.22	0.08	17.2	0.22	0.105	17.2	0.084
L1/45/25	20.5	0.22	0.082	21	0.22	0.073	15.5	0.22	0.09	18.9	0.082
L0/65/25	15.0	0.22	0.089	13.1	0.22	0.08	11.6	0.22	0.075	13.3	0.081
L1/65/25	14.3	0.22	0.085	16.7	0.22	0.083	17.5	0.22	0.07	16.2	0.079
L0/45/45	9.36	0.4	0.17	9.5	0.4	0.173	10.6	0.4	0.174	9.8	0.172
L1/45/45	12.5	0.4	0.18	12.1	0.4	0.156	12.8	0.4	0.168	12.5	0.168
L0/65/45	8.4	0.4	0.18	8.7	0.4	0.178	8.7	0.4	0.168	8.6	0.175
L1/65/45	10.2	0.4	0.192	9	0.4	0.214	9.6	0.4	0.186	9.6	0.197

In Figure 4.2, the reinforced beams with and without steel fibers were compared. As expected, specimens with 25 mm cover depth bear more load as moment capacity increases, whereas specimens with 45 mm cover depth cracks more, yielding ductile behavior, which is a significant parameter for earthquake regions.

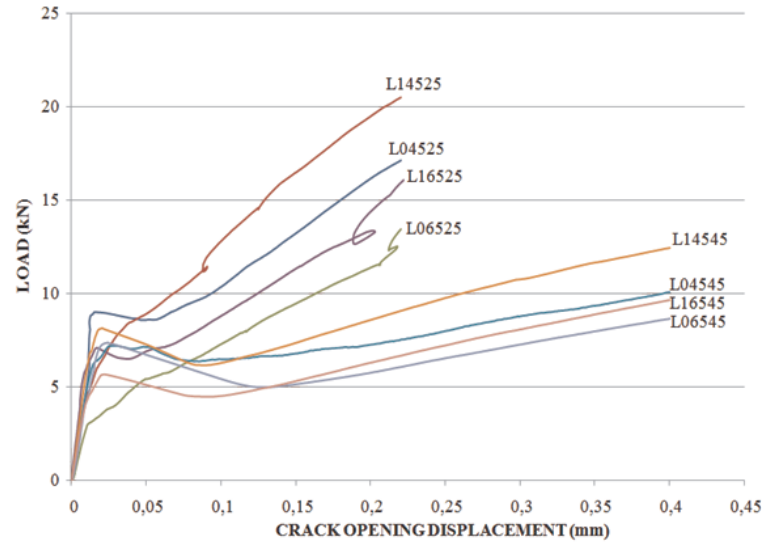


Figure 4.2. Static loading results.

4.5. Results for the Beams Cracked under Dynamic Load

Beams were loaded statically at first and then they were loaded dynamically under three point loading by using MTS loading machine. Test results are given in Table 4.5, Table 4.6 and Figure 4.3 to Figure 4.10. Loading procedure was explained in the experimental procedure part (Section 3.4.4). According to the results, dynamic loading caused greater crack opening displacement. COD values were found to increase with an increase in cover depth, as expected. Steel fiber addition also influences the dynamic loading results, as expected. For fiber reinforced specimens with a water-to-cement ratio of 0.45, COD values were found to be approximately 2 times less than COD for normal reinforced concrete.

Table 4.5. Dynamic loading specimens (cover depth= 25 mm).

Beam Specimens (100 * 100 * 500mm)							
MIXTURE	STATIC LOADING			DYNAMIC LOADING			
	LOAD (kN)	COD (mm)	AFTER STATIC LOAD REMOVAL (mm)	MAXIMUM COD (mm) DURING DYNAMIC	AFTER DYNAMIC LOAD REMOVAL CYCLE (mm)	MIN LOAD (kN)	MAX LOAD (kN)
L0/45/25-1	16,32	0,22	0,074	0,286	0,117	3,00	16,32
L0/45/25-2	14,00	0,22	0,084	0,303	0,133	3,00	14,00
L0/45/25-3	13,90	0,22	0,075	0,310	0,150	3,00	13,90
AVERAGE	14,74	0,22	0,078	0,299	0,133	3,00	14,74
L1/45/25-1	19,05	0,22	0,090	0,216	0,114	3,18	14,74
L1/45/25-2	23,48	0,22	0,065	0,172	0,078	3,18	14,74
L1/45/25-3	20,43	0,22	0,084	0,206	0,112	3,18	14,74
AVERAGE	20,99	0,22	0,080	0,198	0,101	3,18	14,74
L0/65/25-1	14,00	0,22	0,078	0,269	0,115	2,54	14,00
L0/65/25-2	14,72	0,22	0,087	0,319	0,152	2,54	14,72
L0/65/25-3	13,70	0,22	0,084	0,303	0,133	2,54	13,70
AVERAGE	14,14	0,22	0,083	0,297	0,133	2,54	14,14
L1/65/25-1	14,53	0,22	0,089	0,291	0,143	2,78	14,14
L1/65/25-2	13,49	0,22	0,087	0,314	0,147	2,78	14,14
L1/65/25-3	13,91	0,22	0,099	0,380	0,193	2,78	14,14
AVERAGE	13,98	0,22	0,092	0,328	0,161	2,78	14,14

Table 4.6. Dynamic loading specimens (cover depth= 45 mm).

Beam Specimens (100 * 100 * 500mm)							
MIXTURE	STATIC LOADING			DYNAMIC LOADING			
	LOAD (kN)	COD (mm)	AFTER STATIC LOAD REMOVAL (mm)	MAXIMUM COD (mm) DURING DYNAMIC	AFTER DYNAMIC LOAD REMOVAL CYCLE (mm)	MIN LOAD (kN)	MAX LOAD (kN)
L0/45/45-1	10,30	0,40	0,182	0,541	0,281	2,67	10,30
L0/45/45-2	9,38	0,40	0,182	0,518	0,278	2,67	9,38
L0/45/45-3	10,43	0,40	0,151	0,527	0,254	2,67	10,43
AVERAGE	10,04	0,40	0,171	0,529	0,271	2,67	10,04
L1/45/45-1	16,32	0,40	0,155	0,336	0,185	2,90	10,04
L1/45/45-2	12,95	0,40	0,172	0,405	0,225	2,90	10,04
L1/45/45-3	12,38	0,40	0,155	0,424	0,211	2,90	10,04
AVERAGE	13,88	0,40	0,161	0,388	0,207	2,90	10,04
L0/65/45-1	9,45	0,40	0,161	0,542	0,269	2,19	9,45
L0/65/45-2	8,71	0,40	0,170	0,540	0,279	2,19	8,71
L0/65/45-3	9,00	0,40	0,168	0,546	0,276	2,19	9,00
AVERAGE	9,05	0,40	0,166	0,543	0,275	2,19	9,05
L1/65/45-1	12,93	0,40	0,176	0,383	0,220	2,22	9,05
L1/65/45-2	10,81	0,40	0,179	0,458	0,248	2,22	9,05
L1/65/45-3	9,10	0,40	0,179	0,567	0,307	2,22	9,05
AVERAGE	10,95	0,40	0,178	0,469	0,259	2,22	9,05

Figures from Figure 4.3 to Figure 4.10 represent the number of cycles vs. crack opening displacement relations for different specimens. It should be noted that the graphs do not include data for 20000 cycles. To make data more visible, maximum and minimum of 10 COD values were used for every 500th cycle before 5000th cycle and maximum and minimum of 15 COD values were used for every 1000th cycle after 5000th cycle. Meaning, 10 COD data starting from cycle 0 to 5000 with an increment of 500 and 15 COD data starting from cycle 5000 to 20000 were used with an increment of 1000.

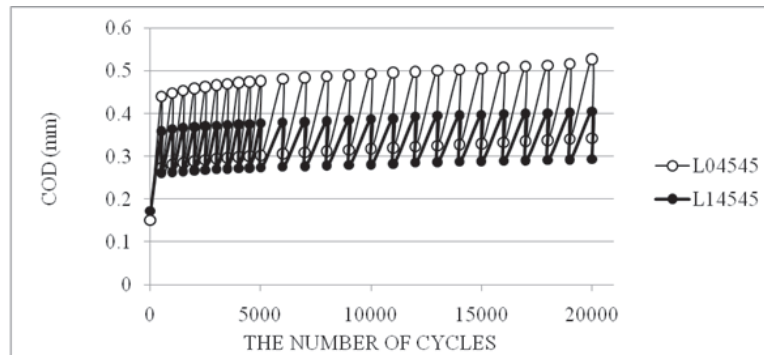


Figure 4.3. Dynamic loading results (cover depth= 45 mm).

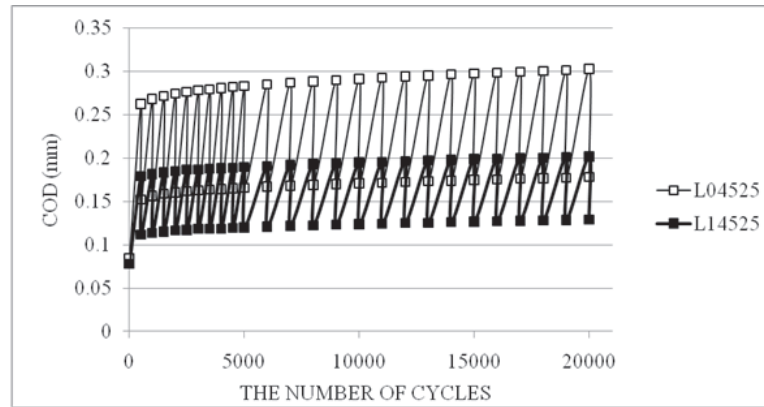


Figure 4.4. Dynamic loading results (cover depth= 25 mm).

Figure 4.3 and Figure 4.4 represent COD vs. number of cycles for specimens with a water-to-cement ratio of 0.45. According to the above results, effects of fibers can easily be seen on the crack behavior of specimens both with a cover depth of 45 mm and 25 mm. Crack opening for L1/45/45 was found to be 27% less than L0/45/45 during the dynamic cycle. Likewise, crack opening for L1/45/25 was found

to be 34% less than L0/45/25 owing to the contribution of fibers. At 20000th cycle, under the same load (it should be remembered that non-fiber reinforced and fiber reinforced specimens are tested under the same maximum loads) the COD increased 0.111 mm when load was varied from minimum to maximum for L1/45/45, whereas the COD variation for L0/45/45 was 0.185 mm. Similarly, under the same load the COD variation for L1/45/25 was 0.072 mm, while the COD variation for L0/45/25 was 0.124 mm. Another result is seen when the specimens with two different cover depths were compared. When cover depth is 25 mm, maximum crack opening is further restricted by rebar and maximum opening is limited to 0.3 mm, on the other hand for the same specimens crack opening can go up to 0.5 mm when the cover depth is equal to 45 mm.

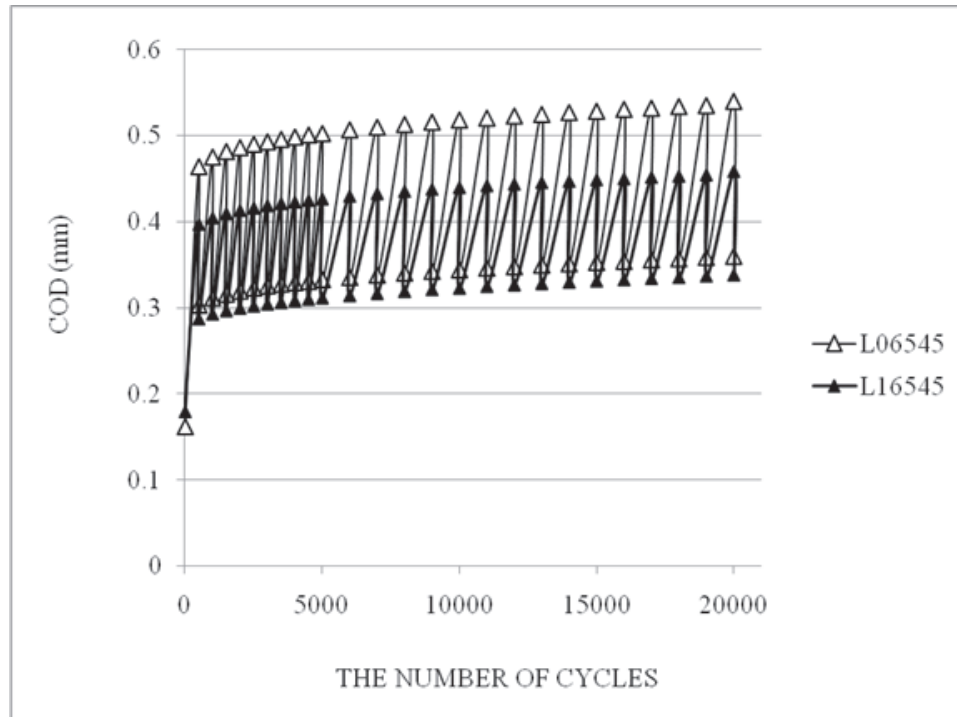


Figure 4.5. Dynamic loading results (cover depth= 45 mm).

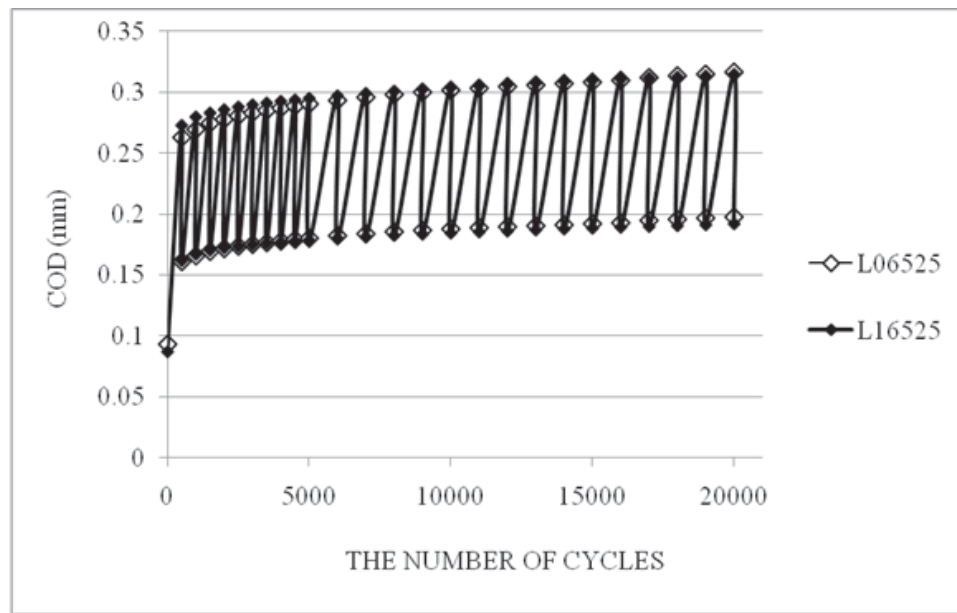


Figure 4.6. Dynamic loading results (cover depth= 25 mm).

Figure 4.5 and Figure 4.6 show COD vs. number of cycles for specimens with a water-to-cement ratio of 0.65. As can be seen from the figures, fiber effect is much less pronounced when water-to-cement ratio is increased. This result could be attributed to the fact that adherence between fiber and concrete matrix decreases when water-to-cement ratio increased and as a consequence fibers may slip out of the matrix before they reach to their maximum capacity. Crack opening for L1/65/45 was found to be 14% less than L0/65/45 during the dynamic cycle. On the other hand, crack opening for L1/65/25 was found to be 9% more than L0/65/25 during the dynamic cycle. COD for L1/65/25 was nearly the same as L0/65/25. At 20000th cycle, under the same load the COD increased 0.120 mm when load was varied from minimum to maximum for L1/65/45, whereas the COD variation for L0/65/45 was 0.180 mm. However, under the same load the COD variation for L1/65/25 was 0.122 mm, while the COD variation for L0/65/25 was 0.12 mm. An affect similar to the one mentioned above is seen when the effect of cover depth is examined. Maximum crack opening can go up to 0.5 mm when cover depth is 45 mm, while it is limited to 0.3 mm due to restriction created by rebar when the cover depth is 25 mm.

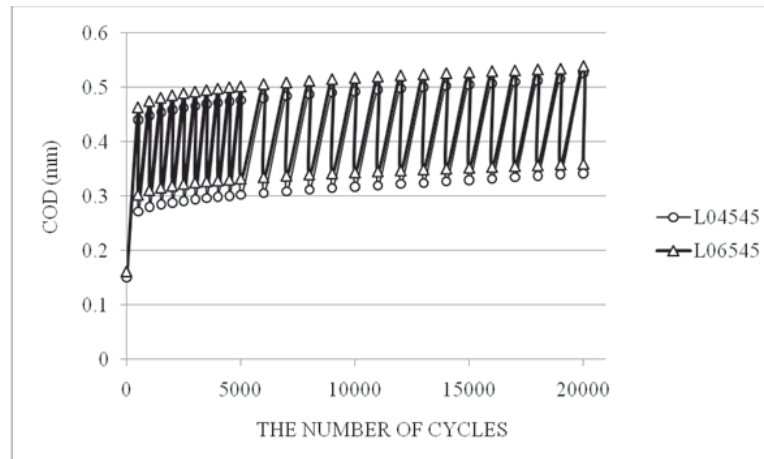


Figure 4.7. Dynamic loading results (cover depth= 45 mm).

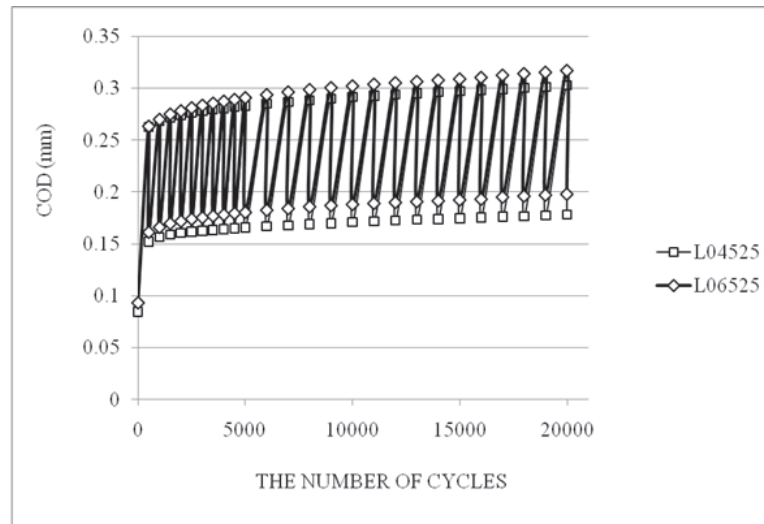


Figure 4.8. Dynamic loading results (cover depth= 25 mm).

Figure 4.7 and Figure 4.8 show COD vs. number of cycles for specimens with non-fibrous reinforced concrete. According to the figures, the COD was not influenced by water-to-cement ratio for specimens with both 25 mm and 45 mm cover depths when steel fibers do not exist in the mixes.

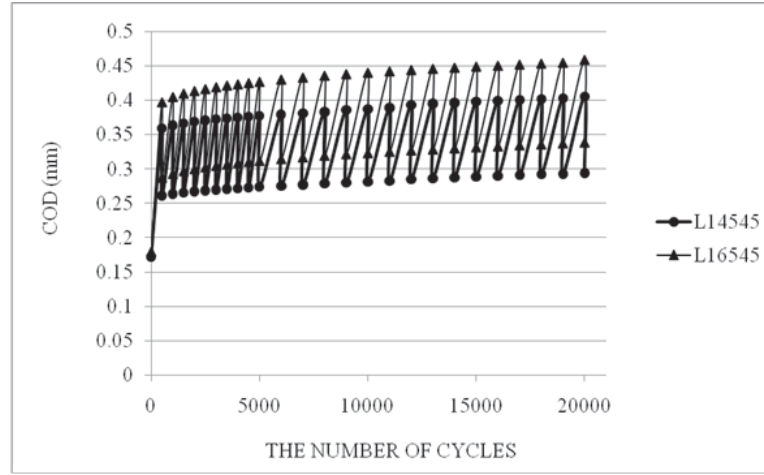


Figure 4.9. Dynamic loading results (cover depth= 45 mm).

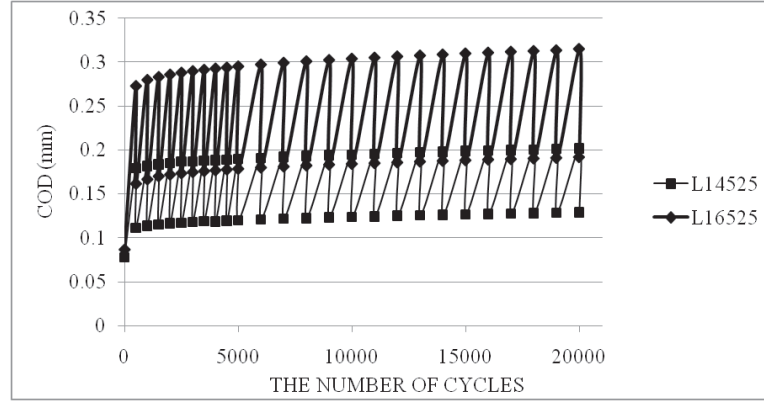


Figure 4.10. Dynamic loading results (cover depth= 25 mm).

Figure 4.9 and Figure 4.10 show COD vs. number of cycles for specimens with fiber reinforced concrete. According to the above figures, water-to-cement ratio influences the COD during dynamic cycle when steel fibers are used in the mixes. Crack opening for L1/45/45 was found to be 17% less than L1/65/45 during the dynamic cycle. Likewise, crack opening for L1/45/25 was found to be 40% less than L1/65/25 during the dynamic cycle. At 20000th cycle, the COD increased 0.111 mm when load was varied from minimum to maximum for L1/45/45, whereas the COD variation for L1/65/45 was 0.120 mm. However, under the same load the COD variation for L1/45/25 was 0.07 mm, while the COD variation for L1/65/25 was 0.12 mm.

4.6. Corrosion Test Results

Corrosion tests were undertaken for following specimens; uncracked, cracked under static loading, cracked under dynamic loading. Twenty four specimens were used for each type of loading. Conditioning of specimens was explained in detail in section 3.5.

4.6.1. Corrosion Test Results for Uncracked (Control) Beams

In this section measurements were taken by using two equipments. First, the potential measurements (mV) were obtained with a copper/copper reference electrode (half cell potential technique). Then a more sophisticated method/equipment was used to measure further data related to corrosion. As explained in Section 3.5.2, an equipment which works based on Galvanostatic pulse technique (trademark name Galvapulse - from here on trademark name of the equipment will be used) was used to measure the potential (mV) and corrosion rate ($\mu\text{A}/\text{cm}^2$). Results are given below. It should be noted that these measurements were started one week later after the beginning of wetting and drying cycles. Unfortunately, due to a breakdown in Galvapulse, it was not possible to obtain measurements for the following 3 weeks. After fixing the problem in the equipment, measurements were started again in the 5th week.

4.6.1.1. Half - Cell Potential Measurements for uncracked beams. Potential test results with a copper/copper reference electrode for uncracked beams are given in Figure 4.11. These measurements were started three weeks after the beginning of wetting and drying cycles.

Table 4.7 presents the potential results for uncracked beams with copper/copper sulphate reference electrode and the specimens were classified according to their corrosion conditions with respect to ASTM C876 as given below.

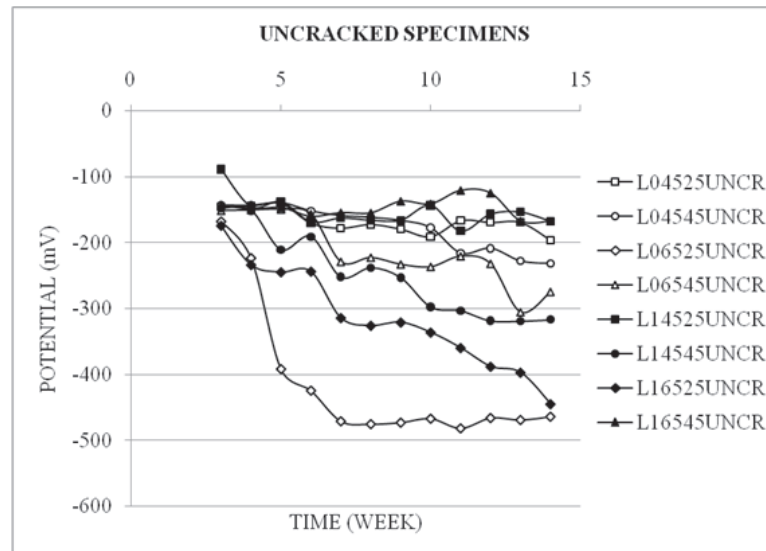


Figure 4.11. Potential measurements for uncracked beams.

Table 4.7. Interpretation of potential results for uncracked specimens in accordance with copper/copper sulphate reference electrode.

ASTM C876 Potential Evaluation Technique		Corrosion Potentials of Specimens				
Cu/CuSO ₄	Interpretation	3 rd week	6 th week	9 th week	12 th week	14 th week
$E > -200\text{mV}$	Greater than 90% probability that no corrosion is occurring	L14525 L16545 L04545 L04525 L14545 L06545 L06525 L16525	L04545 L06545 L16545 L14525 L04525 L14545	L16545 L14525 L04545 L04525	L16545 L14525 L04525	L14525 L16545 L04525
$-200\text{mV} < E < -350\text{mV}$	Corrosion activity is uncertain		L16525	L06545 L14545 L16525	L04545 L06545 L14545	L04545 L06545 L14545
$E < -350\text{mV}$	Greater than 90% probability that corrosion is occurring		L06525	L06525	L16525 L06525	L16525 L06525

According to ASTM C876 standard given in Table 4.7 (first and second columns), if potential values are more positive than -200 mV , corrosion risk is very low. In Figure 4.11, after 3 weeks of exposure to wetting and drying cycles, potential values were positive than -200 mV , which means that at the beginning stages of wetting and

drying cycles, no corrosion is occurring in the specimens. However, with an increased number of wetting and drying cycles corrosion risk increased as is seen in Figure 4.11.

After 14 weeks of exposure to wetting and drying cycles, potential values for L0/65/25 and L1/65/25 specimens were negative than -350 mV, showing high risk of corrosion. Water-to-cement ratio seems to be the main factor for corrosion of uncracked specimens. In addition, cover depth also affects corrosion results. As can be seen from the Table 4.7, potential value of L1/65/45 specimen was positive than -200 mV, showing low risk of corrosion, whereas potential value of L1/65/25 specimen was negative than -300 mV, showing high risk of corrosion. Moreover, specimens with a water-to-cement ratio of 0.45 and a cover depth of 25 mm yielded higher potential values showing high resistance to corrosion. Fiber reinforcement does not seem to show a effect on corrosion potential when the specimens are uncracked.

4.6.1.2. Measuring Potential Values based on Galvanostatic Pulse Technique. Potential test results with Galvapulse for uncracked beams are given in Figure 4.12 for 14 weeks. Reinforcement potential results are generally evaluated by using a classification method given by ASTM C876.

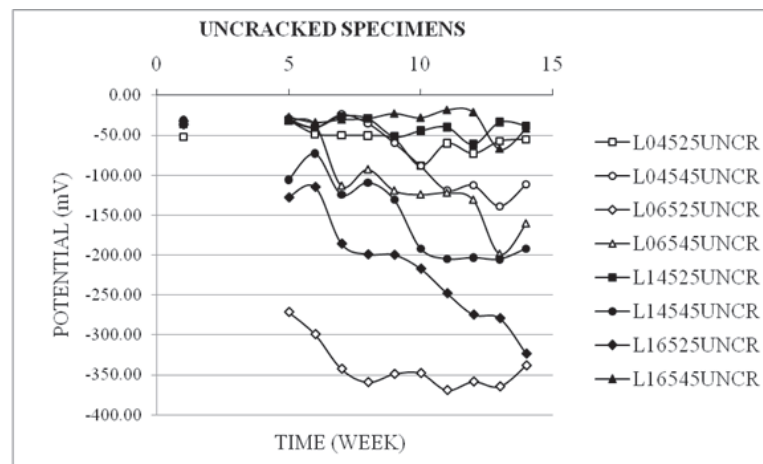


Figure 4.12. Potential measurements for uncracked beams with Galvapulse.

Table 4.8 presents the potential results for uncracked beams with Galvapulse and the specimens were classified according to their corrosion states with respect to ASTM

C876. If the results obtained from the two techniques are compared, it is seen that copper/copper sulphate electrode (half cell potential - Table 4.7) and silver/silver chloride electrode (Galvapulse - Table 4.8) measurements give very similar results.

Table 4.8. Interpretation of potential results for uncracked specimens with Galvapulse (The specimens are listed in an order according to their potential values from low to high).

ASTM C876 Potential Evaluation Technique		Corrosion Potentials of Specimens				
Ag/AgCl	Interpretation	1 st week	5 th week	9 th week	12 th week	14 th week
E>-119mV	Greater than 90% probability that no corrosion is occurring	L16545				
		L06545	L16545			
		L04545	L04545	L16545	L16545	L14525
		L14545	L04525	L14525	L14525	L16545
		L16525	L06545	L04525	L04525	L04525
		L14525	L14525	L04545	L04545	L04545
		L06525	L14545			
		L04525				
-119mV<E<-269mV	Corrosion activity is uncertain		L16525	L06545 L14545 L16525	L06545 L14545	L06545 L14545
E<-269mV	Greater than 90% probability that corrosion is occurring		L06525	L06525	L16525 L06525	L16525 L06525

Galvapulse uses the silver/silver chloride reference electrode. According to ASTM C876 standard given in Table 4.8 (first and second columns), if potential values are more negative than -119 mV, corrosion risk is very low. In Figure 4.12, after one week of exposure to wetting and drying cycles, potential values were in the range of approximately -50 mV, which were more positive than -119 mV, means that at the beginning stages of wetting and drying cycles, no corrosion is occurring for all specimens. However, with an increased number of wetting and drying cycles corrosion risk increased as is seen in Figure 4.8.

According to the above results, water-to-cement ratio seems to affect the corrosion results. After 14 weeks of exposure to wetting and drying cycles, potential values

for L0/65/25 and L1/65/25 specimens were negative than -269 mV, showing high risk of corrosion. However, specimens with a water-to-cement ratio of 0.45 yielded results positive than -269 mV, proving that water-to-cement ratio is very important for corrosion of uncracked specimens.

Apart from this, the effect of cover depth can be easily seen in Table 4.8. Specimens with a 25 mm cover depth and a water-to-cement ratio of 0.65 showed corrosion activity, whereas specimens with a 45 mm cover depth and a water-to-cement ratio of 0.65 showed no corrosion activity as can be seen in Table 4.8.

4.6.1.3. Corrosion Rate Measurements with Galvapulse. Corrosion rate measurements with Galvapulse for uncracked beams are given in Figure 4.13. There are various methods used for evaluating corrosion rate results offered by different researchers as explained in section 2.3.8 (Clear, 1989; Frolund, 2002). In this study Frolund technique is used since his measurements and evaluations are based on the same equipment used in this work.

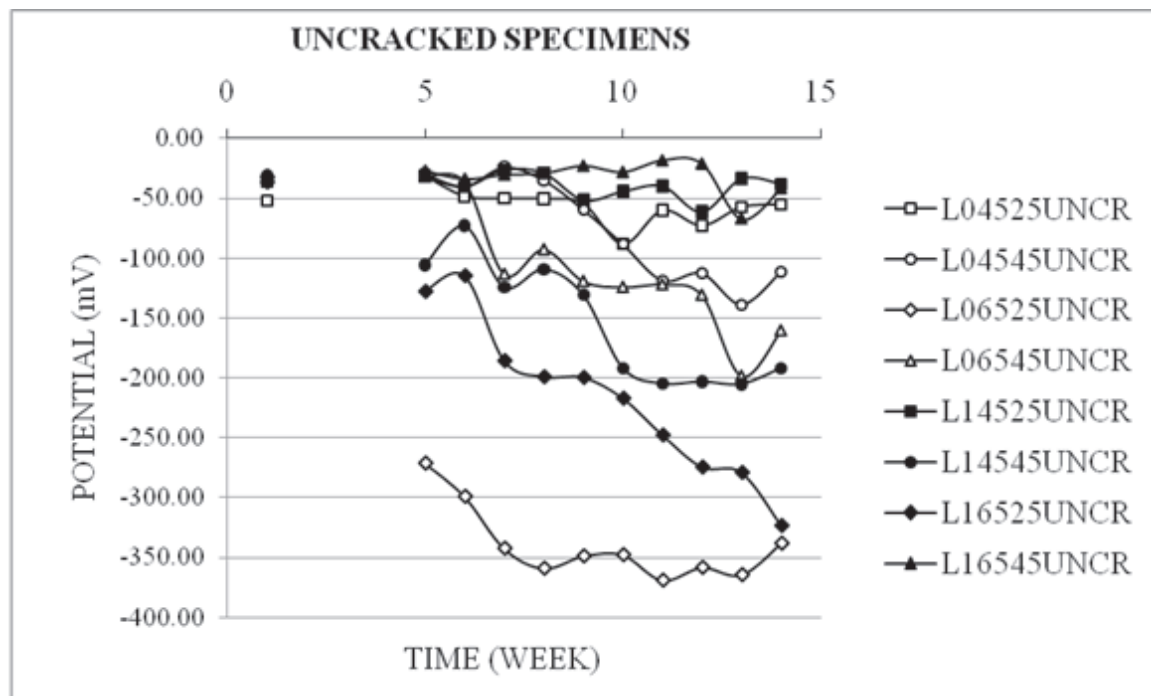


Figure 4.13. Corrosion currents for uncracked beams with Galvapulse.

According to Frolund evaluation technique given in Section 2.3.8, if current values are more than $0.5 \mu\text{A}/\text{cm}^2$, corrosion activity begins. This evaluation technique is given below in Table 4.9 (first and second columns). In the below table, specimens were classified with the used technique for corrosion currents (Frolund, 2002).

Table 4.9. Interpretation of corrosion currents for uncracked specimens (The specimens are listed in an order according to their corrosion current values from low to high).

Frolund Corrosion Current Evaluation Technique		Corrosion Currents of Specimens				
Corrosion Current (Icorr) ($\mu\text{A}/\text{cm}^2$)	Interpretation	1 st week	5 th week	9 th week	12 th week	14 th week
Icorr<0.5	Passive areas					
0.5<Icorr<2	Negligible corrosion activity	L14525	L04525	L16545	L04545	L04525
		L04525	L14545	L14525	L04525	L14525
		L16525	L16545	L04525	L14545	L16545
		L06545	L16545	L04545	L06545	L04545
		L06525	L06525	L06545	L14525	L14545
		L16545	L04525	L14545	L16545	L06545
		L04545	L16525	L16525	L16525	
		L14545	L14525			
2<Icorr<5	Low corrosion activity			L06525	L06525	L16525 L06525
5<Icorr<15	Moderate corrosion activity					
Icorr>15	High corrosion activity					

According to Figure 4.13, initial corrosion current values for specimens are higher than $0.5 \mu\text{A}/\text{cm}^2$, but less than $2 \mu\text{A}/\text{cm}^2$, and therefore at the beginning stage, the negligible corrosion activity is observed. In the following weeks, the corrosion current values were found to fluctuate. This may be resulted from the change in temperature and humidity (Frolund *et al.*, 2000). Especially, corrosion current values for L0/65/25 and L1/65/25 specimen showed more change (more than $2 \mu\text{A}/\text{cm}^2$) during the measurements. On the other hand, other specimens showed variations between 0.5 and $2 \mu\text{A}/\text{cm}^2$, which was the range of the negligible corrosion activity. After 14 weeks of

exposure, corrosion current values for L0/65/25 and L1/65/25 specimens are observed to be equal or more than $2 \mu\text{A}/\text{cm}^2$, meaning low corrosion activity, whereas the other specimens remained in the negligible corrosion activity range. This result confirms that water-to-cement ratio and cover depth are the effective factors for corrosion when there is no crack.

4.6.2. Corrosion Test Results for the Beams Cracked under Static Load

In this section measurements were taken by using half cell potential technique and Galvapulse as it was done for uncracked specimens. First, the potential measurements (mV) were obtained with a copper/copper reference electrode (half cell potential technique). Then again Galvapulse was used to evaluate corrosion state of the specimen in more detail (potential (mV) and corrosion rate ($\mu\text{A}/\text{cm}^2$)). It should be noted that these measurements were started one week later after the beginning of wetting and drying cycles.

In addition to this, crack widths for static specimens after load removal were given in Table 4.10 so as to interpret the corrosion results with respect to crack widths. Results are given below.

Table 4.10. Crack widths for the beams cracked under static load.

SPECIMENS CRACKED UNDER STATIC LOAD	
SPECIMEN	CRACK WIDTH AFTER LOAD REMOVAL
L0/45/25	0.08 mm
L1/45/25	0.08 mm
L0/65/25	0.08 mm
L1/65/25	0.08 mm
L0/45/45	0.17 mm
L1/45/45	0.17 mm
L0/65/45	0.18 mm
L1/65/45	0.20 mm

According to the results, specimens with 25 mm cover depth has a crack width of approximately 0.08 mm, whereas specimens with 45 mm cover depth has a crack width of approximately 0.18 mm. This means that effect of using fibers is surpassed by the constraining effect of rebar.

4.6.2.1. Half- Cell Potential Measurements with a Copper/Copper Reference Electrode for the Beams Cracked under Static Load. Potential test results with a copper/copper reference electrode for the beams cracked under static load are given in Figure 4.14. These measurements were started three weeks after the beginning of wetting and drying cycles. Measurements were evaluated with respect to ASTM C876 as is done before for uncracked (control) specimens.

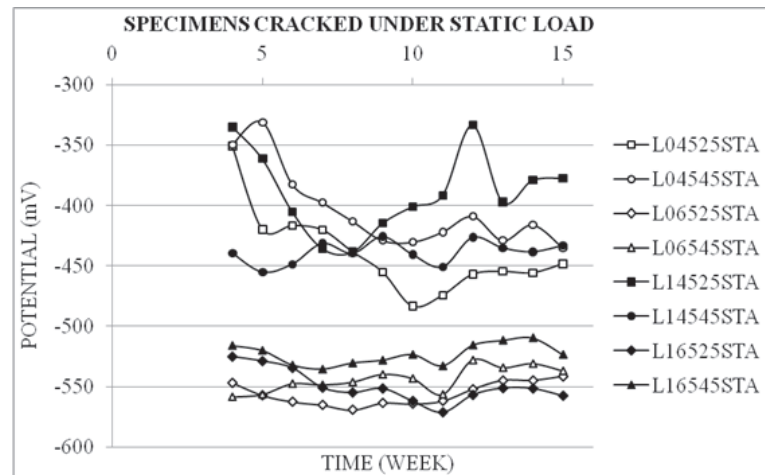


Figure 4.14. Potential measurements with copper/copper sulphate reference electrode for the beams cracked under static load.

Table 4.11 presents the potential results for the beams cracked under static load with copper/copper sulphate reference electrode and the specimens were classified according to their corrosion states with respect to ASTM C876.

Table 4.11. Interpretation of potential results for the beams cracked under static load in accordance with copper/copper sulphate reference electrode.

ASTM C876 Potential Evaluation Technique		Corrosion Potentials of Specimens				
Cu/CuSO ₄	Interpretation	4 rd week	6 th week	9 th week	12 th week	15 th week
E < -200mV	Greater than 90% probability that no corrosion is occurring					
200mV < E < -350mV	Corrosion activity is uncertain	L14525 L04545 L04525			L14525	
E < -350mV	Greater than 90% probability that corrosion is occurring		L04545 L14525 L04525 L16545 L16525 L06525 L06545 L16525	L14525 L04545 L14545 L04525 L16545 L16545 L06545 L16525 L06525	L04545 L14545 L04525 L16545 L06545 L06525 L16525	L14525 L04545 L14545 L04525 L04525 L06545 L06525 L06525 L16525

In Figure 4.14, after 3 weeks of exposure to wetting and drying cycles, potential values for L0/45/25, L1/45/25 and L0/45/45 were negative than -200 mV but positive than -350 mV, meaning that the risk of corrosion is still uncertain for these specimens. On the other hand, potential values for other specimens were negative than -350 mV, showing high risk of corrosion. However, with an increased number of wetting and drying cycle corrosion risk increased for all specimens as is seen in figure. After 15 weeks of exposure to wetting and drying cycles, potential values for all specimens were negative than -350 mV, showing high risk of corrosion. However, the specimens with a water-to-cement ratio of 0.45 yielded smaller potential values when compared to the specimens with high water-to-cement ratio.

If these results are compared with the results obtained from uncracked specimens, it is seen that corrosion risk is very high from the 6th week for all the specimens cracked under static load. Water-to-cement ratio seems to be more effective than crack width for the first 15 weeks (The specimens with a water-to-cement ratio of 0.65 shows greater potential values). Potential values were found to be very similar for the same water-to-

cement ratio when the cover depth is varied from 25 mm to 45 mm (crack widths 0.08 mm and 0.18 mm, respectively). The values for L1/65/45 (0.17 mm) and L0/65/45 (0.17 mm) were found to be more positive than L0/65/25 (0.08 mm) and L1/65/25 (0.08 mm) although the crack widths are greater for the first two specimens. This result is probably due to the fact that it takes more time for aggressive agents to reach reinforcement when the cover depth is greater. No clear effect or enhancement due to fiber addition is seen since cracks are constrained by rebar. Effects of fibers may be seen in the upcoming weeks.

4.6.2.2. Potential Measurements with Galvapulse for the Beams Cracked under Static Load. Potential test results with Galvapulse for the beams cracked under static load are given in Figure 4.15. These measurements were started one week later after the beginning of wetting and drying cycles.

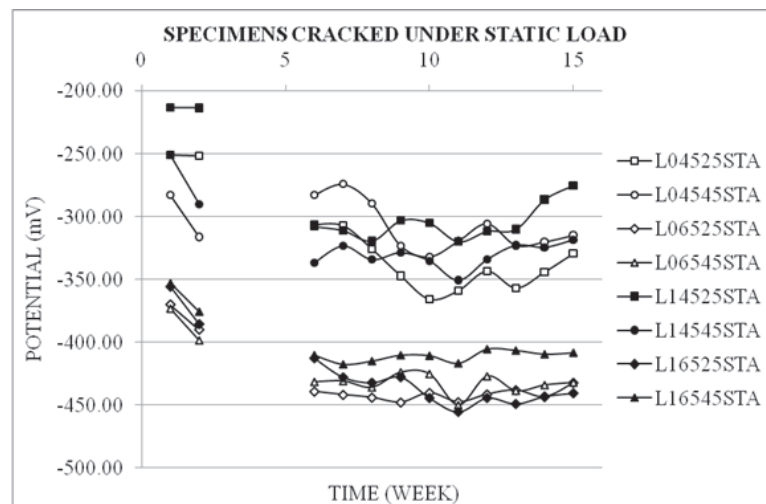


Figure 4.15. Potential measurements with Galvapulse for the beams cracked under static load.

Table 4.12 presents the potential results for the beams cracked under static load with Galvapulse and the specimens were classified according to their corrosion states with respect to ASTM C876. If the results obtained from the two techniques are compared, it is seen that copper/copper sulphate electrode (half cell potential - Table 4.11) and silver/silver chloride electrode (Galvapulse - Table 4.12) measurements yield very similar results.

Table 4.12. Interpretation of potential results with Galvapulse for the beams cracked under static load (The specimens are listed in an order according to their potential values from low to high).

ASTM C876 Potential Evaluation Technique		Corrosion Potentials of Specimens				
Ag/AgCl	Interpretation	1 st week	6 th week	9 th week	12 th week	15 th week
$E < -119\text{mV}$	Greater than 90% probability that no corrosion is occurring					
$-119\text{mV} < E < -269\text{mV}$	Corrosion activity is uncertain	L14525 L04545 L14545				
$E < -269\text{mV}$	Greater than 90% probability that corrosion is occurring		L04545 L04525 L14525 L16545 L16525 L06525 L06545 L06525	L14525 L04545 L14545 L04525 L16545 L06545 L16525 L06525	L04545 L14525 L14545 L04525 L16545 L06545 L06525 L16525	L14525 L04545 L14545 L04525 L16545 L06545 L06525 L16525

As can be seen from Table 4.12, all the specimens exceeded the limit value for high corrosion probability before the 6th week. Water-to-cement ratio is again seems like the dominating parameter when Figure 4.15 is examined.

4.6.2.3. Corrosion Rate Measurements with Galvapulse for Beams Cracked under Static Load. Corrosion rate measurements with Galvapulse for beams cracked under static load are given in Figure 4.16. As explained above in section 4.5.3, the corrosion rates were evaluated with respect to Frolund findings and these findings were also adopted by Germann Instrument, which is the producer company of the equipment used in this study (Galvapulse)(Frolund, 2002).

In Table 4.13, Frolund evaluation technique is given (first and second columns) and the specimens were classified with the used technique for corrosion currents (Frolund, 2002).

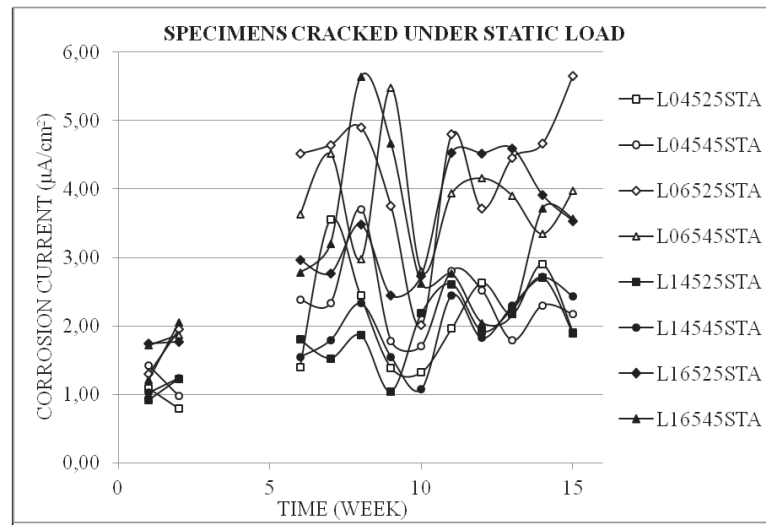


Figure 4.16. Corrosion currents with Galvapulse for the beams cracked under static load.

Table 4.13. Interpretation of corrosion currents for the beams cracked under static load (The specimens are listed in an order according to their corrosion current values from low to high).

Frolund Corrosion Current Evaluation Technique		Corrosion Currents of Specimens				
Corrosion Current (I _{corr}) ($\mu A/cm^2$)	Interpretation	1 st week	6 th week	9 th week	12 th week	15 th week
I _{corr} < 0.5	Passive areas					
0.5 < I _{corr} < 2	Negligible corrosion activity	L14525 L14545 L04525 L16545 L06525 L04545 L06545 L16525	L04525 L14545 L14525	L14525 L04525 L14545 L04545	L14545 L14525	L14525 L04525
5 < I _{corr} < 15	Moderate corrosion activity			L06545		L04525
I _{corr} > 15	High corrosion activity					

In Figure 4.16, the beginning corrosion current values are more than $0.5 \mu A/cm^2$ but less than $2 \mu A/cm^2$, and therefore at the beginning stage, the negligible corrosion

activity is observed. In the following weeks, the corrosion current values were found to fluctuate (Frolund *et al.*, 2000). Especially, corrosion current values for L0/65/25 (0.08 mm), L0/65/45 (0.18 mm) and L1/65/45 (0.20 mm) specimens showed more variation (more than $5 \mu\text{A}/\text{cm}^2$) during the measurements.

If corrosion rates for uncracked and cracked (under static load) specimens are compared (Table 4.9 and Table 4.13), it is seen that the initiation of corrosion for the cracked specimens takes place earlier than uncracked specimens, as expected.

After 15 weeks of exposure to wetting and drying cycles, L0/65/25 (0.08 mm) specimen showed the moderate corrosion activity, L0/45/25 (0.08 mm) and L1/45/25 (0.08 mm) specimens showed negligible corrosion activity and the other specimens showed low corrosion activity. According to these results, water-to-cement ratio is again seems like the dominating parameter.

4.6.3. Corrosion Test Results for the Beams Cracked under Dynamic Load

In this section measurements were taken by using half cell potential technique and Galvapulse as it was done for uncracked specimens and specimens cracked under static load. First, the potential measurements (mV) were obtained with a copper/copper reference electrode (half cell potential technique). Then again Galvapulse was used to evaluate corrosion state of the specimen in more detail (potential (mV) and corrosion rate ($\mu\text{A}/\text{cm}^2$)). It should be noted that these measurements were started one week later after the beginning of wetting and drying cycles.

In addition to this, crack widths for specimens cracked under dynamic load after load removal were given in Table 4.14 so as to interpret the corrosion results with respect to crack widths. Results are given below.

Table 4.14. Crack widths for the beams cracked under dynamic load.

SPECIMENS CRACKED UNDER DYNAMIC LOAD	
SPECIMEN	CRACK WIDTH AFTER LOAD REMOVAL
L0/45/25	0.13 mm
L1/45/25	0.10 mm
L0/65/25	0.13 mm
L1/65/25	0.16 mm
L0/45/45	0.27 mm
L1/45/45	0.21 mm
L0/65/45	0.27 mm
L1/65/45	0.26 mm

According to the results, specimens with 25 mm cover depth has a crack width of approximately 0.13 mm, whereas specimens with 45 mm cover depth has a crack width of approximately 0.26 mm. As is seen in Table 4.14, rebar seems to constraint crack width. Effect of fibers is not as pronounced at still seen for all specimens except L1/65/25.

4.6.3.1. Half - Cell Potential Measurements with a Copper / Copper Reference Electrode for the Beams Cracked under Dynamic Load. Potential test results with a copper/copper reference electrode for beams cracked under dynamic load are given in Figure 4.17. These measurements were started three weeks after the beginning of wetting and drying cycles. Measurements were evaluated with respect to ASTM C876 as is done before for uncracked (control) specimens and specimens cracked under static load.

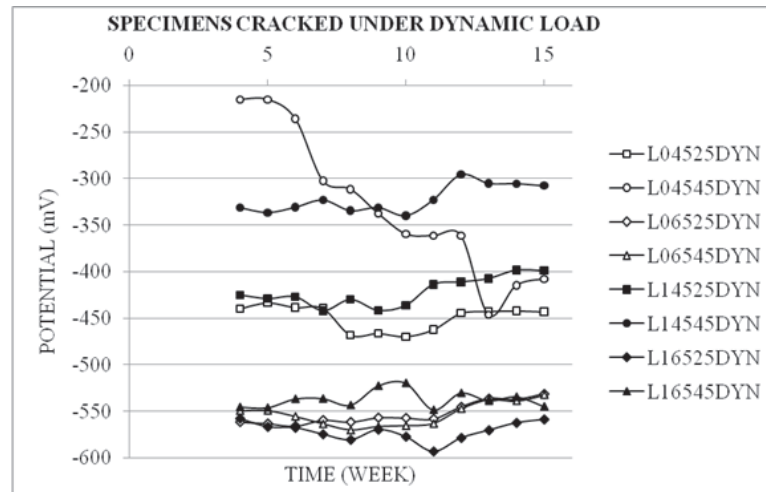


Figure 4.17. Potential measurements with copper/copper sulphate reference electrode for the beams cracked under dynamic load.

Table 4.15 presents the potential results for the beams cracked under dynamic load with copper/copper sulphate reference electrode and the specimens were classified according to their corrosion states with respect to ASTM C876.

Table 4.15. Interpretation of potential results for the beams cracked under dynamic load in accordance with copper/copper sulphate reference electrode.

ASTM C876 Potential Evaluation Technique		Corrosion Potentials of Specimens				
Cu/CuSO ₄	Interpretation	4 rd week	6 th week	9 th week	12 th week	15 th week
$E < -200\text{mV}$	Greater than 90% probability that no corrosion is occurring					
$-200\text{mV} < E < -350\text{mV}$	Corrosion activity is uncertain	L04545 L14545	L04545 L14545	L14545 L04545	L14545	L14545
$E < -350\text{mV}$	Greater than 90% probability that corrosion is occurring	L14525 L04525 L16545 L06545 L16525 L06525	L14525 L04525 L16545 L16545 L06525 L16525	L14525 L04525 L16545 L16525 L06545 L16525	L04545 L14525 L04525 L16545 L06525 L06545 L16525	L14525 L04545 L04525 L06525 L06545 L16545 L16525

In Figure 4.17, after 4 weeks of exposure to wetting and drying cycles, potential values for L0/45/45 (0.27 mm) and L1/45/45 (0.21 mm) were negative than -200 mV but positive than -350 mV, meaning that the risk of corrosion is still uncertain for these specimens. On the other hand, potential values for other specimens were negative than -350 mV, showing high risk of corrosion. However, with an increased number of wetting and drying cycle corrosion risk increased for all specimens as is seen in figure.

After 15 weeks of exposure, potential value for L1/45/45 (0.21 mm) specimen was negative than -200 mV but positive than -350 mV, meaning that the risk of corrosion is still uncertain. On the other hand, potential values for the other specimens were negative than -350 mV, showing high risk of corrosion.

According to the Figure 4.17, potential value for L1/65/45 (0.26 mm) were found to be more positive than L1/65/25 (0.16 mm) although the crack width is greater for the first specimen. Similar results are observed for L0/45/45 (0.27 mm) and L0/45/25 (0.13 mm), and L1/45/45 (0.21) and L1/45/25 (0.10 mm). These results are probably due to the fact that it takes more time for aggressive agents to reach reinforcement when the cover depth is greater.

Water-to-cement ratio again seems to be the most important factor when evaluating the corrosion results for the beams cracked under dynamic load for the first 15 week. As can be easily seen in the above figure, potential values for the specimens with a water-to-cement ratio of 0.65 were negative than the specimens with a water-to-cement ratio of 0.45.

Moreover, fiber effect can also be seen in this figure. It should be reminded that both fiber reinforced concrete and non-fibered reinforced concrete cracked under the same maximum load during dynamic cycles. Therefore, crack widths are smaller for fiber reinforced specimens. Potential value for L1/45/45 (0.21 mm) specimen was 100 mV positive than for L0/45/45 (0.27 mm) specimen. Similarly, potential value for L1/45/25 (0.10 mm) specimen was 50 mV positive than for L0/45/25 (0.13 mm) specimen. On the other hand, fiber effect cannot be seen for specimens with a water-

to-cement ratio of 0.65. It can be concluded that the use of fibers affects corrosion results when water-to-cement ratio is low if the specimens (both fiber and non-fibered) are cracked under similar levels of loads.

4.6.3.2. Potential Measurements with Galvapulse for the Beams Cracked under Dynamic Load. Potential test results with Galvapulse for the beams cracked under dynamic load are given in Figure 4.18.

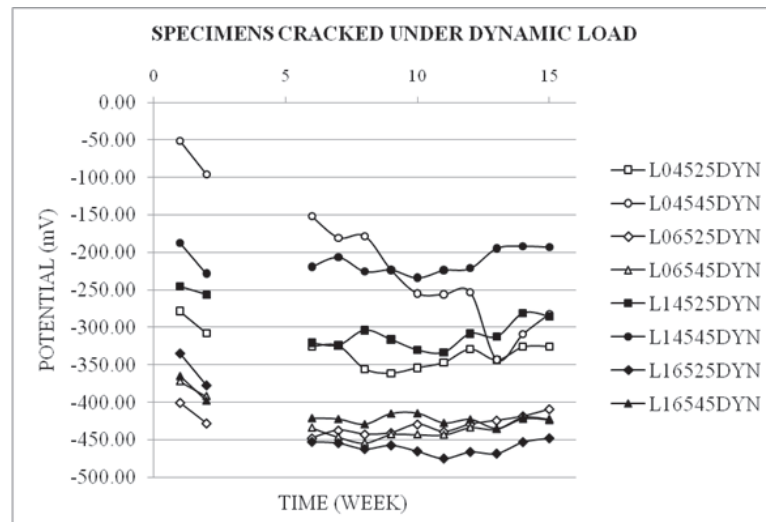


Figure 4.18. Potential measurements with Galvapulse for the beams cracked under dynamic load.

Table 4.16 presents the potential results for beams cracked under dynamic load with Galvapulse and the specimens were classified according to their corrosion states with respect to ASTM C876. If the results obtained from the two techniques are compared, it is seen that copper/copper sulphate electrode (half cell potential - Table 4.15) and silver/silver chloride electrode (Galvapulse - Table 4.16) measurements yield very similar results. Water-to-cement ratio is again found to be the dominating parameter when Figure 4.18 is examined. Contribution of fibers is seen for low water-to-cement ratio.

Table 4.16. Interpretation of potential results with Galvapulse for the beams cracked under dynamic load (The specimens are listed in an order according to their potential values from low to high).

ASTM C876 Potential Evaluation Technique		Corrosion Potentials of Specimens				
Ag/AgCl	Interpretation	1 st week	6 th week	9 th week	12 th week	15 th week
E<-119mV	Greater than 90% probability that no corrosion is occurring	L04545				
-119mV<E<-269mV	Corrosion activity is uncertain	L14545 L14525	L04545 L14545	L14545 L04545	L14545 L04545	L14545
E<-269mV	Greater than 90% probability that corrosion is occurring	L04525 L04525 L16545 L16525 L16545 L06545 L06525	L14525 L04525 L16545 L06545 L06525 L16525	L14525 L04525 L16545 L06525 L06545 L16525	L14525 L04525 L16545 L06525 L06545 L16525	L04545 L14525 L04525 L06525 L06545 L16545 L16525

4.6.3.3. Corrosion Rate Measurements with Galvapulse for Beams Cracked under Dynamic Load. Corrosion rate measurements with Galvapulse for beams cracked under dynamic load are given in Figure 4.19. As explained above in section 4.5.3, the corrosion rates were evaluated with respect to Frolund (2002) findings and these findings were also adopted by Germann Instrument, which is the producer company of the equipment used in this study (Galvapulse).

In Figure 4.19, the beginning corrosion current values are more than $0.5 \mu\text{A}/\text{cm}^2$ but less than $2 \mu\text{A}/\text{cm}^2$ except for L0/65/25, and therefore at the beginning stage, the negligible corrosion activity is occurred for these specimens. On the other hand, corrosion rate for L0/65/25 was more than $2 \mu\text{A}/\text{cm}^2$, so low corrosion activity is occurred for this specimen at the beginning state. In the following weeks, the corrosion current values were found to fluctuate (Frolund *et al.*, 2000). Especially, corrosion current values for L0/65/25, L1/65/25 and L1/65/45 specimen showed more variation (more than $5 \mu\text{A}/\text{cm}^2$) during the measurements.

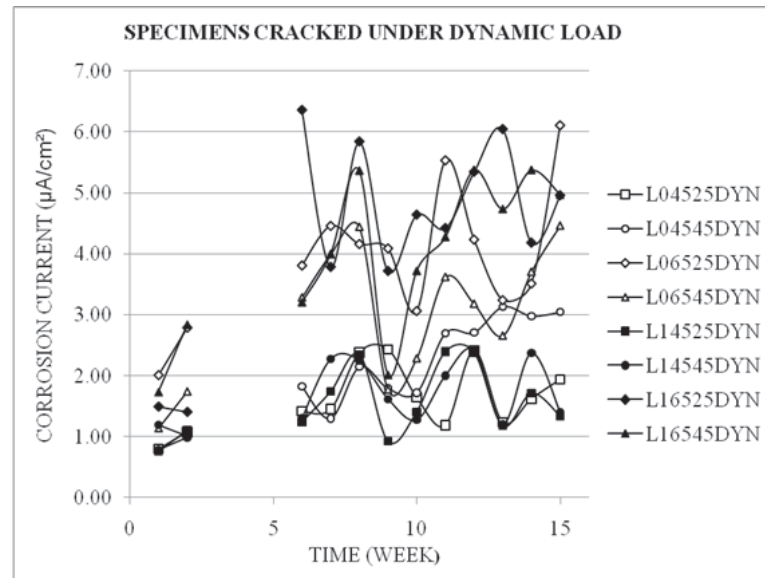


Figure 4.19. Corrosion currents with Galvapulse for beams cracked under dynamic load.

In Table 4.17, after 15 weeks of exposure to wetting and drying cycles, L0/65/25 (0.13 mm), L1/65/25 (0.16 mm) and L1/65/45 (0.26 mm) specimens showed moderate corrosion activity, L1/45/25 (0.10 mm) and L1/45/45 (0.21 mm) specimens showed negligible corrosion activity and the other specimens showed low corrosion activity. According to these results, water-to-cement ratio is again found to be the dominating parameter. Corrosion current for specimens with a water-to-cement ratio of 0.65 showed moderate corrosion activity, except for L0/65/45 (0.26 mm). This specimen was also near the range of moderate corrosion activity. Corrosion currents for specimens with a water-to-cement ratio of 0.45 were fluctuated between 1 and 2 $\mu\text{A}/\text{cm}^2$, whereas current values fluctuated between 3 and 6 $\mu\text{A}/\text{cm}^2$ for the specimens with a water-to-cement ratio of 0.65. It is concluded that corrosion rates for specimens with a water-to cement ratio of 0.45 were nearly steady, whereas corrosion rates for specimens with a water-to-cement ratio of 0.65 were imbalanced.

Table 4.17. Interpretation of corrosion currents for the beams cracked under dynamic load (The specimens are listed in an order according to their corrosion current values from low to high).

Frolund Corrosion Current Evaluation Technique		Corrosion Currents of Specimens				
Corrosion Current (I_{corr}) ($\mu A/cm^2$)	Interpretation	1 st week	6 th week	9 th week	12 th week	15 th week
$I_{corr} < 0.5$	Passive areas					
$0.5 < I_{corr} < 2$	Negligible corrosion activity	L14525 L04545 L04525 L06545 L14545 L16525 L16545 L16525	L14525 L14545 L04525 L04545	L14525 L14545 L06545 L04545		L14525 L14545
$2 < I_{corr} < 5$	Low corrosion activity	L06525	L16545 L06545 L06525	L16545 L04525 L16525 L06525	L14525 L14545 L04525 L04545 L06545 L06525	L04525 L04545 L06545
$5 < I_{corr} < 15$	Moderate corrosion activity		L16525		L16525 L16545	L16525 L16545 L06525
$I_{corr} > 15$	High corrosion activity					

5. CONCLUSIONS

The main objective of this study was to determine the importance and influence of the crack size, depth of concrete cover and water-to-cement ratio, under different loading conditions (static and dynamic) and with and without the use of steel fibers on the chloride-induced corrosion activity of steel reinforcement by using galvanostatic pulse technique. As explained in the experimental procedure and given in Table 3.6, following specimens are obtained as a result of the casting and loading procedures; (i) 24 (three specimens in each 8 different series) control specimens with and without fibers (with no cracks), (ii) 24 specimens (three specimens in each 8 different series) cracked under static load with and without fibers, (iii) 24 specimens (three specimens in each 8 different series) cracked under dynamic load with and without fibers. First mechanical test were applied on these specimens and later they are exposed to wetting and drying cycles and corrosion measurements are carried out on the specimens throughout the process. In this section overall corrosion conclusions will be given and evaluated.

5.1. Failure Test Results

Flexural tests were carried out on beam specimens with dimensions of 100x100x500 mm under three point loading by using MTS loading machine. Three beams were tested until failure for each mixture, and maximum load and crack opening displacement were recorded. Finally, the average values of these results were calculated and they were compared to each other. As a result of these tests, it was found that as water-to-cement ratio increases, the failure stress of beams decreases. On the other hand, as water-to-cement ratio increases, the crack opening displacement also increases (except L0/45/45). The use of steel fibers increases the failure capacity and decreases crack opening displacement of the beams, as expected. Apart from this, the effect of steel fibers seems to increase as cover depth increases for the specimens with a water-to-cement ratio of 0.45. This could be explained as follows. When cover depth is 25 mm, maximum crack opening is further restricted by rebar, and fibers may actively act until crack opening is constrained by the rebar.

5.2. Results for the Beams Cracked under Static Load

Beams were loaded statically under three point loading by using MTS loading machine as is done before. Three beams were tested until the required crack opening displacement was reached for each mixture. Crack width values had been selected 0.4 mm and 0.22 mm for specimens that have 45 mm and 25 mm cover depth, respectively, so that crack width-cover depth ratio is equal to 0.0089 for all specimens (detailed explanation was given in Section 3.4.3). Following the removal of load, crack width decreased as expected for all the specimens. Final crack width values were read from LVDTs.

Crack openings decreased from 0.22 mm to approximately 0.08 mm on average for beams that have 25 mm cover depth, whereas cracks openings decreased from 0.4 to approximately 0.18 mm on average for beams that have 45 mm cover depth after load removal. Decrease in crack width following the removal of load was found to be 10% less when the cover depth was 45 mm. It is hypothesized that as crack width increases, plastic deformation also increases, therefore, cracks of the specimens with 45 mm cover depth were closed approximately 10% less than the cracks of the specimens with 25 mm cover depth.

Moreover, when crack widths were kept constant (0.22 and 0.4 mm), the required loads needed for cracking of beams were more for the fiber reinforced concrete specimens. This required load decreases, as water-to-cement ratio and cover depth increases.

5.3. Results for the Beams Cracked under Dynamic Load

Beams were loaded under static load at first and then they were loaded dynamically under three point loading by using MTS loading machine. Loading procedure was explained in Section 3.4.4.

Dynamic loading caused greater crack opening displacement, as expected. COD

values were found to increase with an increase in cover depth. Steel fiber addition also influences the dynamic loading results. For fiber reinforced specimens with a water-to-cement ratio of 0.45, COD values were found to be approximately 2 times less than COD for normal reinforced concrete.

According to the results, effects of fibers can easily be seen on the crack behavior of specimens both with a cover depth of 45 mm and 25 mm. Crack opening for L1/45/45 was found to be 27% less than L0/45/45 during the dynamic cycle. Likewise, crack opening for L1/45/25 was found to be 34% less than L0/45/25 owing to the contribution of fibers. Another result is seen when the specimens with two different cover depths were compared. When cover depth is 25 mm, maximum crack opening is further restricted by rebar and maximum opening is limited to 0.3 mm, on the other hand for the same specimens crack opening can go up to 0.5 mm when the cover depth is equal to 45 mm. Moreover, fiber effect is much less pronounced when water-to-cement ratio is increased. This result could be attributed to the fact that adherence between fiber and concrete matrix decreases when water-to-cement ratio increased and as a consequence fibers may slip out of the matrix before they reach to their maximum capacity. Furthermore, the COD was not influenced by water-to-cement ratio for specimens with both 25 mm and 45 mm cover depths when steel fibers do not exist in the mixes.

5.4. Corrosion Test Results

Table 5.1 presents the potential results for uncracked beams, beams cracked under static load, and beams cracked under dynamic load (measured with Galvapulse). The specimens were classified according to their corrosion states with respect to ASTM C876.

In Table 5.1, last week of corrosion potential results were evaluated based on the related standard. According to the results, existence of cracks affects corrosion results. Potential values for uncracked specimens (15th week) were positive than -269 mV, except for L0/65/25 and L1/65/25.

Table 5.1. Corrosion potential evaluation of uncracked specimens, specimens cracked under static load and specimens cracked under dynamic load.

ASTM C876 Potential Evaluation Technique		Uncracked Specimens	Specimens Cracked under Static Load	Specimens Cracked under Dynamic Load
Ag/AgCl	Interpretation	15 rd week	15 th week	15 th week
E > -119mV	Greater than 90% probability that no corrosion is occurring	L14525 L16545 L04525 L04545		
-119mV < E < -269mV	Corrosion activity is uncertain	L06545 L14545		L14545 (0.21 mm)
E < -269mV	Greater than 90% probability that corrosion is occurring	L16525 L06525	L14525 (0.08 mm) L04545 (0.17 mm) L14545 (0.17 mm) L04525 (0.08 mm) L16545 (0.20 mm) L06545 (0.18 mm) L06525 (0.08 mm) L16525 (0.08 mm)	L04545 (0.27 mm) L14525 (0.10 mm) L04525 (0.13 mm) L06525 (0.13 mm) L06545 (0.27 mm) L16545 (0.26 mm) L16525 (0.16 mm)

As can be seen in Table 5.1 this means high risk of corrosion. On the other hand, potential values for specimens cracked under static load (15th week) were negative than -269 mV, meaning that corrosion risk is high for all specimens. Likewise, potential values for specimens cracked under dynamic load (15th week) were negative than -269 mV, except for L1/45/45. These results represent the effect of crack size on corrosion potential.

According to Table 5.1, potential values for uncracked specimens (15th week) with a water-to-cement ratio of 0.45 were found to be smaller when compared to the specimens with high water to cement ratio after 15 weeks of exposure. Effect of water-to-cement ratio is clearly seen when the specimens were uncracked. Effect of cover depth is also seen when the specimens which yielded the most negative potential values

were examined (L1/65/25 and L0/65/25). It should be reminded that in ACI 318, it is suggested to increase concrete quality and concrete cover thickness to protect reinforcement against corrosion. The results given above show that for uncracked specimens, the suggested precautions are meaningful and important. The use of steel fibers also seems to affect reinforcement corrosion. Steel reinforced specimens yielded more positive potential values when compared to non-fibered specimens, except for L1/45/45.

According to Table 5.1, potential values for all specimens cracked under static load (15th week) showed high risk of corrosion. However, the specimens with a water-to-cement ratio of 0.45 yielded smaller potential values when compared to the specimens with high water-to-cement ratio (This result is clearly seen in Figure 4.15). No clear effect was seen when cover depth is varied from 25 mm to 45 mm (crack openings 0.08 mm and 0.18 mm, respectively) when the water-to-cement ratio was kept constant. This is probably due to the fact that, crack opening is smaller for the specimens with smaller cover thickness. The reason for having smaller crack opening for the specimens with a cover depth of 25 mm had been explained as the constraining effect of rebar. Effect of using steel fibers is also not clearly seen however fibers are expected to play a more important role in the upcoming weeks when concrete deterioration increases.

In Table 5.1, potential values for specimens cracked under dynamic load (15th week) showed high risk of corrosion, except for L1/45/45 (0.21 mm). However, the specimens with a water-to-cement ratio of 0.45 yielded smaller potential values when compared to the specimens with high water-to-cement ratio, like specimens cracked under static load (Figure 4.18). The potential curves of the specimens with high water-to-cement ratio are found to be in close proximity of each other. Different from the specimens cracked under static load, cover depth affected potential results when water-to-cement ratio is low for the specimens cracked under dynamic load (when water-to-cement ratio is low (0.45)). The specimens with greater cover thickness had larger crack openings and still performed better. This result shows the importance of having greater cover thickness in the cracked case. This effect is not very well seen when water-to-cement ratio is high. However, it should be reminded that the potential

curves are almost on top of each other for the specimens with high water-to-cement ratio (0.65).

The effect of using steel fibers was also seen in the results for the specimens cracked under dynamic load when water-to-cement ratio is low (0.45). A similar observation could not be made for the specimens with a water-to-cement ratio of 0.65. From this result it can be concluded that fibers are more effective in arresting cracks when the concrete quality is high. This is also seen in the last column of Table 5.1 when crack openings are compared for L1/45/45 (0.21 mm), L1/65/45 (0.26 mm) and L1/45/25 (0.10 mm), L1/65/25 (0.16 mm).

Table 5.2. Corrosion current values of uncracked specimens, specimens cracked under static load and specimens cracked under dynamic load.

Frolund Corrosion Current Evaluation Technique		Uncracked Specimens	Specimens Cracked under Static Load	Specimens Cracked under Dynamic Load
Corrosion Current (I _{corr}) ($\mu A/cm^2$)	Interpretation	15 rd week	15 th week	15 th week
I _{corr} <0.5	Passive areas			
0.5< I _{corr} <2	Negligible Corrosion activity	L04525 L14525 L16545 L04545 L14545 L06545	L14525 (0.08 mm) L04525 (0.08 mm)	L14525 (0.10 mm) L14545 (0.21 mm)
2<I _{corr} <5	Low Corrosion activity	L16525 L06525	L04545 (0.17 mm) L14545 (0.17 mm) L16525 (0.08 mm) L16545 (0.20 mm) L06545 (0.18 mm)	L04525 (0.13 mm) L04545 (0.27 mm) L06545 (0.27 mm)
5<I _{corr} <15	Moderate corrosion		L06525 (0.08 mm)	L16525 (0.16 mm) L16545 (0.26 mm) L06525 (0.13 mm)
I _{corr} >15	High corrosion activity			

Table 5.2 presents the corrosion current values for uncracked beams, beams cracked under static load, beams cracked under dynamic load with Galvapulse and

the specimens were classified according to their corrosion states with respect to Frolund evaluation technique.

In Table 5.2, last week of corrosion current values were evaluated in accordance with the related evaluation technique. According to these results, almost all uncracked specimens could be evaluated in the negligible corrosion activity range during the 15th week, except L0/65/25 and L1/65/25. These two specimens showed low corrosion activity, probably due to high water-to-cement ratio and low cover depth. These results are in conformity with the potential results, which show the same two specimens under high risk of corrosion. If effects of varying water-to-cement ratio, cover depth and fiber use are evaluated, it can be said that water-to-cement ratio seems to be the most important parameter for the corrosion activity in uncracked specimens during the first 15 weeks. Effect of cover depth is not clearly seen since the data obtained highly fluctuate.

According to Table 5.2, most of the specimens cracked under static load showed low corrosion activity in the 15th week. Water-to-cement again found to be the most effective factor. Cover thickness and crack opening size was not clearly seen since the data fluctuates a lot. Crack size seems to be effective for the specimens with a low water-to-cement ratio, however, it may lead readers wrong since curves are very close to each other.

In Table 5.2, most of the specimens cracked under dynamic load showed either low or moderate corrosion activity in the 15th week. Only the effect of water-to-cement ratio is clearly seen when Figure 4.19 is examined. Effect of using fibers is also seen for almost all of the specimens.

If corrosion current values compared to each other for the 15 weeks of exposure, specimens cracked under dynamic load showed greater corrosion current values when they were compared to specimens cracked under static load and uncracked specimens. This result was expected since the main crack sizes for these specimens were greater and, moreover these specimens did not only have one main crack but also many dis-

tributed microcracks. Another reason for having high corrosion current for specimens cracked under dynamic loading may be explained as follows. Dynamic loading may cause a greater detachment of bond between the aggregates and cement paste, which result in a higher penetrability of aggressive ions into the concrete (Jaffer and Hansson, 2009).

The general results could be summarized as follows:

- (i) The relationship between water-to-cement ratio and corrosion rate of steel bars in cracked concrete is observed to be more important than the relationship between crack width and corrosion rate for the first 15 weeks of exposure. This result is also found by both Mohammed and his friends, and Otieno and his friends (Mohammed *et al.*, 2001; Otieno *et al.*, 2010).
- (ii) Contrary to ACI 318-08 suggesting that only crack widths greater than 0.4 mm pose a corrosion risk in reinforced concrete, this study has shown that it is not possible to have such a universal limiting crack width for all concrete types. In this study it is seen that, even for smaller crack openings (<0.4 mm), corrosion rate is negatively affected in the existence of cracks. Concrete quality is a major factor even in the presence of cracks. Therefore, it must be taken into consideration when setting corrosion-related width limits (Otieno *et al.*, 2010).
- (iii) Corrosion rate generally increased with the increasing crack width in the following order: uncracked $<$ specimens cracked under static load $<$ specimens cracked under dynamic load.
- (iv) Steel fiber use decreased corrosion rates when water-to-cement ratio is low, as it decreases the crack widths. This may be attributed to greater bond provided by fibers between the aggregates and fiber matrix.
- (v) Dynamic loading caused more degradation in the aggregate - cement matrix, and aggregate - steel fiber interface. Greater crack openings and increased number of microcracks led to higher corrosion rates.

This study is a part of a TUBITAK (Project code: 110M264) project and corrosion measurements will continue until the end of the project. In this study, corrosion

measurements were carried out for cracked and uncracked specimens for about 4 months of exposure and these measurements will be carried out with the increasing number of wetting and drying cycles, as durability properties of specimens will be expected to change with time.

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