

Evaluation of Corrosion Detection Methods in Reinforced Concrete Structures

by

Aynsley Griffin

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Abstract

The objective of this thesis is to conduct a comprehensive comparative analysis of corrosion rate as determined by non-destructive field-applicable techniques and corrosion rate as determined by destructive gravimetric testing as per ASTM G-1. This is accomplished by a two-part experimental program: (1) a laboratory study that monitors the corrosion of reinforcement in reinforced concrete slabs subjected to an accelerated deterioration in a controlled environment, and (2) a field study based on the condition assessment of the deck of the North Channel Bridge, an existing reinforced concrete structure located in Cornwall, ON. Non-destructive techniques analyzed include half-cell potential, linear polarization resistance, galvanostatic pulse technique, connectionless electronic pulse response analysis and four-point electrical resistivity. Generally, corrosion rate as determined by NDT measurements is in agreement, to the same order of magnitude, with the average corrosion rate determined by destructive means. Measurements using the galvanostatic pulse technique are in better agreement with gravimetric measurements than those obtained from the connectionless electronic pulse response analysis and linear polarization technique. Electrical resistivity measurements proved to be a reliable indicator of the concrete condition; these are in agreement with values reported in the literature. From the results of this research, it is concluded that the industry practice of using half-cell potential measurements in the field should be complemented with other NDT techniques, such as electrical resistivity or connectionless electronic pulse response analysis, to obtain a more accurate assessment of the corrosion activity in a reinforced concrete structure undergoing reinforcement corrosion.

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Notations

<i>CEPRA</i>	Connectionless Electrical Pulse Response Analysis
<i>E</i>	Potential (V)
<i>E_{corr}</i>	Free corrosion potential [V]
<i>ER</i>	Electrical Resistivity
<i>GPT</i>	Galvanostatic polarization technique
<i>I_{corr}</i>	Corrosion current [A]
<i>i_{corr}</i>	Corrosion current density [$\mu\text{A}/\text{cm}^2$]
<i>LPR</i>	Linear polarization resistance
<i>R_p</i>	Polarisation resistance [$\text{k}\Omega\text{m}\times\text{cm}^2$]
<i>R_{Ω}</i>	Ohmic concrete resistance [Ωm]
<i>T</i>	Temperature [C]
<i>t</i>	Time [s]
<i>w/c</i>	Water/cement-ratio
<i>ρ</i>	Concrete electrical resistivity [$\text{k}\Omega\text{m}\times\text{cm}$]
<i>NDT</i>	Non-Destructive Testing

1.0 Introduction

1.1. Motivation

Due to its versatile nature, reinforced concrete is a well-used construction material. From a global perspective, concrete is the most widely used construction material, approximately 12 billion tonnes are manufactured annually (Broomfield J. , 2007). Reinforced concrete has consistently proven to be a durable and strong construction material, resulting in reinforced concrete structures with an expected service life typically greater than 50 years. However, if the reinforced concrete structures are poorly designed, poorly constructed, and/or subjected to a harsh environment with little to no protective measures, deterioration is likely, resulting in shorter service life's and necessary repairs.

Corrosion of reinforcing steel within concrete is a significant global problem. Economically, the global cost of corrosion is estimated to be \$2.5 trillion USD, which is equivalent to 3.4% of the global Gross Domestic Product, according to 2013 figures (NACE, 2016). The most common cause of deteriorated reinforced concrete structures in Canada is de-icing salts. In Canada alone, it has been estimated that on average 5 million tonnes of de-icing salts are used on Canadian roadways annually (Environment Canada, 2012). Reinforcement corrosion, by de-icing salts, in such concrete structures on roadways leads to the following structural concerns, in sequential order: cross sectional loss to the reinforcing steel, de-bonding between the reinforcement and concrete, cracking and spalling of concrete, and finally, reduction in strength and integrity of the structure. This process is relatively slow and symptoms of reinforcement corrosion are easily noticeable thorough condition assessment surveys. This enables engineers to monitor the progress of deterioration, maintain the integrity of the reinforced concrete structure throughout its usable service life, and most importantly, ensure the health and safety of those occupying the structure.

There is an academic and industry wide interest in the prevention of reinforcement corrosion either through improved design criteria or use of improved reinforcing materials. While this is an important endeavor for the future of reinforced concrete, there is an urgent need now for reliable methods to determine the severity of reinforcement corrosion within existing concrete structures and subsequently, determine their remaining service life. This is especially true in the construction industry of the 21st century, as the current environmentally friendly trend is to not build new concrete structures but to extend the service life of existing ones.

The main goal of this study is to continue in this academic search for more reliable methods of measuring the level of reinforcement corrosion in concrete, in the hopes of improving upon current understandings

and subsequently providing practicing engineers with a method to accurately monitor and assess the condition of existing reinforced concrete structures.

1.1. Objective and Scope of Research

The current industry standard to assess the condition of reinforced concrete structures typically includes hammer sounding of the concrete and half-cell potential in-situ measurements. To improve assessments there is a need to complement half-cell potential measurements with in-situ electrical resistivity and corrosion rate measurements.

The purpose of this thesis is to conduct a comprehensive comparative analysis of corrosion rate as determined by non-destructive field-applicable techniques and corrosion rate as determined by destructive gravimetric testing as per ASTM G-1. Additionally, use of half-cell potential and electrical resistivity measurements are used to further the analysis. This is accomplished in two parts: (1) a condition assessment on the North Channel Bridge, an existing reinforced concrete structure, located in Cornwall, ON, and (2) a comprehensive assessment on reinforced concrete slabs that were constructed and tested in a controlled laboratory environment. It should be noted that part of the in-situ and in-lab testing were executed in unison.

The research presented here utilizes well-known techniques to measure the instantaneous corrosion rate and one to measure the resistivity of concrete: Galvanostatic Pulse Technique (GPT), as measured by Germann Instrument's GalvaPulse, Linear Polarization Technique (LPT), as measured by James Instrument's Gecor 6, Connectionless Electrical Pulse Response Analysis, as measured by Giatec Scientific's iCOR, and finally, four-point electrical resistivity, as measured by Proceq's Resipod Resistivity Meter. In addition, several laboratory tests were utilized to provide supporting information on the reinforced concrete: moisture content and absorption, concrete compressive strength, reinforcing steel mass loss, concrete carbonation, and concrete chloride content.

1.2. Outline of Thesis

Chapter 2 presents a comprehensive literature review of the most relevant topics on reinforcement corrosion in concrete and its detection in in-service structures.

Chapter 3 presents the experimental program designed to provide corrosion-related data obtained in a laboratory-controlled environment and correlate it to that obtained from an existing in-service structure.

Chapter 4 presents the data collected from the laboratory study and field study. Simultaneously, the experimental data is compared and trends are discussed in detail.

Chapter 5 summarizes the research findings and provides suggestions for future work.

2.0 Literature Review

This chapter provides a background on the theory of reinforcement corrosion and methods used to measure corrosion rate. It is important to note that this chapter does not provide a complete background on all information related to reinforcement corrosion and measurement techniques. However, it attempts to provide a comprehensive look of a few techniques and how these techniques perform in-situ and in-lab.

2.1. Corrosion of Steel in Concrete

Concrete is an alkaline environment. It has microscopic pores containing high concentrations of soluble calcium, sodium, and potassium ions. In the presence of water, during the hydration of cement, hydroxides are formed, raising the pH of the concrete to around 12.5-13. This alkaline condition causes the formation of a passive and protective layer surrounding the steel reinforcement embedded in concrete. Along with other minor chemical protectors, this layer protects the steel reinforcement from oxidation (corrosion). In an uninterrupted environment, this passive layer will maintain and repair itself (Broomfield, 2007). However, due to the porous nature of concrete, unwanted gases, liquids, and substances penetrate into the concrete. These unwanted and mostly aggressive agents that are found within the environment, break down this passive layer and corrosion is initiated. While carbonation is presented in this literature review, the focus of this thesis is on chloride-induced reinforcement corrosion.

2.1.1. The Corrosion Process – Electrochemistry

It is important to understand corrosion of steel is an electrochemical process, i.e., chemical reactions produce electricity. The electrochemical mechanism of corrosion can be broken down into four partial processes: (1) oxidation of iron (anodic reaction) which releases electrons and corresponds to the formation of iron ions whose hydrolysis produces acidity, (2) reduction of oxygen (cathodic reaction) which consumes those released electrons and produces alkalinity, (3) transportation of electrons within the metals from the anodic areas (available to transport) to the cathodic areas (able to consume), which produces an electrical current flow from the cathodic area to the anodic area, and finally (4) the flow of ions through the concrete pore solution from the anodic areas to the cathodic areas, to complete the electrical circuit (Bertolini L. , 2013). This cycle of electrochemical partial processes can be seen in Figure 2.1.

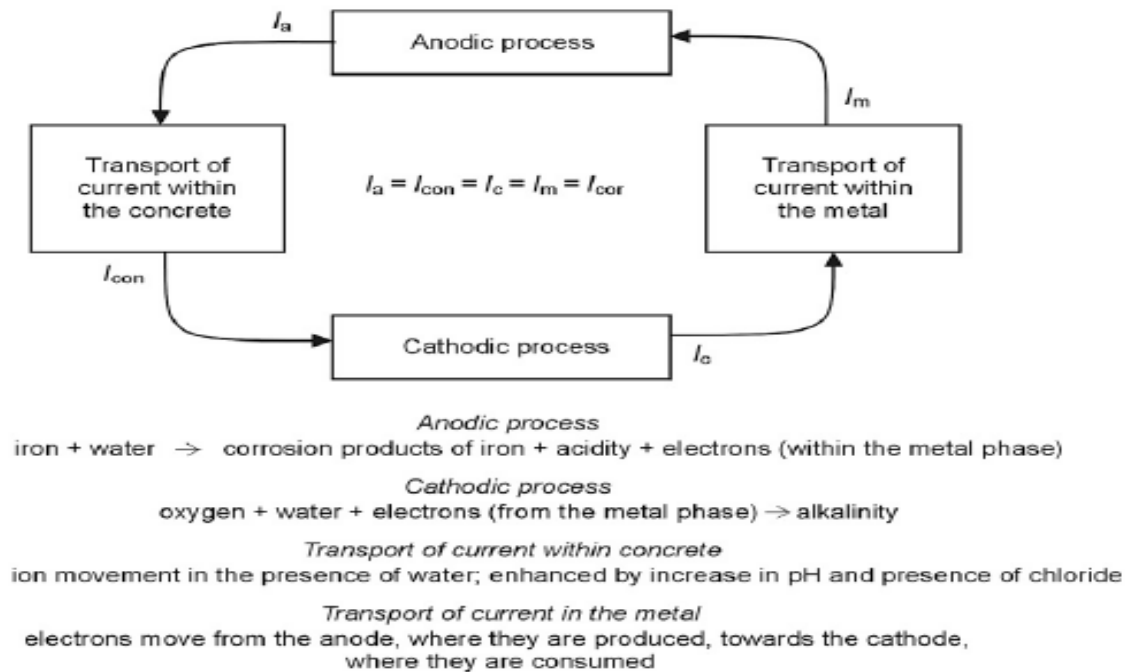


Figure 2.1: Electrochemical Mechanism of Corrosion (reproduced from Pedferri & Bertolini, 2000 as cited in Bertolini, 2013)

These four processes occur at the same rate, as shown Figure 2.1. The common value of these currents (I_{corr}), in electrochemical units, is the rate of the overall process of corrosion. The relationship between the anodic current and the corrosion rate expressed as loss of mass in time can be obtained from Faraday's first law of electrochemical stoichiometry (Bertolini L., 2013). This relationship is later discussed in Section 2.4.1. As these four processes are complementary, the corrosion rate will be determined by the slowest of the four partial processes.

It is important to define the following terms related to negligible corrosion rate:

- Passive Control; the anodic process is labored due to passive reinforcement (either due to concrete without chlorides or non-carbonated concrete)
- Oxygen diffusion control; the cathodic process is labored as the flow of oxygen to the surface of the reinforcement is slow
- Ohmic control; the electrical resistivity of the concrete is high (to be discussed in Section 2.3.5.3)

The rate of the two electrochemical reactions (anodic and cathodic) depends on the electrochemical potential. Polarization curves are used to relate potential to anodic or cathodic current density (corrosion

behavior). It is important to first discuss anodic and cathodic polarization curves for an alkaline concrete (non-carbonated and chloride free) so to establish the baseline polarization curves.

The anodic polarization curve for reinforcement in a passive state is represented in Figure 2.2. A potential of -800 to +600 mV SCE (standard calomel reference electrode) is known as the range of *passivity*. The typical range of passivity has a low anodic current of 0.1 mA/m^2 and consequently the dissolution rate of iron is negligible ($0.1 \text{ }\mu\text{m/year}$). This is due to the presence of a passive film which covers the reinforcement. When the potential is above passivity, +600 mV SCE, the reinforcement is brought to a condition of transpassivity. In this condition, oxygen may be produced on the steel surface according to the anodic reaction of oxygen evolution, which in turn produces acidity (Bertolini L. , 2013). As the anodic reaction is oxygen evolution, the passive layer is not destroyed; however, over time, the amount of acidity can be sufficient to neutralize the alkalinity of the concrete in contact with the reinforcement and thus destroy the passive layer (Bertolini L. , 2013). It is important to note that this only occurs during external polarization.

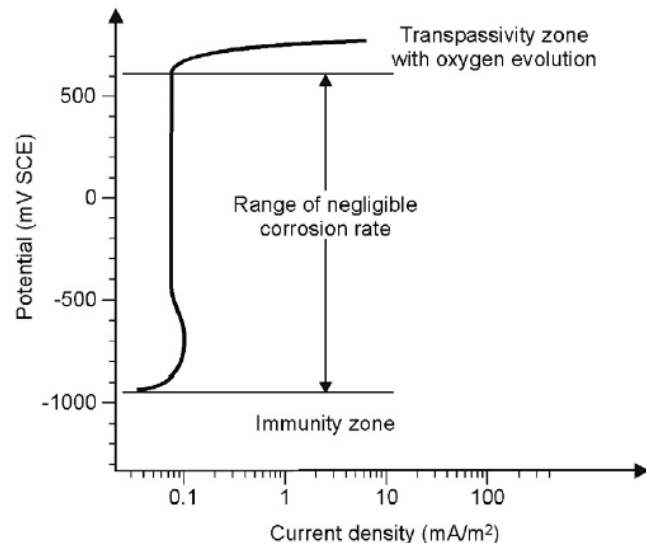


Figure 2.2: Passive State Anodic Polarization Curve (Bertolini L. , 2013)

The cathodic polarization curve to represent oxygen reduction is shown in Figure 2.3, where curve a, b, and c are represented by semi-dry concrete, submerged concrete, and oxygen absent concrete, respectively. As shown, the rate of the cathodic process is significant at potentials below 0 mV SCE and is dependent on the availability of oxygen at the surface of the reinforcement. For a given potential, the current density decreases by 2-3 orders of magnitude as the moisture content of the concrete transitions from semidry (relative humidity of 70%) to saturated (oxygen absent) (Bertolini L. , 2013). For potentials

below -900mV SCE, oxygen reduction is accompanied with *hydrogen evolution*, a process by which an electron is produced, resulting in an additional increase in the cathodic current density (Bertolini L. , 2013).

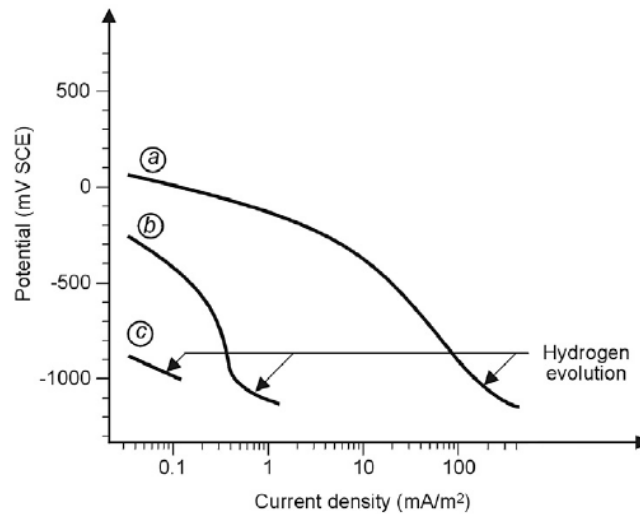


Figure 2.3: Cathodic Polarization Curves for Oxygen Reduction (Bertolini L. , 2013)

An overlay of the anodic and cathodic polarization curves addressed above can be seen in Figure 2.4. From the overlay, the conditions of corrosion of reinforcement in passive state at various situations can be determined. This overlay shows the intersection of the anodic and cathodic curves for the presented conditions, semi-dry concrete, submerged concrete, and oxygen-absent concrete (ignoring the ohmic drop effect). As can be seen, the decrease in oxygen supply results in a decrease in potential. At all conditions, however, reinforcement is subjected to a negligible corrosion rate ($<0.1 \text{ mA/m}^2$).

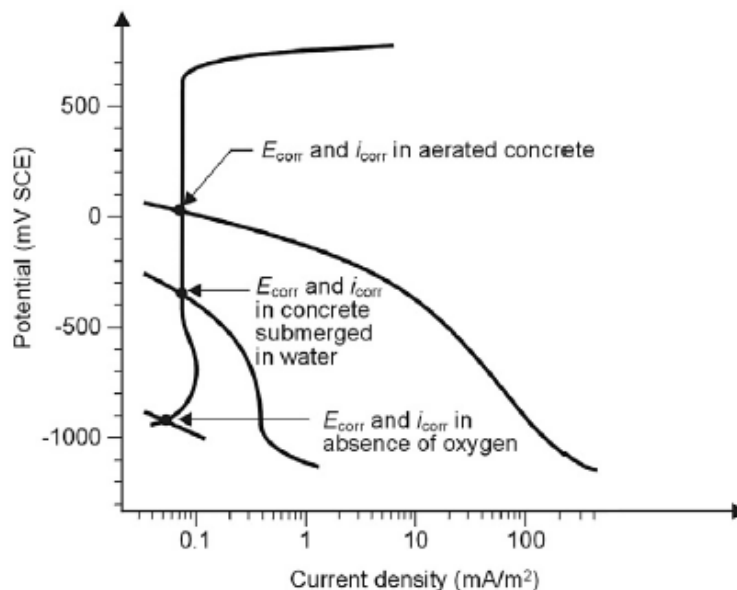


Figure 2.4: Corrosion Conditions of Passive Steel in Concrete (Bertolini L. , 2013)

The presence of chlorides in concrete alter the anodic behaviour of steel, thus modifying the anodic polarization curve previously discussed, see Figure 2.5. The anodic polarization curve, in the presence of chlorides, introduces an upper limit (denoted E_{pit}), the *pitting potential* or *breakdown potential*, which decreases as the chloride content increases. This upper limit defines this pitting potential range from -500 mV SCE for high chloride content to +600 mV SCE for uncontaminated concrete (Page & Treadaway, 1982).

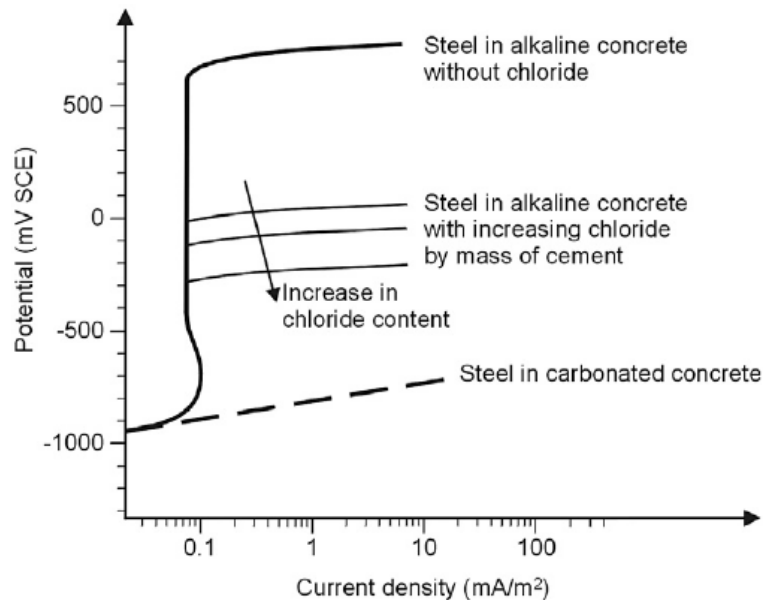


Figure 2.5: Anodic Polarization Curve with Differing Chloride Contents (Bertolini L. , 2013)

The breakdown potential provides a boundary at which chloride ions interact differently with the reinforcement. The presence of chloride ions produces a localized breakdown of the passive layer for potentials more positive than the breakdown potential. At a certain point, the chloride content reaches compatibility with the condition of passivity, this is defined as the *critical chloride content* or *chloride threshold* (see Section 2.2.1.4). This threshold is typical within the range of 0.4% to 1% of the cement content (Bertolini L. , 2013). The relationship between potential of reinforcement and critical chloride content is expressed by Pedefferri's Diagram. See a simplified version of Pedefferri's diagram in Figure 2.6. As seen, the areas of pitting corrosion (potential breakdown) and passivity are shown. The division between pitting corrosion and passivity is considered to represent the chloride threshold for a given steel potential or vis versa. It is important to note that this correlation has not been perfected by experimental research, as such variability exists (Bertolini L. , 2013).

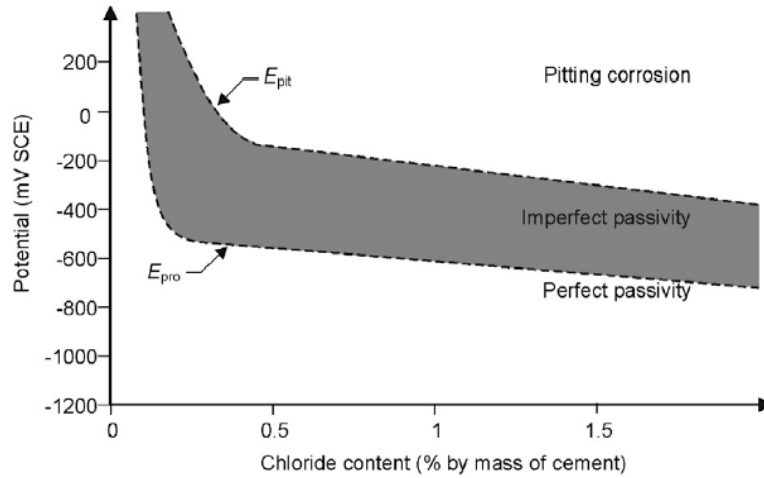


Figure 2.6: Simplified Pedefferri's Diagram (reproduced from Pedefferri, 1993 as cited in Bertolini, 2013)

The principles behind polarization curves, as they relate to corrosion of steel in concrete, will be seen later in Section 2.3 when reviewing electrochemical condition evaluation techniques.

2.1.2. Common Chemical Reaction of Corrosion

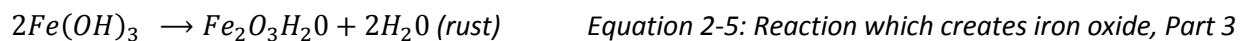
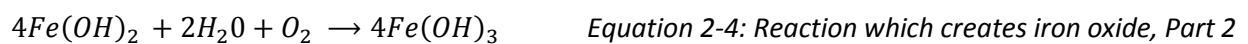
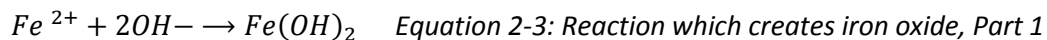
The most common chemical reactions that can occur during this electrochemical process are described below. As steel in concrete corrodes, it dissolves in the pore water and gives up electrons as seen in the anodic reaction below:



The released electrons must be consumed elsewhere to preserve electrical neutrality of the system. This is transformed into the cathodic reaction:



Hydroxides are produced with the consumption of water and oxygen. If this was the end of the chemical process, then the resulting hydroxyl ions would strengthen the passive layer with an increase in local alkalinity (Broomfield, 2007). However, the ferrous ion from the anodic reaction combines with the hydroxyl ion from the cathodic reaction to start a chain reaction that creates rust (iron oxide):



The resulting rust takes up significantly more volume of the steel it replaces (Broomfield, 2007). This volume increase exerts expansive stresses on the surrounding concrete which results in cracking and spalling of the concrete cover. Once these cracks reach the surface, it allows easier transport of water, oxygen, and chlorides to the reinforcement within, subsequently feeding the corrosion process. An illustration of this phenomenon can be seen in Figure 2.7.

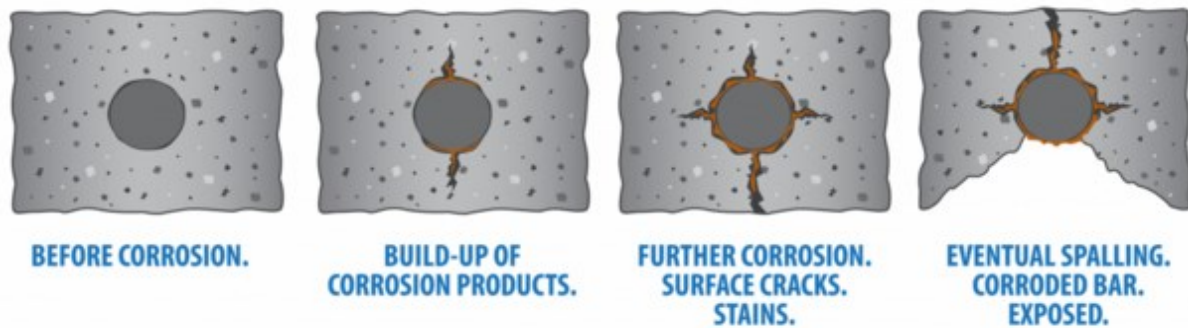


Figure 2.7: Corrosion of Steel Reinforcement in Concrete (reproduced from Carbonation of Concrete (Corrosion), 2010, website)

It is important to note that the above anodic and cathodic reactions are the most common. There are many different anodic and cathodic reactions combination that depend on the pH of the cement paste and the availability of oxygen (ACI Committee 222, 2001). This is shown in the Pourbaix diagram for iron in an aqueous solution as seen in Figure 2.8, which outlines the thermodynamic areas of stability for various reactions as a function of electrochemical potential, since the rate of anodic and cathodic processes depends on the electrochemical potential, as discussed previously.

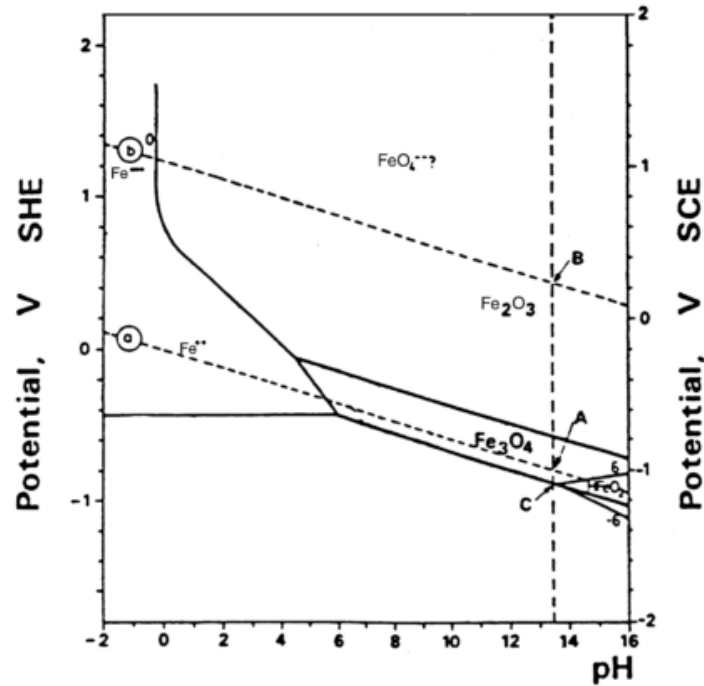


Figure 2.8: Pourbaix Diagram (reproduced from Pourbaix, 1974 as cited in ACI Committee 222 2001)

The iron oxides Fe_3O_4 and Fe_2O_3 , shown above within the region of sound concrete, pH range of 13-13.5, form as solid phases and may develop in the passive layer. A decrease in pH, illustrated as a leftward shift in the Pourbaix diagram, shifts the reactions from anodic to cathodic, leading to the formation of corrosion products. Two examples of this in the above Pourbaix diagram: the cathodic reaction, $2\text{H}_2\text{O} + \text{O}_2 + 4\text{e}^- \rightarrow 4(\text{OH})^-$, which occurs when the potential is below the upper dashed line, and the cathodic reaction, $2\text{H}^+ + 2\text{e}^- \rightarrow \text{H}_2$, which occurs when the potential is below the lower dashed line (ACI Committee 222, 2001).

2.2. Causes of Corrosion of Steel in Concrete

The main causes of corrosion of steel in reinforced concrete are carbonation of the concrete cover and chloride attack. As chloride attack is the main corrosion mechanism looked at in this research, only an overview of this process is presented below.

2.2.1. Chloride Attack

Chloride affects reinforced structures by means of chlorides cast into the concrete, either through contaminated aggregates, use of seawater, or accelerators, or by means of chlorides diffused into the concrete from environmental factors, either through de-icing salts, chemicals, or contact with seawater. As the concrete is an initially alkaline environment, these aggressive agents penetrate the concrete and as the concentration of the chloride ions reach a threshold value, the passive layer breaks down (Bertolini

L. , 2008). It is important to note that corrosion due to chloride attack is known as a self-feeding process which makes it more severe over time.

2.2.1.1. Service Life of a Reinforced Concrete Structure Subjected to Corrosion

Since the first service life diagram idealized by Tuutti (1982), the corrosion of reinforcing steel has been considered as a process with two main phases. The *initiation phase* is when the reinforcement as a whole is passive but environmental attacks can lead to some loss of passivity. At the end of this phase, corrosion begins (steel becomes depassivated). When the passive layer breaks down, the *propagation phase* begins. This phase continues until the effects of corrosion can no longer be tolerated by the reinforced structure (Bertolini L. , 2013). An illustration of the periods in the service life of a reinforced structure can be seen in Figure 2.9.

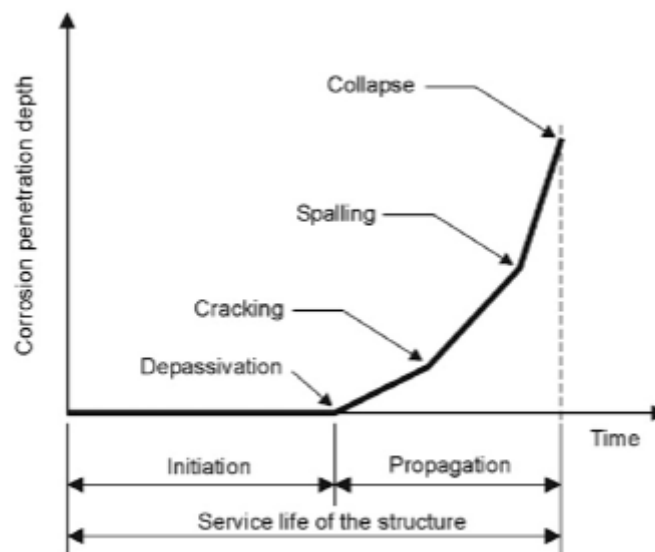


Figure 2.9 Service Life Periods (modified from Tuutti, 1982 as cited in Bertolini, 2013)

2.2.1.2. Chloride Penetration

The *initiation phase* corresponds to the period when the reinforcement as a whole is passive but environmental attacks can lead to some loss of passivity. The initiation period for chloride attack depends on the rate of chloride penetration, the chloride threshold value, and the thickness of the concrete cover (Bertolini L. , 2008).

As the aggressive agents penetrate through the concrete, it is common that the chlorides follow a concentration profile similar to that seen in Figure 2.10. In Figure 2.10, the classical diffusion curve is shown, as a function of chloride depth by mass percent of cement. It shows that chloride concentration is the highest at the concrete surface and decreases with depth (Broomfield, 2007). It is important to note

that most reinforced structures follow a more complicated and erratic penetration curve due to a combination of other ingress mechanisms in addition to diffusion, but the general concept of decreasing chloride concentration with increased cover thickness remains the same.

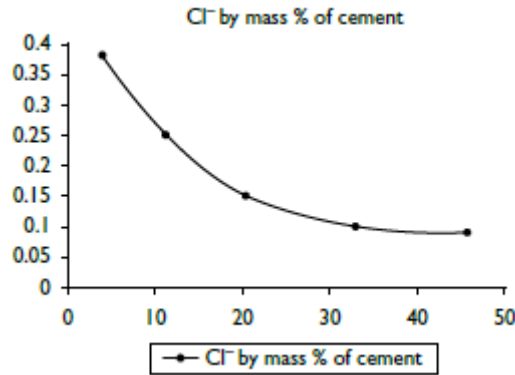


Figure 2.10: Classical Chloride Diffusion Curve (Broomfield J. P., 1992; note: x-axis represents concrete depth in mm, while y-axis represents the chloride content)

The rate of chloride ingress is affected by many mechanisms including diffusion, capillary suction, permeation, migration mechanisms, and chemical reactions that occur within the concrete that hinders the chloride movement (Bertolini L. , 2008). Understanding the penetration of chloride ions is a matter of selecting the relevant mechanisms of chloride penetration, finding an appropriate value of the parameter describing the rate of penetration, and finally calculating the evolution of the chloride concentration in time as it propagates through the concrete depth. Unfortunately, determining the true chloride penetration profile is more complicated than that, because all transport mechanisms are dependent on the complex concrete microstructure. In addition, chloride transport usually involves a combination of several transport mechanisms (Bertolini L. , 2008).

However, in general, the chloride profile can be reasonably described by an analytical closed-form solution of Fick's 2nd law of diffusion, as seen in Eq. 2-6:

$$C(x, t) = C_s \left[1 - \operatorname{erf} \left(\frac{x}{2\sqrt{D_{app}t}} \right) \right] \quad \text{Equation 2-6: Fick's 2}^{nd} \text{ Law of Diffusion}$$

where $C(x, t)$ is the chloride concentration at depth x and time t , D_{app} is the apparent diffusion coefficient taken from experimental data, and C_s is the surface chloride content, which is dependant on the composition of the concrete, the position of the structure, and the orientation of its surface and micro-environment, the chloride concentration in the environment, and finally the general conditions of

exposure (Bertolini L. , 2008). This equation represents nonstationary diffusion and several assumptions are made:

- the concentration of the diffusing substance on the concrete surface is constant with time
- the coefficient of diffusion does not vary with time
- the coefficient of diffusion does not vary with depth
- there are no chlorides initially

A chloride profile can be reasonably described with Fick's Second Law of Diffusion after fitting the error function to experimental data to find suitable values for C_s and D_{app} (Bertolini L. , 2008). It is important to note that this reasonable description only applies solely with diffusion. The apparent diffusion coefficient, D_{app} , is often used as a durability indicator to determine the performance of concrete against chloride ingress (Lataste J.-F. , 2010). The rate of diffusion of chlorides from the environment is dependent on the following factors: the water-to-cementitious materials ratio, type of cement, the temperature, the maturity of the concrete, saturation level, and the specific cation associated with the chloride (ACI Committee 222, 2001).

2.2.1.3. Chloride Binding

It is important to note that not all chlorides present in the concrete contribute to corrosion. The term "total chlorides" refers to all chlorides in the concrete. From the total chlorides, a portion of these chlorides become bound to constituents of the cement paste to form "bound chlorides". For example, chlorides react with calcium aluminates to form calcium chloroaluminates (ACI Committee 222, 2001). Another portion of the chlorides becomes physical trapped by unconnected pores or by absorption (Bentz, Evans, & Thomas, 1996 as cited in ACI Committee 222, 2001). The final portion of the chlorides available in the pore solution that contribute to the corrosion process are termed "free chlorides". The following factors attribute to the amount of free chlorides present: pH, water-to-cementitious material ratio, tricalcium aluminate and tetracalcium aluminoferrite contents, and finally whether the chlorides were added to the original mix or entered the concrete from the environment (ACI Committee 222, 2001).

There is an ongoing discussion on whether chlorides that are dissolved in the pore solution are only involved in corrosion, while chlorides bound to constituents of the cement paste are not. It is important to note that in practice, total chloride content is easily measured, so for this reason, a chloride threshold value is expressed as critical total chloride content, where critical total chloride content is expressed as a percentage of chlorides with respect to the mass of cement (Bertolini L. , 2013).

2.2.1.4. Threshold Value

Chlorides can be present in the concrete while the passive layer is intact. With that said, at what point does the percentage of chlorides within the concrete initiate pitting corrosion? This concentration is the chloride threshold, defined as the chloride content required for depassivation (the transition from corrosion initiation to corrosion propagation) (Bertolini L. , 2013).

Several correlations have been linked to the chloride threshold: the concentration of hydroxyl ions, the electrochemical potential of the steel, and the presence of voids at the interface of the reinforcement and concrete (Bertolini L. , 2013). In terms of chloride/hydroxyl ratio, the chloride threshold is defined when the chloride concentration exceeds 0.6 of the hydroxyl concentrations (Hausmann, 1967). This phenomenon is illustrated in Figure 2.11, where experimental results are related to predictions by a Monte Carlo simulation.

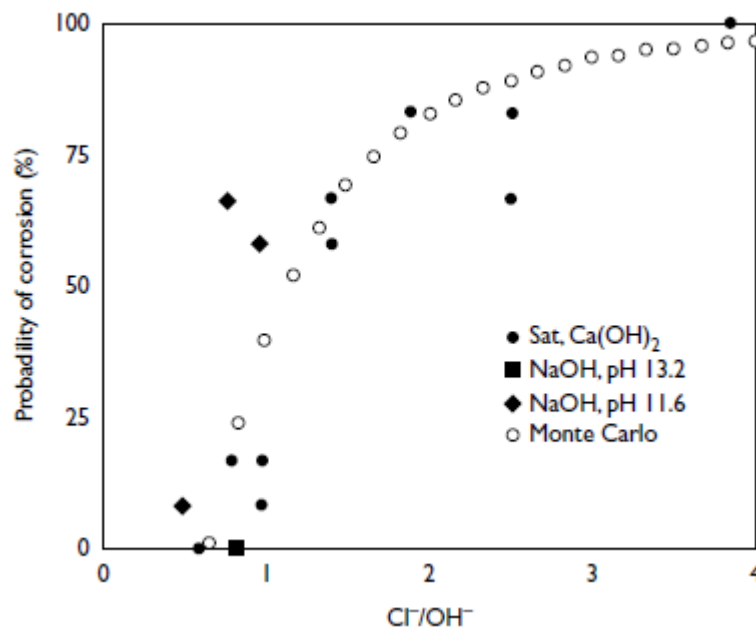


Figure 2.11: Chloride/hydroxyl ratio as Related to Probability of Corrosion (reproduced from Hausman, 1967 as cited in Broomfield, 2007)

A similar relationship is illustrated in Figure 2.12, which shows the general relationship between the corrosion rate and the molar ratio of chloride and hydroxides. It is important to note that the hydroxyl ion concentration depends on the type of cement and admixture. Therefore, it has been summarized that threshold values based on chloride/hydroxyl ions should be defined by statistical data and not by a single value, as true for any threshold value (Bertolini L. , 2013).

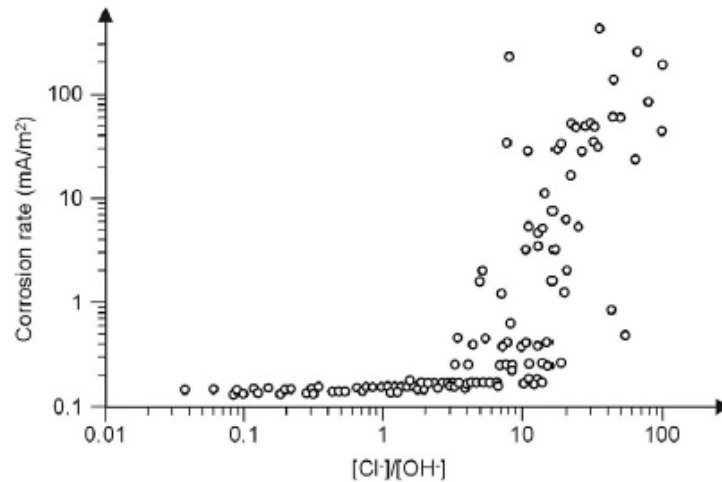


Figure 2.12: Corrosion Rate vs. $[Cl^-]/[OH^-]$ (reproduced from Page, Lambert, & Vassie, 1991 as cited in Bertolini L. , 2013)

The electrochemical potential of steel is mainly related to the pH of the pore solution, availability of oxygen at the surface of the steel, and thus to the moisture content of the concrete. It has been found that the chloride threshold increases and the potential of the steel decreases, by more than one order of magnitude (Bertolini, et al., 2009). Refer to Figure 2.6, which illustrated this relations as it is expressed by Pedefferri's diagram.

In addition, chloride threshold is dependent on the presence of voids as poor compaction of the concrete may weaken a layer of cement hydration products at the steel surface leading to local acidification that favours sustained propagation (Page, 2009). Experimental results have found that decreasing the volume of entrapped air at the interface of steel to concrete from 1.5% to 0.2% (by volume) results in an increase of the chloride threshold from 0.2% to 2% (by mass of cement) (Glass & Buenfeld, 1997)

2.2.1.5. Corrosion Propagation

Once the chlorides have reached the reinforcement surface in sufficient quantities, overpassing a critical total chloride threshold, the protective passive layer is broken down. The two most common results of chloride attack are microcell and macrocell formation. Microcell formation, commonly referred to as localized chloride attack or pitting, occurs when the high levels of chlorides are concentrated on a reasonable small area of reinforcement (Bertolini L. , 2008). This area, no longer protected by the passive layer, becomes the active zone or anode. The unaffected surrounding area, on the other hand, acts like the passive zone or cathode, refer to Figure 1.13 (Bertolini L. , 2013).

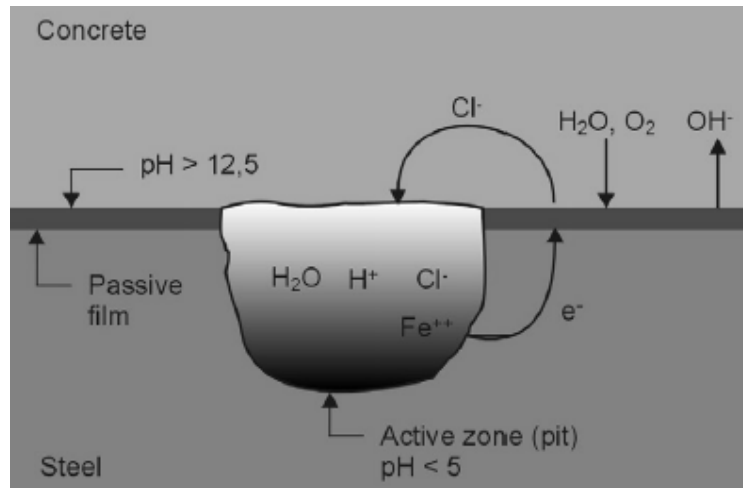


Figure 2.13: Schematic Representation of Pitting (Bertolini L. , 2008)

Electrical current flow from the anodic areas to the cathodic areas increases the chloride content, followed by a decrease in the alkalinity. On the other end, the passivity later in the cathodic region is strengthened since the chlorides, being negatively charged, tend to move towards the anodic area (Bertolini L. , 2013). This stabilization accelerates the corrosion process, leading to drastic cross-sectional loss of the reinforcement, and it may have dangerous consequences to the structure if left unattended.

When there is a separation between the anodic and cathodic regions, seen when there are small corroded areas followed with significantly larger areas of non-corroded reinforcement, macrocell formation occurs. For ions to transport between such large distances, high levels of moisture within the pore structure of the concrete must be present, as seen in chloride attack (Broomfield, 2007). An overall increase in the corrosion rate on the active steel is a strong indication that macrocell formation has occurred (Bertolini L. , 2008).

2.3. Concrete Condition Evaluation

Evaluation of any reinforced concrete structure usually involves two steps:

1. Preliminary survey to define the nature of the problem
2. Detailed survey which confirms the nature of the problem and determines the extent of the problem.

It is important to note that this research closely examines one possible deterioration mechanism, chloride-induced corrosion. However, there are far more mechanisms found, including but not limited to: alkali-silica reactivity, sulphate attack, Thaumasite attack, freeze thaw, delayed ettringite formation, thermal movement and settlement (Broomfield, 2007).

There are various techniques used in practice today that provide valuable on-site or in-lab information on the condition of a reinforced concrete structure. Figure 2.14 provides a good summary of the most commonly used methods, what each method detects, and the approximate speed at which information can be gathered. For the purpose of this review, an overview of measurements of half-cell potential and electrical resistivity follows.

<i>Method</i>	<i>Detects</i>	<i>Use</i>	<i>Approximate speed^a</i>
Visual	Surface defects	General	1 m ² s ⁻¹
Hammer/chain	Delaminations	General	0.1 m ² s ⁻¹
Cover meter	Rebar depth and size	General	1 reading in 2–5 min 50–500 m ² per day
Phenolphthalein	Carbonation depth	General	1 reading in 5 min 80–250 per day
Chloride content	Chloride corrosion	General	Core in 10 min 7–80 per day (small or large diameter) or drillings in 2 min 100–250 per day + lab or special site analysis
Reference electrode (half cell) survey	Corrosion risk	General/specialist	1 reading in 5 s after connecting to steel 50–500 per day
Linear polarization	Corrosion rate	General/specialist	1 reading in 5 to 30 min depending on equipment used
Resistivity	Concrete resistivity/corrosion risk	General/specialist	1 reading in 20 s 10 to 20 per day.
Permeability (ISAT)	Diffusion rate	General/specialist	1 reading in 5 min or core + lab 6–12 per day
Impact/ultrasonics	Defects in concrete	Specialist	1 reading in 2 min 20–30 per day
Petrography	Concrete condition, etc.	General	Core in 10 min 7–80 per day (small or large diameter) + lab
Radar/radiography	Defects, steel location, condition	Specialist	>1 m ² per sec for vehicle system or 1 m ² in 20 s for hand system + interpreting

Figure 2.14: Common Methods for Condition Surveying (Broomfield J. , 2007)

2.3.1. Reference Electrode Potential

Also known as half-cell potential mapping, reference electrode potential is the most widely accepted form of non-destructive corrosion monitoring and condition assessment that can be used before symptoms of corrosion are seen on the concrete surface. This technique follows the (C876, 2015). The concept of half-cell potential stems from the understanding that non-corroding and corroding reinforcement have a difference in corrosion potential, thus half-cell testing measures the potential distribution on the concrete surface. In order to determine the location of corroding reinforcement, a reference electrode is placed on the concrete surface, and an electric field coupled with the corrosion current between the passive and active areas of the reinforcement is measured, resulting in equipotential lines (Bertolini L. , 2013). The

half-cell device follows the concept of a simple electrical Daniell cell. A schematic of the half-cell on-site test setup can be seen in Figure 2.15.

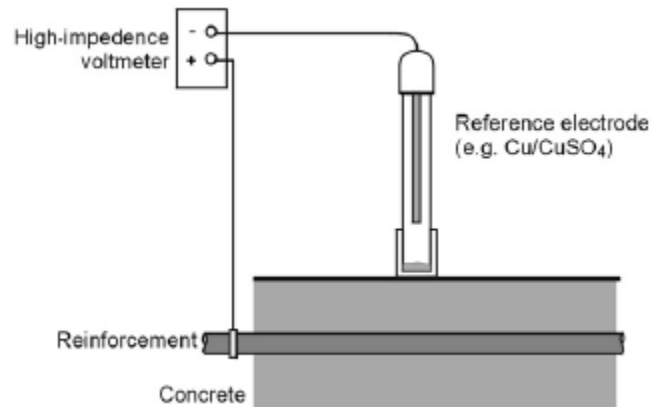


Figure 2.15: Schematic of Half-Cell Potential Setup (Bertolini L. , 2013)

The values acquired from the half-cell device depend on the reference electrode used. The most commonly used reference electrode is copper/copper sulfate (CSE). Regardless of the reference electrode used, the potential difference to the standard hydrogen electrode (SHE) is required so that the recorded data takes into account the offset due to different reference electrodes (Broomfield, 2007). It is important to note that the condition of the concrete, e.g., saturation level, drastically alters the half-cell potential readings. This must be taken into account when using half-cell potential as a method for assessing the condition of reinforced concrete. Table 2.1 provides a broad summary of the typical ranges of half-cell potentials of rebar in concrete.

Table 2.1: Typical Ranges of Half-Cell Potentials for Different Concrete Conditions (154-EMC, 2003)

Condition	Potential range (mV/CSE)
Humid, chloride free concrete	100 to -200
Wet, chloride contaminated concrete	-400 to -600
Water saturated concrete without oxygen	-900 to -1000
Humid, carbonated concrete	100 to -400
Dry, carbonated concrete	200 to 0
Dry concrete	200 to 0

In general, locations of corroding reinforcement are found when the half-cell device reads potential values in the negative potential range. Broad summaries of the potential measurements values along with the probability of corrosion are seen Figure 2.16, taken from the industry standard, ASTM C876, for reinforced concrete structures exposed to the atmosphere.

Table 2.2: Potential Measurements (ASTM C876 , 2015)

Measured Potential E (mV CSE)	Probability of Corrosion
E > -200	< 10%
-200 > E > -350	Unknown
E < -350	> 90%

It is important to note that the potential ranges provided in ASTM C876 were derived empirically from a set of specific structures. In addition, these potential ranges do not take into account all factors that influence half-cell potential readings such as moisture content, chloride content, temperature during test, carbonation of concrete, and cover thickness. There is an inverse relationship between half-cell potential and concrete cover as the potential difference between the position above the anode and a distant cathode becomes smaller with increasing concrete cover (154-EMC, 2003) Refer to Figure 2.16. For these reasons, experimental studies looking into the former mentioned factors have shown that the ASTM ranges are indeed inaccurate (Verma, Bhadauria, & Akhtar, 2014).

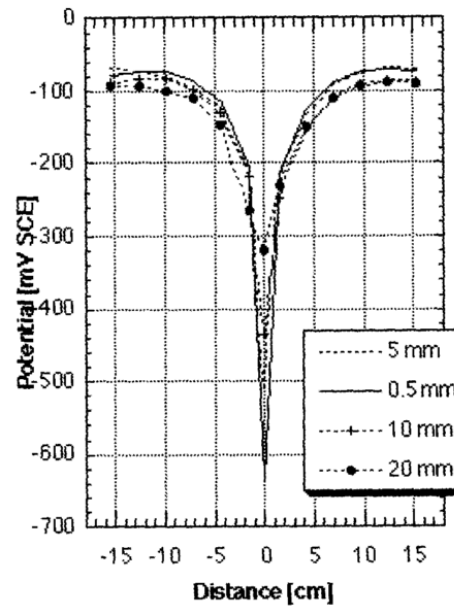


Figure 2.16: Influence of Concrete Cover on Half-Cell Potential (Elsener & Bohni, 27-31 July 1997)

Measured potential E (mV CSE)	Probability of corrosion
$E > -200$	<10%
$-200 > E > -350$	unknown
$E < -350$	>90%

Multiple studies have shown that a more comprehensive understanding of the half-cell potential data is needed to fully grasp the condition of the reinforced concrete. For example, Pour-Ghaz et. al (2009) showed that more realistic results could be retrieved from a reinforced concrete structure if availability of oxygen, resistivity, cover thickness, and the anode-to-cathode area ratio are considered. In addition, it was observed that for the same corrosion rate, half-cell potential measurements can differ with different resistivity values. More interesting, it was assessed that with an increase in cover thickness, the potential distribution on the concrete surface will not provide accurate information about the potential distribution at the concrete/steel interface (Pour-Ghaz, O. Burkan, & Ghods, 2009).

It is important to note that the ASTM C876 (2015) requires data presentation to be in the form of equipotential contour mapping or cumulative frequency distribution (C876, 2015). The latter incorporates statistics in the analysis of the corrosion data. Research into improving the half-cell test has shown that these statistical analyses of potential mapping results, which produce potential boundaries between active and passive steel, are more accurate (Bertolini L. , 2013). In addition to incorporating statistical

analysis, a good application practice for this half-cell potential technique is to develop a history for the reinforced structure by performing the half-cell potential test at regular intervals. This collection of data as it relates to time provides for a more detailed understanding of the corrosion activity (ACI Committee 222, 2001).

Half-cell potential measurements cannot be directly related to rate of corrosion but instead provide an indication of the state of corrosion (ACI Committee 222, 2001). Vast amounts of studies have shown that there are many factors that affect the half-cell potential results, and that it is important to consider the condition of the reinforced concrete at the time when the test is performed. With all this stated, it is common for engineers to expose sections of the reinforced structure that show very negative potential measurements, so that the condition of the structure is defined with greater certainty.

2.3.1.1. Potential Sources of Error

It is possible for half-cell potential drift to occur. This is defined as a half-cell potential reading that shows systematic change in potential over time (154-EMC, 2003). This could be due to changes in moisture content within the concrete. It is important to monitor the concrete surface of wet patches. Any drift less than 20 mV per minute can be ignored (154-EMC, 2003). In addition, if half-cell potential readings experience random fluctuations, this is an error and is usually due to a loss of electrical continuity (154-EMC, 2003). It is important to ensure that all contact areas are sufficiently wet, and that there is good connection to the reinforcement. It is important to note that fluctuating readings could also be due to the presence of stray currents (154-EMC, 2003).

2.3.2. Electrical Resistivity

Resistivity is defined as “a measure of the ability of an electrical current to flow within a material, and it is thus an indicator of a material’s transfer properties” (Lataste J.-F. , 2010). In mathematical terms, electrical resistivity is the ratio between applied voltage and resulting current in a unit cell (154-EMC, 2001). At its most basic, electrical resistivity (ER) is an indication of the amount of moisture in the concrete pore structure, and the tortuosity and size of that pore structure. The pore structure is related to the concrete mix properties like cement content, water/cement ratio, curing, additives used, and degree of hydration. As it relates to chlorides, chlorides encourage concrete to hold water, and therefore ER can also be an indication of chloride content (Broomfield, 2007).

It is important to note that there is a distinct difference between resistivity and resistance. Resistivity, ρ , is an inherent characteristic of a material, while resistance, R , is stated as voltage/current (from Ohm’s Law). In addition, both are related through the Eq. 2-7 for resistivity:

$$\rho = k R$$

Equation 2-7: Electrical Resistivity

where k is a geometric factor (Layssi H. , Ghods, Alizadeh, & Salehi, 2015). Units are typically given as ohm meter ($\Omega \text{ m}$). Depending on the device used, the resistivity equation may be modified to account for electrode spacing on the device or the method the device takes a measurement.

Generally, electrical resistivity values on their own provide little information on the condition of the reinforced concrete structure. However, electrical resistivity can be correlated to an estimation of corrosion rate severity (refer to Table 2.2).

Table 2.3: Resistivity Ranges for Electrical Resistivity (Giatec Scientific Inc., iCOR User Manual, 2016)

Resistivity ($\Omega \cdot \text{m}$)	Corrosion Rate Severity
>200	Low
100 – 200	Low to moderate
50 – 100	High
< 50	Very high

A more informative method is to compare electrical resistivity data on the structure in question to existing data for a similar concrete type (Polder, 2000). Figure 2.17 provides an example of existing data.

Global reference values at 20°C for the electrical resistivity of dense-aggregate concrete of mature structures (age > 10 years); conditions in square brackets are corresponding laboratory climates [29]

Environment	Concrete resistivity ρ_{concrete} ($\Omega \text{ m}$)	
	Ordinary Portland cement (CEM I)	Blast furnace slag cement (> 65% slag) or fly ash (> 25%) or silica fume (> 5%)
Very wet, submerged, splash zone, [fog room]	50–200	300–1000
Outside, exposed	100–400	500–2000
Outside, sheltered, coated, hydrophobised (not carbonated) [20°C/80%RH]	200–500	1000–4000
Ditto, carbonated	1000 and higher	2000–6000 and higher
Indoor climate (carbonated)[20°C/50%RH]	3000 and higher	4000–10 000 and higher

Figure 2.17: Concrete Resistivity Reference Values (Polder, 2000)

2.3.2.1. Factors Affecting Electrical Resistivity

Electrical resistivity is affected by a large variety of intrinsic and extrinsic factors. Intrinsic factors related to concrete microstructure include, but not limited to: porosity, water/cement ratio, aggregates, and

supplementary cementitious materials (SCMs). The most prominent extrinsic factors affecting ER are temperature and moisture content (and accompanying relative humidity). The most influencing factors are porosity and moisture. For this purpose of this review, only the factors listed above are discussed here. It is important to note that while it is well-established that the above noted factors influence electrical resistivity, the mechanism and extent of influence is not well understood and the focus of on-going research.

Porosity is the main intrinsic factor that affects ER. In general, the higher the porosity, the lower the resistivity is. As a subset of porosity, correlations can also be made for pore distribution, pore volume, pore radii, and the pore network, where pore network is defined by interconnectivity and tortuosity (Lataste J.-F. , 2010). Porosity is a good indication of the microstructure and mechanical properties of the concrete. Concrete with a vast pore network of interconnected large-volume pores decreases the ability to resist aggressive external conditions and agents, subsequently more porous concrete allows for easier flow of electrical current provided that the concrete is moist. If the concrete is dry, resistance is higher as air conducts less electricity than a cement matrix. As a result, the link between resistivity and porosity provides a reasonable indication of the level of deterioration and can be used in the evaluation of concrete condition.

Electrical resistivity and w/c are inversely related: the higher the w/c ratio, the more porous the concrete, and thus the lower the resistivity, for a moist concrete condition (Lataste J.-F. , 2010). This relationship was experimentally researched by (Project SENSO, 2009) where water saturated Ordinary Portland Cement concrete was used, resulting in resistivity at porosity of 14.2 to 18.1%. Refer to Figure 2.18.

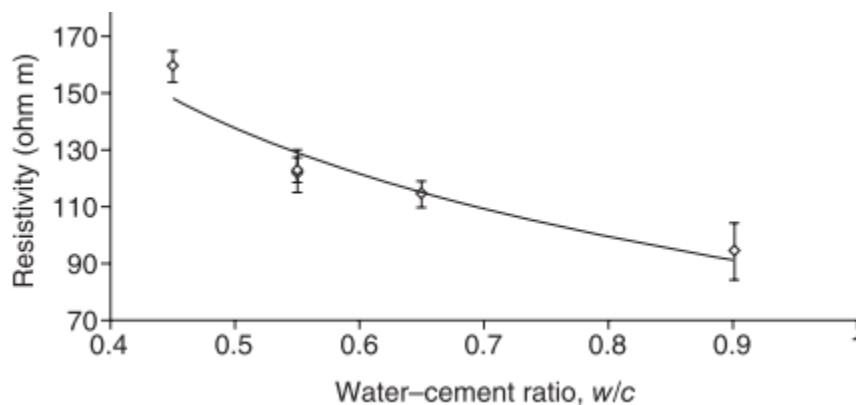


Figure 2.18: Resistivity vs. Water/Cement (Project SENSO, 2009)

Cement type is one of the more complex factors, because there are many different cement types with various chemical compositions available. At a basic level, cement is the source of chemical elements and ions that support the electrical flow, elements and ions such as Ca^{2+} , Na^+ , K^+ , OH^- , and SO_4^{2-} . It is for this reason that studies relating electrical resistivity to corrosion usually provide data in categories separated by the cement type, where such electrical resistivity ratios range from one to one hundred for multiple cement types tested under the same conditions and methods (Neville, 1996).

For the most part, aggregates have been labeled to be non-conducting particles surrounded by ionically conducting cement paste (McCarter, Ford, & Whittington, 1981 as cited in Lataste, 2010). Aggregates can be subdivided by the following properties: aggregate mass/cement mass, mineralogy, and aggregate distribution. Studies have shown that these properties have various effects on ER, but the generally accepted correlation between electrical resistivity and aggregates is that the more aggregate in a mix, the more the concrete is resistive.

Finally, supplementary cementing materials have been found to play a critical role in electrical resistivity measurements. Experimental studies have shown that, in general, electrical resistivity increases, drastically, with the presence of admixtures. This correlation has been shown experimentally where two typical admixtures, fly ash (CV) and silica fume (FS) are compared to Ordinary Portland Cement (OPC), as shown in Figure 2.19 (Lataste, Breyse, Sirieix, & Naar, 2006). The concrete strength in MPa is provided in each sample identifier, i.e., M25 is a concrete with 25 MPa compressive strength.

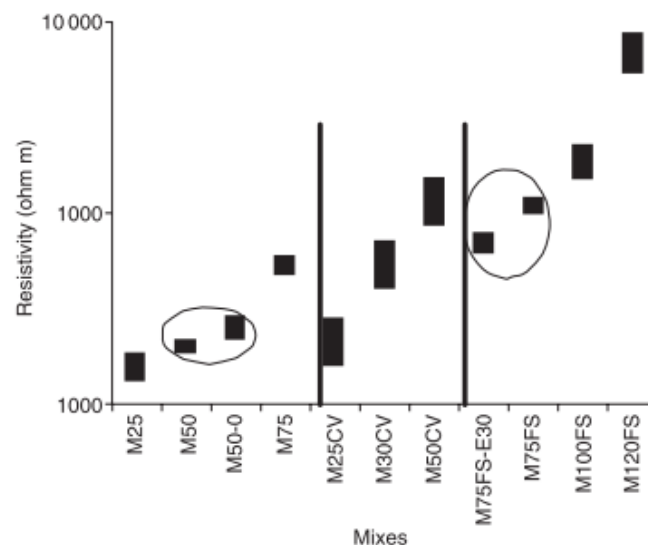


Figure 2.19: Electrical Resistivity Ranges for Concrete Mixes with and without Admixtures (Lataste, Breysse, Sirieix, & Naar, 2006)

As silica fume generally consists of particularly fine particle size and pozzolanic characteristics, the use of silica fume results in a fine pore structure and low ionic concentration within the pore structure (Berke, Scali, Regan, & Shen, 1991), resulting in a significant increase in resistance over concrete with fly ash and OPC.

As an external factor affecting electrical resistivity, changes in temperature have been found to have a drastic effect on ER. For a constant humidity, temperature (up to 40°C) and electrical resistivity are inversely related. That is to say that as one increases, the other decreases (Polder, 2000). At the microstructure level, this is due to changes in the mobility of the ions and the ion-solid interactions in the cement paste (Bertolini L. , 2013). This relationship is seen in Figure 2.20.

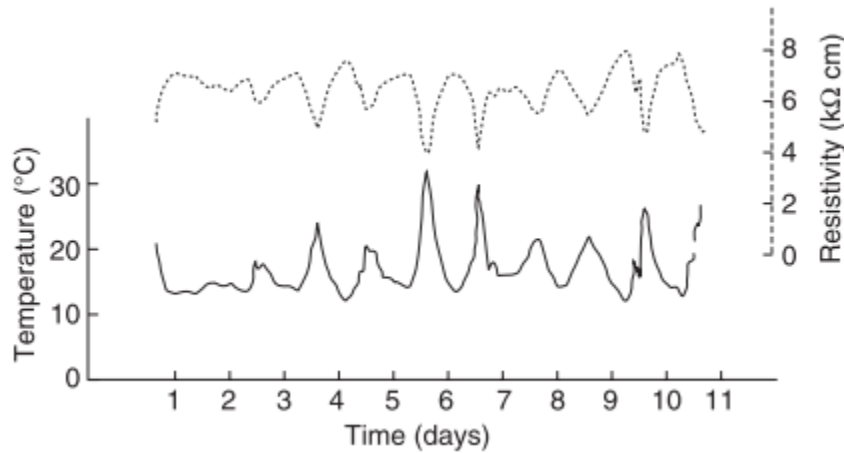


Figure 2.20: Electrical Resistivity vs. Temperature (Millard, 1991)

Arrhenius equation, Eq. 2-8, is the basic equation used to describe the relationship between temperature and ER:

$$\sigma(T_i) = \sigma(T_o) \exp \left(b \left(\frac{1}{T_o} - \frac{1}{T_i} \right) \right) \quad \text{Equation 2-8: Arrhenius Equation}$$

where: σ is conductivity (inversely related to electrical resistivity), $1/\Omega \text{ m}$, T_o is the reference temperature in K, T_i is the actual temperature in K, and b is an empirical factor found to be between 1,500 to 4,500 K. However, under steady state conditions, b may increase with decreasing relative humidity for a given concrete, cement paste, or mortar (154-EMC, 2001).

As electrical conduction through concrete is mainly electrolytic, concrete electrical resistivity is highly influenced by the presence of water content, as dictated by relative humidity (RH) of the surrounding environment. Research by Hunkeler (1996) has shown that water in concrete is absorbed and non-mobile below 40% relative humidity (or saturation rate). For conditions above this 40% threshold, electrical resistivity will decrease as there is an increase in moisture content and subsequently an increase in relative humidity, approximately one order of magnitude (Hunkeler, , 1996). This relationship is illustrated in Figure 2.21, where saturation rate is related to resistivity for OPC with varying porosity percentages. It is important to note that relative humidity is a non-steady state parameter, constantly fluctuating with environmental conditions. This makes determining the effect of relative humidity on existing concrete structures more convoluted.

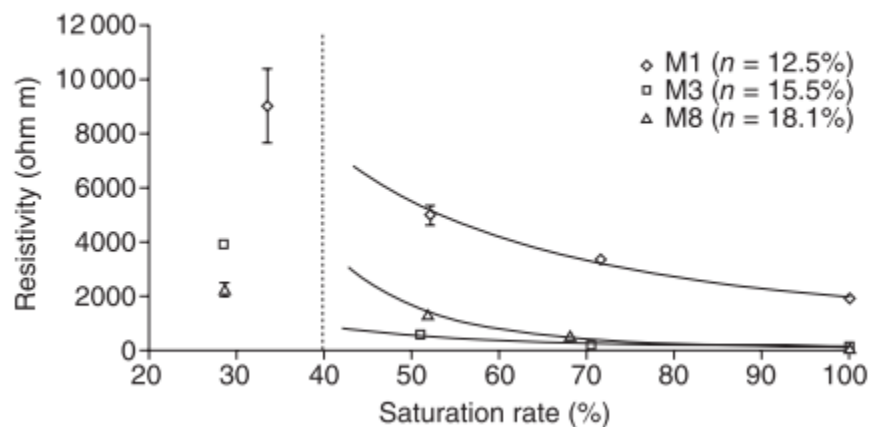


Figure 2.21: Saturation Rate vs Resistivity for OPC with varying porosity (Project SENSO, 2009)

2.3.2.2. Potential Sources of Error

As measuring electrical resistivity requires good contact between the surface of the concrete and the electrode, it must be ensured that good electrical contact is made. For most devices, this is ensured by either wetting the concrete surface, using wet sponges, using conductive gel, or a combination of these. The extent to which the contact is ensured will alter the electrical resistivity of the concrete. A thin film of water on the concrete surface or extensive prewetting of the surface should be avoided (154-EMC, 2001). On the other hand, completely dry surface should be avoided as measuring may not be possible or electrical resistivity values could be greatly skewed.

Good mechanical contact between concrete and probe should be maintained during testing to avoid variability in measurements (Baraghel-Bouny, 2004 and Newlands, Jones , Kandasami, & Harrison, 2008).

Electrical resistivity measurements taken close to edges of concrete may affect the device's interpretation of the measurements. Measurements close to edges limit the volume of concrete investigated as a concentration of current density leads to an overestimation of concrete resistivity (Lataste J.-F. , 2010). For on-site investigations, a correction should be made to measurements at edges. For laboratory testing, if the samples are similar in shape and size, this bias can be considered a constant without disruption of any relative interpretations (Lataste J.-F. , 2010).

The presence of conductive embedded metal in concrete has an effect on electrical current flow (Lataste J.-F. , 2010). Electrical resistivity is used to assess the probability of reinforcement corrosion in concrete. However, for the study of concrete exclusively, the presence of reinforcing steel must be taken into account and measurements corrected. Typically, this effect can be mitigated by taking ER measurements far from any reinforcing bars. However, in the instance when this is not possible, measurements should be taken perpendicular to the reinforcing bar (Lataste J.-F. , 2010). Computation of the influence of reinforcing bars on ER measurements is possible, however complicated (Lataste, Sirieix, Breyse, & Frappa, 2003).

2.4. Corrosion Measurements Techniques

Quantifying the rate of deterioration in a reinforced concrete structure is an engineer's best tool to determine the structure's life span, and to determine if rehabilitation (restore to proper condition) or repair (replacing of fixing parts) is needed. There are numerous techniques of measuring the corrosion rate of reinforcement, but the three that are the focus of this research are linear polarization resistance (LPR), galvanostatic pulse, and connectionless electrical pulse response analysis (CEPRA).

At the most basic level, the corrosion rate can be determined as a corrosion current by measuring the rate at which electrons are removed from the metal in the anodic reaction. This corrosion current can then be converted to a mass of steel consumed at the anode due to reinforcement corrosion by way of Faraday's Second Law, refer to Eq. 2-9.

$$m = \frac{M}{zF} I_{corr} t \quad \text{Equation 2-9: Faraday's Second Law}$$

where: m is the mass loss in grams, M is the metal molar mass (55.85 g/mol for iron), I_{corr} is the corrosion current in amperes, t is the time elapsed since the start of corrosion in seconds, z is the valency of iron formed as a result of iron oxidation ($z = 2$ for $\text{Fe} \rightarrow \text{Fe}^{2+} + 2\text{e}^-$), and F is Faraday's Constant ($F = 96,485 \text{ C/mol}$) (Bertolini L. , 2008)

For the corrosion current density, in $\mu\text{A}/\text{cm}^2$, refer to Eq. 2-10.

$$i_{corr} = \frac{I_{corr}}{\pi d_b L}$$

Equation 2-10: Corrosion Current Density

where d_b is the diameter of the reinforcement and L is the length of the reinforcement.

Faraday's Second Law assumes that the current completely passes through the electrodes. Faraday's Constant stems from the charge carried by one mole of electrons, labeled as 1 faraday (F). The faraday is related to other electrical units, since the charge on a single electron is equal to 1.6×10^{-19} C/electron. By multiplying the electronic charge by the Avogadro number, 6.02×10^{23} electrons/mole electrons, it is found that 1 F is equal to 96,485 C (Kelly & John R. Scully, 2003), or that Faraday's Constant is approximately 96,500 coulombs/equivalent mass (ACI Committee 222, 2001).

Unfortunately, the corrosion current density (i_{corr}) cannot be determined directly. Therefore, an external potential must be imposed on the system to displace it from equilibrium, and the resultant net current then measured by means of potentiostatic measurements. The difference between the original corrosion potential E_{corr} and the applied potential E is termed polarization (η). In the absence of passivity, the current density would decrease with cathodic polarization, resulting in the lower curve, and increase with anodic polarization, resulting in the upper curve, as seen in Figure 2.22. Using the following equation, it has been determined that for values of η between ± 100 to 22 mV, η is directly proportional to the logarithm of the current density (ACI Committee 222, 2001):

$$\eta = a + b \cdot \log(i_{corr})$$

Equation 2-11: Tafel Constant

where: a is a constant, and b is the Tafel slope. The corrosion current density value, i_{corr} , is obtained by extrapolating the linear part of the curves to E_{corr} as shown in the Figure 2.22 for an actively corroding system without limits from oxygen diffusion.

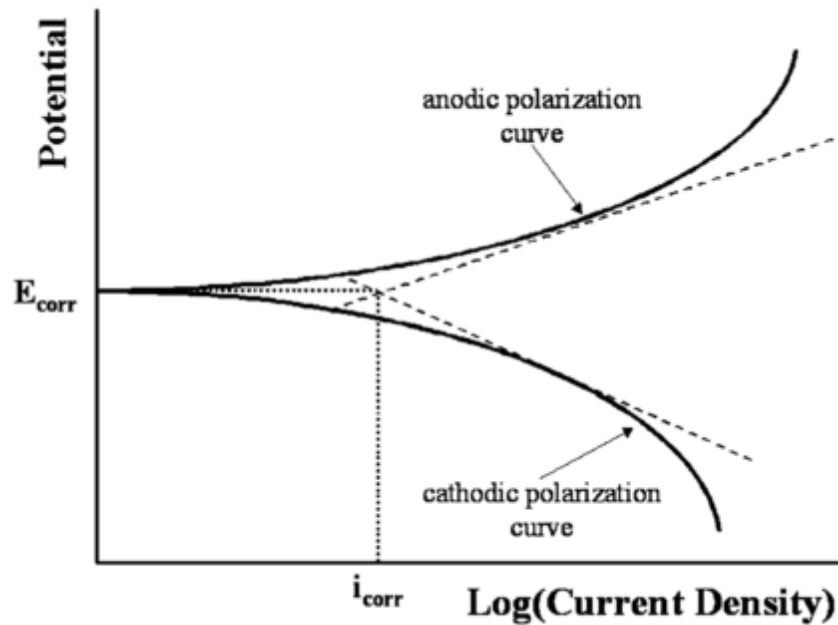


Figure 2.22: Polarization Curve (ACI Committee 222, 2001)

2.4.1. Gravimetric Mass Loss

As stated previously, during the corrosion process, there is a buildup of iron oxide content which over time results in cross-sectional loss of the reinforcement. As means of determining the average corrosion rate of reinforcing bars, gravimetric mass loss testing needs to be conducted. This test allows for the determination of the total weight of corrosion products produced. Following ASTM G1-03 “Preparing, Cleaning, and Evaluating Corrosion Test Specimens” (2017), corroded rebar specimens are subjected to several cycles in an acid bath in order to chemically remove the corrosion products. From the theoretical initial weight of the reinforcement, the total mass loss can be used to calculate the average corrosion rate. Taking Faraday’s Second Law, Eq. 2-9, for the mass of steel consumed and substituting the corrosion current density, Eq. 2-10, the equation can be rearranged to express corrosion current density in $\mu\text{A}/\text{cm}^2$:

$$i_{corr} = \frac{m z F}{M t (\pi d_b L)} \quad \text{Equation 2-12: Current Density, expressed in terms of Faraday's Law}$$

where m is the total mass loss, in grams, of removed corrosion products as determined by ASTM G1.

Taking the total mass loss and dividing by the density, the mass can be converted to thickness of the dissolved layer. Commonly seen for iron or steel, $1\mu\text{A}/\text{cm}^2 = 11.6 \mu\text{m}/\text{yr}$ (Broomfield J. , 2007). Therefore, the thickness of the dissolved layer x can be determined from:

$$x = 0.0116 i_{corr} \quad \text{Equation 2-13: Penetration Rate}$$

where: x is in units of mm/year and i_{corr} is in $\mu\text{A}/\text{cm}^2$ (Andrade & Alonso, 2004).

It is very important to note that a distinction needs to be made between the current (I_{corr}), the rate of a reaction, and current density (i_{corr}), the area-specific reaction rate. The current has units of amperes and represents an electrical flux, whereas the current density has units of amperes per unit area or A/cm^2 (Kelly & John R. Scully, 2003).

For the purpose of this review, it is important to highlight that this study determined the total mass loss of reinforcing bars subjected to corrosion-inducing environment in both a controlled laboratory environment and an un-controlled site environment. The total mass loss for the corroded reinforcing bars from both environments was determined by means of Faraday's Law; the average corrosion current density was calculated for the pre-determined (controlled) and estimated (site) time at which the reinforcing bars were exposed to corrosion-inducing conditions. The term "average" is used as any fluctuations to the corrosion current, by means of temperature, relative humidity, etc., that may or may not have occurred during this time are accounted for in the measurement of the total mass loss. Gravimetric data is the only reliable data that provides an accurate measurement of the amount of steel that has corroded, for both controlled and non-controlled exposure environments. However, care should be taken to ensure that the time of depassivation is accurately determined for gravimetric data calculations. For the determination of service life for existing reinforced concrete structures, this can be difficult.

It is important to highlight the differences between *average corrosion rate* (or corrosion current density) and *instantaneous corrosion rate*. As previously stated, average corrosion rate is measured by determining the total mass loss, or loss of cross-section, over a pre-determined period of time, as done using ASTM G1. The instantaneous corrosion rate (i_{corr}) is measured using electrochemical techniques (discussed in the following section) and provides the corrosion rate value at the time of measurement. The instantaneous corrosion rate is typically taken at different points during the exposure time. These values must be corrected for the influence of factors that affect corrosion current density (Bertolini L. , 2013). In a controlled environment, such influences are non-existent. To compare the average corrosion rate, as determined by ASTM G1, with instantaneous corrosion rates, these instantaneous corrosion rates need be integrated over time to provide for an average corrosion rate and thus allow for comparison (Thomas Frolund, 2002).

2.4.2. Linear Polarization Resistance Technique

The linear polarization resistance technique (LPR), steady-state, technique is based on the observation that the polarization curve close to the corrosion potential is linear (Bertolini L. , 2013). In other words, the change in potential (ΔE , in V) divided by the change in applied current density (Δi , units of A/m²), is defined as the polarization resistance (R_p), which has a slope with a linear relation. Refer to Eq. 2-14.

$$R_p = \frac{\Delta E}{\Delta i}$$

Equation 2-14: Polarization Resistance

This relationship is seen in the Figure 2.23.

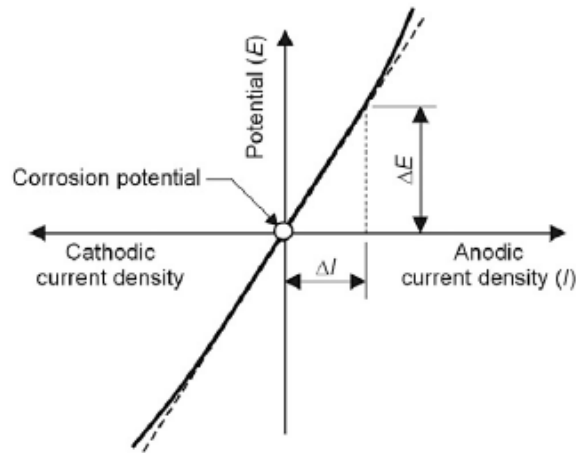


Figure 2.23: Linear Portion of Polarization Curve (Bertolini L. , 2013)

The LPR technique setup incorporates a working electrode (the reinforcement), a counter electrode (nonreactive metal), and a reference electrode. An LPR device applies a voltage to the working electrode via the counter electrode, and the corresponding voltage response is measured. Refer to Figure 2.24. The reference electrode measures the initial corrosion potential and any shift in potential of the working electrode. It is recommended that the potential sweep should not go beyond ± 12 mV on either side of the open-circuit potential to ensure that the linear relationship is held (Chabi, 2012).

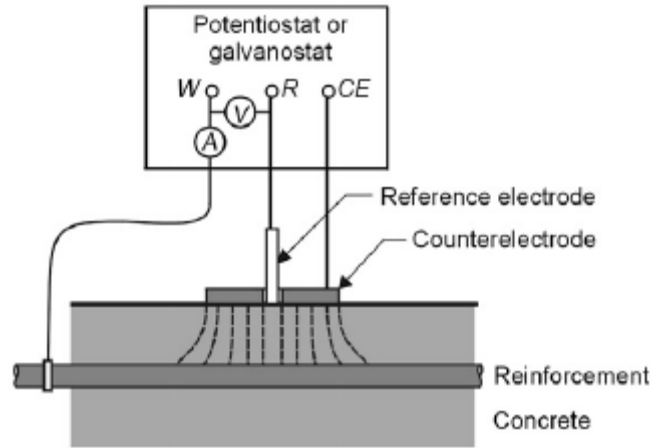


Figure 2.24: Schematic Setup for LPR (Bertolini L. , 2013)

It is important to note that the linear relationship between potential and current can be obtained using two different approaches: potentiostatic sweep and galvanostatic sweep. Potentiostatic sweep increases the voltage with a constant sweep rate while recording the current. Galvanostatic sweep increases the current with a constant sweep rate while recording the potential (D.W. Law, 2000).

The polarization resistance (R_p) is inversely related to the corrosion current density (i_{corr}) as seen in the Stern-Geary equation Eq. 2-15 (Stern & Geary, 1957):

$$i_{corr} = \frac{K}{R_p} \quad \text{Equation 2-15: Stern-Geary}$$

where: K is the proportionality constant, which is a function of the anodic and cathodic Tafel slopes (i.e., the relationship between current and voltage levels outside the linear region) and is represented by Eq. 2-16:

$$K = \frac{\beta_a \beta_c}{2.3(\beta_a + \beta_c)} \quad \text{Equation 2-16: Proportionality Constant}$$

where: β_a and β_c are the anodic and cathodic Tafel constants, respectively (ACI Committee 222, 2001). Units for the above parameters are as follows: i_{corr} is in units of $\mu\text{A}/\text{cm}^2$ (the instantaneous corrosion current density), K in units of mV, and R_p in units of $\Omega \cdot \text{cm}^2$. Two values for K are commonly used: for actively corroding steel, $K=26$ mV and for passive steel, $K=52$ mV (Andrade & Gonzalez, 1978).

Within the research community, the values for K are heavily disputed (Song G. , 2000). When considering the anodic Tafel slope under a passive condition, this slope approaches infinity, as seen in Figure 2.25.

When assuming the cathodic Tafel slope for the rebar in the passive state, the slope is about $\beta_c = 230$ mV/dec. the value of K for Eq. 2-16 becomes approximately 100 mV ($K = \frac{\beta_c}{2.3}$). Therefore, corrosion rate values calculated with $K = 26$ mV or $K = 52$ mV are an underestimation for rebars in the passive state (Chabi, 2012).

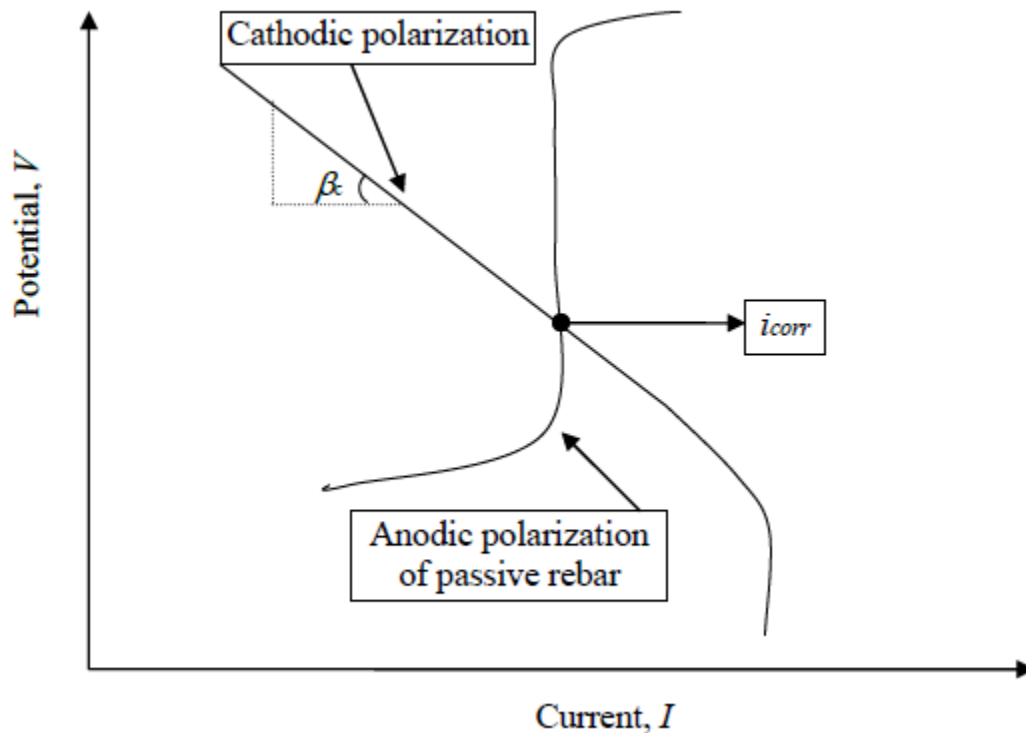


Figure 2.25: Cathodic and Anodic Polarization Curves of Rebar in the Passive State (Giatec Scientific Inc., 2008)

It has been found that the assumption for $K = 26$ mV for actively corroding steel does stand, but mostly for conditions when there is oxygen reduction and not enough oxygen, as in the case when reinforced concrete is in a highly saturated state (Elsener, 2005).

As can be inferred, using the widely accepted K values when calculating the corrosion rate will yield some error. This is due to the fact that the Tafel slopes, which depend on the concrete pore solution and the environmental effects, are difficult to estimate (Ge & Isgor, 2007). With that stated, to efficiently determine the corrosion rate of reinforcing steel with linear polarization resistance techniques requires a K value. The actual K value used should be carefully considered.

It is important to note several additional items:

- to maintain a linear relation, the change in potential must be kept to less than 20 mV for non-corroding steel and 50 mV for corroding steel (Andrade C. , 1973)
- concrete resistance between the reference electrode and the working electrode, known as iR drop, needs to be accounted for
- to avoid a large current spread-out for more passive steel, guard rings, should be utilized

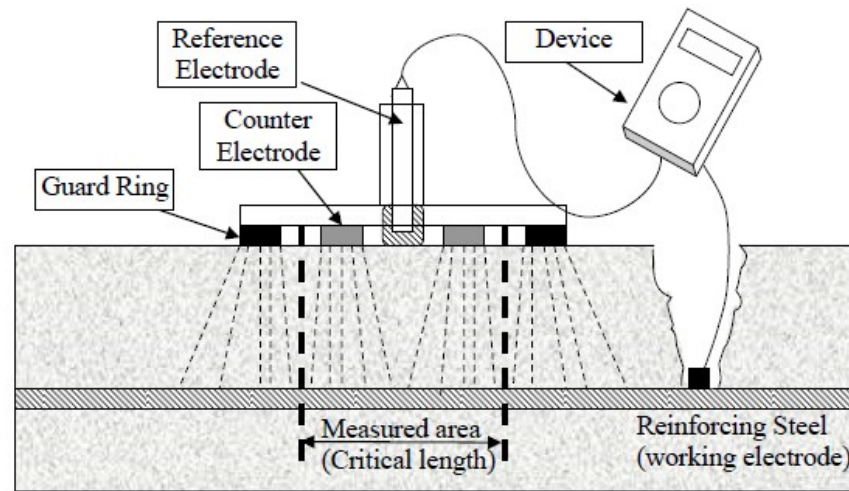
The polarization resistance R_p is the sum of the resistance associated with the actual corrosion process and the resistance associated with the electrolyte resistance. For this reason, the use of LPR must compensate for this resistance associated with the electrolyte resistance, or more commonly called ohmic (iR) drop, R_c , (for concrete resistance). Compensation can be made by two methods: positive feed-back and current interruption (Andrade & Alonso, 2004).

In order to determine the actual polarization resistance of the rebar, the resistance associated with the electrolyte resistance must be subtracted from the resistance associated with the actual corrosion process R_T , as seen the Eq. 2-17:

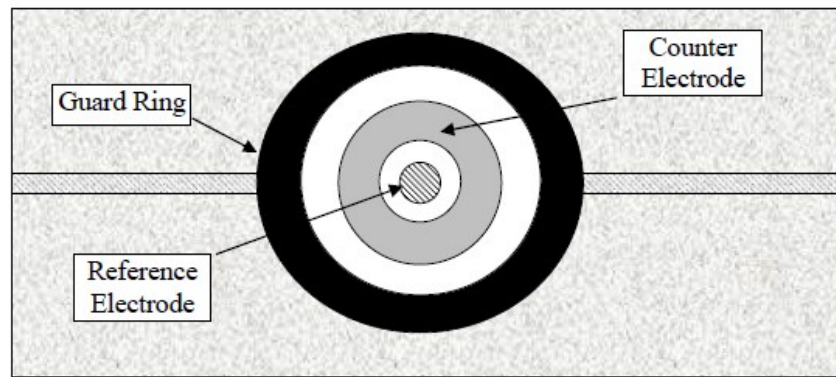
$$R_p = R_T - R_c \quad \text{Equation 2-17: Polarization Resistance, Ohmic Drop Accounted}$$

The actual polarization resistance, R_p , is the total measured polarization resistance, R_T , minus the ohmic resistance.

Most devices that use LPR incorporate a “guard ring” to accurately confine the area of measurement. This confinement is achieved by means of two twin reference electrodes, located between the central electrode and the external guard ring in order to achieve the required counterbalancing electrical field, as seen in Figure 2.26.



a) Cross section view



b) Plan view

Figure 2.26: Sensor Schematic – Illustration of Guard Ring (Chabi, 2012)

The two twin electrodes permanently control the external rings by way of detecting the current lines coming from the counter electrode in order to adjust them within a predetermined area. There are several advantages behind using a guard ring. Since the guard ring delimitates the area of polarized steel, the inherent error due to the relative area sizes is reduced. In addition, using a guard ring minimizes the effect of macrocells (Andrade & Alonso, 2004). It is important to note that the effectiveness of guard rings is the subject of much debate (Flis, Pickering, & Osseo-Asare, 1995) (Gepreags & Hansson, 2004).

2.4.2.1. Typical Corrosion Rates

Typical corrosion range classifications along with the corresponding corrosion current density and corrosion penetration rate from LPR measurements devices can be seen in Table 2.3.

Table 2.4: LPR Typical Corrosion Rates (Sadowski L. , 2010)

Corrosion Classification	Corrosion Current Density i_{corr}	Corrosion Penetration Rate
Passive/Very Low	Up to $0.2 \frac{\mu A}{cm^2}$	Up to $2 \frac{\mu m}{year}$
Low/Moderate	$0.2 \frac{\mu A}{cm^2}$ to $0.5 \frac{\mu A}{cm^2}$	$2 \frac{\mu m}{year}$ to $6 \frac{\mu m}{year}$
Moderate/High	$0.5 \frac{\mu A}{cm^2}$ to $1.0 \frac{\mu A}{cm^2}$	$6 \frac{\mu m}{year}$ to $12 \frac{\mu m}{year}$
Very High	Greater than $1.0 \frac{\mu A}{cm^2}$	Greater than $12 \frac{\mu m}{year}$

2.4.2.2. Advantages and Disadvantages

The result from on-site LPR testing, usually in conjunction with half-cell potential readings, can provide the reasonable precise locations of high corrosion activity and thus allows engineers to predict future deterioration and the overall service life of the structure (Bertolini L. , 2013).

As previously stated, the assumed K values used in the Stern-Geary equation are not accurate for all the different conditions of reinforced concrete structures and will therefore provide erroneous corrosion rate values.

Use of guard rings is required to ensure current confinement to a specified length of steel reinforcement. However, care should be taken to ensure effective confinement.

Any device that uses this technique must connect to the reinforcement. In addition, the diameter of the reinforcement must be known. This can be problematic for in-situ testing.

2.4.3. Galvanostatic Pulse Technique

The Galvanostatic Pulse Technique (GPT) is quite similar to the LPR technique in the determination of the polarization resistance; however, instead of waiting for an electrochemical potential equilibrium (as in the case of LPR), the transient response perpetrated by a galvanostatic pulse is measured. The technique works as follows. An anodic current pulse is galvanostatically applied to the rebar from the counter electrode, which is placed on the surface of the concrete. This anodic current changes the potential of the rebar, relative to the reference electrode placed on the surface of the concrete, with respect to time. The rebar is polarized in the anodic direction starting from the open-circuit potential. This transient response potential is shown in Figures 2.27 and 2.28.

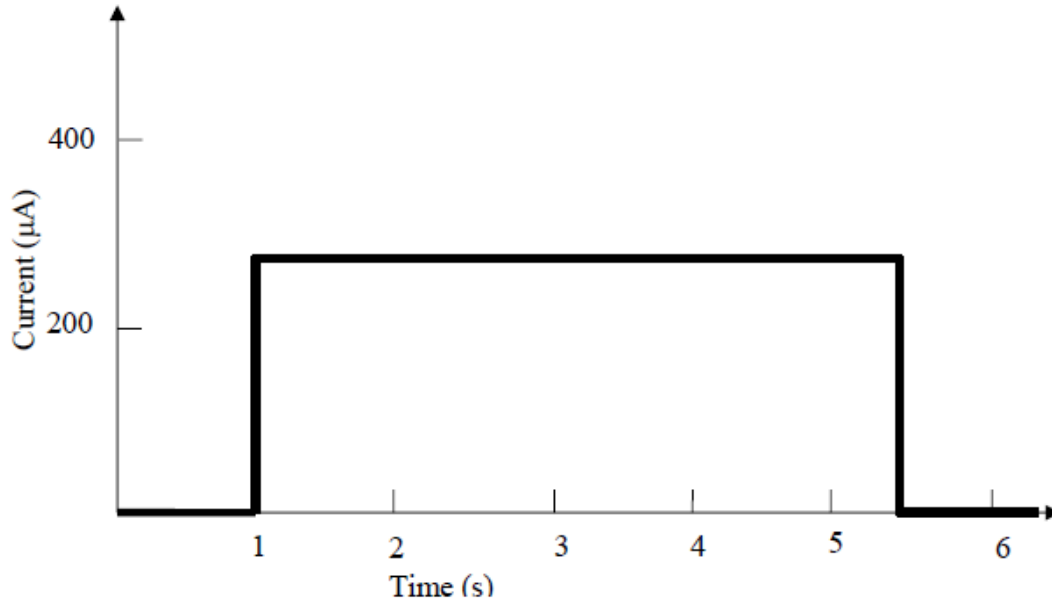


Figure 2.27: Galvanostatic Pulse Technique – Typical Current Pulse (Chabi, 2012)

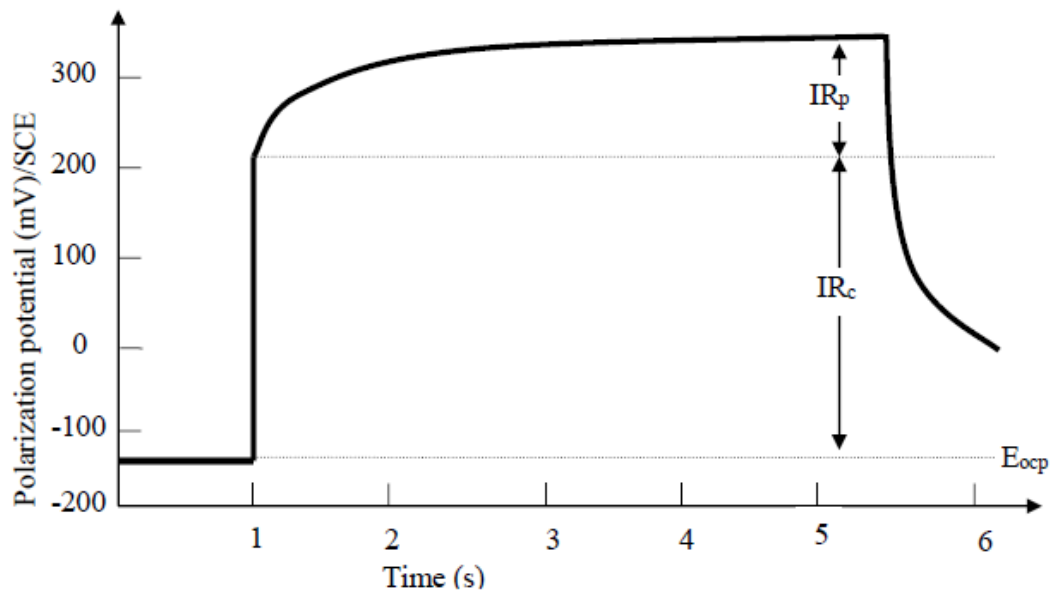


Figure 2.28: Galvanostatic Pulse Technique- Typical Potential Response (Chabi, 2012)

Typical ranges for the magnitude of the applied current is from 2 to 400 μA , and for the duration of the pulse, typically 10 seconds or less.

Under the assumption that this transient, or non-stationary, behaviour follows a Randles Circuit, refer to Figure 2.29, the potential of the rebar at any given time, $V(t)$, can be expressed by Eq. 2-18 (Newton & Sykes, 1988):

$$V(t) = I_{app} \left[R_p \left[1 - \exp\left(\frac{-t}{R_p C_{dl}}\right) \right] + R_c \right]$$

Equation 2-18: Potential, function of Time

where: I_{app} is the constant current applied to the system and C_{dl} is the double layer capacitance (double layer of charge ions on the surface of the steel bar formed as ions from the pore solution are absorbed onto the reinforcing steel surface).

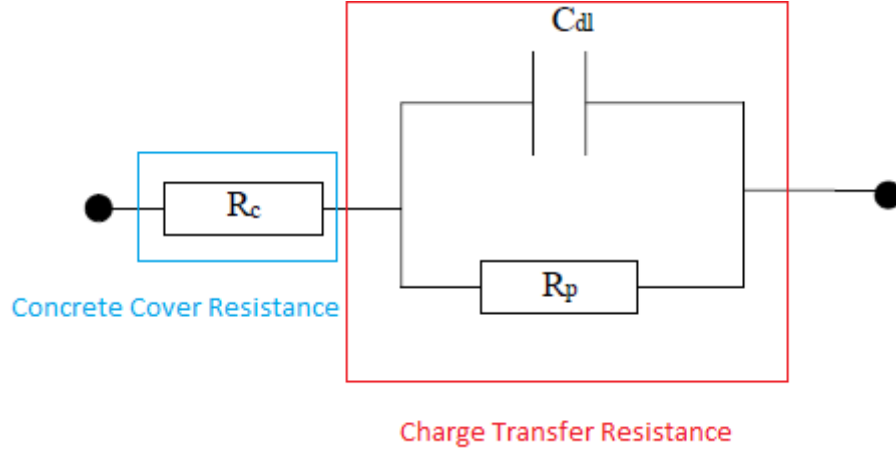


Figure 2.29: Representation of Steel Polarization in Concrete by Randles Circuit (modified from Chabi 2012)

In order to obtain the values for R_p , C_{dl} , and R_c , several different methods have been proposed in the literature: linearity, exponential curve fitting, and nonlinear regression analysis (galvanostatic pulse uses linearization technique). Using the linearity method, Eq.2-19 can be transformed into a linear form (Elsener, et al., 1997):

$$\ln(V_{max} - V(t)) = \ln(I_{app} R_p) - \frac{t}{R_p C_{dl}} \quad \text{Equation 2-19: Potential, expressed in linear form}$$

where: V_{max} is the final steady potential value reached after long polarization. Extrapolation of this straight line to $t = 0$, using least square linear regression analysis, yields an intercept corresponding to $\ln(I_{app} \times R_p)$ with a slope of $1/(R_p \times C_{dl})$. The remaining over-potential corresponds to $(I_{app} \times R_c)$, which is the ohmic drop (Elsener, et al., 1997). With the determination of the polarization resistance, R_p , the corrosion current is then calculated from the Stern Geary equation given by Eq. 2-15.

Similar to the LPR technique, the galvanostatic pulse technique instrument generally consists of a portable device that generates a galvanostatic pulse and a disc probe that is applied to the concrete surface over the rebar.

2.4.3.1. Typical Corrosion Rates

Typical corrosion range classifications along with the corresponding corrosion current density from a GPT measurements device can be seen in Table 2.4.

Table 2.5: Corrosion Rate Ranges for GPT (Germann Instruments, 2011)

Corrosion Activity	Corrosion Rate ($\mu\text{A}/\text{cm}^2$)
Passive area	< 0.5
Negligible	0.5 – 2
Low corrosion	2 – 5
Moderate	5 – 15
high	> 15

2.4.3.2. Advantages and Disadvantages

Generally speaking, this technique is fast, with testing times close to one minute. Most devices are quite portable and easy to use for large scale testing projects.

As previously stated, the assumed K values used in the Stern-Geary equation are not accurate for all the different conditions of reinforced concrete structures and will therefore provide erroneous corrosion rate values.

Any device that uses this technique must connect to the reinforcement. In addition, the diameter of the reinforcement must be known. This can be problematic for in-situ testing.

2.4.4. Connectionless Electrical Pulse Response Analysis

In an attempt to move corrosion measurement techniques away from the need to connect to the reinforcement, the study of connectionless or non-destructive techniques has flourished.

Electrochemical impedance spectroscopy (EIS) consists of the measurement of electrical properties of concrete at varying frequencies of the applied AC current with the objective to describe the electrical concrete behaviour, including its resistive and capacitive responses (Lataste J.-F. , 2010). The impedance (Z) is the ratio of applied potential (ΔE) to measured current (i), as shown in Eq 2-20:

$$Z = \frac{\Delta E}{i}$$

Equation 2-20: Impedance

Measurements are carried out at different AC frequencies and as a result, a spectrum of impedance as a function of frequency is obtained. The impedance is a complex number which can be decomposed into imaginary and real parts at each frequency. Results of this technique are typically represented by Nyquist or Bode diagrams where the imaginary part of impedance (Z_i) is a function of the real part of impedance (Z_r) at each excitation frequency (Lataste J.-F. , 2010). The Bode diagram examines the absolute impedance, ($|Z|$), as calculated and the phase shift, θ , of the impedance, each as a function of frequency (Poursaei, 2011). The electrochemical system is represented by a simple Randles circuit, as seen previously in Figure 2.29. This circuit includes a resistor, which represents the electrolyte's resistance towards the polarized current, in parallel, a capacitor, which is the double-layer capacitance, and the resistor, which is the polarization resistance. These two paths indicate that a portion of the current will pass in each, depending on the resistance or impedance values of each and depending on the frequency of the applied current. At higher frequencies, the impedance caused by the double-layer capacitance reaches zero, resulting in a short-circuit element and leading to all current flow through the concrete resistance, allowing for an estimation of the concrete resistance. At lower frequencies, the impedance caused by the double-layer capacitance reaches high values, leading to all current flow through the concrete resistance and polarization resistance, thus allowing for the sum of the concrete resistance and polarization to be estimated. The difference between these two estimations determines the polarization resistance of the reinforcement. See Figure 2.30 for a representation of a typical Nyquist plot for a Randles circuit.

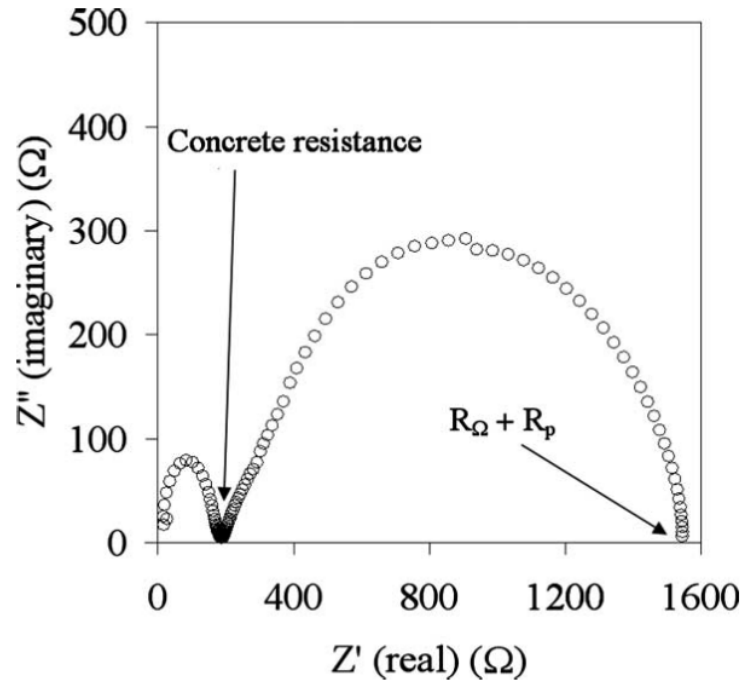


Figure 2.30: Typical Nyquist Plot for Randles circuit (Poursaee, 2011)

Steaming from the concept of electrochemical impedance spectroscopy and electrical resistivity, it was seen that the presence of reinforcing bars in concrete distorts the applied current as current is drawn into the path of low-resistivity or the reinforcement (Layssi H. , Ghods, Alizadeh, & Salehi, 2015). As such, impedance characteristics of the reinforcement can be quantified. By imposing a current at a wide range of frequencies and measuring the potential differences, the interface between the reinforcement and concrete can be distinguishable.

Connectionless Electrical Pulse Response Analysis (CEPRA) is based on the concept that a corroding rebar has a different response to a sweeping frequency of an AC current than a non-corroding rebar, as illustrated in Figure 2.31.

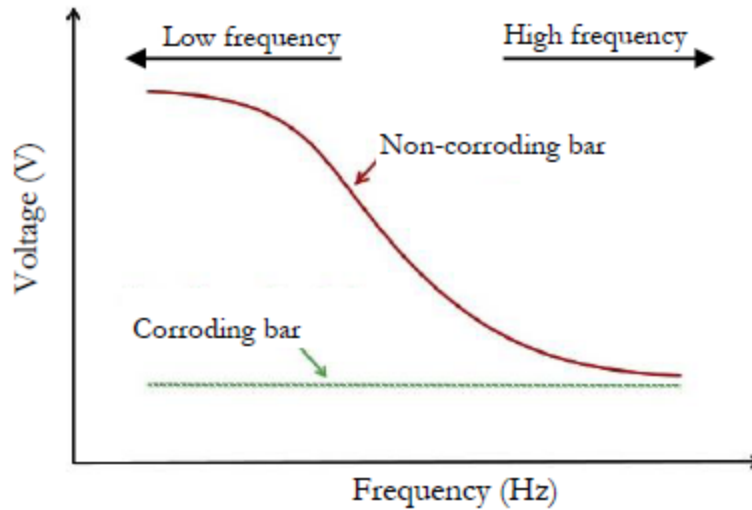


Figure 2.31: AC Current Voltage-Frequency Response (Giatec Scientific Inc., 2016)

Using varying frequency changes allows the locations of corroding reinforcement to be determined along with the extent of corrosion rate.

CEPRA employs a more detailed simple Randles circuit model to illustrate the interaction between concrete and steel properties subjected to varying frequencies, see Figure 2.32.

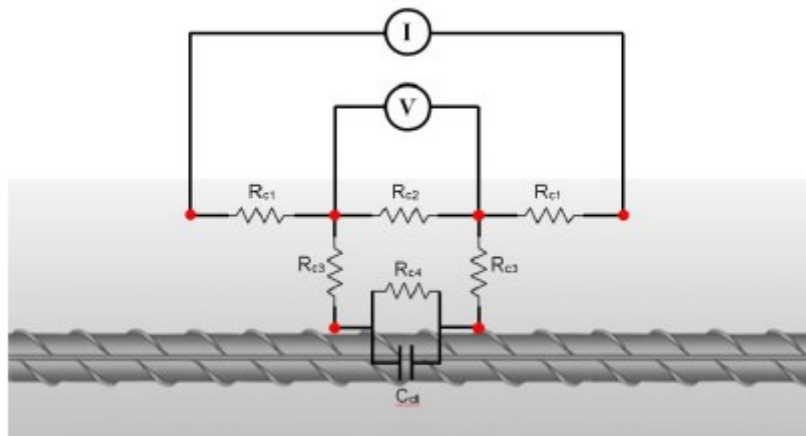


Figure 2.32: Circuit Model (Giatec Scientific Inc., 2016)

In the above circuit, R_{c1} represents the contact resistance of the probe where all current is faced by the resistance of the concrete. Two current flow paths are identified in the concrete through R_{c2} and R_{c3} , where the magnitude of the passing current in each resistor is dependent on the following: magnitude of each resistance, the impedance caused by the capacitance and subsequently the extent of charging of this capacitance, the magnitude of the polarization resistance, the concrete cover, and the reinforcement

diameter. Resistances R_{c2} , R_{c3} , and R_{c4} are all defined as concrete material properties related to intrinsic electrical resistivity. For the application of a narrow and time-restricted AC current pulse or voltage step, the measured voltage response as a factor of time is similar to a charging RC circuit under the assumption that the electrolyte capacitance is negligible, as seen in Eq. 2-21:

$$V_{in}(t) = V_{ex}(A - Be^{-Dt}) \quad \text{Equation 2-21: Potential Difference CEPRA Circuit Model}$$

where V_{ex} is the constant voltage applied through the external electrodes and V_{in} is the potential difference between the inner electrodes. Variables A, B, and D are solved using the Circuit Model in Figure 2.23 by measuring the voltage response over time as described in Eq. 2-11 and fitting an exponential curve to the measured data. Only when the cover depth is known can the circuit be solved. This is due to the fact that the cover depth provides an indirect measure of the ratio of current flow through R_{c2} to that flowing through R_{c3} (Ghods, Fahim, Alizadeh, Decarufel, & Salehi, 2019).

2.4.4.1. Typical Corrosion Rates

Corrosion activity along with the corresponding corrosion rate for the CEPRA technique are seen in Table 2.5.

Table 2.6: Corrosion Rate Ranges for CEPRA (Giatec Scientific Inc., 2016)

Corrosion Activity	Corrosion Rate ($\mu\text{A}/\text{cm}^2$)
Passive/Low	< 0.1
Moderate	0.1 – 0.3
High	0.3 – 1
Severe	> 1

2.4.4.2. Advantages and Disadvantages

The main advantage to this electrochemical device that makes it stand out from the rest is the fact that the technique does not require connection to the rebar for corrosion rate measurements.

Similar to previous corrosion measurement techniques, the diameter of the reinforcement must be known. This can be problematic for in-situ testing.

2.4.5. Factors that Affect Corrosion Rate

At a basic chemical level, corrosion is fed by oxygen and water (cathodic reaction). Thus, both of these environmental factors must be present to feed the corrosion process. In addition, the following primary

factors affect the rate of corrosion of steel reinforcement: pH of the concrete pore water, electrical resistivity, relative humidity, and temperature (ACI Committee 222, 2001).

It is also important to be aware of corrosion during the design and construction of the structure. Care should be taken to ensure that water runoff removes the water from the reinforced structure and avoids splashing of water. In addition, a few ways to reduce the possibility of reinforcement corrosion would be to ensure adequate concrete cover is provided, crack-control measures are established, and corrosion protection systems are incorporated.

More discussion on how temperature, relative humidity, and electrical resistivity influence corrosion rate can be seen below.

2.4.5.1. Temperature

Temperature has a remarkable influence on the corrosion rate within reinforced concrete structures. The oxidation reaction rate is influenced by the amount of heat energy available to push the reaction forward (Broomfield J. , 2007). As temperature is increased, the ions become more mobile and the aggressive agents become more soluble. As temperature decreases, the opposite occurs where the pore water freezes and corrosion stops as the ions are no longer mobile. This phenomenon can be seen in Figure 2.33, where the data was collected during an on-site research experiment (Tuutti, 1982).

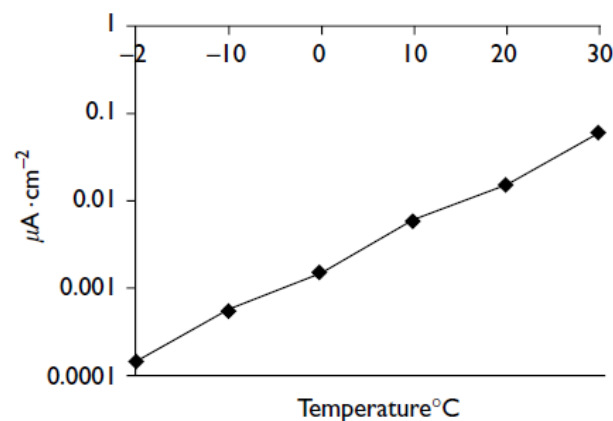


Figure 2.33: Correlation between Temperature and Corrosion Rate (Tuutti, 1982)

2.4.5.2. Relative Humidity

Since the movement of water within the concrete pore structure is a key element in the reinforcement corrosion process, relative humidity has a key effect on the corrosion rate. Even though there is a delicate interaction between the amount of water and oxygen within the pore structure of the concrete and how

those two elements contribute to the corrosion rate of the reinforcement, as a general rule, as relative humidity increases, so does corrosion rate. This phenomenon can be seen in Figure 2.34.

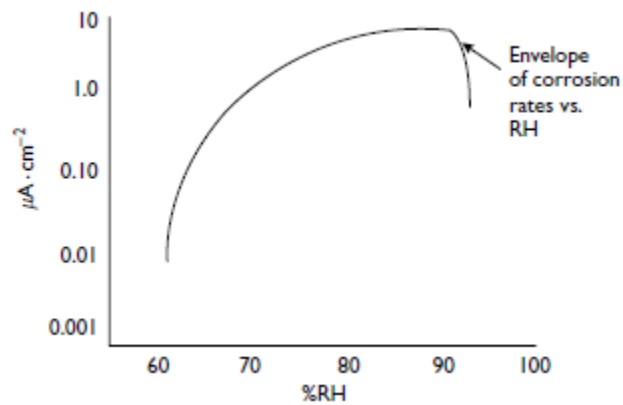


Figure 2.34: Correlation between Relative Humidity and Corrosion Rate in Chloride Contaminated Concrete (Tuutti, 1982)

The maximum value of corrosion rate can be found for a relative humidity in the atmosphere of approximately 95%, seen as the upper limit of the curve in Figure 2.34. Relative humidity above this limit is characterized by high water content and thus a deficient oxygen supply (Bertolini L. , 2008). If the concrete becomes saturated, the corrosion rate will decrease until it nearly becomes zero.

2.4.5.3. Electrical Resistivity

As it relates to reinforcement corrosion within concrete, electrical resistivity (ER) has a bearing on the corrosion rate, since during the corrosion mechanism, ionic current passes from the anodic to cathodic areas (Broomfield, 2007). In terms of electrochemistry terms, resistivity relates to the resistance of ion transport between the cathode and anode in a corrosion cell circuit (Villagran-Zaccardi & Di Maio, 2014).

Generally speaking, low electrical resistivity is related to high risk of corrosion (Polder, 2000). This general relationship is universally accepted throughout the literature, even with variations in experimental studies and studied parameters (Layssi H. , Ghods, Alizadeh, & Salehi, 2015) (Hornbostel, Larsen, & Geiker, 2013) (Polder, 2000) (Lataste J.-F. , 2010). The different parameters found in these include, but not limited to, cement type, corrosion cause, specimen geometry, reinforcement type, cover depth, measurement technique, and exposure conditions. In addition, the analysis of this relationship between resistivity and corrosion rate vary by properties, scales, scatter, and correlations (Hornbostel, Larsen, & Geiker, 2013). A comparison of the regression lines for the relationship between electrical resistivity in concrete and the corrosion rate can be seen in Figure 2.35.

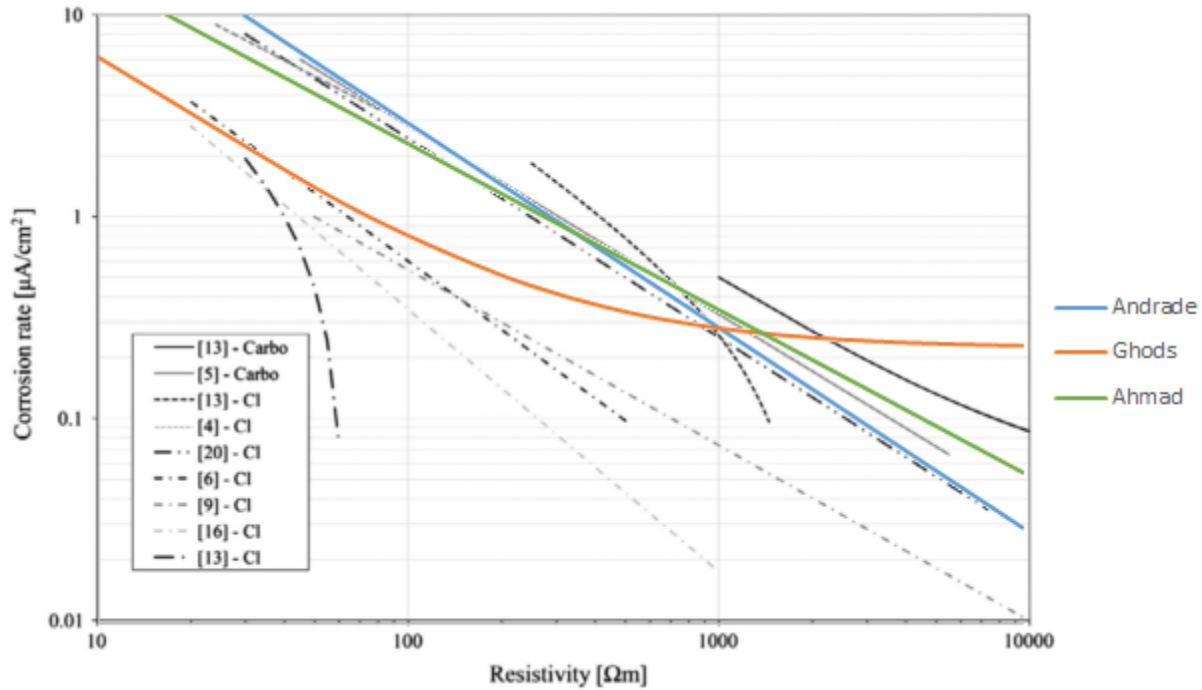


Figure 2.35: Relation between Resistivity and Corrosion Rate (Hornbostel, Larsen, & Geiker, 2013)

It is important to note that the most important parameters affecting the relationship between electrical resistivity and corrosion rate, as outlined by the referenced literature review, are the degree of saturation of the pore structure and temperature (Hornbostel, Larsen, & Geiker, 2013).

Equations relating electrical resistivity to corrosion rate are many. As stated before, there is a generally accepted inverse relationship between the two but not a universally accepted equation. With that said, a sampling of empirically determined equations establishing a relationship between the electrical resistivity and the intensity of the current of corrosion can be seen below in Table 2.7. These equations are superimposed onto the graph of resistivity and corrosion rate in Figure 2.35. As it can be seen, the empirical equations by Andrade & Alonso (2004) and Ahmad (2014) agree with the trends exhibited by the experimental results. Ghods, Isgor, & Pour-Ghaz (2007) depart from the generalize trend for resistivity values greater than 1000Ωm.

Table 2.7: Comparison of Equations Relating Electrical Resistivity and Corrosion Rate

(Andrade & Alonso, 2004)	(Ghods, Isgor, & Pour-Ghaz, 2007)	(Ahmad, 2014)
$I_{corr} = \frac{3 \times 10^4}{\rho}$	$i_{corr} = -1.33 \times 10^{-3} + \frac{3.0}{\rho} - 3.83 \times 10^{-4} \ln(C_{O_2}) + 0.333 \frac{\ln(C_{O_2})}{\rho}$	$I_{corr} = \frac{15.39}{(\rho)^{0.8125}}$
Units: $\mu A/cm^2$; Ωcm	Units: A/m^2 ; Ωm Where: $C_{O_2} = 0.00075$ (average) C_{O_2} is defined as the oxygen concentration	Units: $\mu A/cm^2$; $K\Omega cm$

The relationship between electrical resistivity and corrosion rate requires further exploration. Each of the above equations are dependent on many factors surrounding how the experimental tests were done. From this fact, caution needs to be taken when applying these equations to existing reinforced concrete structures for the purpose of condition evaluation.

In addition to the universally accepted inverse relationship between electrical resistivity and corrosion rate, several resistivity ranges corresponding to risk of corrosion have been published (Lataste J.-F. , 2010). Figure 2.36 presents these relationship values.

Risk of corrosion	Resistivity (ohm m)	
	Feliu <i>et al.</i> , 1996; RILEM TC154EMC, 2004	Langford and Broomfield, 1987
Negligible	>1000	>200
Low	500–1000	100–200
Moderate	100–500	50–100
High	<100	<50

Figure 2.36: Resistivity Range as it related to Risk of Corrosion (Lataste J.-F. , 2010)

2.4.6. Corrosion Rate Range

No matter the electrochemical technique used to measure the corrosion rate, a broad criterion has been established between the corrosion current density or corrosion rate, and the significance of the corrosion level within the reinforced concrete structure, as seen in Table 2.8 for values associated with Ordinary Portland Cement.

Table 2.8: Corrosion Rate Ranges as they relate to Corrosion Level (Andrade & Alonso, 2004)

$I_{corr} \left(\frac{\mu A}{cm^2} \right)$	$x \left(\frac{mm}{year} \right)$	Corrosion Level
≤ 0.1	≤ 0.001	Negligible
0.1 – 0.5	0.001- 0.005	Low
0.5 - 1.0	0.005- 0.010	Moderate
> 1.0	> 0.010	High

2.5. Studies on Determination of Corrosion Rate

As this thesis strives to show the effectiveness of several electrochemical techniques in the assessment of reinforced concrete structures on-site and in-lab, a presentation of experimental studies and case studies is given. A common underlying theme found in most current studies involving non-destructive corrosion determination methods shows that it is recommended to utilize several non-destructive techniques during an assessment to avoid misrepresentation (Bjegovic, Mikulic, & Sekulic, 2000).

2.5.1. On-site

An experimental study that assessed the condition of several reinforced concrete slabs removed from a deteriorating bridge in Hawkesbury, Ontario, using different electrochemical techniques was conducted by Qian & Chagnon (2001). This study focused on four slabs measuring 315 cm x 115 cm x 14 cm reinforced with five 15-mm diameter longitudinal bars, spaced at 23 cm, and six 12-mm transverse bars, spaced at 45 cm. These slabs were subjected to repairs at some point during their life. In order to simulate the environmental conditions to which the slabs were subjected, wet-dry bi-weekly cycles with tap water were used. Three electrochemical techniques were used: half-cell potential, linear polarization by use of Gecor 6, and concrete resistivity, also by Gecor 6. A plot of the corrosion potentials, E_{corr} , versus the corrosion current, i_{corr} , as determined by the Gecor 6 showed a poor correlation between these two values for the slabs. The study attributes this poor correlation to current fluctuation at the active corrosion areas and shifts in potential due to changes in the oxygen concentration at the reinforcing surface. The

study does support the notion that considering both half-cell potential data and corrosion rate measurements can facilitate better interpretation of potential results and provide a more reliable condition evaluation. In addition to the electrochemical techniques, compressive tests were conducted and chloride content was determined to provide additional information on the condition assessment.

A case study by Bjegovic et al. (2002) shows the use of GPT in a marine environment. A reinforced concrete marina located in Croatia was investigated due to its visually deteriorating state. Destructive testing was conducted, compressive strength and chloride content, to assess the condition. It was established that a new structure was required. After the construction of the new marina with migrating corrosion inhibitors, the structure was monitored with GPT. It was noted that the GPT was suitable for on-site measurements as it was rapid and simple. However, it was noted that corrosion rate values are very dependent on the changing temperature and humidity conditions. Therefore, it was concluded that a combination of several NDT methods should be utilized to more reliably assess the reinforced concrete structure.

2.5.2. On-site and In-lab

A critical evaluation of devices used to measure corrosion rate in reinforced concrete structures was conducted by Gulikers & Polder (2003). As part of the critical evaluation, experimental results from on-site and laboratory measurements showed there were noticeable differences in corrosion rate with the use of LPR and GPT, with GPT frequently showing values larger than one order of magnitude or more. In addition, GPT was unable to make a clear distinction between passive and active corroding reinforcement areas. It was found that regions with passive steel yielded corrosion rates that were unrealistically high, frequently exceeding $10 \mu\text{A}/\text{m}^2$. In contrast, LPR provided more reliable information on the actual corrosion rate. In conclusion, Gulikers & Polder (2003) found that corrosion rate measurements should be used to confirm half-cell potential measurements for areas where potential values suspect areas of active corrosion.

A more comprehensive experimental study conducted by Vedalakshmi et al. (2010) compared corrosion rate values measured by the GPT with the GalvaPulse device with that of electrochemical impedance spectroscopy (EIS) technique and gravimetric mass loss testing. In the laboratory, reinforced concrete slabs were constructed with 1.2% chloride by weight of cement and were allowed to cure for a period of 90 days, during which GPT and EIS measurements were performed. After the completion of the testing, mass loss testing on the reinforcement was conducted according to the 2000 ASTM G-1 standard. In the field, three separate reinforced concrete slabs, all with different exposure conditions, periods of exposure, and grade of concrete, were subjected to corrosion rate testing. Condition assessment of the

reinforced concrete slabs involved several measurements: half-cell potential, corrosion rate and resistivity. It was concluded that for field measurements, potential and concrete resistivity values can provide a misleading assessment of the actual condition of the reinforcement. It was also found that corrosion rate determination by GPT compared well with that of mass loss testing when compared to corrosion rate values by EIS, see Figure 2.37. In addition, GPT used with an anodic pulse of 100 μA with a pulse duration of 10 seconds was able to properly discriminate the passive and active areas of the rebar. This is in direct contrast to Gulikers & Polder (2003) as stated previously. It is also important to note that Vedalakshmi et al. (2010) concluded that GPT was unable to predict corrosion rate values less than 1 $\mu\text{m}/\text{year}$, whereas EIS was successful.

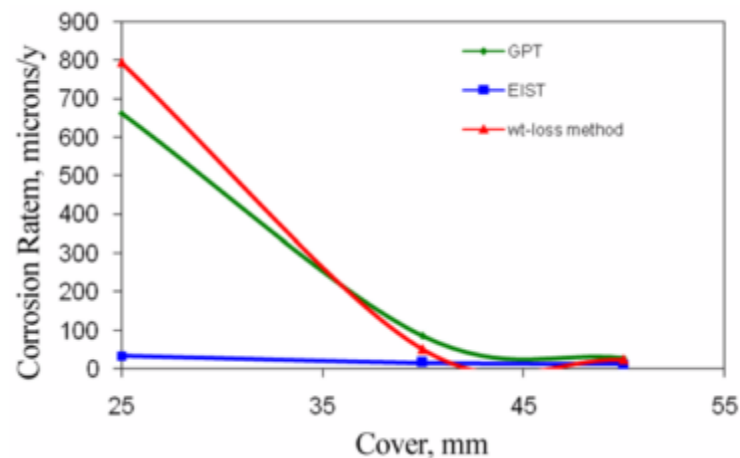


Figure 2.37: Comparison of Corrosion Rate by GPT, EIS, and Mass-Loss Testing (Vedalakshmi, Balamurugan, Saraswathy, Kim, & Ann, 2010)

Frolund et al. (2002) used the GPT equipment produced by FORCE Institute, or the GalvaPulse, for an experimental study that looked at corrosion rate data for a deteriorating bridge in Copenhagen, Denmark, and corrosion rate data for laboratory reinforced concrete test blocks. Frolund et al. (2002) determined that results from the laboratory showed very good correlation between corrosion rates determined by mass loss and corrosion rates as determined by the GPT. In addition, corrosion rates obtained on-site by GPT compared to the average corrosion rate from actual cross-section loss, where the actual corroding area is the same as the confined area, were comparable. Use of the LPR method by use of the Gecor 6 was also used to compare with corrosion rate as determined by GalvaPulse and mass loss, however, the LPR method tended to grossly underestimate the average corrosion rate. Refer to Figure 2.38.. In conclusion, it was recommended that more meaningful results could be obtained if knowledge of the daily

and seasonal changes of corrosion rate on-site are known, with the aid of mounted sensors on the structure.

	Weight loss	GalvaPulse®			Gecor 6		
Description	Avg. Corr.rate $\mu\text{A}/\text{cm}^2$	mV vs. Ag/AgCl	Avg. Corr.rate $\mu\text{A}/\text{cm}^2$	Resistance kOhm	mV vs. Ag/AgCl	Avg. Corr.rate $\mu\text{A}/\text{cm}^2$	Resistance kOhm
bar A	4.6	-345	3.11	0.50	-355	0.21	0.47
bar B	4.8	-334	2.47	0.90	-437	0.23	0.93
bar A and B connected	4.7	-345	5.24	0.50	-343	0.54	0.35

Figure 2.38: Average Corrosion Rate Values Compared to GalvaPulse and Gecor 6 Values

2.5.3. In-Lab

Sadowski (2013) proposed a methodology that combines half-cell potential data with that of concrete resistivity to assess the probability of corrosion in concrete structures. The in-lab experimental study focused on a 750 mm x 750 mm concrete slab with 10-mm diameter reinforcement spaced at 150 mm and a cover thickness of 50 mm. A corrosive environment was established with two-hour spray wetting with a sodium chloride solution cycle followed a twenty-four hour drying cycle. Measurements were taken after ninety days of concrete curing. Three distinct types of areas were established based on ASTM C876 probabilities of corrosion and resistivity ranges. Sadowski (2013) was able to establish that using both half-cell potential data and electrical resistivity data in conjunction provides maximum information about the probability of corrosion within a reinforced concrete structure.

As the current standard for use of half-cell potential data in condition assessment of reinforced concrete structures is based on the probability of corrosion, the interpretation of these half-cell measurements can be very subjective. Pour-Ghaz et al. (2009) showed quantitative relationships that related potential readings on the concrete surface to the corrosion rate through the use of data from concrete resistivity, cover thickness, oxygen demand, and the anode-to-cathode area (A/C). These quantitative relationships attempt to relate the potential distribution at the concrete surface with that of the steel/concrete interface. The parameters of these relationships were investigated and several key conclusions were made: (1) potential readings should be interpreted in accordance with the resistivity of the concrete, for the same corrosion rate, different potentials can be read corresponding to different resistivities; (2) as cover thickness increases, the potential at the concrete surface differs from that of the potential at the

concrete/steel interface; (3) oxygen availability does not greatly affect half-cell potential measurements in an environment where concrete is not completely deprived of oxygen; and, (4) larger A/C ratio shows smaller potential differences between the surface and interface at the anodic site, this implies that half-cell potential is not suitable for the detection of localized corrosion. Pour-Ghaz et al. (2009) also put forth a model that attempts to establish a relationship between half-cell potentials and the rate of localized corrosion. Data from an experimental study carried out by Luping (2002) was used to compare with the model. It was observed that this model predicts corrosion current densities higher than that of the non-destructive device used by Luping. It is noted that the model predicts localized corrosion rates and the non-destructive techniques used by Luping (2002) might have underestimated them.

Sadowski (2010) proposed a new method for reinforced concrete assessment by taking advantage of the short-circuit effect of the reinforcement on the resistivity measurements. This study looks at the effect of using the conventional AC four-point resistivity device and a DC galvanostatic resistivity device (by way of a modified electrode array), both tested directly over and parallel to the reinforcement to maximize the reduction effect of the short-circuiting bar on the resistivity measurements. The AC signal will pass through the capacitance layer regardless of the reinforcement corrosion rate, while the DC signal cannot pass through the capacitance layer after a suitable equilibrium period. The use of DC galvanostatic current allows the effect of the reinforcement on the resistivity measurements to be influenced by the rate of corrosion on the rebar surface. During this experimental study, reinforced concrete slabs were constructed, and corrosion activity was established in the specimens using the LPR technique. Experimental results showed that for rebars with active corrosion current densities (or relatively small charge transfer resistance), DC measurements of resistivity over the rebar gave a resistivity much closer to that of AC resistivity measurements than that of actual concrete resistivity, refer to Figure 2.39.

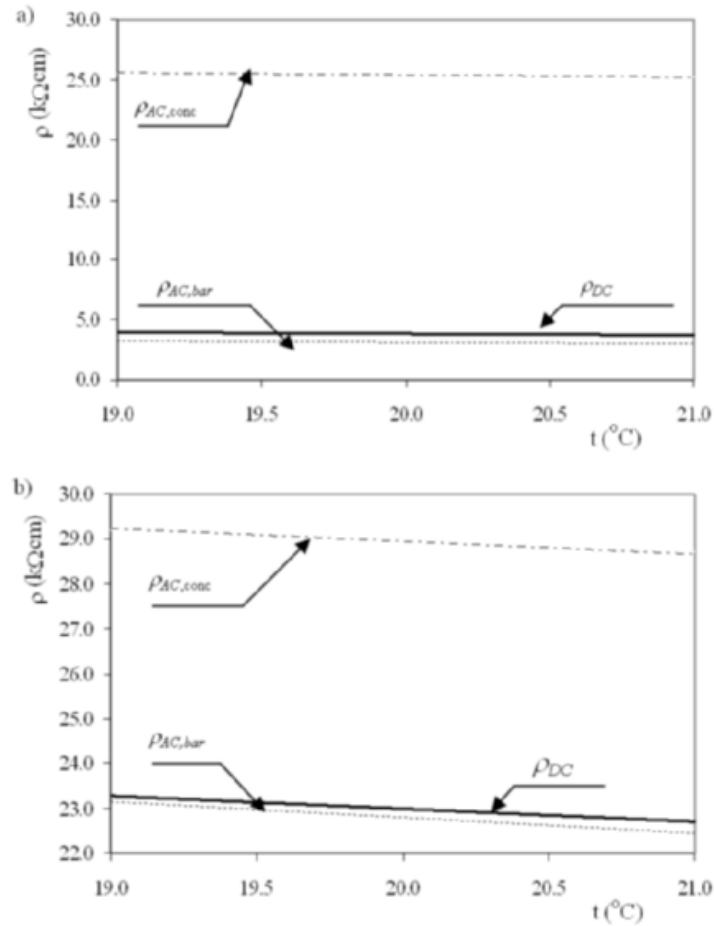


Figure 2.39: Comparison of DC resistivity with AC resistivity of the bar and resistivity of the concrete on actively corroding reinforcement (Sadowski L. , 2010)

In the case of lower corrosion current densities, the presence of the reinforcement had small effect on the DC resistivity measurements. Sadowski (2010) was able to show that short-circuit influence of an embedded reinforcement within concrete can be used to evaluate the rate of ongoing corrosion on the rebar surface. The new method presented does show that the DC resistivity approach in conjunction with the conventional AC resistivity approach can provide for a way to assess the instantaneous corrosion rate easily.

In a more focused look at the galvanostatic pulse transient technique, So and Millard (2007) performed an experimental study that compared both the LPR and GPT techniques. The focus of this study was on the evaluation of separate components: the capacitance values and the associated resistances. In the analysis of these components, it was determined that the corrosion rates calculated from the galvanostatic pulse transient data were generally higher than the rates determined by LPR, with a factor

of 3.0. The results shows that the galvanostatic pulse transient method has the advantage of making the separation of resistance componenets possible and it enable those resistances not associated with corrosion to be discounted. As such, the data showed that the galvanostatic pulse transient technique provided more reliable corrosion rates on the reinforcing bars than that of the LPR technique.

In an experimental study by Dou et al. (2014), GPT was used by means of the GalvaPulse device to measure the corrosion rate of standard specimens with different concrete cover thickness (10, 30, 50, and 70 mm) and different amounts of NaCl content (0, 0.5, 1.0, 1.5, 2.5% NaCl by weight of cement). Testing was conducted with the GalvaPulse on days 5, 10, and 30. Several relationships were shown: (1) corrosion current will generally decrease with increasing cover thickness, but the rate of reduction will decay with the increasing of the cover thickness, as illustrated in Figure 2.40; (2) in general, corrosion current increased with the addition of NaCl, although the relation is not linear, as shown in Figure 2.40; there existed a NaCl threshold, were the addition of more NaCl did not greatly affect the corrosion current, as seen in Figure 2.40, where a cover thickness of 10 mm has a NaCl threshold of 1.5%, and a cover thickness of 50 mm has a NaCl threshold of 1%.

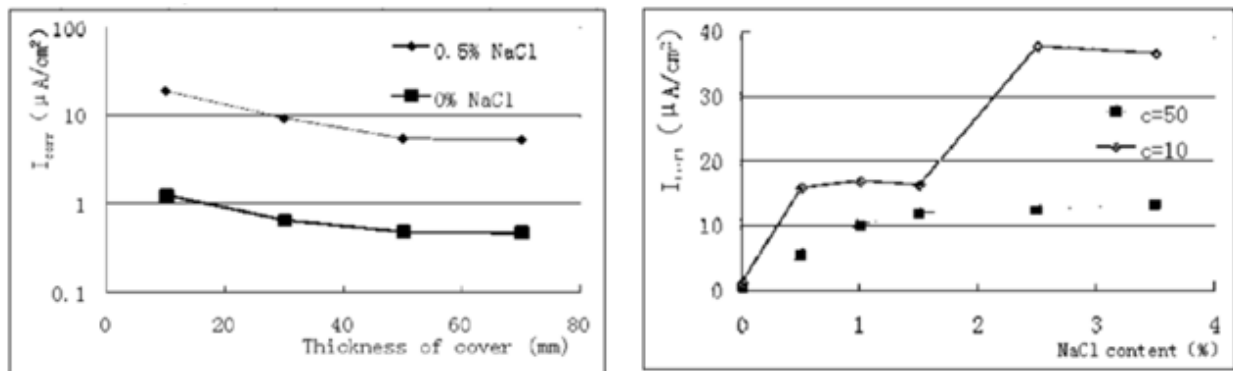


Figure 2.40: Corrosion Rate vs Cover Thickness & Corrosion Rate vs NaCl Content (Dou, et al., 2014)

In contrast to Vedalakshmi et al. (2010), Dou et al. (2014) documented the change in corrosion rate with changes in the pulse magnitude and concluded that an anodic pulse of 50 μA with a pulse duration of 10 seconds is suitable for passive and active areas of the reinforcement.

Luping (2002) embedded reinforcement within concrete specimens of 0, 1.5, 3, and 6 % chloride content (by mass of cement) and monitored the corrosion rate for a year. Measurements techniques LPR by use of the Gecor 6 and GPT by use of the GalvaPulse were evaluated. At the end of the year, gravimetric testing according to ASTM G1 was conducted, and the results compared with those by the electrochemical techniques. The study concluded that corrosion rates by the Gecor 6 were comparable to the true mean

corrosion rate (mass loss divided by the whole exposed area of the steel), while the GalvaPulse provides an overestimation, mostly in passive areas. For the most accurate results, use of a calibration factor of approximately 6 (pitting factor) for the LPR technique can be used. The study also concluded that the gap within the LPR and GPT polarization resistance measurements are due to the polarization time.

In contrast with Luping (2002), Sathiyarayanan et al. (2006) found that the anodic galvanostatic pulse technique gave more reliable results about the state of the reinforcement and the rate of corrosion for reinforced concrete specimens with varying NaCl content. On the other hand, results obtained by LPR were found to provide lower values than that of the actual corrosion rate due to the inclusion of the resistance of concrete in the LPR measurements. These results can be seen in Figure 2.41.

Concrete grade	Chloride concentrations, %	Corrosion rate (mmpy)		
		Galvanostatic pulse technique	Linear polarization resistance method	Gravimetric method
M15	0	4.375E-10	0.00015	Nil
	1	0.036	0.0021	0.0265
	3	0.068	0.0030	0.0875
	5	0.070	0.0010	0.082
M20	0	1.20E-8	0.0002	Nil
	1	0.00001	0.0003	Nil
	3	0.031	0.002	0.057
	5	0.064	0.0015	0.080
M30	0	9.95E-10	0.0004	Nil
	1	9.92E-9	0.0005	Nil
	3	0.0026	0.0015	0.025
	5	0.041	0.0013	0.03
M35	0	1.53E-10	0.00050	Nil
	1	1.79E-8	0.0004	Nil
	3	0.064	0.0015	0.055
	5	0.017	0.0014	0.018

Figure 2.41: GPT, LPR, and Gravimetric Method Comparison (Sathiyarayanan et al. 2006)

In another experimental study, Poursaee (2011) evaluated several electrochemical NDT devices for the purpose of determining the most accurate method for measuring corrosion rate. These devices were then compared with mass loss results on the reinforcement extracted from the concrete specimens. This study concluded that using applied potential (potentiostatic LPR) is more reliable than techniques that are based on applied current (galvanostatic LPR). Such results obtained from techniques based on applied potential showed less variation. It was also noted that the use of potentiostatic LPR is more reasonable on-site as testing times are lower. The results are shown in Figure 2.42.

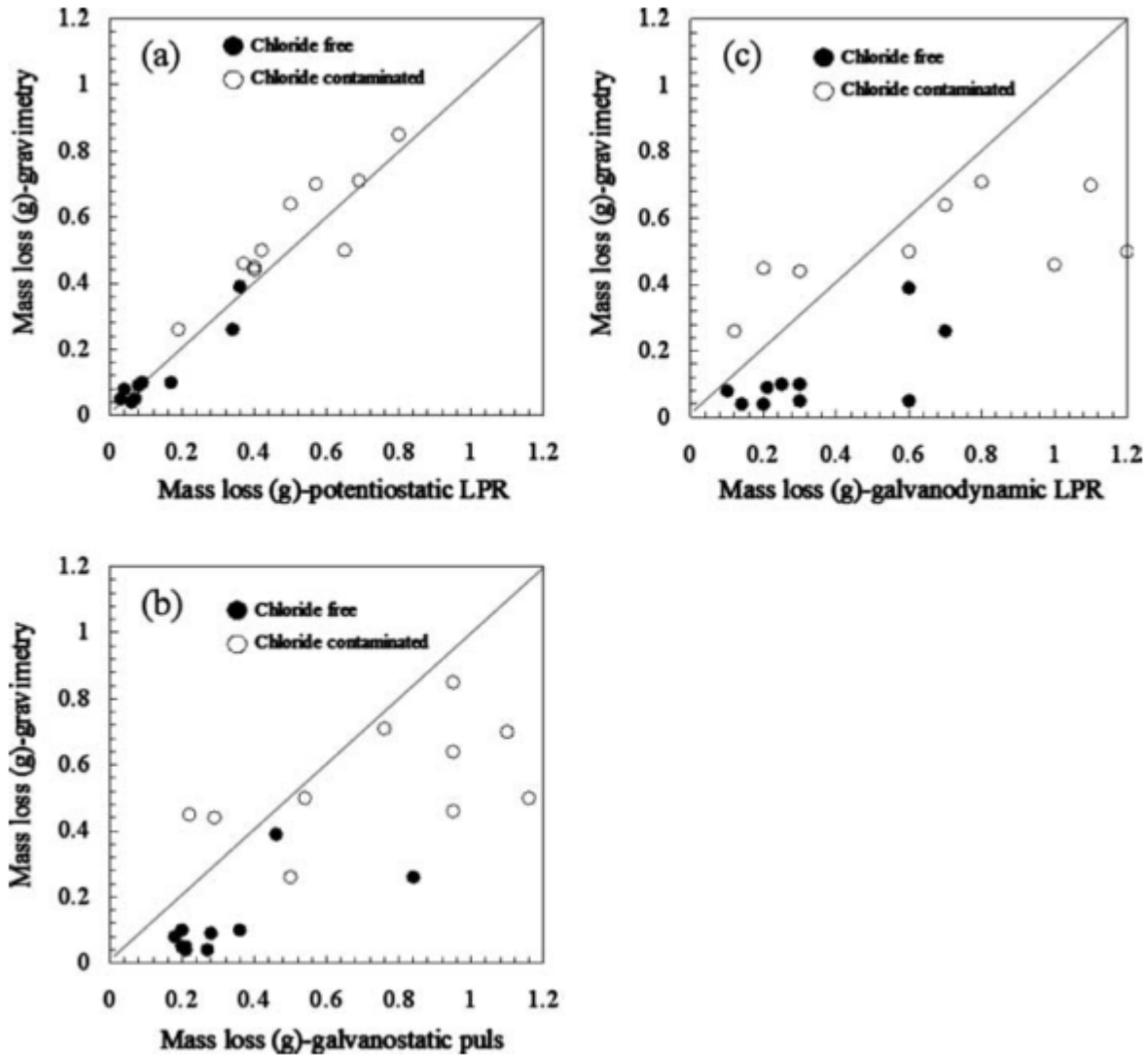


Figure 2.42: Comparison of Results (reproduced from Poursaei, 2011)

Gepreags and Hansson (2005) determined that the main reason behind the varied results with the Gecor 6 and the GalvaPulse devices is not the electrochemical techniques themselves, but instead the results of differing electrode configurations and maybe, the electrode material itself. This was concluded by connecting the Gecor 6 electrode to the GalvaPulse device. The guard ring current/potential changes the polarization of the reinforcement with respect to the central reference electrode. It is assumed that the guard ring electrode is unable to apply a discrete current independent of the counter electrode. It was concluded that this electrode effect is more observed in the Gecor 6. When the corrosion rate data of each device was analyzed independently, it showed that both devices had consistent results when the guard ring was not used.

As measured corrosion rates are a function of both the confinement technique and the electrochemical techniques used, Nygaard and Geiker (2009) presented a quantitative assessment of these differences. This study is a continuation of the experimental work by Gepraegs and Hansson (2005), discussed previously. This experimental work monitored the operation of two devices, the Gecor 6 and the GalvaPulse, and the corresponding current distribution between the devices' electrode assembly on the concrete surface and the reinforcement within. With this study, it was concluded that both the Gecor 6 and the GalvaPulse over-estimate the actual corrosion rate for passive reinforcement: Gecor 6 by a factor of about 10 and the GalvaPulse by a factor of 100. This overestimation was attributed to each device being unable to effectively confine or compensate for the spread-out effect of the counter electrode current. In addition, both devices polarize the reinforcement beyond the linear potential-current region, which yields errors with the corrosion rate calculation. In the case of localized corrosion activity, it was concluded that both devices did not accurately locate the active corroding area due to the phenomenon of "self-confinement", where the current applied from the counter electrode and the guard ring flows into the reinforcement at the location of active corrosion regardless of the location of the sensor on the concrete surface. This occurs because the current follows the path of smallest resistance to the reinforcement, refer to Figure 2.43. However, both devices underestimated the corrosion rates in these active areas; Gecor 6 by a factor of 6 and GalvaPulse by a factor of 2. This underestimation was attributed to both devices using a constant confinement length (seen to be larger than the actively corroding region) and the final obtained confinement.

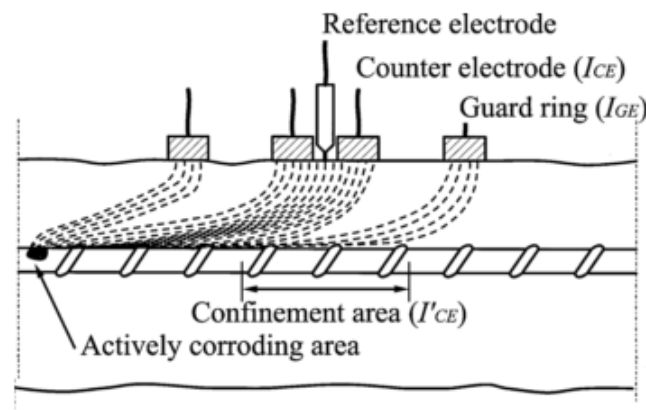


Figure 2.43: Concept of Self-Confinement (Nygaard & Geiker, 2009)

2.5.4. Pitfalls of Electrochemical Techniques

The use of electrochemical techniques for the determination of corrosion rate cannot pick up on the presence of micro-cell action (Gulikers & Polder, 2003). This is of concern because the local dissolution

rate is accelerated greatly due to this large cathode to anode surface area ratio. When reinforcement is subjected to localized corrosion attack, a non-uniform distribution will occur, which will shift the potential over the surface area of the working electrode (Gulikers & Polder, 2003). As a result, an erroneous value of polarization resistance will be determined that may not reflect the local corrosion rate. In addition, by definition, potential readings are measured not on the reinforcement but on the concrete surface itself.

Verma et al. (2014) laid out several limitations with the use of half-cell potential measurements on reinforced concrete structures. Half-cell data only provides a probability of corrosion and no assessment of the corrosion rate. Interpretation of half-cell data requires careful consideration as variations in moisture content, chloride content, and concrete electrical resistance can influence the measurements.

So and Millard (2007) discussed sources of error within the LPR technique: (1) as polarization resistance is not measured from the surface of the reinforcement but from the concrete surface, the concrete resistance must be independently measured and subtracted from the overall polarization resistance measured; (2) it is difficult to choose a suitable equilibrium time for the current response to the applied potential shift at which measurements are taken; and, (3) it is difficult to compensate for all resistance components not directly associated with the corrosion rate.

Gulikers and Polder (2003) provide valuable information on the concerns with GPT. In the application of GPT on-site, the magnitude of the perturbing electrical signal that is applied can be of concern. If low currents are used, it becomes difficult to distinguish the potential response from background noise. In opposition, high currents are better for the pulse transient definition, but are not suitable for the corrosion rate determination. In addition, for larger perturbations, it is less likely that inadequate linearity of the current-potential relationship can be established for the use of the Stern-Geary equation, causing errors in the calculation of the corrosion rate. In conclusion, the magnitude of the applied current should be used to provide for a larger potential shift but remain within the linear region. It is important to note that unrealistically high corrosion rates have been measured with GPT in passive steel.

As far as experimental setups, Poursaee and Hansson (2009) found that there are several factors that should be considered with chlorides mixed in the concrete. Most importantly, this is not an accurate representation of the real-life scenario for deteriorating reinforced concrete structures, as the steel does not have time to passivate before encountering chlorides, and thus the concept of localized passive film breakdown as the first step in corrosion initiation must be considered. In addition the following adverse effects due to the presence of chloride ions can be noted: (1) acceleration of cement hydration which

increases the concrete porosity; (2) increase in the conductivity of the pore solution; (3) changes in the pH of the pore solution; and, (4) changes in the amount of chemically bound chlorides. All of these considerations must be taken into account when evaluating corrosion rate in laboratory settings.

2.6. Gaps in the State of the Art

The information presented in this literature review has shown the following:

- Although use of half-cell potential, as per ASTM C876, is still widely used in practice, half-cell potential data alone cannot be used for condition assessment of reinforced concrete structures.
- Several different condition assessment techniques should be used for a more complete assessment of deteriorating reinforced concrete structures.
- There is significant research to show that the use of electrical resistivity has proven to be a useful tool in condition assessment.
- The use of LPR and GPT in laboratory or on-site experimental studies has shown varying results.
- There is a need for a more reliable and constant electrochemical technique.
- On-site studies are rare and dated. More importantly, experimental studies that compare condition assessment results for both laboratory testing and on-site testing are scarce.

Laboratory studies on corrosion of steel in concrete employ regulated conditions and accelerated corrosion measures to allow for a controlled study. Where this type of study is required to precisely analyze all aspects of the experiment, corrosion measurement techniques for field application require a practical and efficient strategy. As it can be inferred, there is a strong need for experimental programs that assess both laboratory and on-site condition assessment data for the purpose of evaluating non-destructive devices to provide more confidence with measurements of corrosion rate in actual structures. This is the motivation behind this program of study. .

3.0 Experimental Program

This chapter presents a detailed description of the experimental program carried out in this research to make an independent assessment of the accuracy of the Connectionless Electrical Pulse Response Analysis (CEPRA) technique. This experimental program is conducted in two parts, both parts incorporating non-destructive and destructive testing of:

- Portions of existing reinforced concrete slabs, extracted from the North Channel Bridge, Ontario, constructed in 1957, henceforth referred to as the Field Study.
- Multiple, small-scale reinforced concrete slabs, constructed and monitored in a controlled environment, henceforth referred to as the Laboratory Study.

For both Field and Laboratory Study, corrosion rate, half-cell potential, and resistance or electrical resistivity are determined by the following methods:

- Linear Polarization Resistance (LPR), as measured by James Instrument's Gecor 6
- Galvanostatic Pulse Technique (GPT), as measured by Germann Instrument's GalvaPulse
- Connectionless Electrical Pulse Response Analysis (CEPRA), as measured by Giatec Scientific's iCOR
- Four-point electrical resistivity, as measured by Proceq's Resipod Resistivity Meter

In addition, several addition non-destruction and destructive tests were conducted on concrete and rebar specimens to provide supporting information on the condition of the reinforced concrete, namely: moisture content and absorption, compressive strength, steel mass loss, and concrete carbonation and chloride content.

3.1. Evaluation of Deterioration by NDT Devices

3.1.1. Linear Polarization Resistance

The Linear Polarization Resistance technique was employed by the Gecor 6 device, manufactured by James Instruments, to measure the corrosion rate, half-cell potential, and resistance of reinforced concrete slabs. An in-depth explanation of the theory behind the technique and the equations used by the Gecor 6 is provided in Section 2.4.2. The Gecor 6 measures the polarization resistance, R_p , with the use of a "guard ring" to confine the applied current from the counter electrode. The Gecor 6 sensor has two circular stainless steel rings. The inner ring acts as the counter electrode with an inner diameter of 11

mm and an outer diameter of 70 mm. The outer ring acts as the “guard ring” and has an inner diameter of 140 mm and an outer diameter of 180 mm. The diameter of the entire sensor is 180 mm.

The device utilizes a Cu/CuSO₄ reference electrode and CU/CuSO₄ confinement sensor electrode or “guard ring” with a CuSO₄ liquid solution to be filled in the three reservoirs (James Instruments, n.d.). In order to take a measurement, the device needs to be connected to the reinforcement within the concrete. For best contact to ensure a good electrical connection, wetted sponges should be placed in between the sensor and the concrete while a constant pressure is applied. In addition, the concrete surface should be prewetted and conductive gel should be used. This setup can be seen in Figure 3.1.



Figure 3.1: Setup for Gecor 6

Once the device is setup, the Gecor 6 requires that the user inputs the confined area of the reinforcement in cm², as seen in Eq. 3-1:

$$A = 3.14 \times D \times 10.5 \quad \text{Equation 3-1: Confined Area (James Instruments, n.d.)}$$

where D is the diameter of the reinforcement in cm.

Once the polarization area has been inputted, the Gecor 6 begins the test by measuring the corrosion potential, E_{corr} , through the reference electrodes until the difference between the corrosion potential readings become stabilized. After stabilization, an initial current pulse is applied to the system by the counter electrode, and the potential response is recorded so that the magnitude of the optimum current,

i_{ce} , can be calculated and used to acquire the polarization resistance, R_p . Next, a constant current is applied for a duration of 50 seconds through the counter electrode. This current alters the potential of the reinforcement with respect to time. During this polarization time, a secondary current is applied through the guard ring to ensure that the potential difference between the auxiliary reference electrodes is equal to those potential values recorded in a previous step.

The polarization resistance acquired is corrected for the ohmic potential drop, and then the corrosion rate is calculated according to Eq. 3-2, i.e.,

$$i_{corr} = \frac{K}{A \times R_p} \quad \text{Equation 3-2: Corrosion Rate (James Instruments, n.d.)}$$

where i_{corr} is the corrosion rate expressed in $\mu\text{A}/\text{cm}^2$, A is the polarization area as determined in Eq. 3-2, and K is a constant value equal to 26 mV.

Several issues were noted while testing with the Gecor 6: (i) measurements were time-consuming, taking up to five minutes per measurement; (ii) it requires the user to know the diameter of the reinforcement in consideration, which is not always the case for in-situ testing; and, (iii) a “confinement not fully achieved” error message appears frequently, a consequence of a bad electrical connection between the sensor and the concrete or between the rate meter and the reinforcement or the result of high concrete resistivity (James Instruments, n.d.) .

Typical corrosion rate values accompanied with corrosion classification for LPR can be found in Section 2.4.2.1, Table 2.3.

3.1.2. Galvanostatic Pulse Technique

The Galvanostatic Pulse technique was employed by the GalvaPulse device, manufactured by Germann Instruments, to measure the corrosion rate, half-cell potential, and resistance of the reinforced concrete slabs. An in-depth explanation of the theory behind the technique and the equations used by the GalvaPulse is provided in Section 2.4.3.

The GalvaPulse is a portable device which consists of a sensor connected to a power source/data recorder. The sensor consists of an Ag/AgCl reference electrode, a counter electrode and a “guard ring”. A measurement works as follows: a current pulse is imposed on the reinforcement from the counter electrode, which is placed on the concrete surface. The “guard ring” confines the current to an area A (length of 7 cm) of the reinforcement below the central counter electrode. This applied current is typical in the range of 5 to 400 μA , and the typical pulse duration is 5 to 10 seconds. This polarizes the

reinforcement in the anodic direction and is then compared to its free corrosion potential, E_{corr} . This results in a change of the electrochemical potential of the reinforcement, which is recorded by the device, as a function of the time used by the reference electrode. As the current is applied to the system, an ohmic potential drop and a change in potential due to the polarization of the reinforcement occur. These factors are taken into account during the determination of the corrosion rate, which is determined by means of the Stern-Geary equation, Eq. 2-5. The constant K is equal to 26 mV.

The GalvaPulse recommends that the applied current range be from 80 to 100 μA and the duration be from 5 to 10 seconds for active areas. For very active areas, the current can be increased to 400 μA to obtain reasonable polarization. However, it is always recommended to use as low a current and time as possible to minimize the polarization of the reinforcement (Germann Instruments, 2011). For all testing conducted in this work with the GalvaPulse device, a current of 100 μA was applied for a period of 10 seconds.

Similar to the Gecor 6 device, the GalvaPulse needs to be connected to the reinforcement within the concrete. Prewetting of the concrete surface should be done. In addition, wetted sponges and conductive gel should be used to ensure a good electrical connection between the sensor and the concrete surface.

A depiction of the setup for the GalvaPulse device is seen in Figure 3.2.



Figure 3.2: Setup for GalvaPulse

Several issues were noted while testing with the GalvaPulse: (i) calibration of the device was difficult and required several attempts, and (ii) it requires the user to know the diameter of the reinforcement in consideration, which is not always the case for in-situ testing.

Several corrosion rate ranges are provided by the GalvaPulse Manual. Refer to Section 2.4.3.2, Table 2.4.

3.1.3. Connectionless Electrical Pulse Response Analysis

The Connectionless Electrical Pulse Response Analysis (CEPRA) technique is utilized in the iCOR device, manufactured by Giatic Scientific, to measure the corrosion rate, half-cell potential, and electrical resistivity of the reinforced concrete slabs. An in-depth explanation of the theory behind the CEPRA technique used by the iCOR is provided in Section 2.4.4.

Using the CEPRA technique, a DC/AC current pulse is applied to the reinforced concrete for a brief time period, while the voltage response of the system is recorded concurrently (Giatic Scientific Inc., iCOR User Manual, 2016). Using the value of current and voltage applied, along with the recorded low-frequency

impedance response of the reinforcement, the iCOR device provides a corrosion rate value. The electrical circuit model used to predict the different properties of the concrete materials and steel reinforcement can be seen in Section 2.4.4, Figure 2.32. In the figure, R_{c1} is the contact resistance, R_{c2} is the bulk electrical resistance of the concrete, R_{c3} is the electrical resistance of the concrete cover, R_{c4} (R_p) is the polarization resistance of the rebar, and C_{dl} is the double layer capacitance.

The iCOR device employs a mathematical algorithm that incorporates the parameters from the equivalent electrical circuit model to provide a condition assessment of the reinforcement concrete structure (Giatec Scientific Inc., iCOR User Manual, 2016). In addition, the device provides half-cell potential and electrical resistivity results. Electrical resistivity is determined from the surface electrical resistivity technique, where the equation used by the iCOR is modified from Eq. 3-3, i.e.,

$$\rho = 2\pi aR \quad \text{Equation 3-3: Surface Electrical Resistivity}$$

where a is a constant parameter determined from the geometry of the sensor electrodes, and R is the equivalent resistance of the concrete calculated from R_{c2} , R_{c3} , and R_{c4} .

From the equivalent electrical circuit model, Figure 2.3.2, the iCOR determines parameter R_{c4} , using the Stern-Geary equation, Eq. 2-5, the polarization resistance by iCOR is calculated, as seen in Equation 3-4:

$$R_p = A_p R_{c4} \quad \text{Equation 3-4: Polarization Resistance by CEPR}$$

where A_p is the polarized area of the reinforcement, R_{c4} is the charge transfer resistance of the reinforcement, and the constant K is equal to 26 mV.

Unlike the previously mentioned devices, the iCOR device comes with a tablet that has a data monitoring application installed. This application allows for a more user-friendly interface and the exportation of results as contour maps or Excel files. At the start of the project, the application allows for the user to create projects with grids of user-decided specifications. In addition, the application requires the user to input the diameter of both the vertical and horizontal reinforcements. For each grid point, the user can alter the measurement duration, measurement direction, and cover thickness. Outputs are corrosion rate, concrete resistivity, corrosion potential, temperature, and relative humidity. The majority of tests in this research were conducted with 6-second measurement duration. The setup of the iCOR device is seen in Figure 3.4.



Figure 3.3: Setup of iCOR

Unlike the previously mentioned devices, the iCOR device does not need to have a physical connection to the reinforcement within the concrete. Corrosion rate and electrical resistivity assessment can be done without a connection. However, half-cell potential assessment needs an electrical connection to the reinforcement.

Similar to the previous devices, prewetting of the concrete surface should be done. In addition, the sensors should be wetted, and conductive gel should be applied. It has been the experience of the author that if the concrete surface is not sufficiently wet, the iCOR device is unable to take a measurement. In addition, there are times when the device is unable to take measurements, regardless of adequate testing conditions.

Several issues were noted while testing with the iCOR: (i) sensors were highly sensitive to moisture conditions on the concrete surface; (ii) it requires the user to know the diameter of the reinforcement in consideration, which is not always available for in-situ testing; and, (iii) the device is sensitive to the battery status of the sensors even when the sensors are considered fully charged.

Several corrosion rate ranges are provided by the iCOR Manual and can be seen in Section 2.4.4.1, Table 2.5

3.1.4. Electrical Resistivity

Two different forms of electrical resistivity were employed in this experimental program: surface electrical resistivity and bulk electrical resistivity.

3.1.4.1 Surface Electrical Resistivity

The surface electrical resistivity was measured using Proceq's Resipod Resistivity Meter, a four-probe system. The Resipod, following the requirements of ASTM G57 (2012) for the four-probe system, which is also known as the Wenner probe, impresses a current through the concrete using the outer probes, while simultaneously measuring the voltage difference with the inner probes (ASTM G57, 2012). The electrodes touch the concrete surface through wet sponges at the tip of each electrode. A general schematic of a Wenner technology setup can be seen in Figure 3.6.

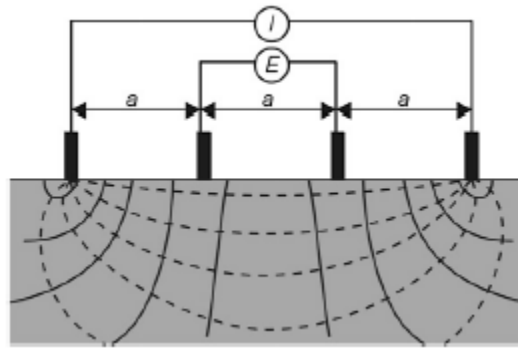


Figure 3.4: Schematic of Wenner Technology (Lataste J.-F. , 2010)

Modified from Eq. 2-1, the surface electrical resistivity is expressed for a semi-infinite, homogenous material as seen in Eq. 3-5:

$$\rho = 2\pi a \frac{V}{I} \quad \text{Equation 3-5: Surface Electrical Resistivity}$$

where: ρ represents the surface resistivity of the concrete, a is the electrode spacing, as illustrated in Figure 3.6, V is the voltage measured between the inner probes, and I is the current applied across the outer probes. The term $\frac{V}{I}$ is known as the resistance (R) or the impedance (Z) value. Modifications to the above equation are needed in situations where the outside probes are not spaced at the same interval as the inner probes (ASTM G57, 2012). During the design of the reinforced concrete slabs for the Laboratory Study, it was ensured that the maximum aggregate size of the concrete was not larger than the probe spacing a , as this would record the electrical resistivity of a single aggregate piece (Broomfield, 2007). Additionally, it is important to note that reinforcement actually conducts current better than concrete. For this reason, measurements taken near the reinforcement will yield results that are not ideal and

erroneous (Polder, 2000). Therefore, the surface electrical resistivity measurements were taken parallel to and in-between the rebars.

Typical ranges for concrete surface resistivity provided by the Resipod, expressed in $\Omega\cdot m$, can be seen in Table 3.4.

Table 3.1: Corrosion Rate Ranges for iCOR (Giatec Scientific Inc., iCOR User Manual, 2016)

Resistivity ($\Omega\cdot m$)	Corrosion Rate Severity
>200	Low
100 – 200	Low to moderate
50 – 100	High
< 50	Very high

3.1.4.2 Bulk Electrical Resistivity

The bulk electrical resistivity was measured using Giatec Scientific's RCON device. The RCON applies an alternating current to the concrete, which allows the dipoles of ions in the pore solution to direct the electric current (Layssi et al., 2015). This results in a non-resistive opposition to the current in the circuit known as reactance, an imaginary component (Giatec Scientific, 2016). This poses a problem in the determination of the electrical resistivity of the concrete, as there is now two oppositions to the current: the normal resistance of the concrete (R) and the reactance (X). This joint opposition to the current is the concept of impedance (Z), which is considered a complex number and is considered the vector sum of the normal resistance (R) on the real axis and the reactance (X) on the orthogonal imaginary axis (Layssi et al., 2015). The impedance is represented as a magnitude $|Z|$, and ϕ represents the phase angle, as shown in Figure 3.7. It is important to note that changing the frequency imposed on the system will change the magnitude of the reactance and phase angle (Giatec Scientific, 2016). It has been recommended that using a phase angle from 0° to 3° allows for a reactance value close to the resistance value. This enables an easier and more precise calculation of the resistivity.

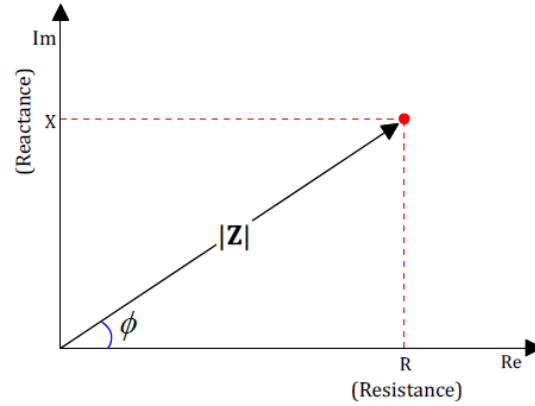


Figure 3.5: Graphical Representation of Impedance (Giatec Scientific, 2016)

Using the alternating current technique discussed above to acquire values of impedance for a concrete specimen, the bulk electrical resistivity can be determined from Eq. 3-6:

$$\rho = \frac{A}{L} Z \quad \text{Equation 3-6: Bulk Electrical Resistivity}$$

where: ρ represents the bulk resistivity, A is the cross-sectional area of the specimen, L is the length of the specimen, and Z is the impedance (resistance). The RCON device used in this research is shown in Figure 3.7.

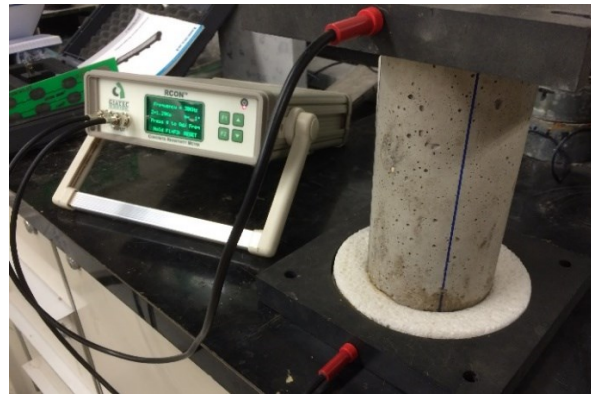


Figure 3.6: Setup for RCON

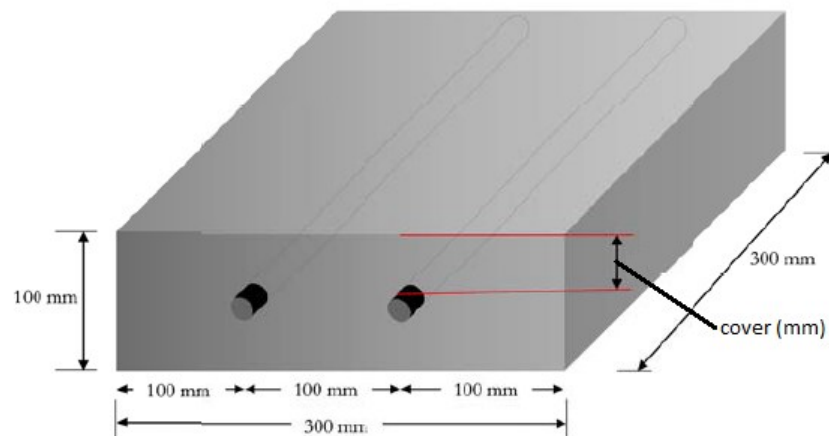
3.2. Laboratory Study

The Laboratory Study involved the construction of multiple small-scale reinforced concrete slabs that were cast and tested using the devices previously mentioned over a period of nine months, to monitor the progress of reinforcement corrosion. These reinforced concrete slabs vary in cover thickness, rebar diameter, and initial chloride contents, so that various degrees of corrosion activity on the steel reinforcement could be achieved.

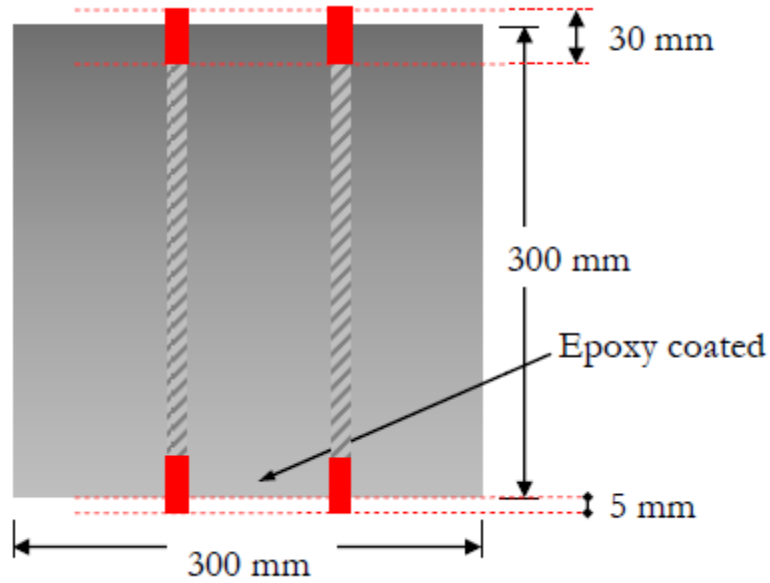
This section presents a detailed description of the slab specifications, concrete mix design and material properties, material testing, and deconstruction of the slab specimens for destructive measurements.

3.2.1. Slab Specifications

Eighteen (18) small-sized reinforced concrete slabs were built and tested for the Laboratory Study. The slabs are rectangular with dimensions of 300 mm x 300 mm x 100 mm (see Figure 3.9 (a) and (b)). All slabs have two industry standard ribbed reinforcing bars. To determine the effect of rebar diameter d_b on the measurements, two rebar sizes are used: 10 M or 20 M. These rebars are located at two different cover thickness: 30 mm and 60 mm. The 60 mm cover thickness is specified in CSA A23.1-14, which requires concrete slabs exposed to chlorides (Class C-XL) to have a minimum cover of 60 mm (A23.1-14/A23.2-14, 2014). Two different concrete mixes are used, one with a water-to-cement ratio (w/c) of 0.62 and the other with a water-to-cementitious material (w/cm) ratio of 0.47. The 0.47 w/cm ratio is in agreement with CSA A23.1-14 Table 2 requirement for concrete in a C-XL or A-XI exposure class, or concrete exposed to chlorides (A23.1-14/A23.2-14, 2014). In addition, the spacing of the reinforcement is in agreement with CSA A23.1-14 clause 6.6.5.2, which requires that the clear distance between parallel bars shall not be less than $1.4 d_b$, $1.4 \times$ nominal maximum size of coarse aggregate, and 30 mm (A23.1-14/A23.2-14, 2014). The clear distance between parallel bars is 88.7 mm for the slabs with 10M rebars and 80.5 mm for the slabs with 20 M. Corrosion was induced by mixing sodium chloride into the concrete mix. Three different chloride contents are used: 0% Cl^- as the control specimen, 1.5% Cl^- and 3% Cl^- by weight of cement.



(a) 3D view



(a) Top view

Figure 3.7: Reinforced Concrete Slab Dimensions

In order to ensure that corrosion of the rebar ends did not occur, Performix Plasti Dip® Multi-Purpose Rubber Coating was applied to the rebar ends, as illustrated in Figures 3.8 (b) and 3.9.



Figure 3.8: Pre-casting formwork

A summary of the varying parameters seen in this experimental setup, with the corresponding identifier, can be found in Tables 3.5 and 3.6.

Table 3.2: Mix #1 Experimental Testing Grid

Mix #1			
Chloride content (by weight of cement)	cover thickness (mm)		
	A* 30 mm	B° 30 mm	C° 60 mm
Control (0% Cl)	10M-MIX1-A-30-0%	20M-MIX1-B-30-0%	20M-MIX1-C-60-0%
1.5% Cl	10M-MIX1-A-30-1.5%	20M-MIX1-B-30-1.5%	20M-MIX1-C-60-1.5%
3.0% Cl	10M-MIX1-A-30-3%	20M-MIX1-B-30-3%	20M-MIX1-C-60-3%
* A series are made with 10M bars ° B and C series are made with 20M bars			

Table 3.3: Mix #2 Experimental Testing Grid

Mix #2			
Chloride content (by weight of cementitious material)	Cover thickness (mm)		
	A* 30 mm	B° 30 mm	C° 60 mm
Control (0% Cl)	10M-MIX2-A-30-0%	20M-MIX2-B-30-0%	20M-MIX2-C-60-0%
1.5% Cl	10M-MIX2-A-30-1.5%	20M-MIX2-B-30-1.5%	20M-MIX2-C-60-1.5%
3.0% Cl	10M-MIX2-A-30-3%	20M-MIX2-B-30-3%	20M-MIX2-C-60-3%
* A series are made with 10M bars ° B and C series are made with 20M bars			

3.2.2. Concrete Mix Design and Materials

Two concrete mix designs with normal density concrete were used. Mix #1 was provided by the industry partner and has a w/c of 0.62. Mix #2 was produced at the University of Ottawa and has a w/cm of 0.47.

Essentially a w/c of 0.62 provides for a poorer concrete and enables an environment where the reinforcement corrosion can thrive. On the opposing end, according to CSA A23.1-14, reinforced concrete exposed to a chloride environment should have a w/c with a maximum value of 0.4. Both concrete mix designs can be found in Table 3.7.

Table 3.4: Mix Designs (per m³ of concrete)

Mix #1 – Industry Partner		
GU/Type 10 Cement	265	kg/m ³
20-mm Aggregate	1055	kg/m ³
Sand	940	kg/m ³
Water	165	kg/m ³
Mix #2 – University of Ottawa		
GU/Type 10 Cement	183	kg/m ³
Slag	138	kg/m ³
Fly Ash, type unknown	69	kg/m ³
20-mm Aggregate	1047	kg/m ³
Sand	824	kg/m ³
Water	183	kg/m ³
Plasticizer	280	ml / 100 kg of cement

The water content of the mix was adjusted to account for the moisture state of the aggregates. To do so, the fineness modulus, relative density and absorption, bulk density, and moisture content for both fine and coarse aggregates were tested according to CSA A23.2-14 2A, 6A, 10A, and 12A procedures. Full details of the concrete mix design, material properties, and laboratory test results can be found in Appendix A.

3.2.3. Casting and Curing

In order to meet the specifications needed for the slabs, the formwork was constructed, by hand, with standard plywood and screws. The constructed formwork with the epoxy coated rebars placed inside the formwork are seen in Figure 3.10.



Figure 3.9: Completed formwork ready for concrete pouring

Concrete mixing and pouring took place over a period of two days. Mix #2 was mixed and poured first. This was done in three batches, as there were three concentrations of NaCl. The NaCl was mixed into the water. In addition, three cylinders were poured from each batch for compression testing. The slabs cured for 24 hours, while covered in burlap. On the second day, the formwork was taken apart, the slabs removed, and the formwork re-constructed. Mix #1 was then mixed and poured into the re-constructed formwork and new cylinders were cast. Again, three batches were mixed to reflect the three NaCl concentrations. These slabs were also allowed to cure for 24 hrs under a burlap.

As expected, the slabs were not constructed exactly as specified by the experimental plan. Upon removal from the forms, it was determined that two slabs from Mix #1, 10M-MIX1-A-30-0% and 20M-MIX1-C-60-0%, were poorly cast, unsuitable for testing, and were recast the following week. Refer to Appendix A for details and photos. In addition, a few slabs from Mix #1 and #2 were poorly vibrated as voids were discovered at the underside of the slab. For these slabs, a standard 2:1 sand/cementitious material mortar was used to fill the voids. In addition, the concrete cover was found to slightly deviate from the one specified. However, the exact cover thickness was inputted into the non-destructive devices when applicable.

The temperature in the laboratory stayed around 21°C, and the relative humidity stayed around 50%. Please see Appendix A for further details of the concrete mix production.

After removal, the slabs were placed in sealable plastic containers. The slabs rested on wooden supports to prevent contact with the pool of tap water at the bottom of the sealable plastic container. Water was placed under the slabs to simulate 100% relative humidity. The slabs were removed from the sealed environment at week 28, so that the effect of a drier environment could be monitored as well. It is important to note that the confined environment was broken during weekly testing. Tap water was occasionally added to the sealable container to maintain 100% relative humidity.

3.2.4. Material Testing

3.2.4.1. *Compression Test*

Concrete cylinders were cast along with the slabs to test the concrete compressive strength at 28 days. Following CSA A23.2-9C, the nine concrete cylinders corresponding to each mix design were placed inside a concrete grinder and subjected to several cycles to ensure that the ends were flat and smooth. For a good portion of the cylinders, poor mixing was seen. To ensure that the load applied during the compression testing was evenly distributed, rubber caps were placed on the ends of the cylinders during testing. The average compression strength of Mix #1 was 30.1 MPa and for Mix #2 46.7 MPa. As the concrete mix design for Mix #2 has a w/cm of 0.47 while Mix #1 has 0.62, the compression strength averages reflect the expectation that mix #2 is stronger than mix #1. All results can be seen in Appendix C.

3.2.4.2. *NDT Testing*

For a period of nine months, non-destructive testing was conducted on the laboratory slab specimens. For a detailed explanation of how each NDT test was performed, see Section 3.1. As all devices required a connection to the reinforcement to acquire all information relevant to this study, stainless steel self-tapping screws were drilled into the epoxy-coated ends. Half way through the testing period, the screws were replaced. To ensure that the polarization of the reinforcement from a previous test would not affect the polarization of a future test, the testing schedule in Table 3.8 was followed. The grid points specified in Table 3.8 are shown in Figures 3.11 and 3.12.

Table 3.5: NDT Testing Schedule and Grid

Day (out of 7 day schedule)	Test	Grid Point
1	Resipod	C & D & E
1	iCOR	A & B
2	GalvaPulse	A & B
3	Gecor	A & B

Corrosion rate, half-cell potential, and resistance or electrical resistivity were measured using different methods: linear polarization resistance (as measured by the Gecor 6), galvanostatic pulse technique (as measured by the GalvaPulse), connectionless electrical pulse response analysis (as measured by the iCOR), and finally four-point electrical resistivity (as measured by the Resipod). All corrosion rate assessment techniques require that the sensors be placed directly over both rebars within the concrete. This can be seen in Figure 3.11. As previously stated, the four-point probe provides best electrical resistivity results with the sensor placed parallel to and in-between the reinforcement, as illustrated in Figure 3.12.

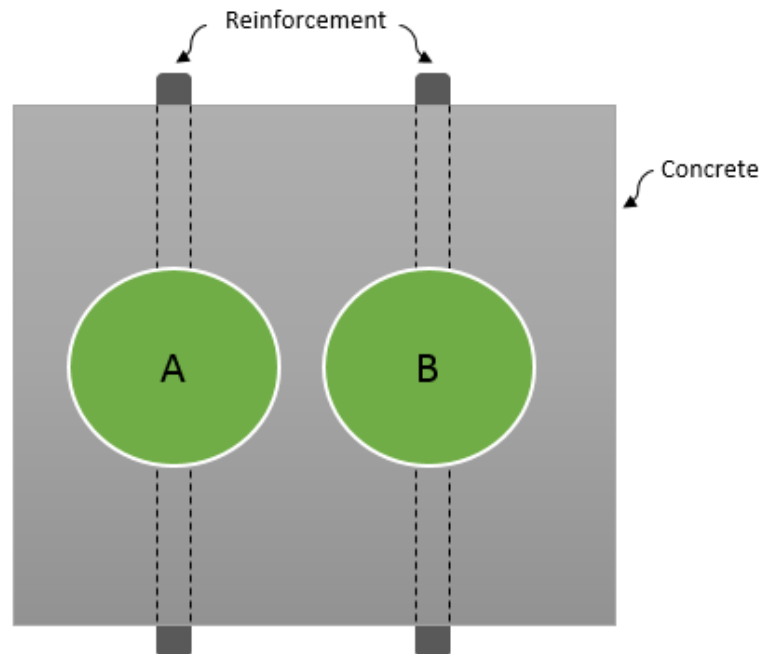


Figure 3.10: Laboratory NDT Testing Grid: Overtop of Reinforcement

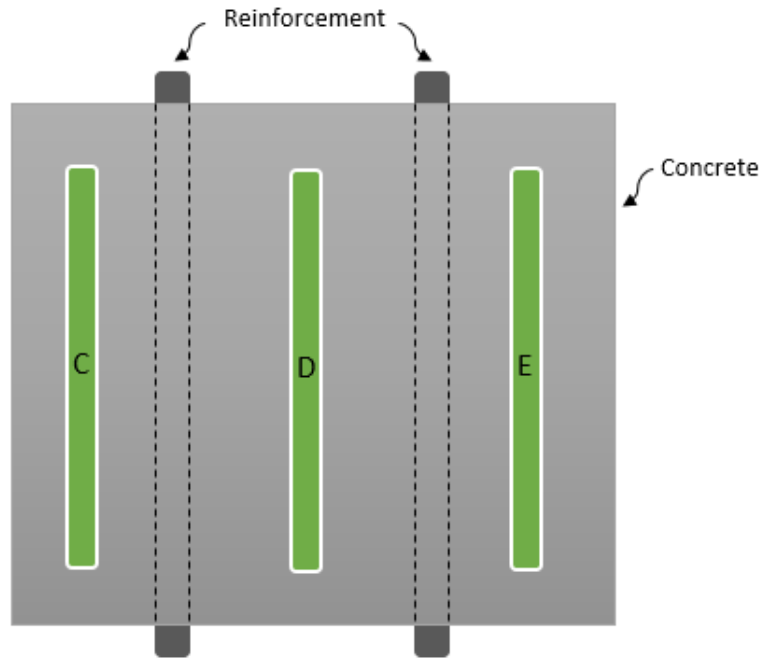


Figure 3.11: Laboratory NDT Testing Grid: In-between Reinforcement.

3.2.4.3. Gravimetric mass loss

After the nine-month non-destructive testing period, the rebars within the concrete slab specimens were removed. This was accomplished by subjecting the slabs to a modified split cylinder test, see Figures 3.14 and 3.15. This modified split cylinder test subjected the reinforced concrete slabs to a concentrated compressive force parallel to the reinforcing bars resulting in the concrete “splitting” along the cast-in reinforcing. This allowed for the rebars to be easily removed from the concrete. Rebar specimens were labelled and photographed. The epoxy-coated ends were cut off, so that just the area of the reinforcement exposed to the corrosive environment was tested. Gravimetric mass loss testing was conducted according to ASTM G1-03 (2011). Corroded rebar specimens were subjected to several cycles in an acid bath in order to remove the rust products. Several control rebars were subjected to the acid bath as well (see Figure 3.16). For a more detailed explanation of the theory behind gravimetric mass loss see Section 2.4.1. For a step-by-step detailed testing procedure and accompanying explanation of how to use the raw results as seen in ASTM G1, see Appendix D.



Figure 3.12: Modified Split Cylinder Test for Laboratory Rebar Removal



Figure 3.13: Laboratory Rebars Within Split Concrete



Figure 3.14: Gravimetric Mass Loss Testing according to ASTM G1-03.

As stated previously, the corroded rebars were photographed prior to gravimetric mass loss testing in the acid bath. Two photos for each rebar were taken, top and bottom. These photos were run through an image analysis software, *ImageJ*, a Java-based image processing program developed by the National Institute of Health and the Laboratory for Optical and Computational Instrumentation. This photo analysis allowed for a rough estimate of the percentage of the rebar that was corroded and the percentage of the rebar covered with mortar products. An image of one of the original rebars is seen in Figures 3.16. Figure 3.17 is a snapshot of the *ImageJ* application showing the percentage of mortar on the surface of the rebar. Note that the percentage of mortar content on the surface is determined as a percentage of the total threshold of the image. Figure 3.18 is a snap shot of the *ImageJ* application showing the percentage of corroded products, calculated on the opposite threshold range for the image.



Figure 3.15: ImageJ Illustration, original image

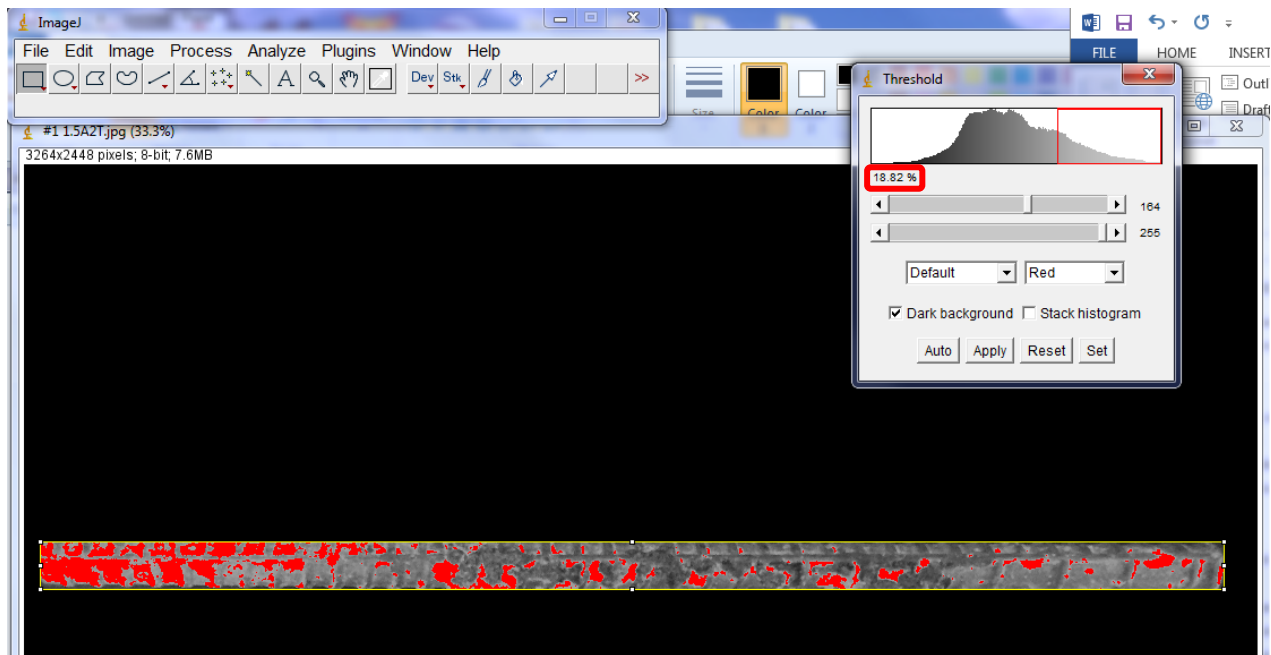


Figure 3.16: ImageJ Illustration, mortar content

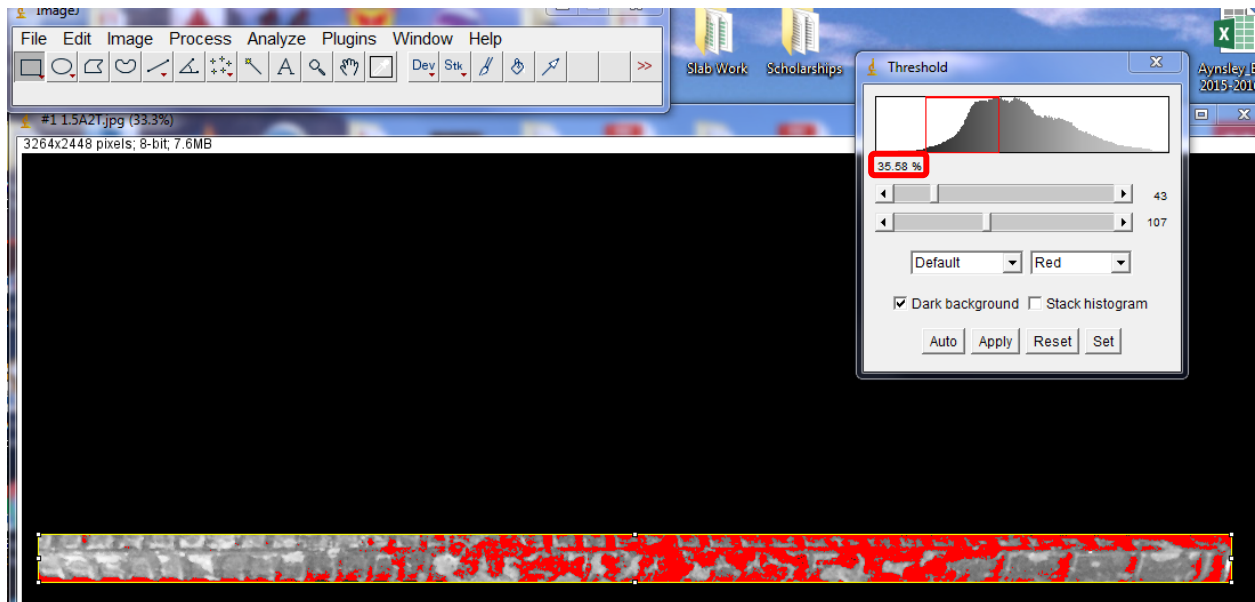


Figure 3.17: ImageJ Illustration, corrosion products

3.3. Field Study

The Field Study consisted of one trip to Cornwall, Ontario to retrieve a comprehensive collection of data from four slabs, S11, S12, S13, and S14, taken from random locations along the North Channel Bridge as seen in Figure 3.19. This collection of data included: data on corrosion rate by GPT and CEPRA, half-cell potential, and resistance or electrical resistivity by use of CEPRA, GPT, and four-point electrical resistivity.

In addition, concrete core samples and the corroded reinforcement were retrieved from the slabs, taken back to the laboratory and subjected to testing that included: compression testing, carbonation and chloride penetration depth, chloride concentration, and finally, the level of corrosion by gravimetric mass loss.



Figure 3.18: North Channel Bridge Slabs

3.3.1. Background Information

The North Channel Bridge, more commonly referred to as the Seaway International Bridge, crossed the St. Lawrence River and North Cornwall Channel providing an international crossing from Canada to the United States. Completed in July of 1962, the North Channel Bridge was constructed with an arched truss superstructure and concrete substructure to support long continuous spans. Additionally, the North Channel Bridge was construction high off the ground to allow for large shipping barges to cross underneath. The concrete deck was typically 200 mm in thickness and 8.2 metres wide. The top reinforcing consisted of 20M reinforcing bars spaced at 300mm each way with an average in-situ cover of 50 mm. Overtime the superstructure fared well against the elements, however, the concrete deck deteriorated. The cost to replace the deck was substantial and it was decided to demolition and replace the bridge. Demolition was completed July 2015.

3.3.2. North Channel Bridge Slabs

Non-destructive testing was performed on the slabs using Giatec Scientific's XCell and iCOR, collecting data on corrosion rate, half-cell potential, and electrical resistivity. In addition, corrosion rate, half-cell potential, and resistance were determined with GPT by use of the GalvaPulse device. Surface electrical resistivity was determined with the four-point Resipod probe. As most devices require that the sensor be placed on top of intersections of reinforcement, the top level of vertical and horizontal rebars were located to establish a grid for measurement, as seen in Figures 3.21 to 3.24. These grids are in units of centimetres. Two rebar sizes were discovered, 15M (illustrated in light grey) and 20M (illustrated in dark grey). In addition, it was discovered that these slab specimens were subjected to patch work repair at some point during the life of the bridge, or it was observed that non-concrete deck material was left on the slab surface. As these surface materials do not allow for NDT devices to make a measurement, these surface areas needed to be recorded and are illustrated as blue hatching in Figures 3.20 to 3.23. Moreover, the location of attempted retrieved core samples are located as green markers in the figures. It is important to note that not all attempts were successful as several cores broke during retrieval.

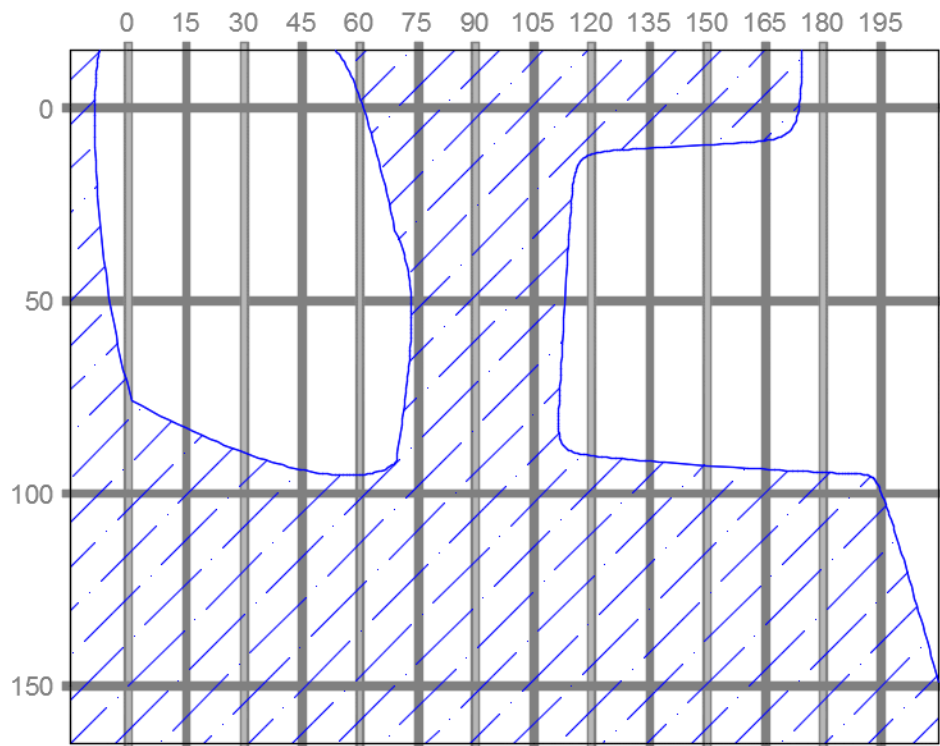


Figure 3.19: North Channel Bridge Slab S11 Grid

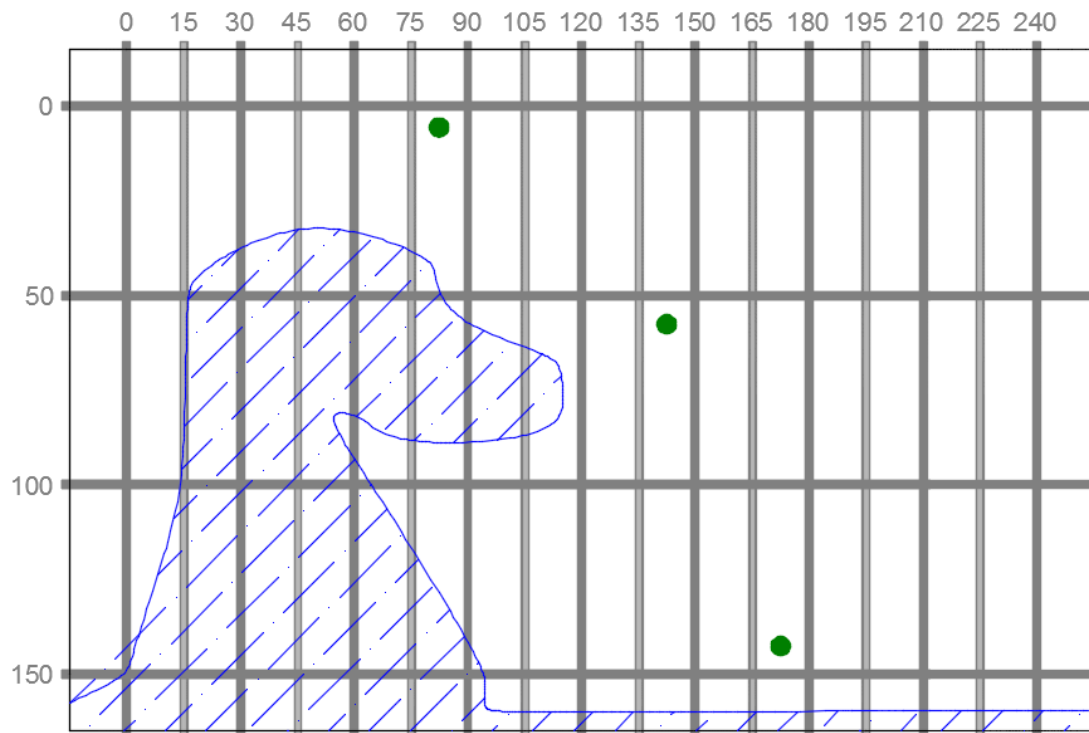


Figure 3.20: North Channel Bridge Slab S12 Grid

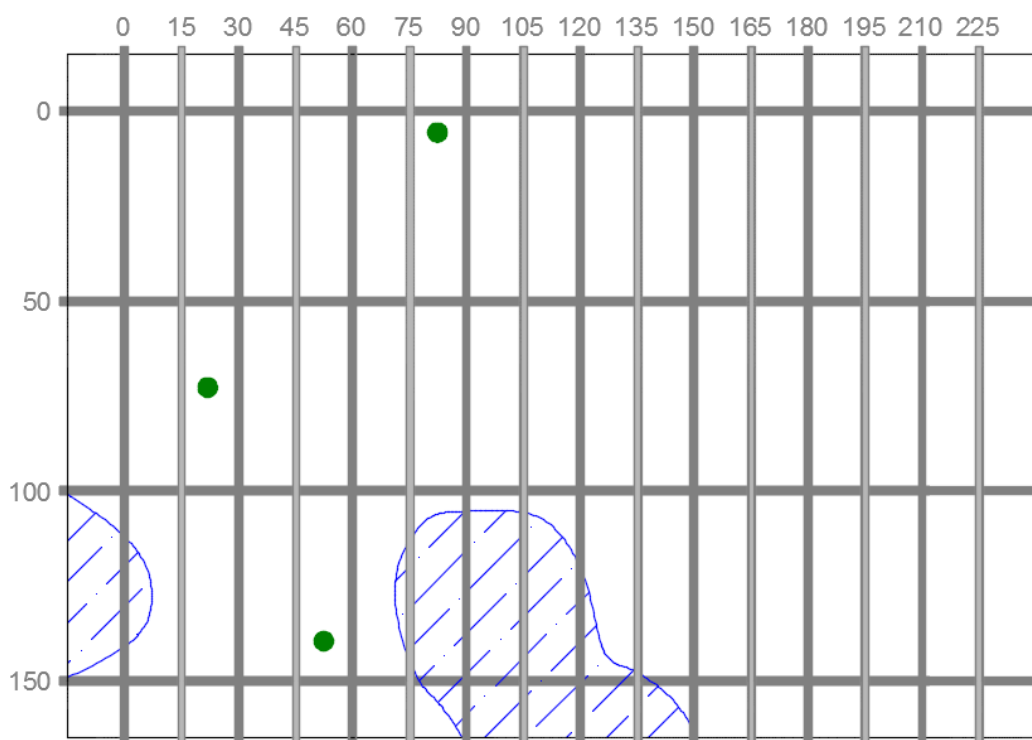


Figure 3.21: North Channel Bridge Slab S13 Grid

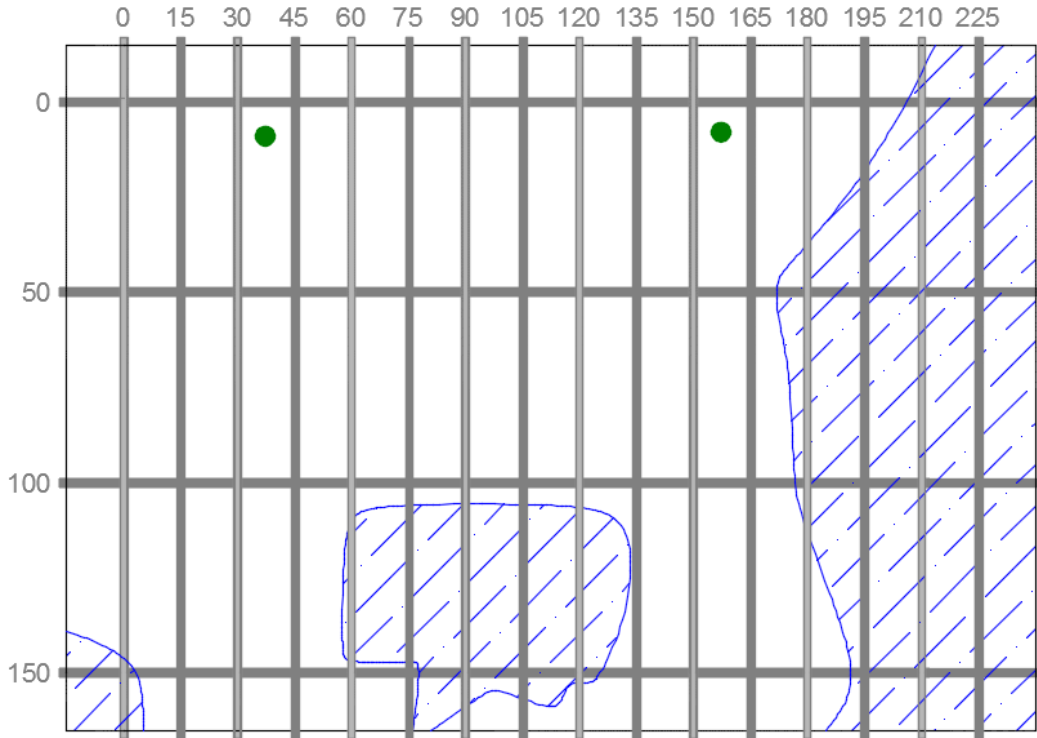


Figure 3.22: North Channel Bridge Slab S14 Grid

The North Channel Bridge slabs were cut from larger sections of the bridge deck. As a result of this, the locations of the reinforcement and the cover thickness could be seen from the cut sections. Utilizing this, the concrete cover of the vertical reinforcing bars was determined as an average of the concrete cover seen from the North and South cut ends. The results of this are tabulated in Table 3.9 and 3.10.

Table 3.6: North Channel Bridge Slabs S11 and S12 Concrete Cover Thickness

	Slab S11			Slab S12		
Rebar Location- X Direction	South Depth (cm)	North Depth (cm)	Average Depth (cm)	South Depth (cm)	North Depth (cm)	Average Depth (cm)
0	52	50	51	49	45	47
15	52	46	49	52	53	53
30	47	45	46	42	46	44
45	49	46	48	44	50	47
60	48	42	45	40	47	44
75	48	41	45	48	49	49

90	49	40	45	50	52	51
105	59	49	54	41	50	46
120	48	33	41	52	46	49
135	51	37	44	55	47	51
150	42	35	39	62	49	56
165	49	15	32	67	46	57
180	50	20	35	65	50	58
195				83	53	68
210				64	50	57
225				61	58	60

Table 3.7: North Channel Bridge Slabs S13 and S14 Concrete Cover Thickness

	Slab S13			Slab S14		
Rebar Location- X Direction	South Depth (cm)	North Depth (cm)	Average Depth (cm)	South Depth (cm)	North Depth (cm)	Average Depth (cm)
0	50	45	48	51	51	51
15	48	47	48	49	45	47
30	47	45	46	51	49	50
45	45	49	47	44	38	41
60	45	45	45	50	44	47
75	41	43	42	46	40	43
90	51	51	51	45	40	43
105	50	46	48	42	32	37
120	55	48	52	44	39	42
135	49	42	46	46	37	42
150	48	46	47	42	40	41
165	43	41	42	35	35	35

180	49	39	44	41	41	41
195	40	33	37	41	37	39
210	42	37	40	51	50	51

At the conclusion of the non-destructive testing, concrete core samples were retrieved from the slabs ,refer to Figure 3.25, so that compressive strength, carbonation front, moisture content, and absorption could be determined in the laboratory. The concrete cores measured approximately 95 mm in diameter. At the conclusion of the NDT and core retrieval, the top layer of 20 M rebars were removed mechanically, tagged, photographed, and brought back to the laboratory for gravimetric mass loss testing.

It is important to note that as the North Channel Bridge Slab S11 was considerably compromised by surface patch work and leftover concrete deck material, it was decided that non-destructive testing would be performed on reinforcement locations that were not compromised. However, as the number of measurement points were few, core samples were not taken and rebars were not removed for Slab S11, as there would be too few NDT data points to use for comparison. Additional photos of the Field Study testing in Cornwall can be found in Appendix B.



Figure 3.23: Field Study – Cornwall NDT Testing



Figure 3.24: Field Study – Cornwall Destructive Testing

3.3.3. Testing of North Channel Bridge Specimens

3.3.3.1. Absorption and Moisture Content

In order to determine the density, absorption, and voids in hardened concrete, cylindrical specimens with diameter of 95 mm and length of 50 mm were cut from the core samples taken from the North Channel bridge slabs, refer to Figure 3.26. Samples were sealed so the condition of the concrete during in-situ measurements could be determined. Following (ASTM C642, 2013), several concrete properties were determined: absorption, bulk density, apparent density, and volume of permeable pore space. The procedure is outlined in Appendix E.

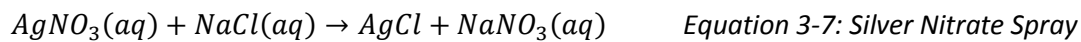


Figure 3.25: Specimens for ASTM 642 Testing

3.3.3.2. Compression Strength, Carbonation Front and Chloride Penetration Depth

From the core samples retrieved from the North Channel Bridge slabs, one specimen from slabs S12, S13, and S14 was used for compression testing. All specimens were altered to have a diameter of 95 ± 5 mm and a length of 200 ± 5 mm. Once the compression test was performed, and the specimen was split in two, freshly exposing the interior of the concrete specimen, the carbonation depth was determined with a spray of phenolphthalein indicator. This indicator is made with a mix of 1 gram of phenolphthalein and 100 ml of alcohol/water (50:50 mix). The phenolphthalein indicator will remain clear where concrete is carbonated and turn pink when the concrete is still alkaline (Broomfield J. , 2007). Since the concrete surface was extremely dry, a spray of tap water was applied prior to carbonation testing.

Using the second half of the split specimen, the chloride penetration depth was determined using the colourimetric method. This methods is a fast and easy way to roughly determine the average penetration depth of free chlorides present in the concrete sample (C. Andrade, 1999). The presence of chlorides is determined by applying a 0.1N (mole/L) silver nitrate spray on the freshly-broken concrete surface. The silver nitrate reacts with the free chlorides present in the concrete to form a white gray precipitate (silver chloride) as seen in the chemical reaction below (Karam, 2014):



This colourimetric methods was used to determine the average chloride penetration depth, so that concrete powder samples could be retrieved within the penetration depth and later used for chloride concentration using titration. Figure 3.27 shows the split core samples after phenolphthalein and silver nitrate were sprayed on the concrete.



Figure 3.26: Split Cylinders Tested for Carbonation Front (left) and Chloride Penetration Depth (right)

For the carbonation and chloride penetration depth tests, it was determined that concrete is still alkaline and that there is not clear indication of chlorides within the concrete. However, concrete samples were removed for chloride content determination by titration.

3.3.3.3. Chloride Content

From the core samples, powder samples were retrieved for chemical analysis to determine the free chloride concentration of the concrete. Following ASTM C1152/C1152M (2012), see Appendix F for the complete procedure, a 5-g sample size is recommended. Using the silver nitrate spray as an indication of the chloride penetration depth, powder samples were retrieved to a minimum of 50 mm from the concrete deck surface and 40 mm from the bottom of the concrete core sample. Using an adjustable rotary impact drill, powder samples were retrieved at 4 or 5 mm intervals. With each interval, penetrating further into the concrete specimen, the drilled area was decreased so to ensure that the drill would not remove concrete from the wall sides of the previous interval, see Figure 3.28. In between drilling intervals, the drilling equipment and drilled hole was cleaned with compressed air to ensure that cross-contaminated between intervals would not occur. Retrieved samples were placed in sealable bags and labelled for transportation and further testing.



Figure 3.27: Profile Grinding Process

3.3.3.4. Gravimetric Mass Loss

The 20M vertical reinforcement retrieved from the North Channel Bridge Slabs S12, S13, and S14 was brought back to the laboratory for gravimetric mass loss testing according to ASTM G1-03 (2011), similar to that conducted in the Laboratory Study of this experimental program. At the locations of in-situ non-destructive testing, the reinforcement was inspected, and locations with visible deterioration were cut in segments of approximately 10 cm in length as seen Figure 3.29. These specimens were photographed, from both the top and bottom of each rebar, and run through the image analysis software, *ImageJ*, to extract a rough estimate of the percentage of the rebar that was corroded and a percentage of the rebar that was covered with mortar products. After the completion of this, the cut specimens were subjected to gravimetric mass loss testing. See Appendix D for a complete list of steps and equations for gravimetric testing according to ASTM G1-03 (2011).



Figure 3.28: North Channel Bridge Reinforcement Specimens

4.0 Experimental Results and Discussion

As the quantity of tests performed and the resulting volume of data is extensive, this chapter presents the experimental results and corresponding discussion within the same section. As seen previously, this chapter is broken into two major components, the results from the Laboratory Study and the results from the Field Study.

4.1. Laboratory Study

4.1.1. NDT Testing

Over the course of nine months, the deterioration of the 18 reinforced concrete slabs was monitored with four different non-destructive devices, each recording results for four different parameters. The following sections present the trend results of this nine-month process. These plots show the trend results for corrosion rate ($\mu\text{A}/\text{cm}^2$), electrical resistivity ($\text{ohm}\cdot\text{m}$) or electrical resistance (kohm), and half-cell potential (mV) plotted against the concrete age (weeks) for each individual slab. The results are presented in order of technique: CEPRA, GPT, and LPR. Refer to Figure 4.1, for the coloured legend which is consistent for all trends presented.

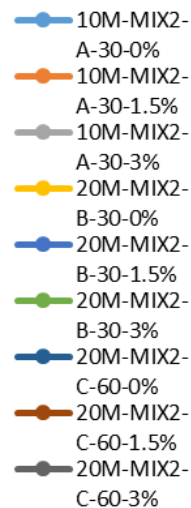


Figure 4.1: Coloured Legend for Laboratory Trend Graphs

4.1.1.1. Half-Cell Potential

For each technique, the half-cell potential measurements are graphically represented against the concrete age in weeks. Refer to Figures 4.2 to 4.7 for half-cell potential trends for CEPRA Mix 2, CEPRA Mix 1, GPT Mix 2, GPT Mix 1, LPR Mix 2, and LPR Mix1, respectively. Results from GPT were adjusted to mV/CSE for comparison.

Half-cell potential measured over a period of weeks starting from the concrete pour is expected to increase from a more negative potential to a more positive potential. This is a result of concrete hydration and curing process, where water present in the concrete microstructure slowly hydrates to form calcium silicate hydrate. As expected, this is the general trend observed in the lab. All three techniques measured potentials starting as negative as -650 mV to -200 mV at the start of testing. During concrete curing and under saturated condition, the potential steadily increased to potentials ranging from -250 mV to -100 mV at 28 weeks. At 28 weeks, the concrete was in a saturated surface dry (referred to as “dry” within figures) condition until 38 weeks. In a saturated surface dry condition, it is expected that half-cell potential to be between 0 mV to 200 mV (154-EMC, 2003). As expected, during the 28 to 38-week period, all techniques measured a potential that increased to approximately 0 mV. As the half-cell potential measurements moved closer to a positive potential, the probability of corrosion decreased as per ASTM C876. It is important to note that the slabs with the highest corrosion rates also generally measured half-cell potentials values greater than -350mV, corresponding to smaller probability of corrosion activity.

It is expected that half-cell potential measurements for concrete with a larger concrete cover would have less negative half-cell potential values (Elsener & Bohni, 27-31 July 1997). This trend is seen for all techniques and all slabs with a 60-mm concrete cover.

The use of supplementary cementitious materials (SCMs) may drastically alter half-cell potential measurements, with the change in half-cell potential depending on the SCMs used. Generally, electrical resistivity increases with the presence of SCMs (Project SENSO, 2009). Half-cell potential measurements in this experimental study follow a similar trend. Mix #2 with a w/cm of 0.62 produced potential measurements generally more negative, at similar concrete age than the potential measurements for mix #1 with a w/c of 0.47.

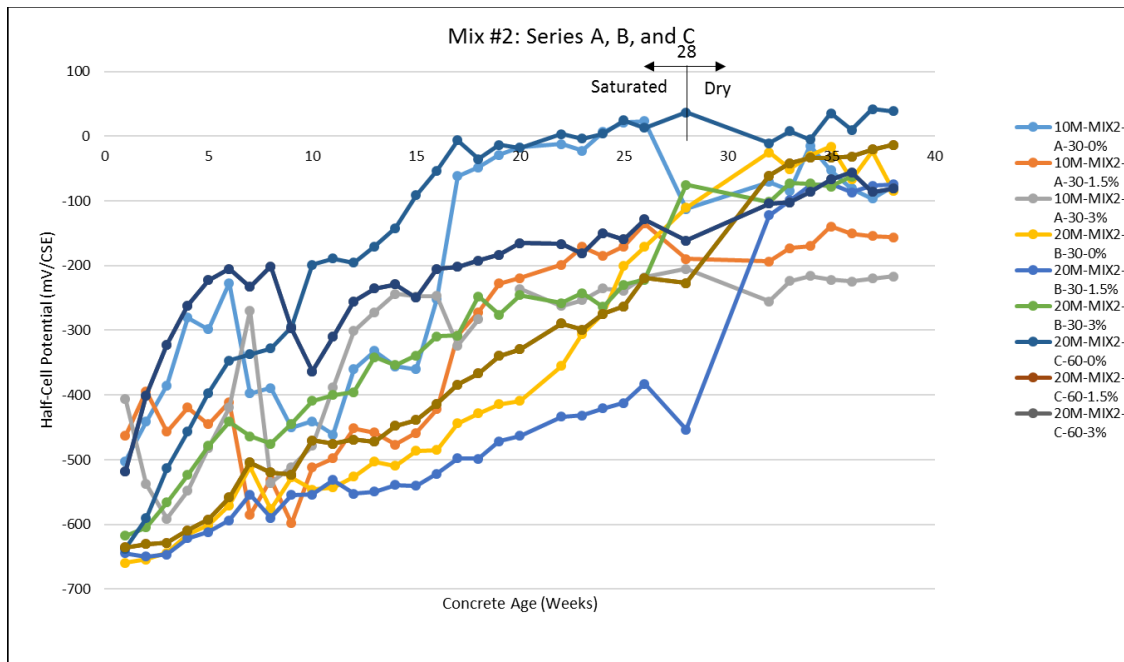


Figure 4.2: Laboratory Trend Results: Half-Cell Potential CEPRA Mix 2

In three instances, the half-cell potential measurements for CEPRA Mix 1 (Figure 4.3) and GPT Mix 1 (Figure 4.5) illustrate a few abnormal values, with half-cell results drastically greater than results of similar concrete age for the same specimen. These anomalies can be attributed a malfunction of the device at the time of measurement or poor electrical contact with the concrete surface.

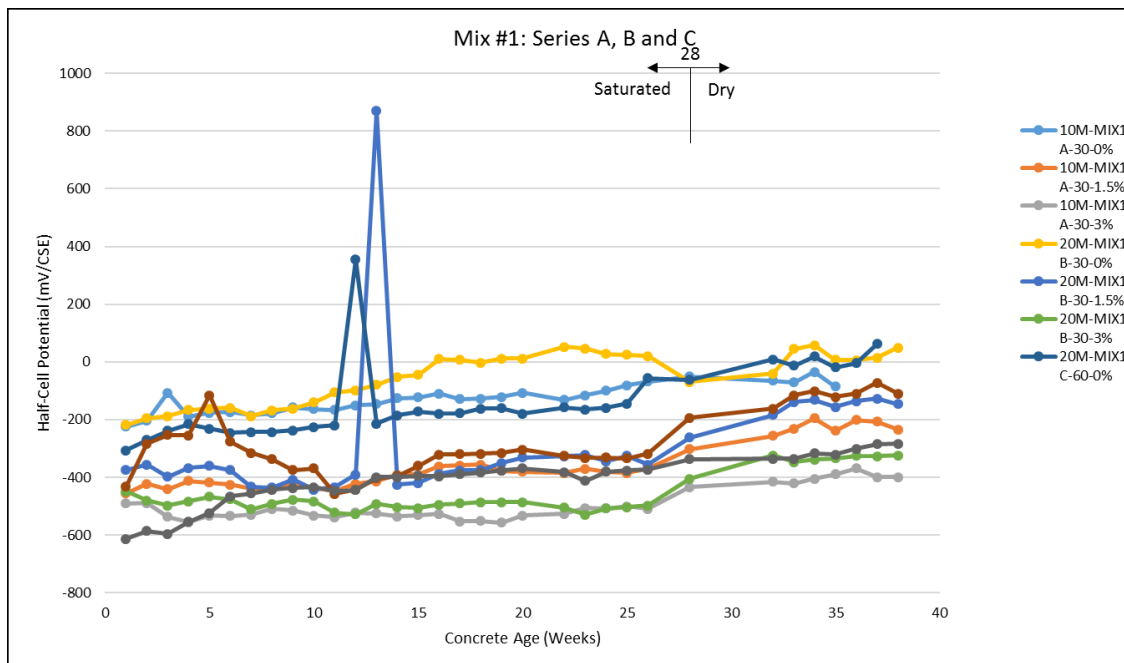


Figure 4.3: Laboratory Trend Results: Half-Cell Potential CEPRA Mix 1

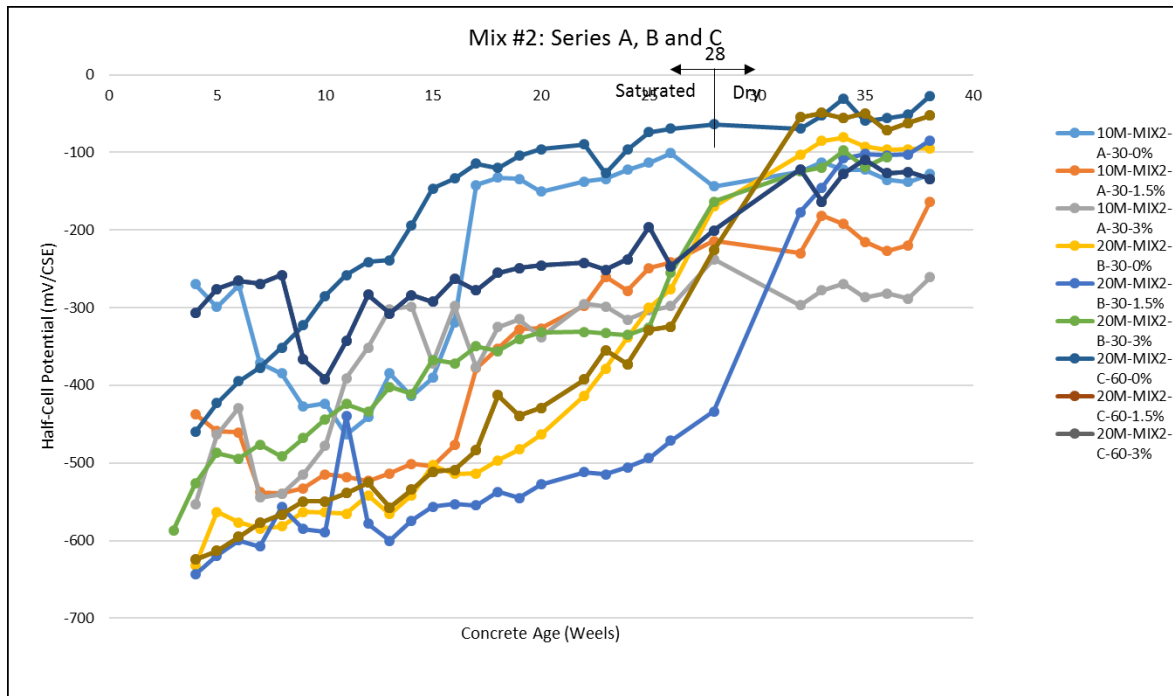


Figure 4.4: Laboratory Trend Results: Half-Cell Potential GPT Mix 2

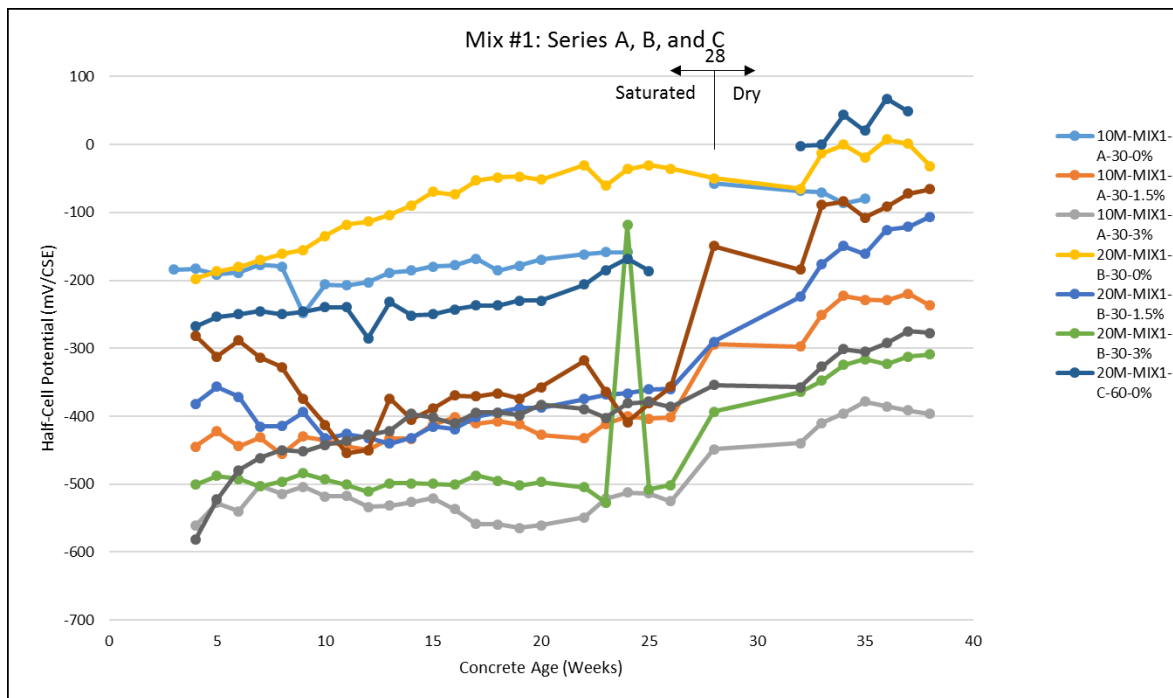


Figure 4.5: Laboratory Trend Results: Half-Cell Potential GPT Mix 1

At week 7, the Gecor device malfunctioned. As the malfunction was beyond the repair capabilities of the user, the device was sent back to the manufacturer for repairs and calibration. The functioning device was

returned at week 22. As such, measurements for LPR testing are missing from this experimental program from week 7 to 22. Refer to Figures 4.6 and 4.7.

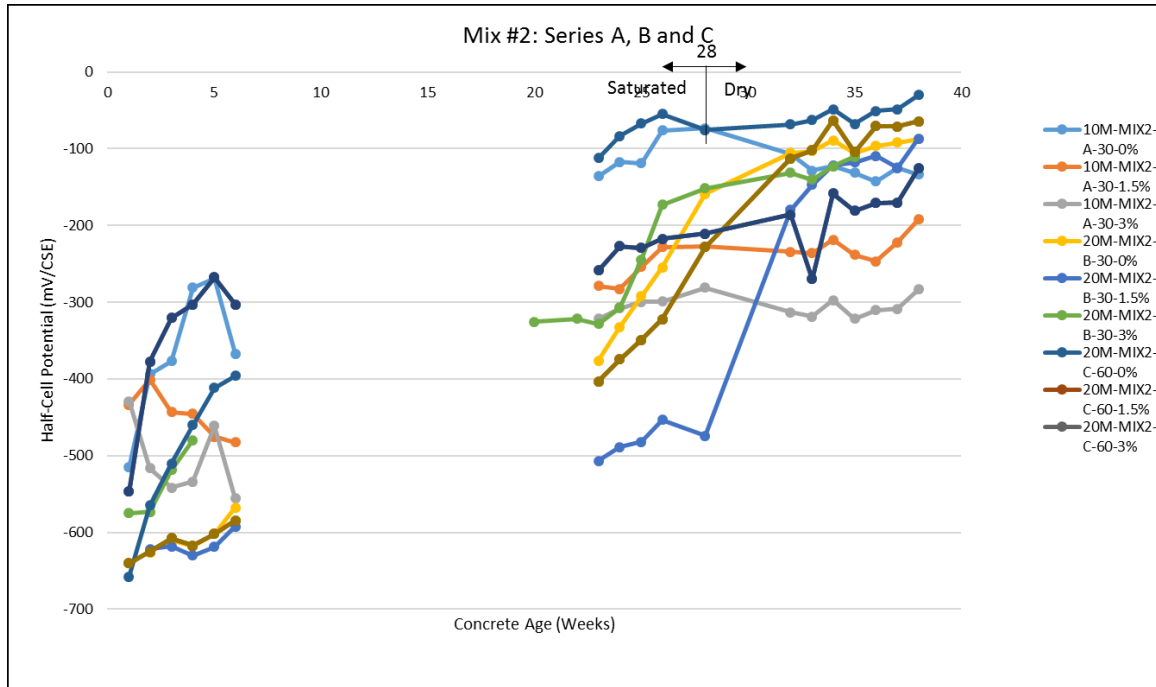


Figure 4.6: Laboratory Trend Results: Half-Cell Potential LPR Mix 2

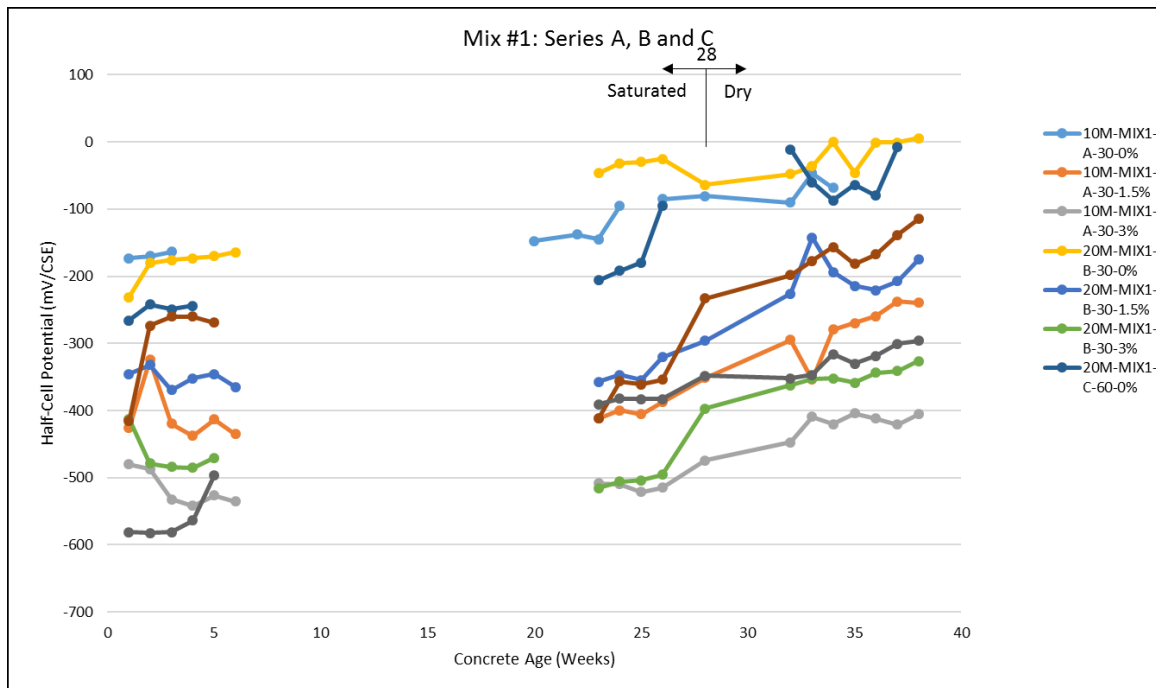


Figure 4.7: Laboratory Trend Results: Half-Cell Potential LPR Mix 1

Half-cell potential measurements between devices are generally similar for the same concrete age, as all three devices use similar techniques and probe arrangement. To quantify this similarity, the average half-cell potential for CEPRA when compared to the average half-cell potential for LPR have a linear relationship with a coefficient of determination range of 0.95 for saturated surface dry condition and 0.64 for saturated conditions.

4.1.1.2. Corrosion Rate and Electrical Resistivity

Figures 4.8 to 4.25 show the laboratory results for each technique: CEPRA, corrosion rate and electrical resistivity, GPT, corrosion rate and electrical resistance, and LPR, corrosion rate and electrical resistance for each concrete mix and slab series. Note that clustered columns represent the corrosion rate trend, and the line represents the electrical resistivity or resistance trend. The concrete age is in weeks.

High risk of corrosion is generally accompanied with lower electrical resistivity (Layssi H. , Ghods, Alizadeh, & Salehi, 2015) (Hornbostel, Larsen, & Geiker, 2013) (Polder, 2000) (Lataste J.-F. , 2010). Different parameters affect this relationship and include, but not limited to, cement type, corrosion cause, specimen geometry, reinforcement type, cover depth, measurement technique, and concrete saturation. The most apparent parameter affecting the relationship between corrosion rate and electrical resistivity in this experimental program is concrete saturation. The shift from saturated to saturated surface dry at week 28 results in drastically higher electrical resistivity measurements and lower corrosion rate measurements.

With the addition of chloride content to the concrete mix, the corrosion rate should be lower for 0% chloride content and increase for 3% chloride content. This trend is prevalent for all Mix 1, Series A (refer to Figures 4.8, 4.14 and 4.20) . This general trend is not, however, illustrated for the remaining specimens, Series B and C, for both Mix 1 and 2, where 20-mm diameter reinforcing bars are used. The corrosion rate for specimens with 1.5% chloride content was generally higher than specimens with 3% chloride content.

Figures 4.8 to 4. 13 represent the trends as recorded by CEPRA.

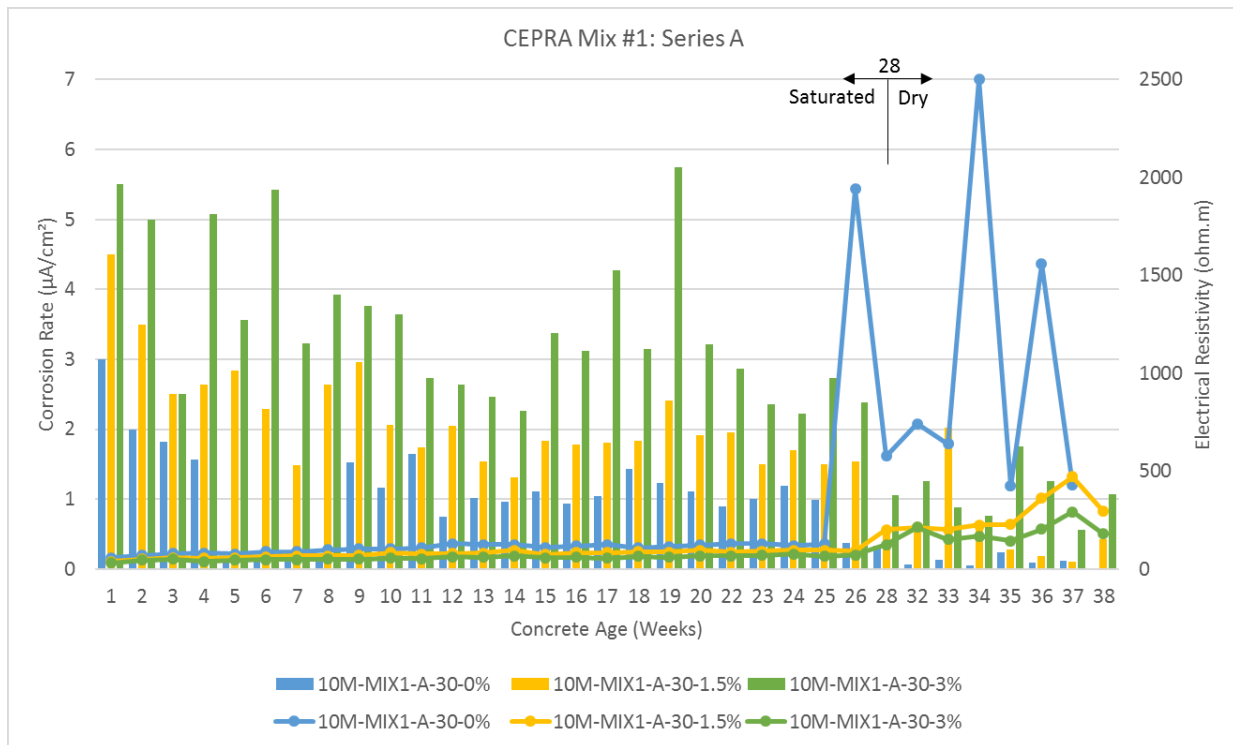


Figure 4.8: Laboratory Trend Results: CEPRA Mix 1: Series A

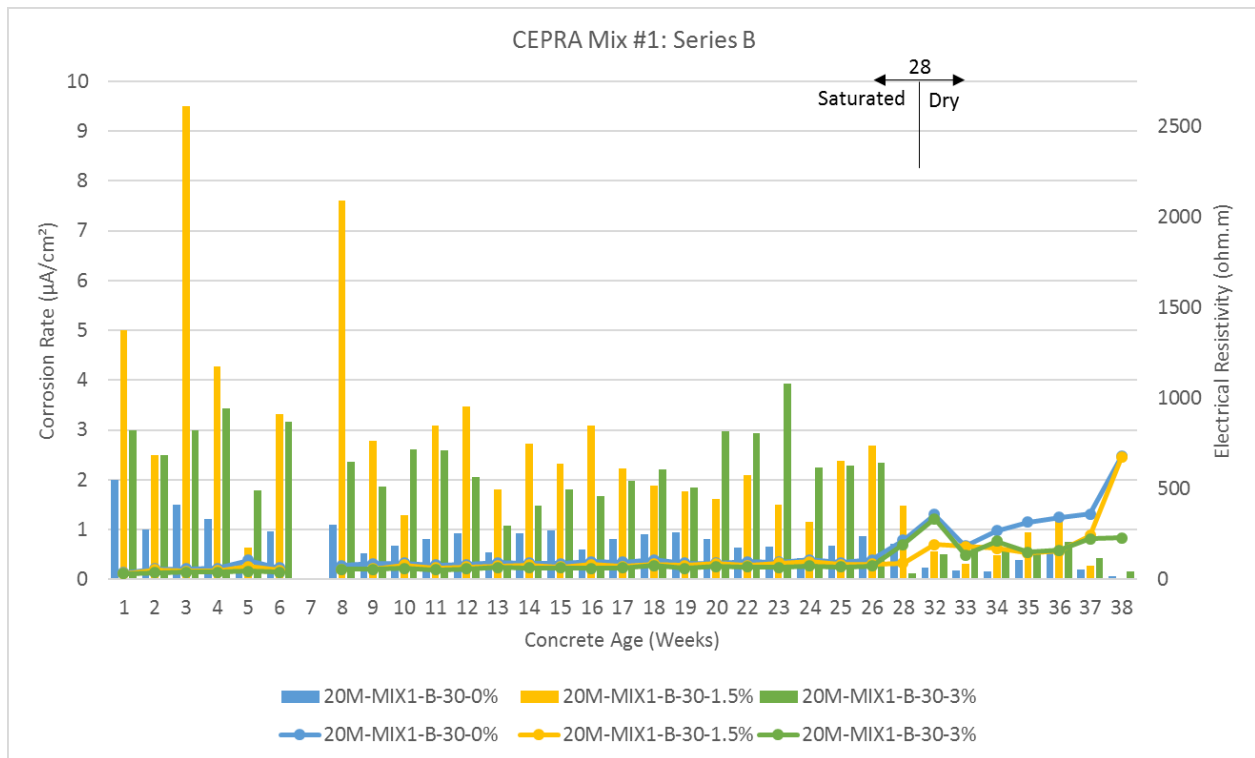


Figure 4.9: Laboratory Trend Results: CEPRA Mix 1: Series B

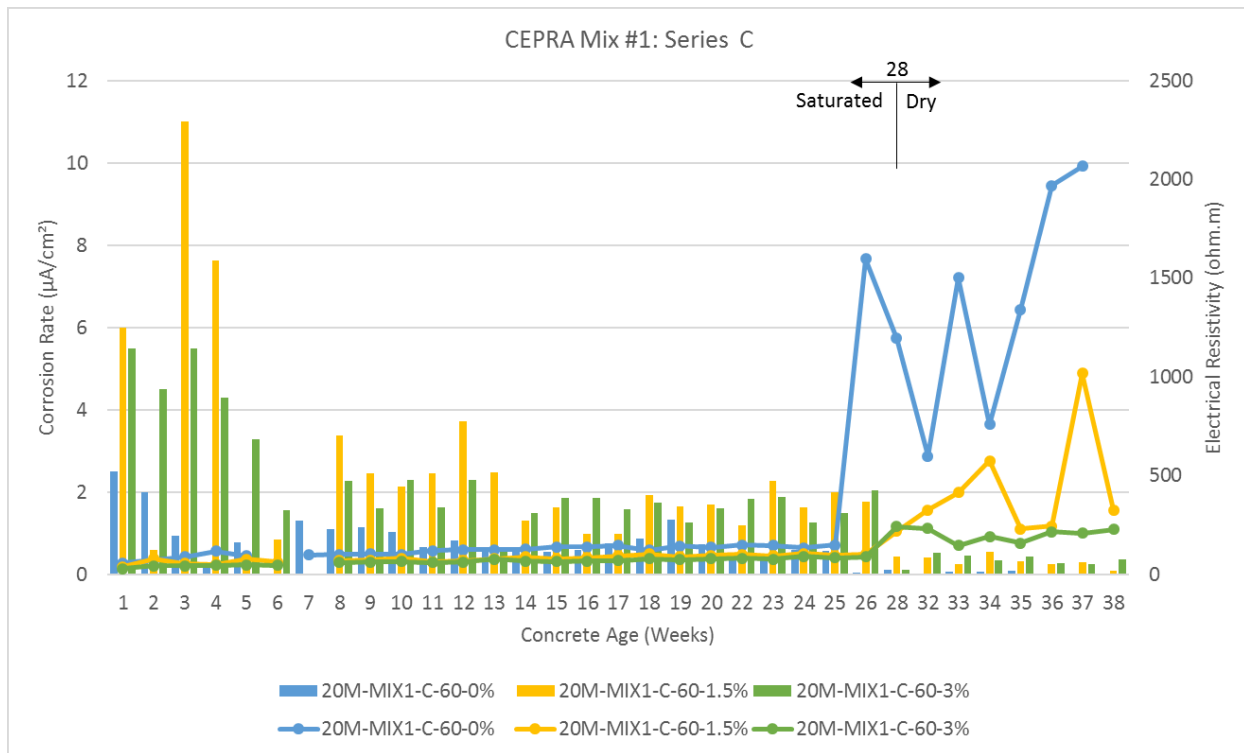


Figure 4.10: Laboratory Trend Results: CEPRA Mix 1: Series C

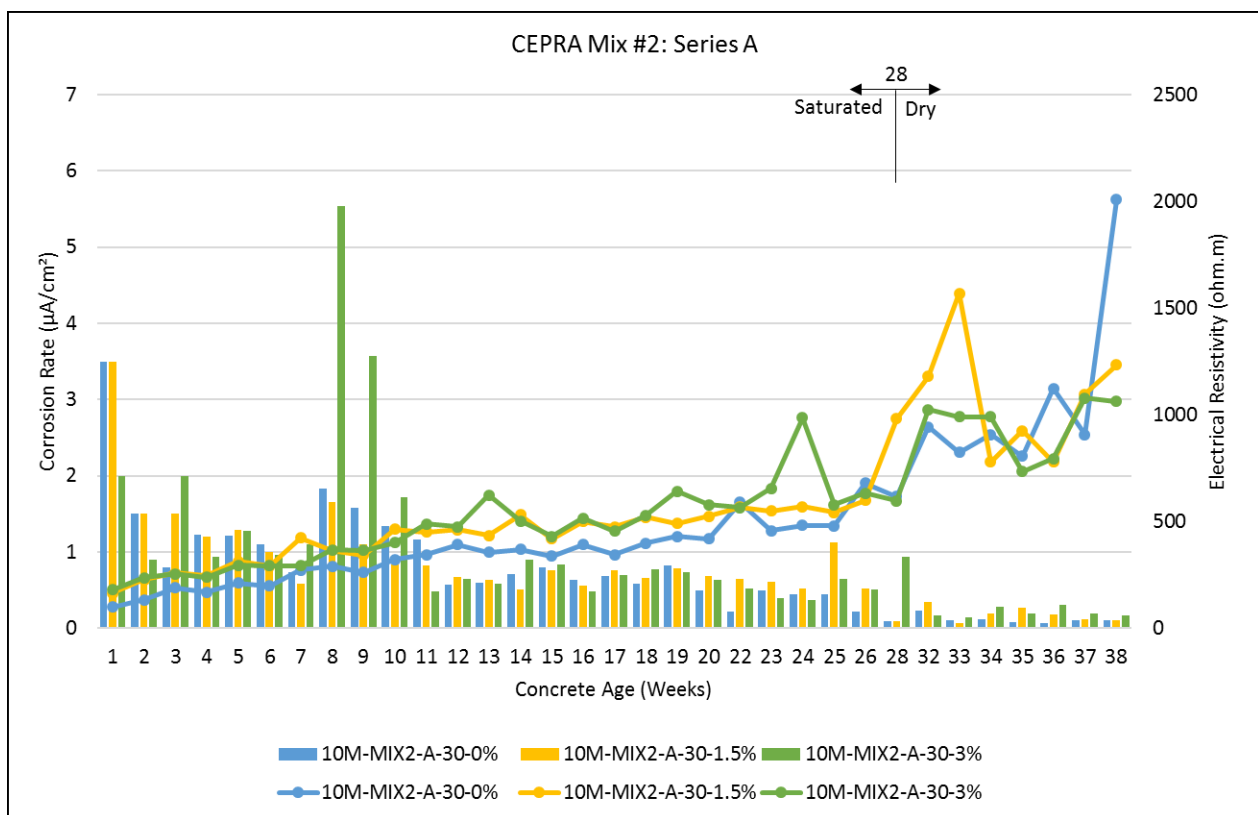


Figure 4.11: Laboratory Trend Results: CEPRA Mix 2: Series A

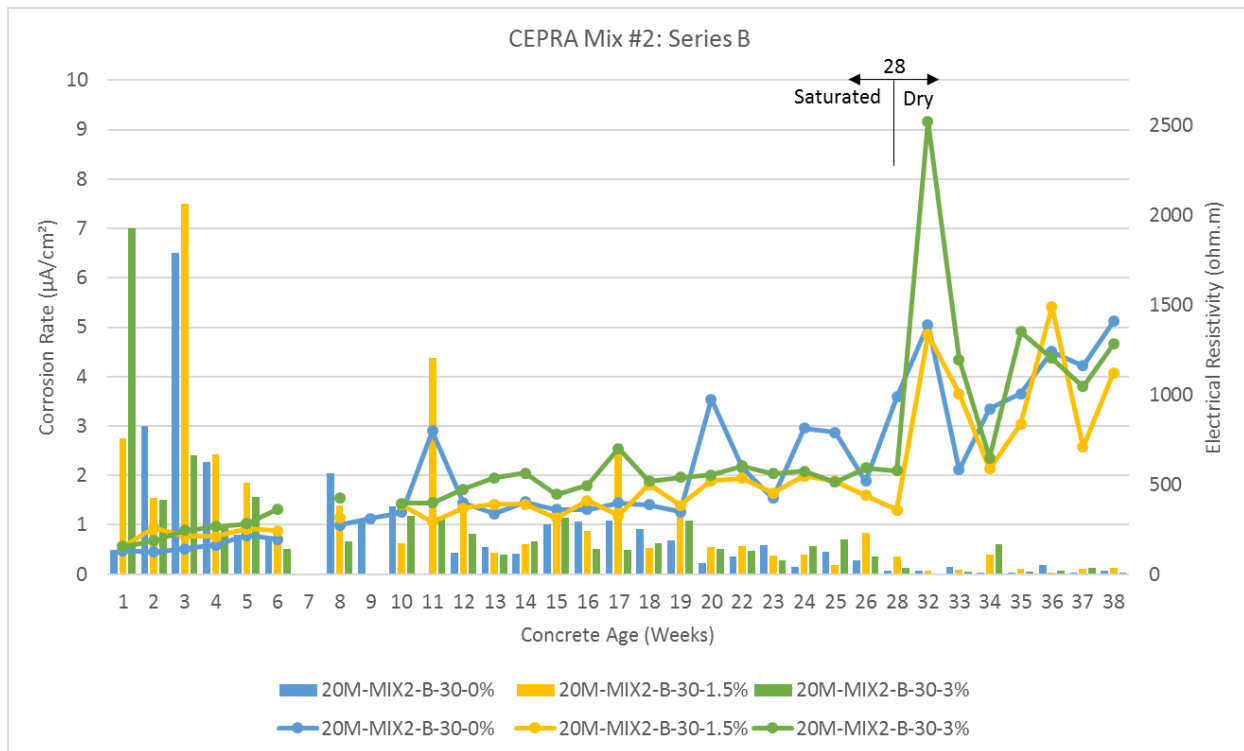


Figure 4.12: Laboratory Trend Results: CEPRA Mix 2: Series B

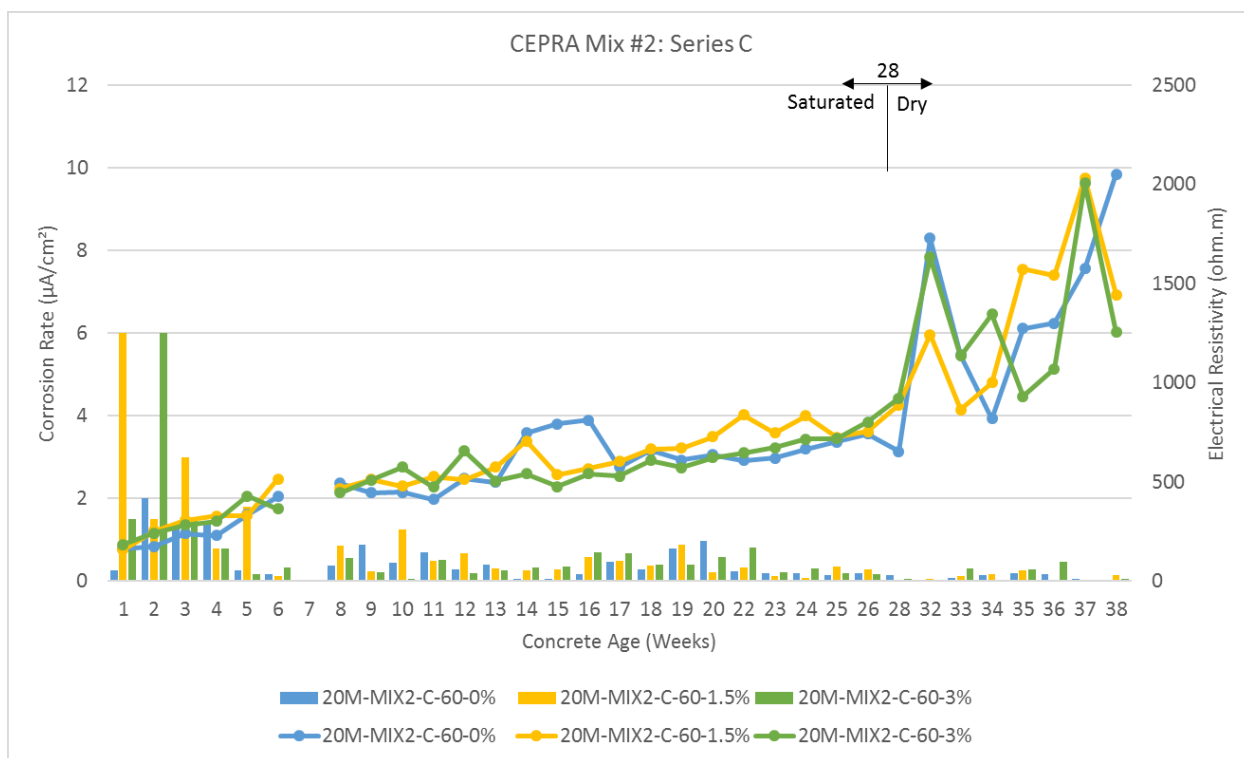


Figure 4.13: Laboratory Trend Results: CEPRA Mix 2: Series C

The trend results for GPT are seen in Figures 4.14 to 4.19.

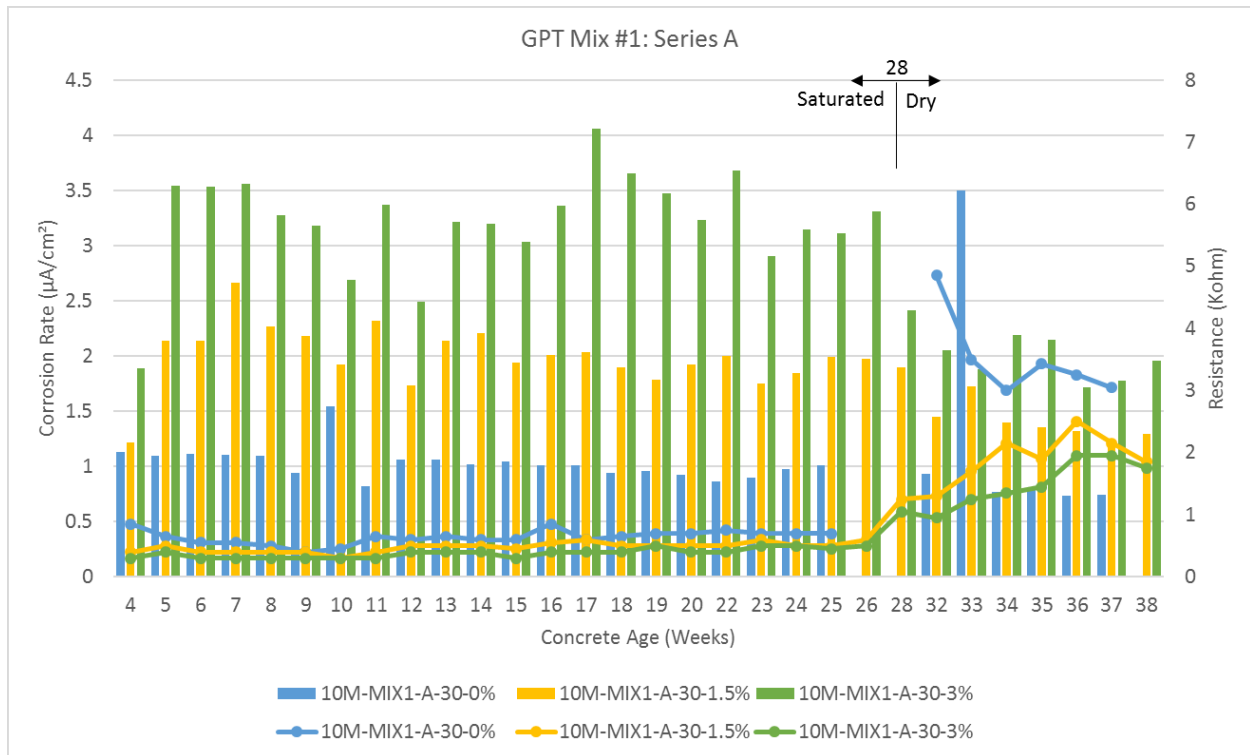


Figure 4.14: Laboratory Trend Results: GPT Mix 1: Series A

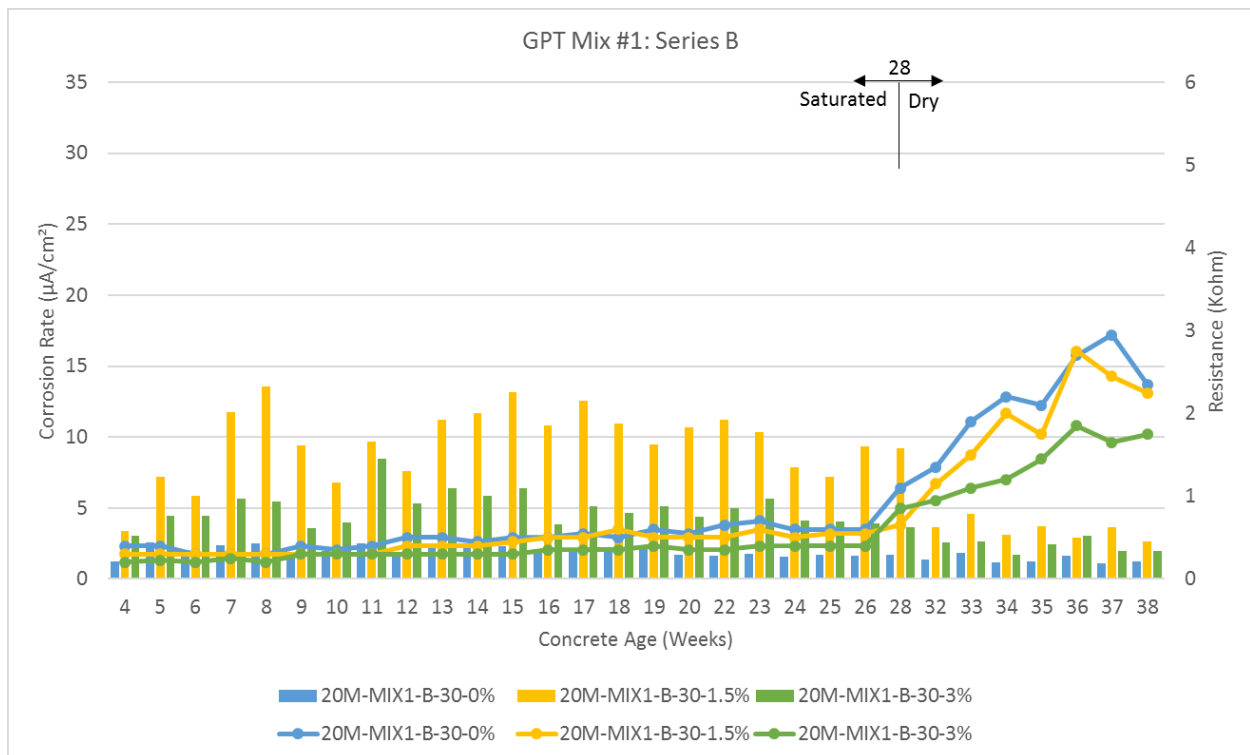


Figure 4.15: Laboratory Trend Results: GPT Mix 1: Series B

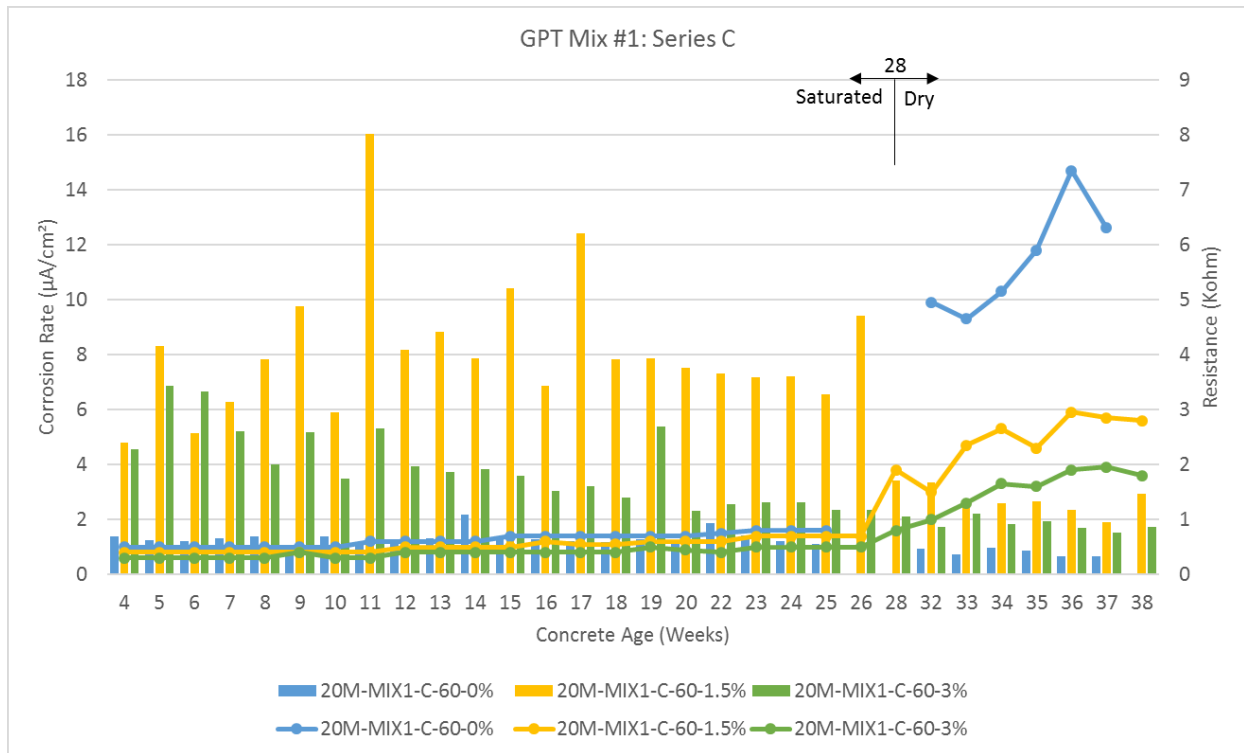


Figure 4.16: Laboratory Trend Results: GPT Mix 1: Series C

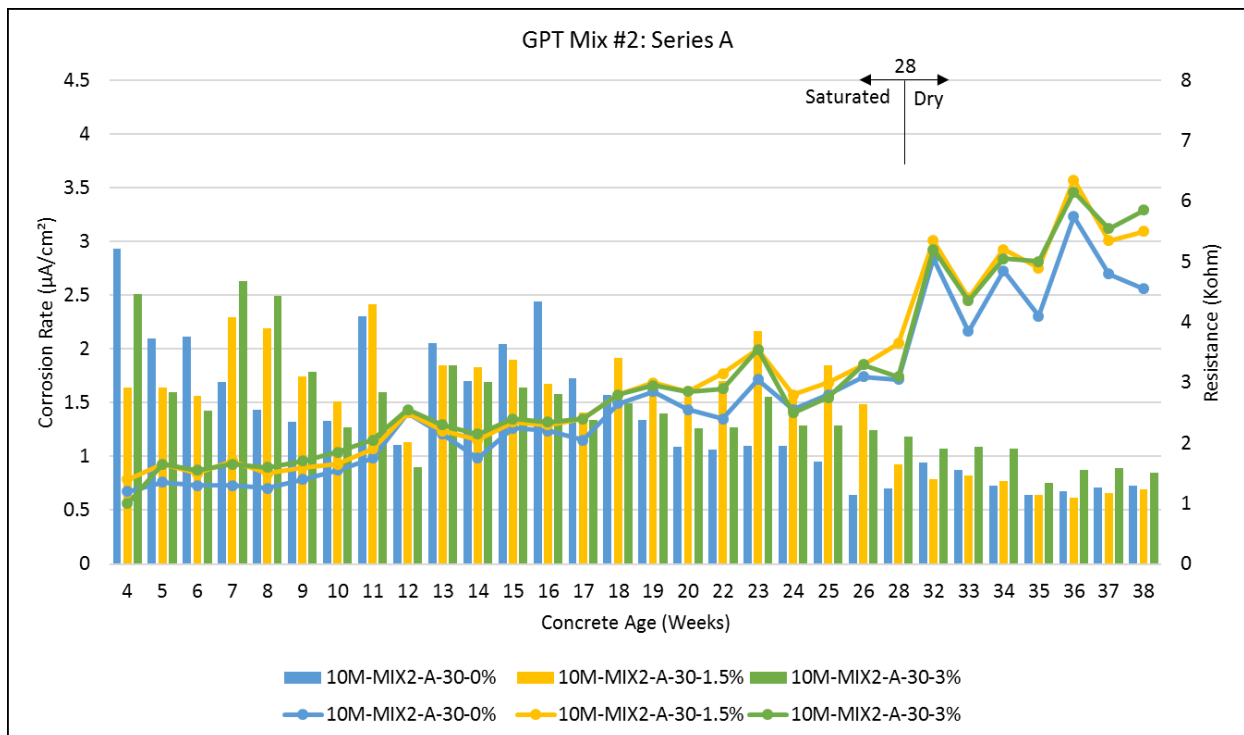


Figure 4.17: Laboratory Trend Results: GPT Mix 2: Series A

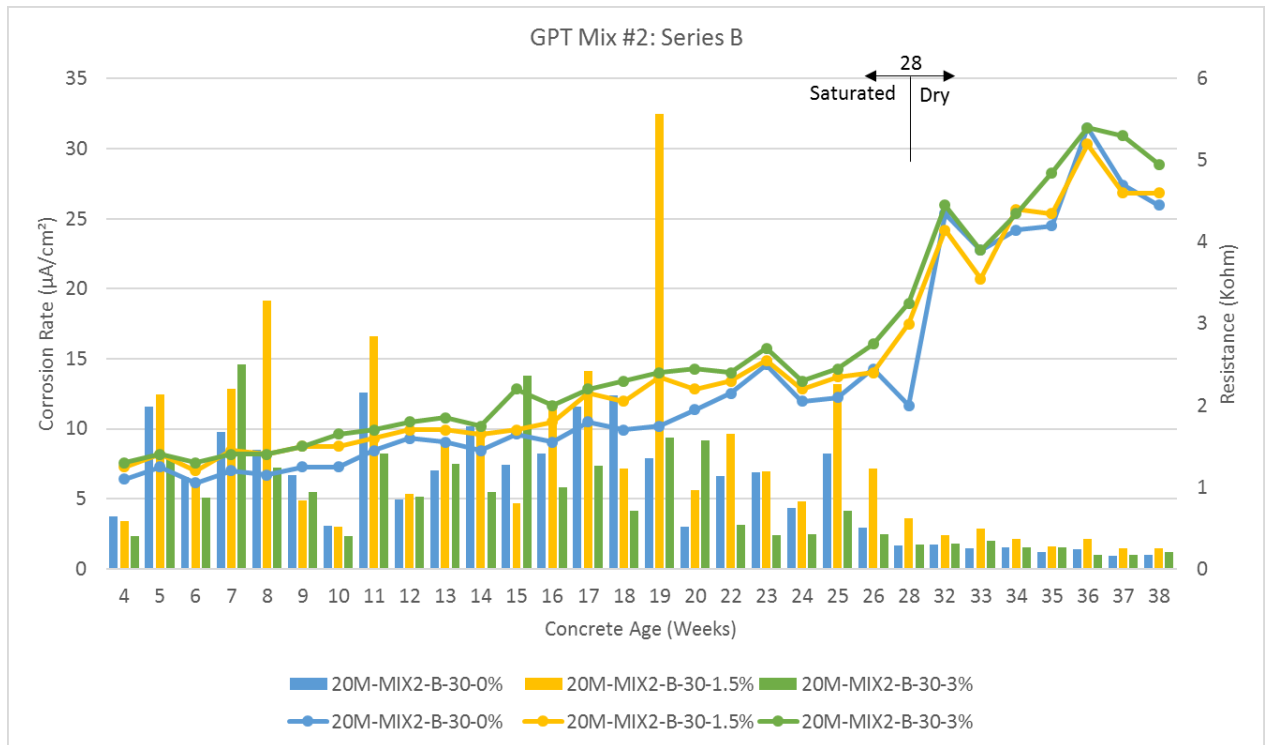


Figure 4.18: Laboratory Trend Results: GPT Mix 2: Series B

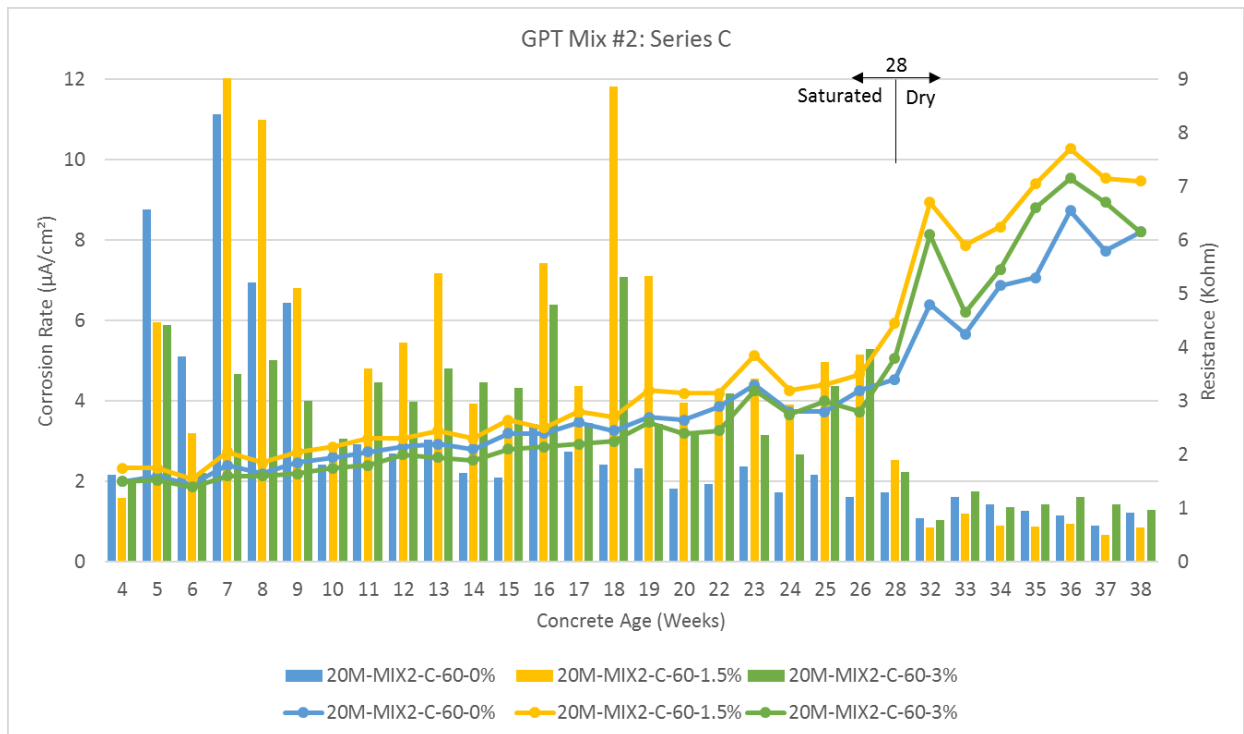


Figure 4.19: Laboratory Trend Results: GPT Mix 2: Series C

The trend results for LPR are seen in Figures 4.20 to 4.25. As previously mentioned, at week 7, the Gecor device malfunctioned. As such, measurements for LPR testing are missing from this experimental program from week 7 to 22.

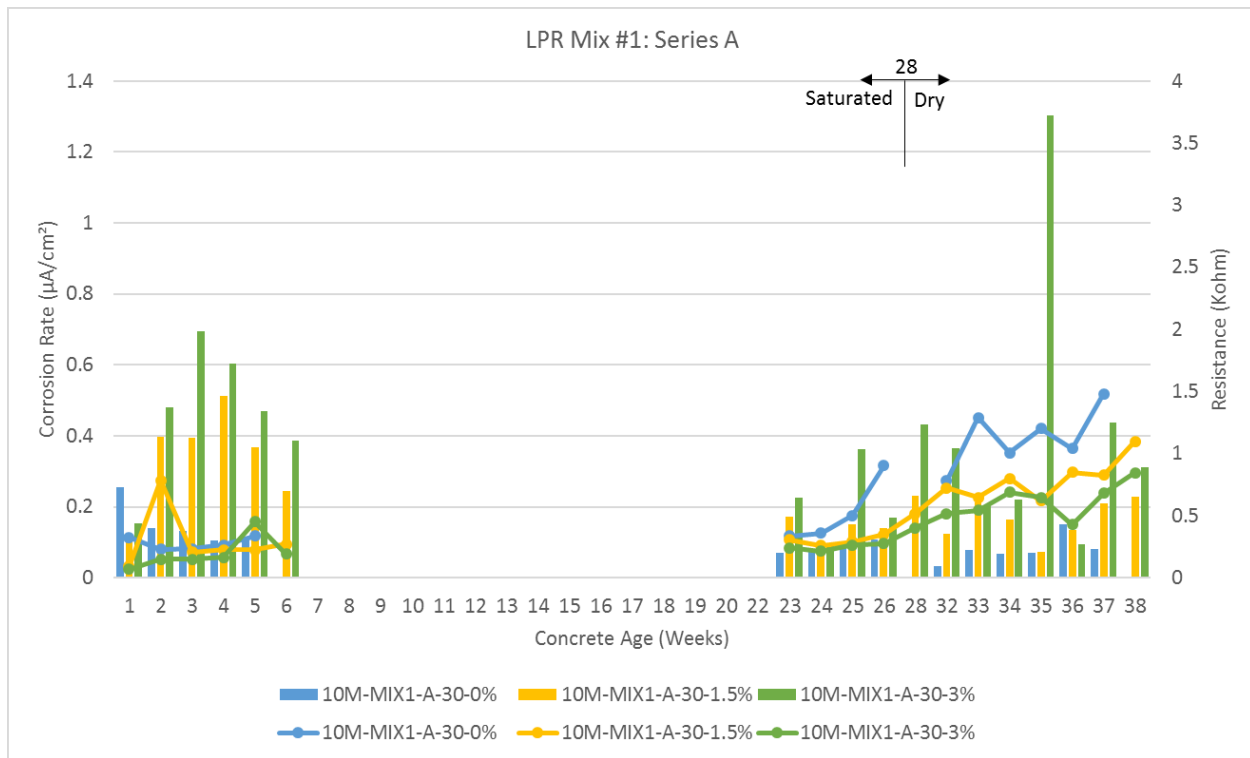


Figure 4.20: Laboratory Trend Results: LPR Mix 1: Series A

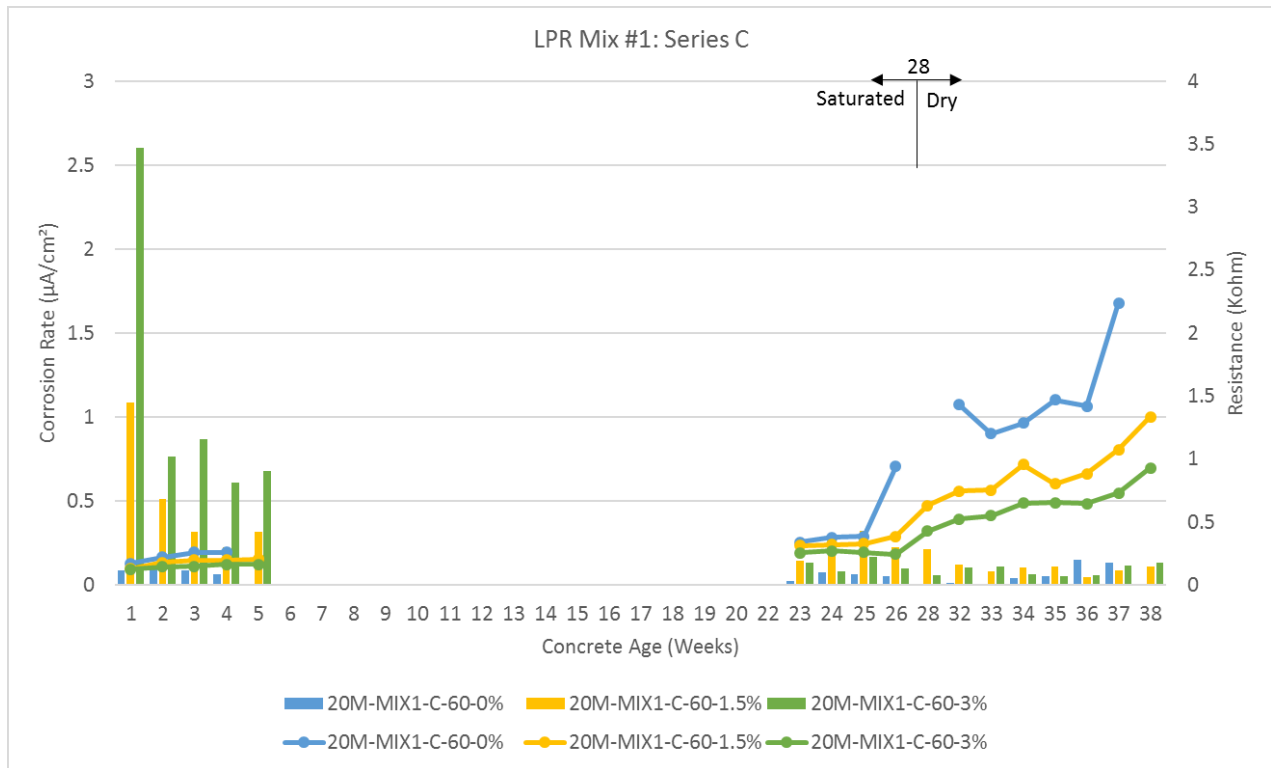


Figure 4.21: Laboratory Trend Results: LPR Mix 1: Series B

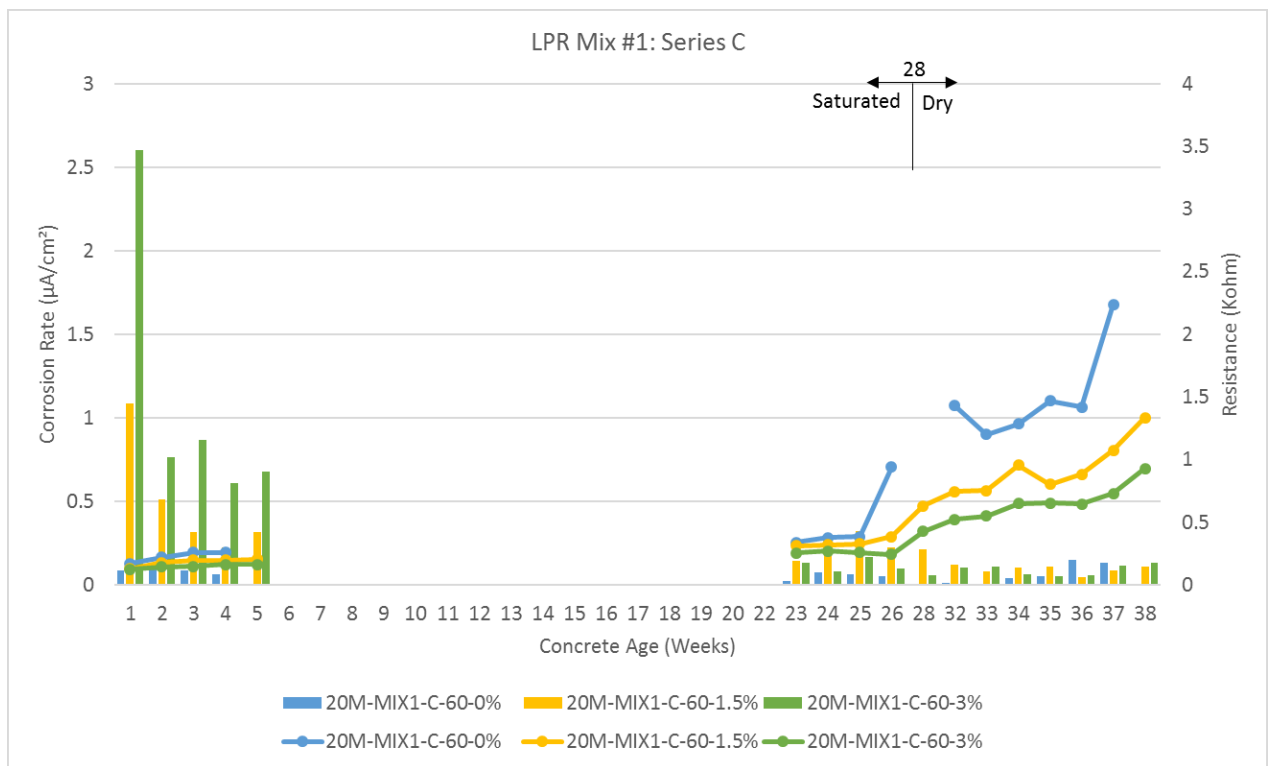


Figure 4.22: Laboratory Trend Results: LPR Mix 1: Series C

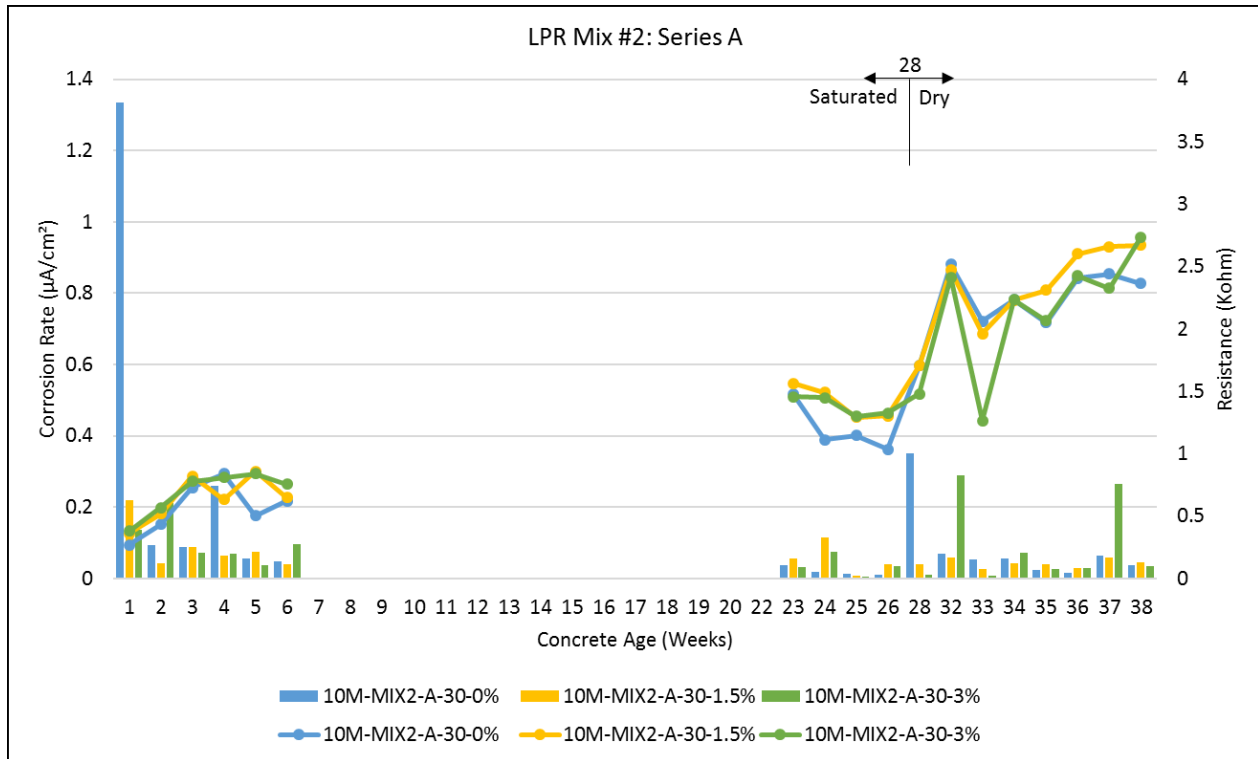


Figure 4.23: Laboratory Trend Results: LPR Mix 2: Series A

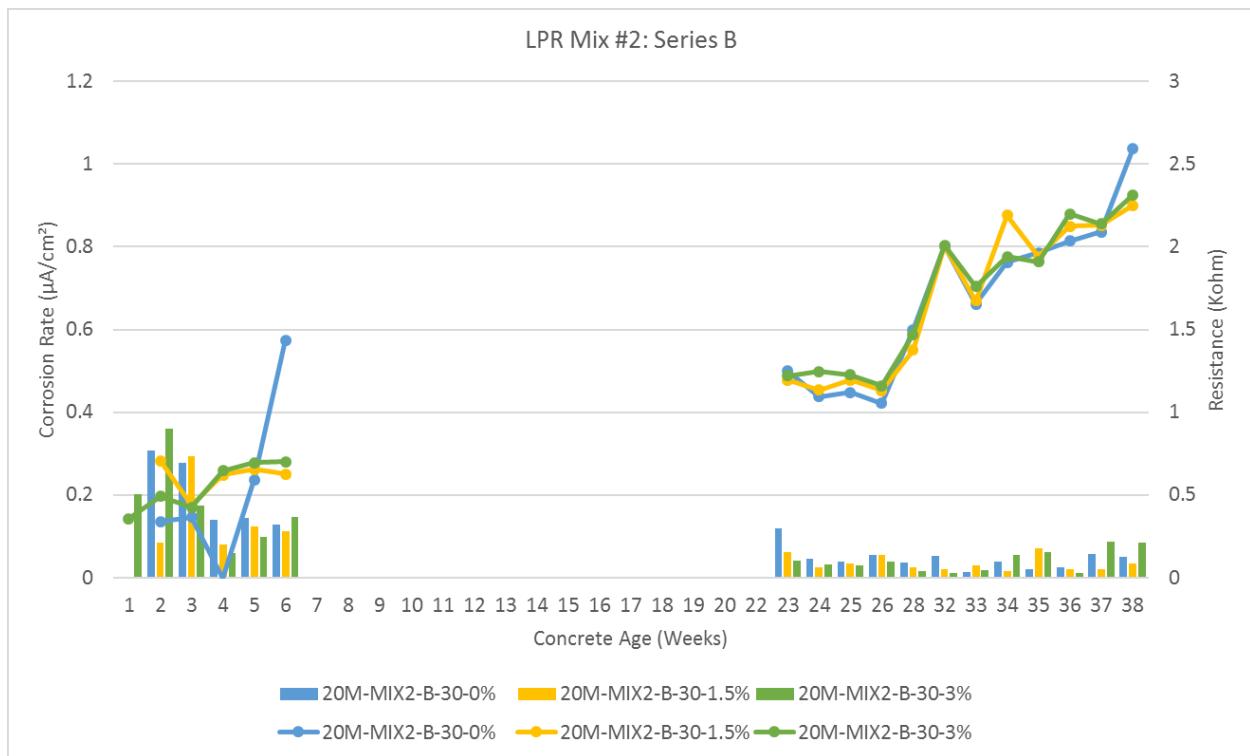


Figure 4.24: Laboratory Trend Results: LPR Mix 2: Series B

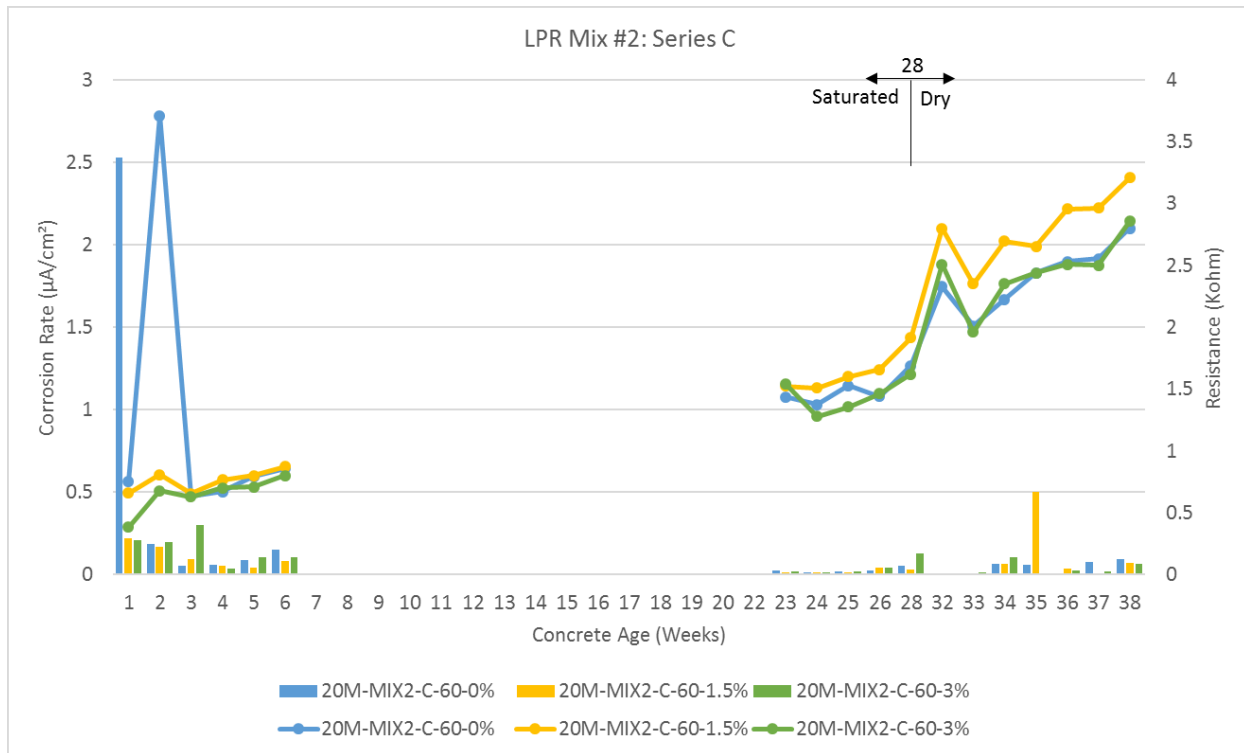


Figure 4.25: Laboratory Trend Results: LPR Mix 2: Series C

As to be expected, the general trend, regardless of the device used, includes the following observations:

- The corrosion rate measured by the CEPRA, GPT, and LPR techniques for Series A slabs built with mix 1 was higher for slabs with a 3% chloride content than those with 1.5% and 0%.
- There is a spike in the corrosion rate during the week of testing following the casting of the concrete slabs.
- At 28 weeks, all concrete slab specimens were removed from high relative humidity conditions resulting in a spike in electrical resistivity or resistance and a decrease in corrosion rate.
- The measurements by CEPRA, GPT, and LPR collected up to week 7 for series B and C display a higher corrosion rate for slabs with 1.5% chloride content than those with 3%. Similar to the conclusions by Luping (2002), it is reasonable to assume that high concentrations of chloride content have led to lower amounts of dissolved oxygen resulting in lower corrosion rates. Additionally, for ease of construction, concrete forms were constructed based on the size of reinforcing bars and not based on the percentage of chloride content. As such a single form, constructed to pour three slabs, had concrete with 0%, 1.5% and 3% chloride content. There is a concern that chloride-laden water leached through the wooden forms into the adjacent concrete

slabs resulting in corrosion rate for 1.5% chloride content slabs greater than 3% chloride content slabs.

Figure 4.26 represents the corrosion rate as an average of measurements for each device over the testing period. The overall trend indicates that GPT measures corrosion rates greater than both CEPRA and LPR. This trend is in agreement with Gulikers & Polder (2003) research, which indicated corrosion rates larger than one order of magnitude when compared to specimens tested with LPR. Additionally, this general trend was strengthened by the findings of So & Millard (2007) as corrosion rates calculated from the galvanostatic pulse transient data were generally higher than the rates determined by LPR by a factor of 3.0. Corrosion rates by CEPRA fall above those by LPR and below GPT.

The corrosion rate trend for Mix #1, with a w/c of 0.62, generally is greater than the corrosion rate for Mix #2, with a w/cm of 0.47. This is as expected since Mix #2 has a smaller w/cm and included SCMs leading to a more durable concrete.

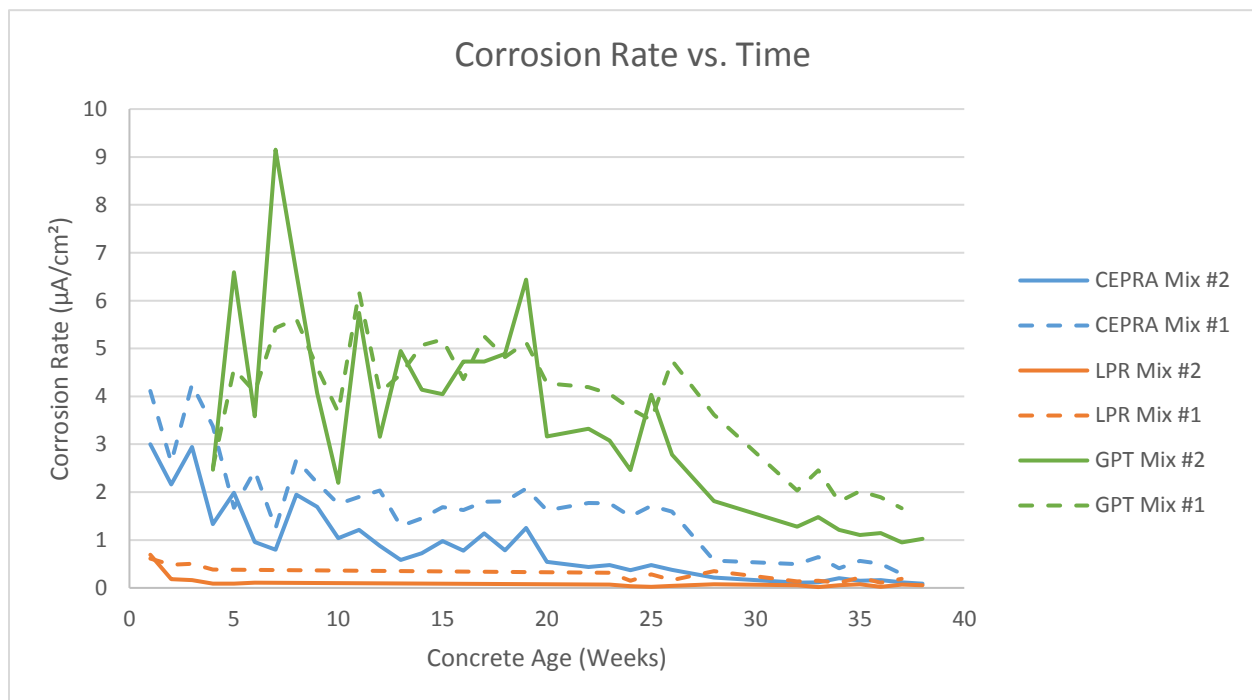


Figure 4.26: Laboratory Trend Results, Average Corrosion Rate Over Time

4.1.1.3. Electrical Resistivity by Four-Point Probe

As a means of monitoring electrical resistivity independently from the electrical resistivity results collected by each technique (CEPRA, GPT, and LPR), the four-point probe was used. The trend results for the nine-month monitoring period for the four-point Wenner probe are presented in Figures 4.27 to 4.29. Figure

4.27 presents the results for all nine Mix #2 slabs. Figure 4.28 and 4.29 present the results for all nine Mix #1 slabs. However, to better illustrate the change, these results are broken into results collected during the first 25 weeks, while the slabs were in a saturated condition, and after 25 weeks, while the slabs were in a saturated surface dry condition.

As expected, the electrical resistivity consistently increases over the 38-week testing period. The electrical resistivity of Mix #2 is approximately six times greater than Mix #1; this is consistent during the testing period. At 28 weeks, there is a spike in electrical resistivity for all slabs, a slight decrease followed by a continuation of the general increasing trend. The maximum electrical resistivity measured was roughly 1,700 ohms* m for Mix #2 with 20M reinforcing bars, 60-mm cover, and 1.5% chloride content. The maximum electrical resistivity measured for Mix #1 was 1,200 ohms*m with 20M reinforcing bars, 60-mm cover, and 1.5% chloride content. An outlier of 2,600 ohms*m for Mix #1 with 20M reinforcing bars, 30mm cover and 3% chloride content has been removed from further analysis. Additionally, all electrical resistivity measurements for this slab after 28 weeks appear to be significantly greater than the general trend for Mix #1 leading to the conclusion that there is an error in measurements for Mix 1, Series C slabs with 1.5% chloride content. This is possibly due to a spill of CuSO_4 liquid solution from the Gecor device creating a disrupted electrical circuit at the interface of the concrete. For both Mix #2 and Mix #1, the electrical resistivity for slabs with a 60-mm cover is consistently higher than the electrical resistivity for slabs with a concrete cover of 30 mm, a trend consistent with the findings of Elsener & Bohni (1997).

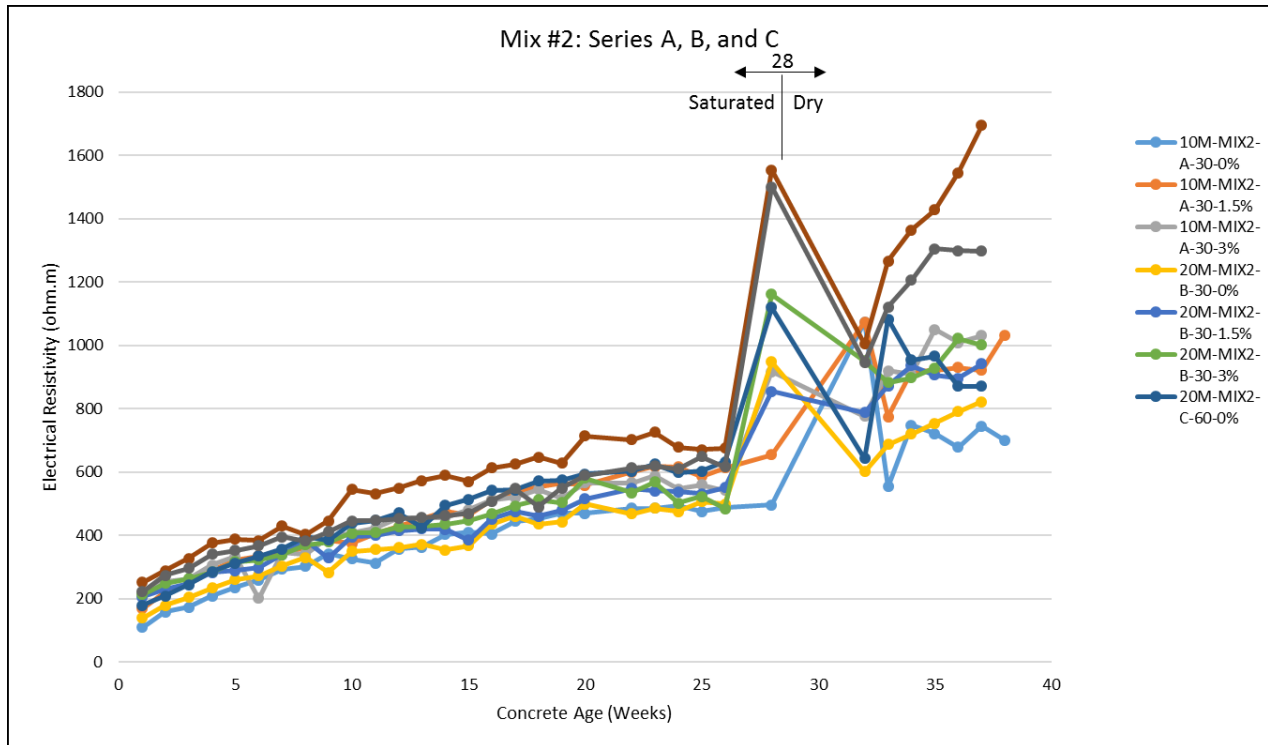


Figure 4.27: Laboratory Trend Results: Four-Point Probe Mix 2

In two instances, the measurements provided by the four-point probe for Mix 1 under saturated condition illustrate an abnormal measurement (Figure 4.28), with electrical resistivity significantly less than the results of similar concrete age for the same specimen. These anomalies can be attributed to malfunction at the time of measurement or poor electrical contact with the concrete surface.

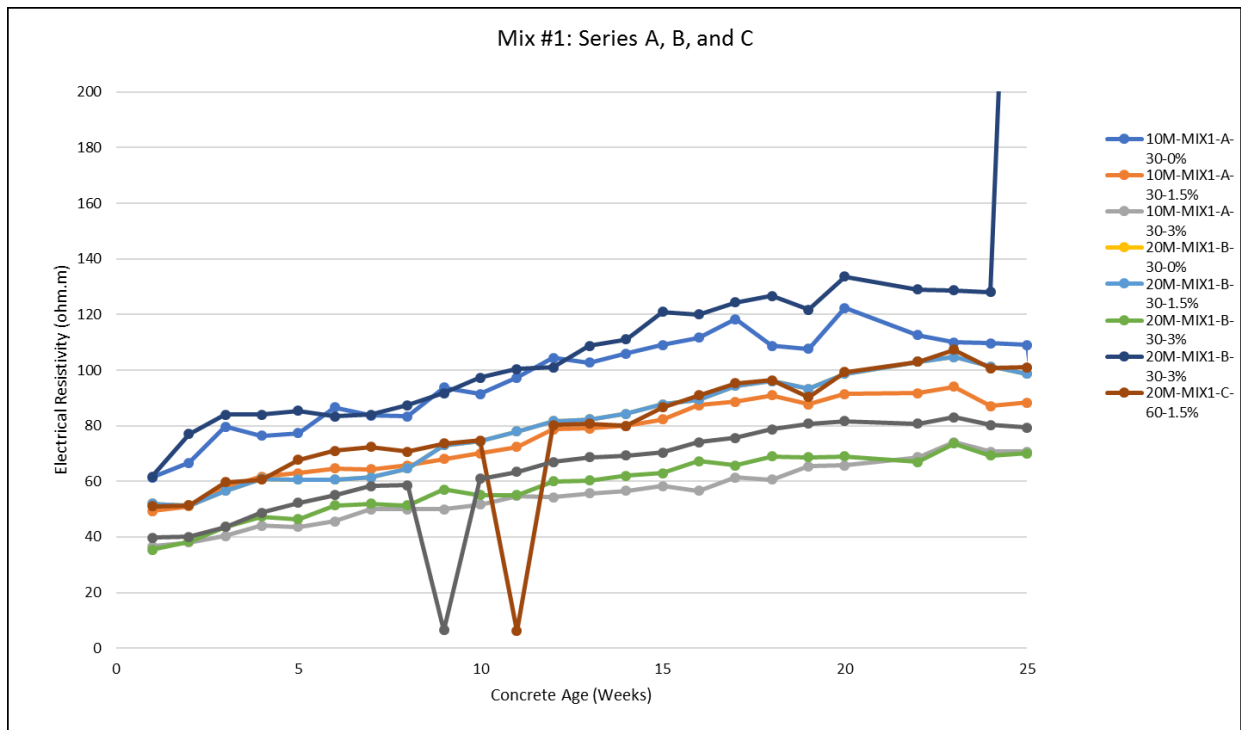


Figure 4.28: Laboratory Trend Results: Four-Point Probe Mix 1. Saturated Condition

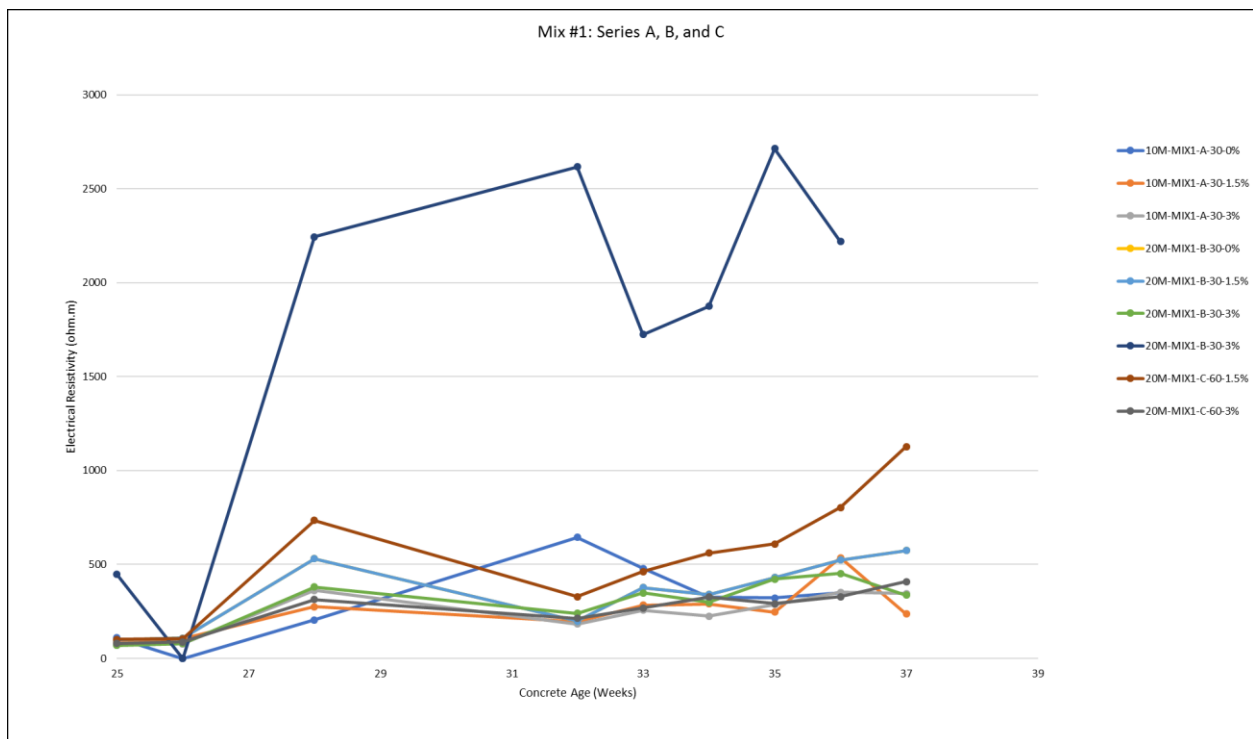


Figure 4.29: Laboratory Trend Results: Four-Point Probe Mix 1. Saturated Surface Dry Condition

Similar to the trend of electrical resistivity and resistance by CEPRA, GPT, and LPR, the average electrical resistivity gradually increases by 10% (maximum) per week as the concrete slabs cure. This is in agreement with the generalized relationship between electrical resistivity and degree of concrete hydration, as concrete cures and strengthens, the electrical resistivity increases (Lataste J.-F. , 2010). Additionally, the electrical resistivity of mix #2 is vastly greater than the resistivity of mix #1. This is due to the improved concrete microstructure of mix #2, which has a water-to-cementitious ratio of 0.47, whereas mix #1 has 0.62. Additionally, Mix#2 included slag and fly ash (whereas mix #1 does not), thus providing for a more isolated pore network and low ionic concentration, which decreases the ability for electrical current to flow throughout the concrete (Berke, Scali, Regan, & Shen, 1991). These comparisons are easily highlighted in Figure 4.30, which provides the average electrical resistivity for each technique over the testing period. To allow for comparable data, resistance by GPT and LPR values are converted to electrical resistivity by Eq. 2-7 using a geometric factor of $K = (1/0.578)$ (Layssi et al, 2015). Measurements by LPR, for both Mix #2 and Mix #1, are significantly lower than that of GPT, CEPRA and Four-Point Probe.

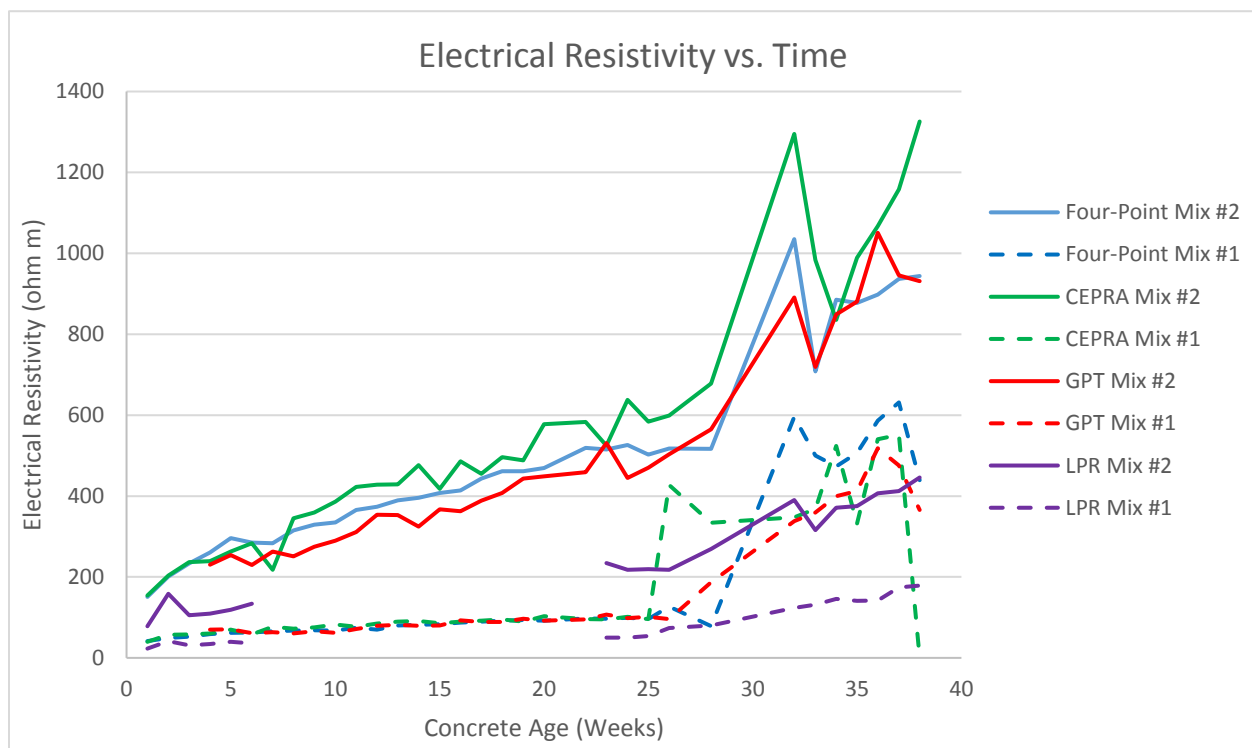


Figure 4.30: Laboratory Electrical Resistivity vs Time

4.1.1.4. Electrical Resistivity vs Corrosion Rate

The relationship between electrical resistivity and corrosion rate has been correlated by empirical relationships, such as those provided by Andrade & Alonso (2004), Ghods, Isgor, & Pour-Ghaz (2007) and

Ahmad (2014). The corrosion rate, taken as an average for the measurements up to 28 weeks (saturated condition) and after 28 weeks (saturated surface dry condition) is plotted against electrical resistivity in Figure 4.31. The relationships established by the above-mentioned researchers, which are given in Table 2.7, are plotted in Figure 4.31 as well. The relationship between electrical resistivity and the corrosion rate is affected by many parameters including cement type, specimen geometry, and cover depth. As the water to cement/cementitious material ratio provides the most pronounced variable, these results are broken down by mix #2 (w/cm of 0.47) and mix #1 (w/c of 0.62) for this comparison. Additionally, the presence of SCMs increases the electrical resistivity of the concrete (Polder, 2000). As CEPRA technology directly provides electrical resistivity measurements, this comparison is only provided from the results obtained using the CEPRA device.

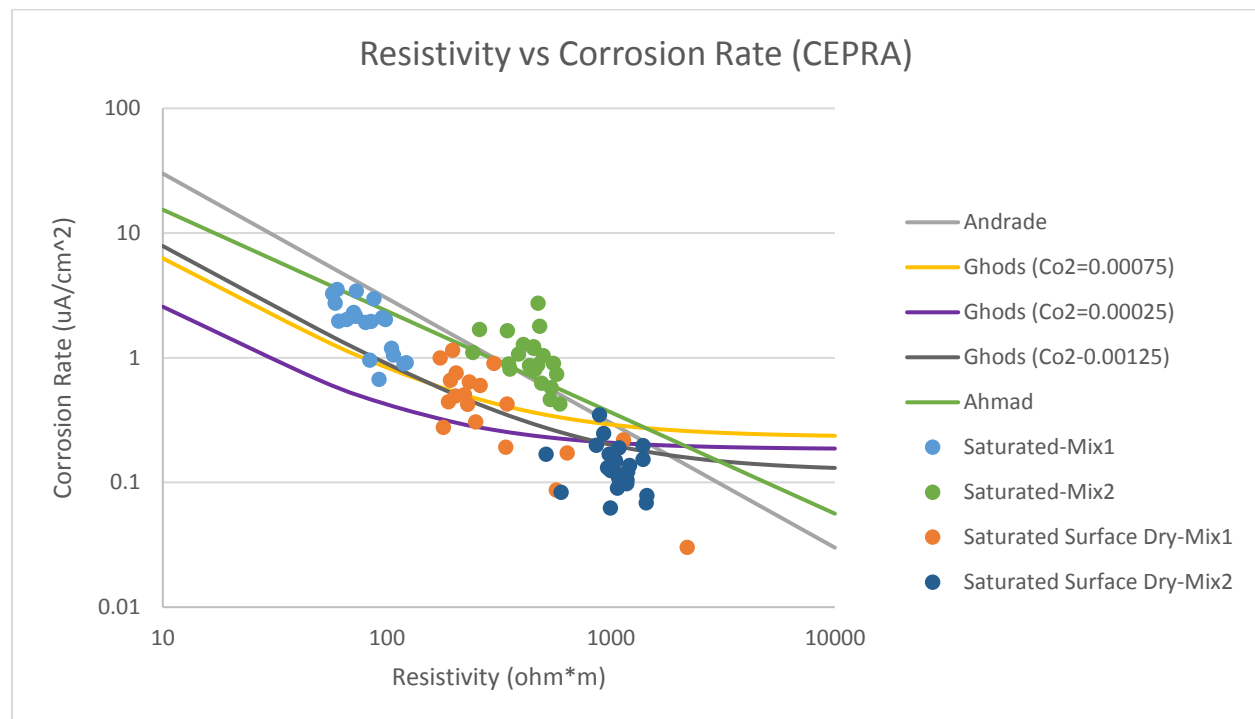


Figure 4.31: Laboratory Electrical Resistivity vs. Corrosion Rate

For higher corrosion rates, the electrical resistivity is anticipated to be lower. The opposite is also established for lower corrosion rates, where the electrical resistivity is anticipated to be higher (Lataste J.-F. , 2010). This generalized relationship is seen for measurements from the CEPRA technique. Furthermore, both mixes, each for saturated and semi-dry conditions, consistently show clusters of data points following the trends proposed by Andrade & Alonso (2004) and Ahmad (2014). The most agreeable trend can be found for Mix #1 and Mix #2, where data from saturated and saturated surface dry condition is combined for comparison. Refer to Figure 4.32.

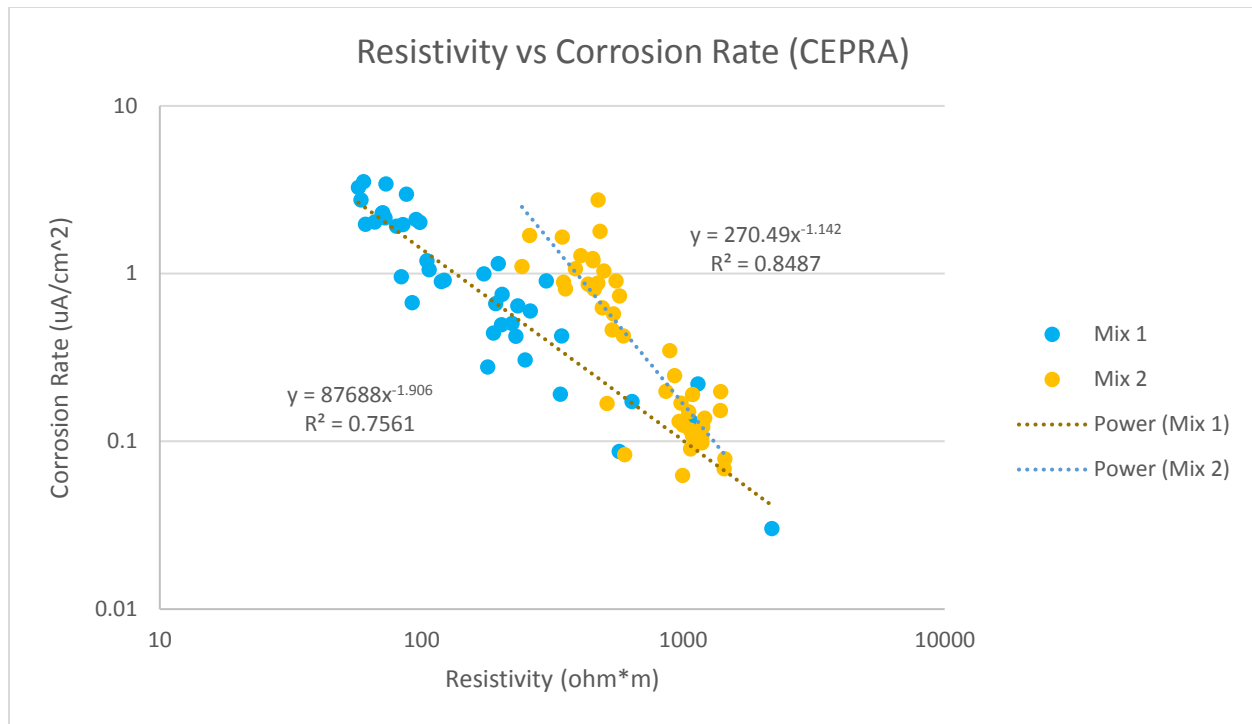


Figure 4.32

For Mix #1, a power trend was established as shown in Eq. 4-1 with a coefficient of determination of 85%. For Mix #2, the results also follow a power trendline, as shown in Eq. 4-2 with a coefficient of determination of 76%. These results indicate the CEPRA technology has a good correlation between average electrical resistivity and average corrosion rate for both Mix #1 and Mix #2.

$$y = 270.49x^{-1.142}$$

$$R^2 = 0.8487$$

Equation 4-1: Laboratory Equation Mix 1

$$y = 87688x^{-1.906}$$

$$R^2 = 0.7561$$

Equation 4-2: Laboratory Equation Mix 2

4.1.2. Gravimetric Mass Loss

After the nine-month period in which the reinforced concrete slabs deteriorated, the reinforcement was removed. Upon initial inspection, the rebars within Mix #1, the poorer concrete mix, appeared to have more corrosion products than those rebars on Mix #2. Pictures of the reinforcement were taken, both sides of the rebar, and can be found in Appendix I. The weight of the corroded reinforcing bars, length, and indication of any unusual defects on the rebars after extraction were recorded. The original weight of the reinforcing bars, prior to placement in the slabs, was not taken. No rebars were excluded from the analysis due to unusual defects.

The results from gravimetric mass loss testing conducted according to ASTM G1-03 (2011) are found in Appendix J. Mass loss for each specimen was adjusted by four 10M control and seven 20M control specimens. Adjustments by the control mass loss account for the presence of mill scale on the surface of the reinforcing bar. Additionally, adjustments by the control mass loss account for deviation in reinforcing steel density from that anticipated by standard reinforcing steel manufacturers. Based on mass loss and specimen length, the average weight by linear distance for 10M and 20M control reinforcing steel is 0.77 kg/m and 2.28 kg/m, respectively.

From the results of the mass loss testing, the corrosion rate based on Faraday's Second Law, refer to Eq. 2-12, was determined. The surface area for each specimen was determined based on measured length and radius. Three measurements were taken for both length and radius; the average for both parameters was used in the calculation of the gravimetric corrosion rate. Time of exposure for Mix #1 and #2 was 38 weeks and 5 days. For measurements by LPR, the time of exposure was less as testing did not occur during weeks 7 to 22. In addition, the corroded area as determined from the *ImageJ* software was also determined. Ultimately, the average gravimetric corrosion rate, which used the complete surface area and not the corroded surface area, was used in the comparison.

Non-destructive testing by CEPRA, GPT, and LPR measured instantaneous corrosion rate over the 38 week testing period. The average electrochemical-predicted corrosion rate was determined by integrating the instantaneous corrosion rates over the testing period (38 weeks) for each reinforcing specimen and each non-destructive testing technique. Integration was accomplished by the use of the Trapezoid Rule.

The average corrosion rate obtained from each device is compared to the gravimetric corrosion rate in Figures 4.33 (10M reinforcing bar) and 4.34 (20M reinforcing bar). Standard deviations for 0% chloride content 10M reinforcing bars for CEPRA, GPT, and LPR are 0.08, 0.035, and 1.42, respectively. Standard deviations for 1.5% chloride content for 10M reinforcing bars for CEPRA, GPT, and LPR are 0.64, 1.07, and 25.4, respectively. Standard deviations for 3% chloride content for 10 M reinforcing bars for CEPRA, GPT, and LPR are 0.28, 0.48, and 27.6, respectively. Standard deviations for 0% chloride content 20M reinforcing bars for CEPRA, GPT, and LPR are 10.8, 5.7, and 221.8, respectively. Standard deviations for 1.5% chloride content for 20M reinforcing bars for CEPRA, GPT, and LPR are 5.0, 1.32, and 188.01, respectively. Standard deviations for 3% chloride content for 20M reinforcing for CEPRA, GPT, and LPR are 6.89, 3.05, and 126.60, respectively.

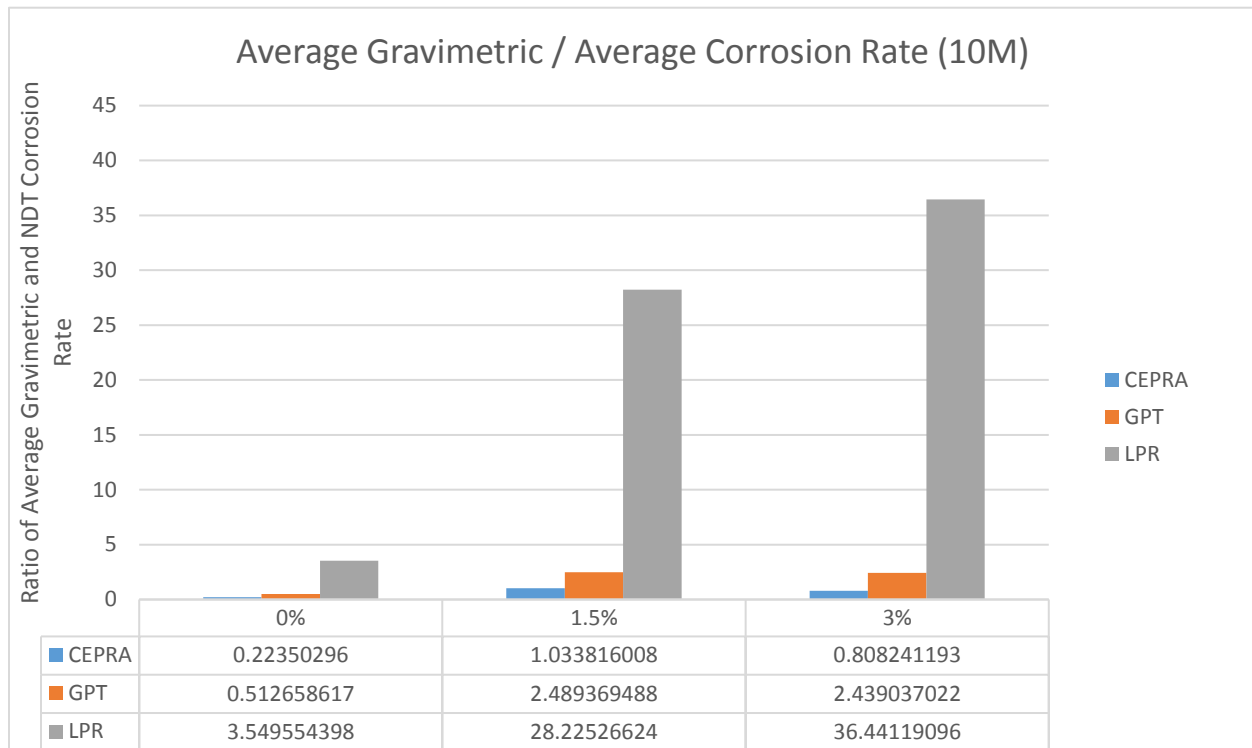


Figure 4.33: Average Ratio of Gravimetric to Average Corrosion Rate – For 10M Reinforcing Bars

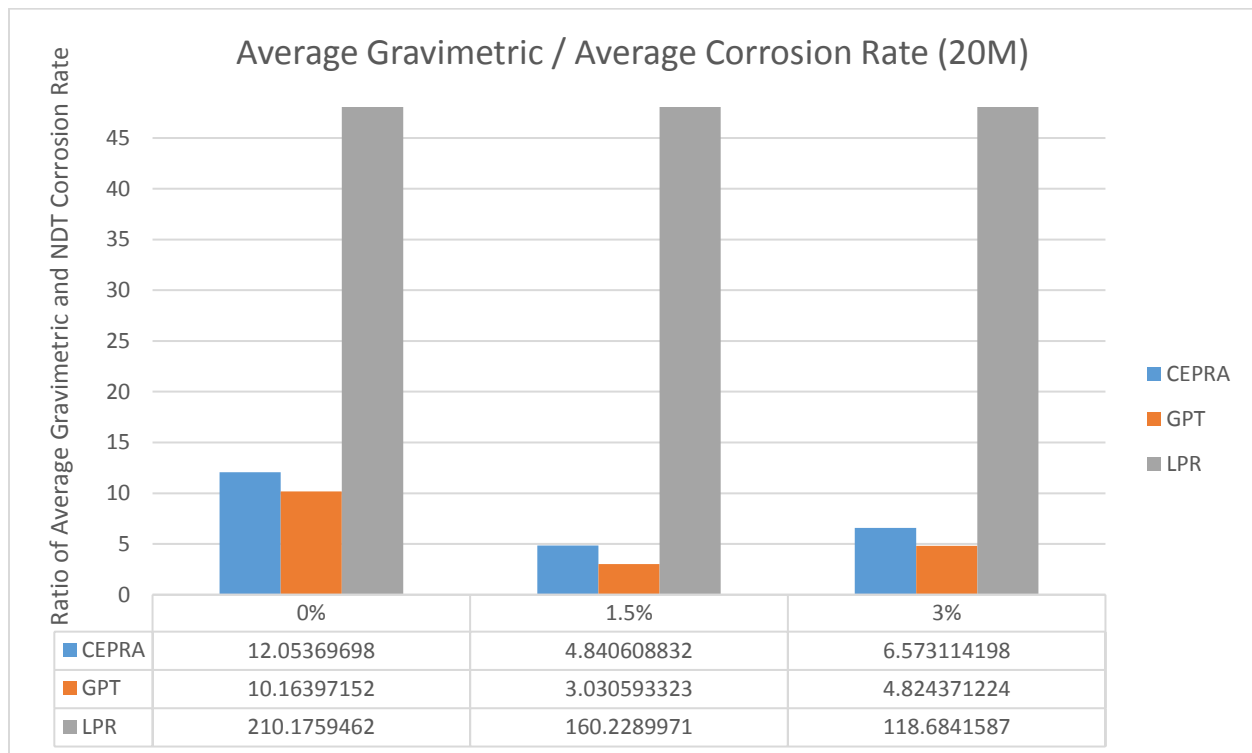


Figure 4.34: Average Ratio of Gravimetric to Average Corrosion Rate – For 20M Reinforcing Bars

From Figures 4.33 and 4.34, it is observed that the LPR technique grossly underestimates the actual corrosion rate by an average ratio greater than 40 for both 10M and 20M reinforcing bars. Although other experimental studies by So and Millard (2007) and Sathiyarayanan et al. (2006) agree with this conclusion, the magnitude of this comparison is significantly greater. It is anticipated that had measurements been successfully recorded for the entire testing time (weeks 7-22 where Gecor device was unavailable), this magnitude in error would be less. This constitutes an error in this study. It is important to emphasize that the average corrosion rate for the LPR technique was determined based on the time where measurements were recorded and not the entire 38 weeks.

Generally, the gravimetric corrosion rate is greater than the actual corrosion rate, indicating that all devices underestimate the corrosion rate to varying degrees. This may be due to the fact that original weight of rebar was not recorded before casting the concrete sample. The average weight by linear distance for 10M and 20M reinforcing bars was determined by the control bars. However, this does not provide for the best analysis and may be the main reason for such a high values and scattered data. This conclusion is further strengthened by the larger average ratio of gravimetric to actual corrosion rate for 0% chloride content specimens.

The accepted trend for deterioration of reinforcing bars due to the presence of chlorides is that as the percentage of chloride content increases, the corrosion rate increase in tandem. This positive correlation is consistent with experimental studies by Luping (2001), Sathiyarayanan (2006), and Dou (2014). The average corrosion rate results presented in Figure 4.34 do not follow this accepted trend; reinforcing bars subjected to an environment with 0% chloride content have higher average corrosion rate than 1.5% and 3%. Therefore, there is significant error with these results for the 20M reinforcing bars. This error may be attributed to leaching of chloride infused water from one slab to another during concrete curing or human error during construction of the slabs. As previously noted, corrosion products on 20M reinforcing bars were visibly less present than those of 10M. Due to this, there is a possibility that mass loss testing inaccurately represented the mass loss of corrosion products as there were minimal corrosion products present. As these results are grossly inconsistent with the anticipate trend, the data collected for the 20M reinforcing bars will not be used for futhur analysis.

The average gravimetric corrosion rate is plotted against the average corrosion rate by each device for 10M reinforcing bars only. Refer to Figure 4.35. Outliners are identified within the plot and removed from further analysis. The two lines besides the equality line indicate the upper and lower limit when

considering a tolerance factor of 2; this is commonly used when comparing electrochemical techniques (Luping, 2002).

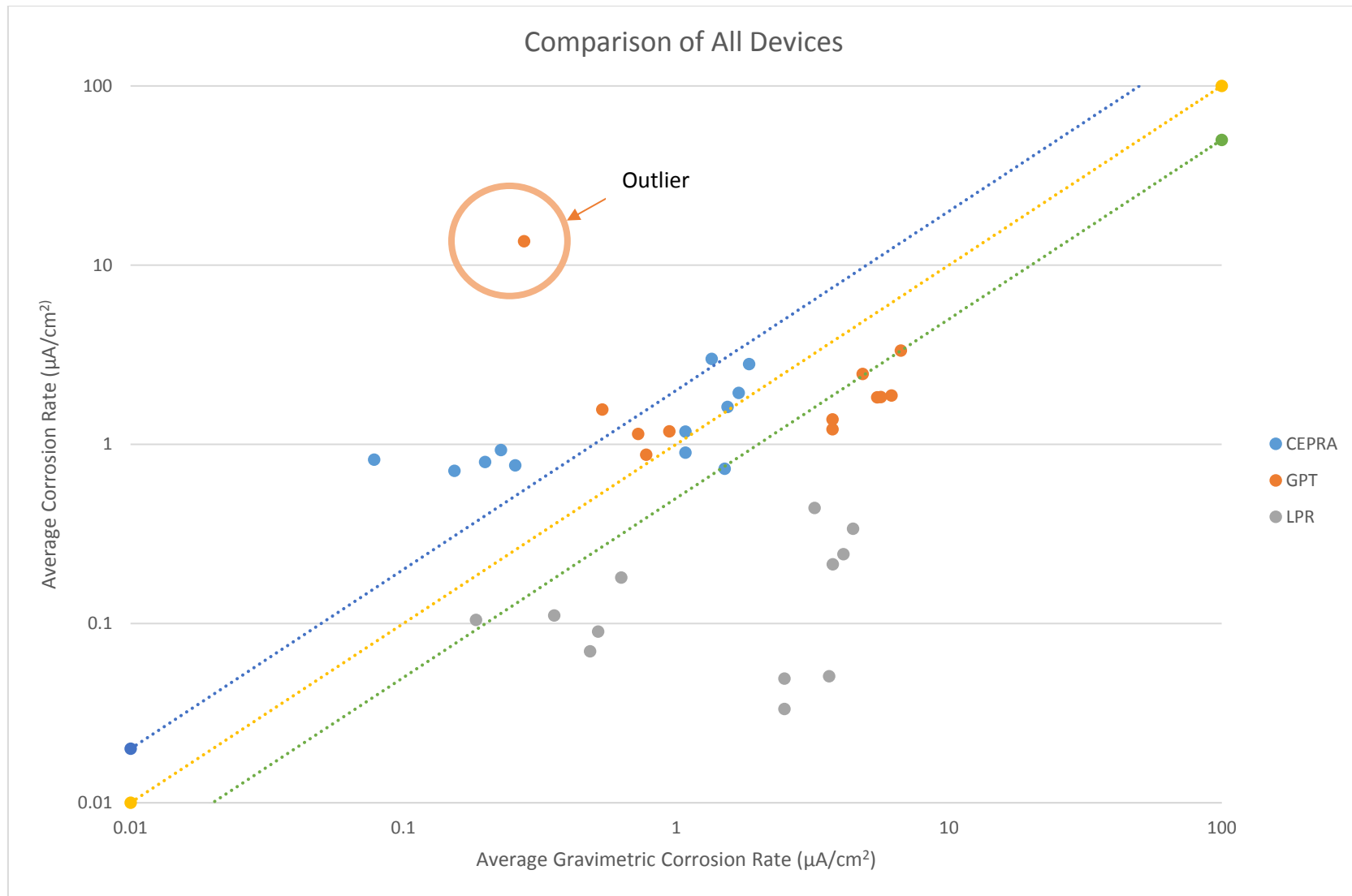


Figure 4.35: Laboratory Gravimetric vs Average Corrosion Rate – Comparison of all Devices, 10M Reinforcing Bars Only

To address the inconsistent polarization areas used by each non-destructive device and the length of specimen used for gravimetric testing, the corrosion rates were adjusted. CEPRA intends to polarize 95 cm² of lateral surface area. GPT intends to polarize 7 cm of reinforcement length. LPT intends to polarize 10.5 cm of reinforcement length. Gravimetric testing was conducted on reinforcement approximately 250 mm in length. All corrosion rates were adjusted to a comparable polarization area. This was accomplished by multiplying the average corrosion rate by the ratio of polarization area as used by each device and lateral surface area of each specimen used to determine gravimetric mass loss.

A comparison of gravimetric and average corrosion rate indicates a relatively good correlation for CEPRA and GPT, while LPR generally underestimates actual corrosion rate. Refer to Table 4.1 which presents the coefficient of determination for each device based on 0%, 1.5%, and 3% chloride content. The coefficient for all 10M reinforcing data points is also included. The coefficient was determined for each group of data in comparison with the intent of determining how strongly each data set relates to the other. A coefficient of 1.0 indicated a perfect positive correlation. A positive coefficient indicated that as one set of data increases so does the other. The coefficient relates the modified gravimetric corrosion rate and the modified corrosion rate for each technique. Overall, GPT coefficients has the strongest relations with 0% chloride content, 1.5% chloride content, and 3% chloride content testing parameters in range from moderate to strong in terms of strength of relationship as all coefficients are above 0.4. CEPRA coefficient for 0% chloride content indicates no correlation while 1.5% and 3% chloride content are within moderate to strong range in terms of strength of relationship. LPR has the weakest correlation with 0% and 1.5% chloride content below 0.5.

Table 4.1: Laboratory Correlation Coefficient

	10M	0% Chloride Content	1.5% Chloride Content	3% Chloride Content
CEPRA	0.48	0.03	0.37	0.6
GPT	0.55	0.4	0.89	0.85
LPR	0.24	0.35	0.33	0.51

For laboratory testing and comparison using GPT, this correlation is in agreement with Vedalakshmi et al. (2010), where anodic pulse and pulse duration are similar to that of this study. Additionally, this

correlation is in good agreement with Frolund et al. (2002), where corrosion rates obtained on-site by GPT compared well to the average corrosion rate from actual cross-section loss, where the actual corroding area is the same as the confined area. So and Millard (2007) showed through an experimental study that GPT provided more reliable corrosion rates on reinforcing bars than that of the LPR technique. LPR providing an underestimation of actual corrosion rate is in agreement with Sathiyarayanan et al. (2006), where results obtained by LPR were found to provide lower values than that of the actual corrosion rate.

Results of the analysis of the data obtained from all devices by percentage of chloride content is presented in Figure 4.36

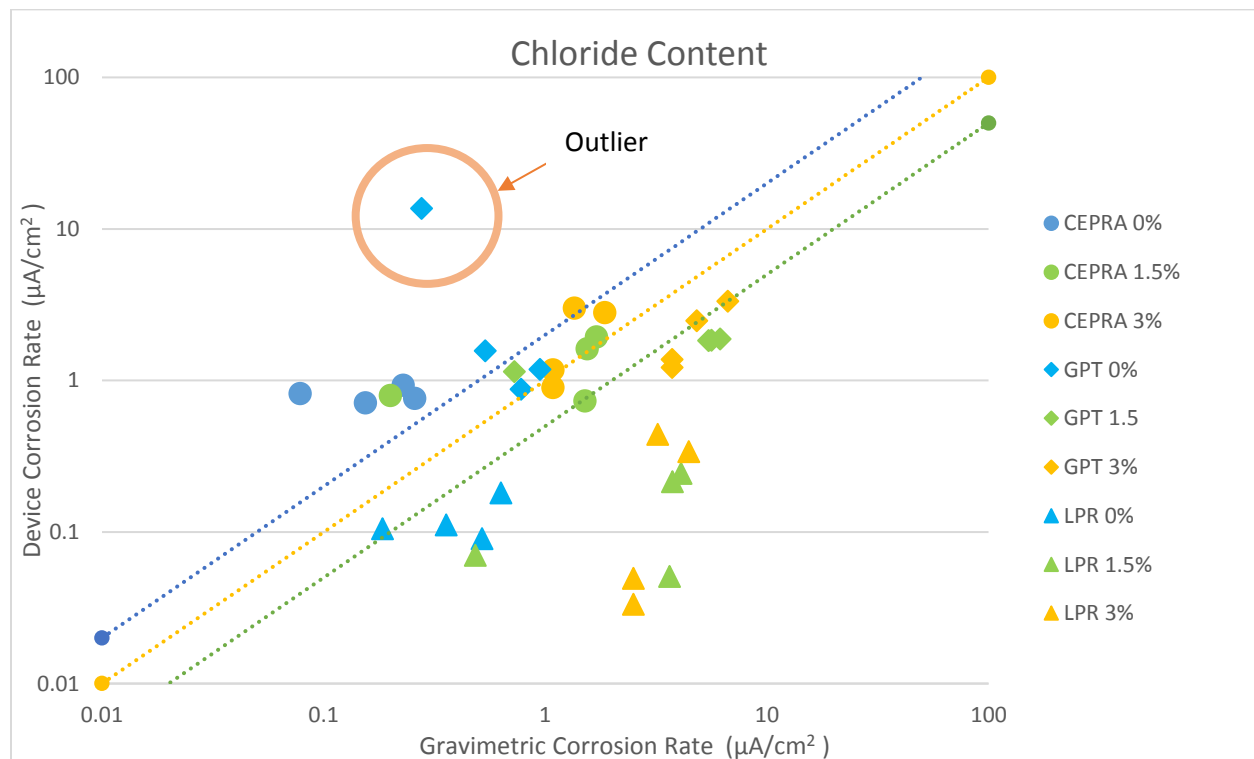


Figure 4.36: Laboratory Gravimetric vs Average Corrosion Rate – CEPRA by Chloride Content

There is good agreement between gravimetric corrosion rate and average corrosion rate for CEPRA and GPT technology. From Figure 4.36, CEPRA and GPT data points for 0% chloride content indicate the device measures higher corrosion rate than the corresponding gravimetric corrosion rate. This is consistent with Gulikers & Polder (2003) as GPT is unable to make a clear distinction between passive and active corroding reinforcement areas. This overestimation of actual corrosion rate for specimens with no chloride content is also in agreement with laboratory and/or field studies by Luping (2002), Gepraegs & Hansson (2005)

and Nygaard & Geiker (2009). For both CEPRA and GPT 1.5% and 3%, results are generally within the 2.0 tolerance factor. As previously indicated, LPR underestimates the average corrosion rate. However, similar to GPT and LPR, there is a clear distinction between passive and active corroding reinforcement. For all devices, corrosion rates data points for 1.5% and 3% chloride content are beyond $1 \mu\text{A}/\text{cm}^2$ which indicates a high corrosion level and corresponds well with the chloride content.

Further investigation to improve upon the correlation between gravimetric corrosion rate and average corrosion rate for the CEPRA technology was undertaken in an attempt to determine possible improvements to the technology. Possibly modifications to gravimetric corrosion rate include utilizing the percentage of corroded area as determined by the *ImageJ* software to refine the surface area for corroded area only and modifying based on the control mass loss from 0% chloride content specimens and not the originally designated control specimens. Possibly modifications to average corrosion rate include modifying polarization by the intended polarization area (assuming device unable to locate active corroding area due to “self-confining”), utilizing corroded surface area by *ImageJ* instead of entire surface area, using the 0% chloride content reinforcing bars as the control bars, assume half polarization area, and modifications of polarization area by percentage of corroded area by *ImageJ* for both gravimetric and device measured corrosion rate. Refer to Figure 4.37 for the average ratio of gravimetric to average corrosion rate for all possible modifications applied to 10M reinforcing bars.

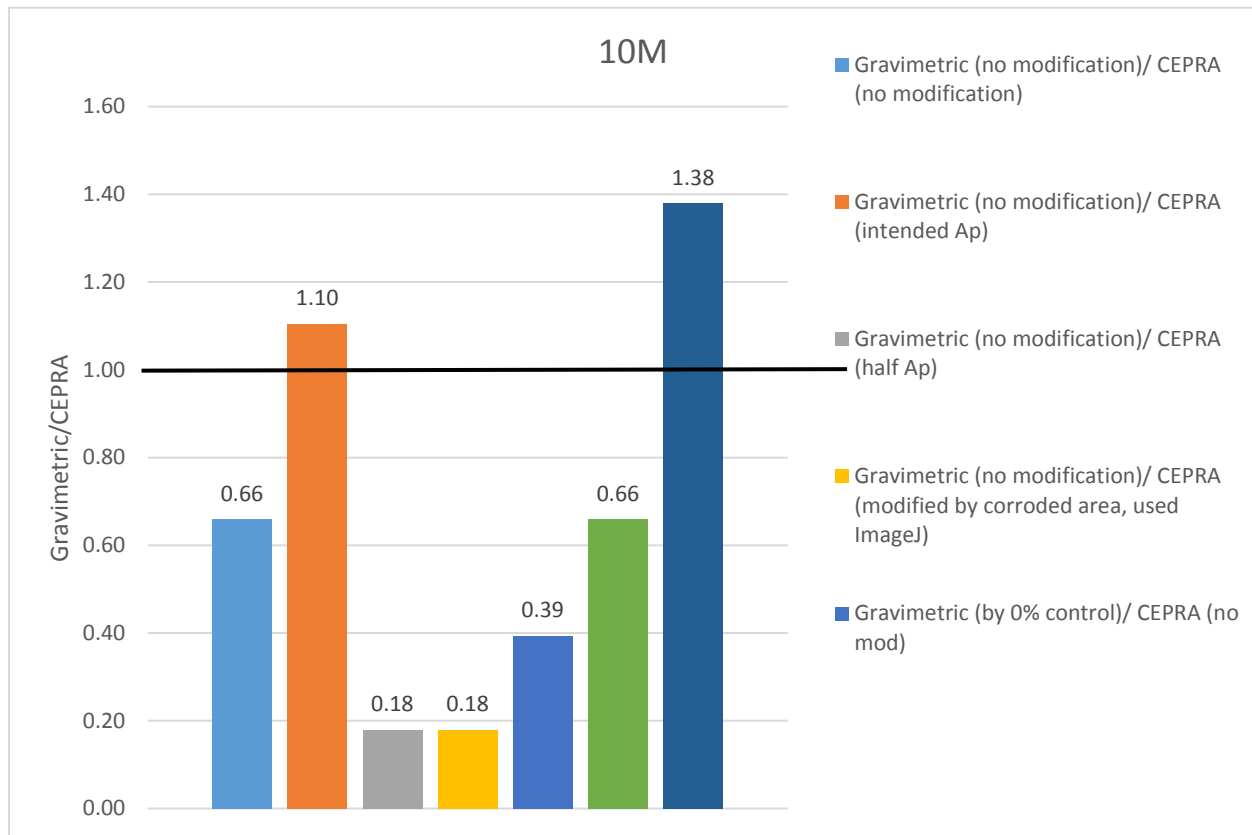


Figure 4.37: Laboratory Average Ratio Gravimetric to Average Corrosion Rate 10M – Possibly Modifications

Out of all possible modifications, as seen in Figure 4.37, it was determined that no modifications to gravimetric corrosion rate by percentage of corroded area on the surface of reinforcing bars or modification to intended polarization area results in an average ratio of 1.10 and standard deviation of 1.04 for 10M reinforcing bars. As a result, the CEPRA technology, measured average corrosion rate accurately as no modification was able to improve the relationship between gravimetric and measured corrosion rate. However, there is a discrepancy between the measured polarization area and the intended polarization area. This is a shortcoming of the CEPRA technology.

It is important to note that all results, graphs, and correlation coefficients presented in Section 4.1.2.1 are based on average gravimetric corrosion rate where the entire surface area of the reinforcing bars was used and average corrosion rate by NDT where no modification was made.

4.2. Field Study

The results and discussion of non-destructive and destructive testing on the four reinforced concrete slabs retrieved from the North Channel Bridge are presented in this section. All non-destructive and destructive testing occurred August 23-25, 2016. Generally, testing conditions varied during the investigation. Non-destructive testing occurred on August 23, 2016, on which the temperature ranged from a low of 12°C to a high of 23°C. Additionally, the relative humidity ranged from 75-92%.

To provide visual representation of the data, *Surfer Version 11 .6* was used to create equipotential contour maps. The default gridding method, the Kriging method, was used to provide a generalized and accurate representation of the results. All contour maps are labelled based on the technique used, and the mapping colour scales are based on the device recommended ranges for corrosion rate, half-cell potential, and resistance or electrical resistivity. Refer to Section 2.4 for more detailed information.

4.2.1. NDT Testing

Non-destructive measurements for instantaneous corrosion rate, half-cell potential, and electrical resistivity or resistance are determined by the following methods:

- Galvanostatic Pulse Technique (GPT), as measured by Germann Instrument's GalvaPulse
- Connectionless Electrical Pulse Response Analysis (CEPRA), as measured by Giatec Scientific's iCOR
- Half-cell potential, as measured by Giatec Scientific's XCell
- Four-point electrical resistivity, as measured by Proceq's Resipod Resistivity Meter

All results are grouped by slab designation: Slab 11, Slab 12, Slab 13, and Slab 14. Refer to Section 3.3 for more information related to each slab.

4.2.1.1. Contour Maps

ASTM 876 (2015) recommends that half-cell potential results be presented on equipotential contour maps with a maximum contour interval of 100 mV. For the contour maps presented here, an interval of 10 mV is used for more accuracy. Half-cell potential results are given in units of mV for all four slabs. Utilizing the Potential Difference Technique and the Numeric Magnitude Technique provided in ASTM 876, the half-cell data is presented on contour maps with colour scales that highlight the potential ranges and the probability of corrosion, refer to Table 2.1 in Section 2.3.1.

Half-cell potential data was also collected using the GPT device, which utilizes a silver/silver chloride (SSE) reference electrode. As the XCell device measures half-cell potential data with potential data referenced

to copper/copper sulfate (CSE), the GPT half-cell data was calibrated to the CSE reference so that the results could be easily compared. Refer to Table 4.4.

Table 4.2: Electrode Potential Differences (Broomfield J. , 2007)

Electrode	Potential (mV SHE)
Calomel (SCE)	+244
Silver/silver chloride (SSE)	+199
Copper/copper sulfate (CSE)	+316
Manganese dioxide	+365
Graphite	+150/-20
Activated titanium	+150/-20
Stainless steel	+150/-20
Lead	-450

The corrosion rate was measured by both the CEPRA and GPT methods. Contour maps showing in-situ corrosion rates measured by CEPRA and GPT for the four slabs are presented in Figures 4.38 to 4.45. All results are presented in $\mu\text{A}/\text{cm}^2$. Typical corrosion rate ranges for the GPT and CEPRA devices are presented in Section 2.4.3.1 and Section 2.4.4.1, respectively. These typical ranges are incorporated in the contour maps and presented in the colour scales seen next to the contour maps.

Concrete resistance was measured by the GPT method and presented in the contour maps in units of $\text{k}\Omega$. Electrical resistivity was measured by the CEPRA method and the Four-Point method and presented in the following contour maps in units of $\text{k}\Omega \cdot \text{cm}$. Refer to Section 2.3.2 for typical ranges for electrical resistivity.

Data points representing half-cell potential, corrosion rate, resistance and electrical resistivity are presented in the contour maps in Figures 4.38 to 4.39 for Slab 11, Figures 4.40 to 4.41 for Slab 12, Figures 4.42 to 4.43 for Slab 13, and Figures 4.44 to 4.45 for Slab 14.

To relate the contour maps with the surface condition of the reinforced concrete slabs, Figures 3.19 to 3.22 from Section 3.3.2 were shadowed behind the contour maps to illustrate where surface obstructions were noted in relation to NDT assessment results.

It is important to note that the relative humidity as recorded by CEPRA ranged from 38-70% while data by Environment Canada recorded environmental relative humidity ranging between 75-92%. As the environmental relative humidity assists the assessment of deteriorated reinforced concrete structures, this large deviation by CEPRA technology will lead to misinterpretation of the condition by users.

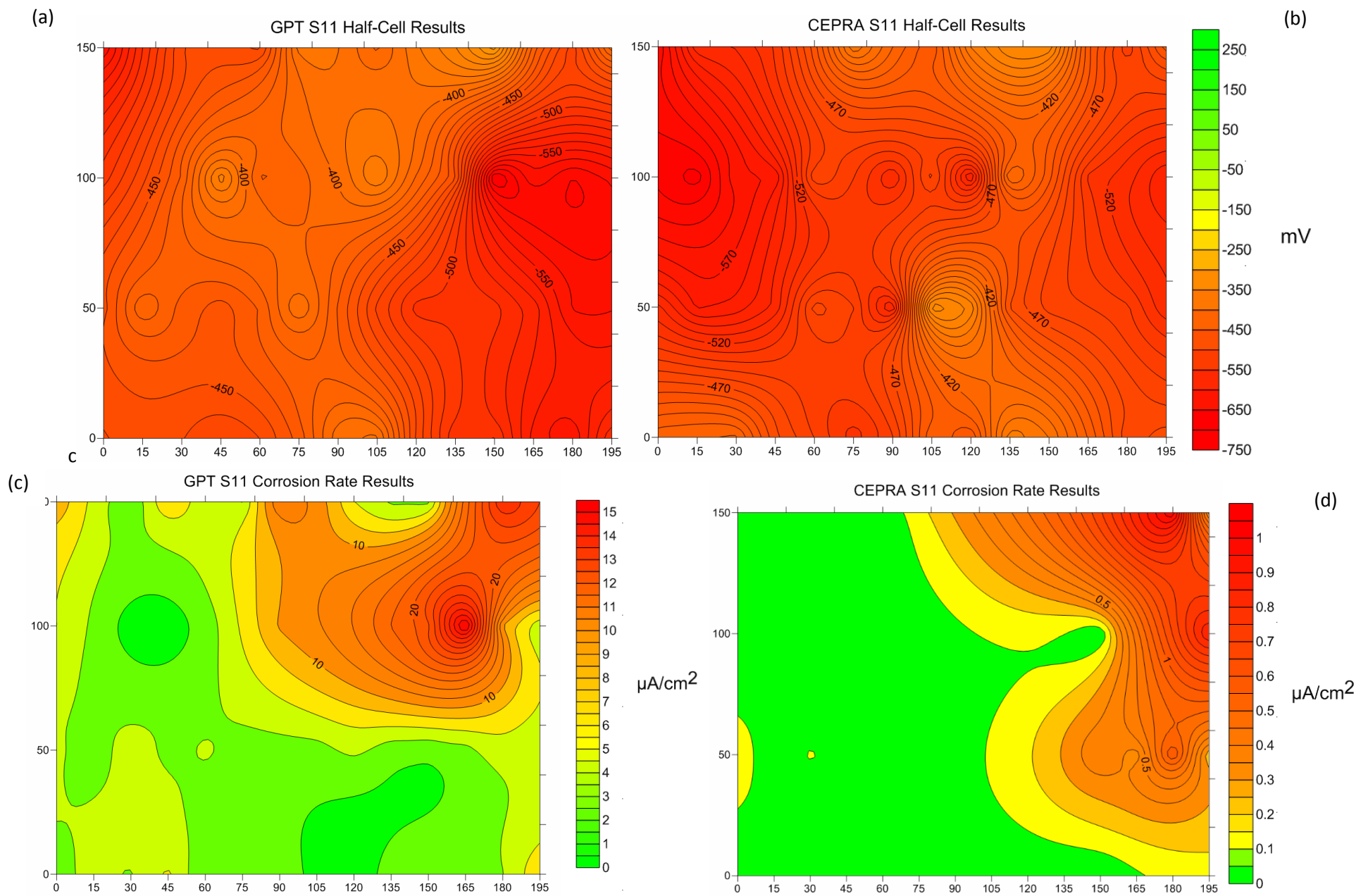


Figure 4.38: S11 Half-Cell (a-b) and Corrosion Rate (c-d) Results

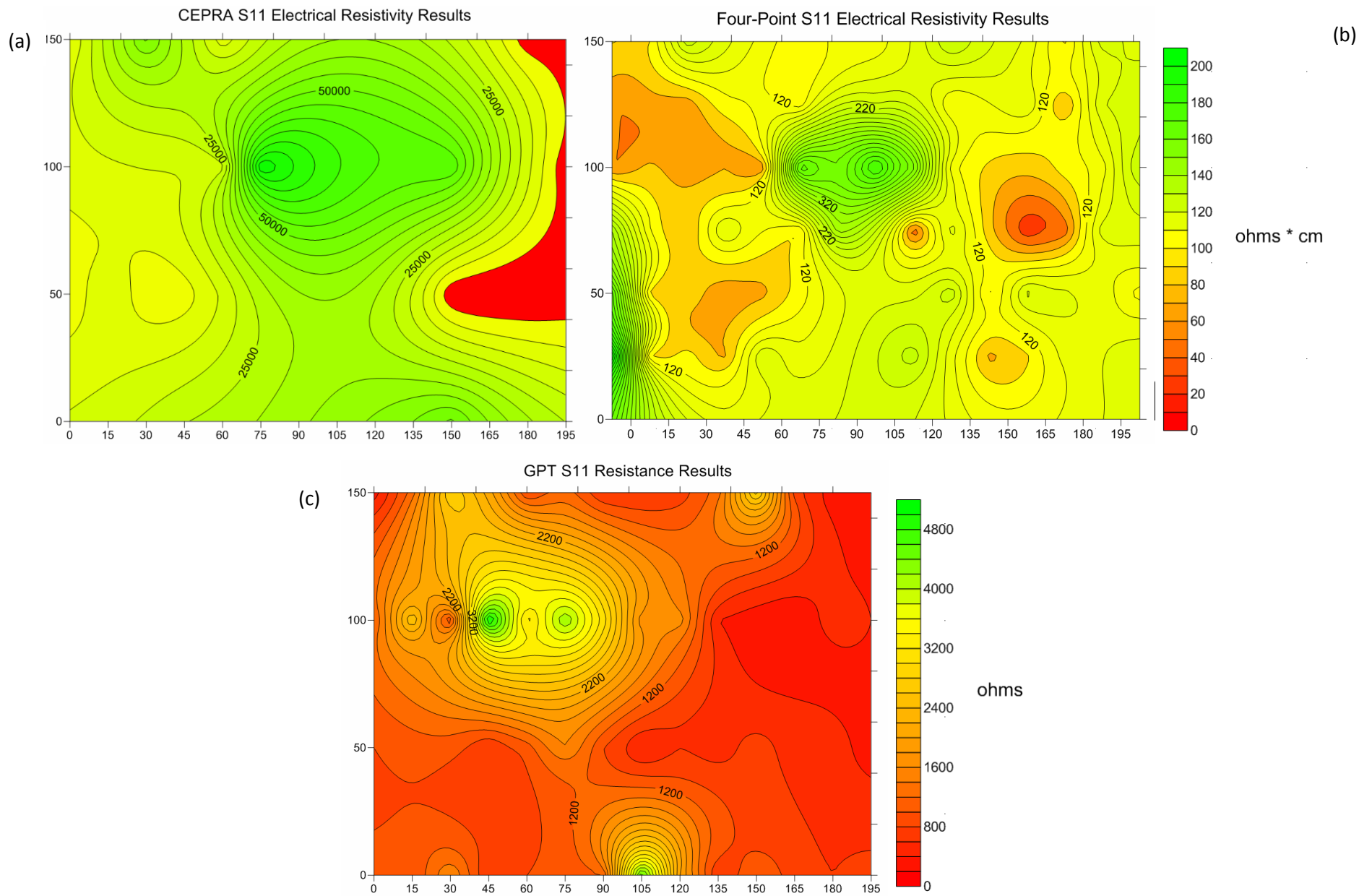


Figure 4.39: S11 Electrical Resistivity (a-b) and Resistance (c) Results

Approximately 75% of the top of the surface of Slab 11 was previously patch repaired and/or covered in non-concrete decking materials, as mentioned in Chapter 3. Refer to the shadowed portion of contour maps. The contour maps in Figures 4.38 to 4.39 illustrate compromised results. Half-cell potential values ranging from -550 mV to -400 mV are noted throughout the entire slab surface, indicating a 90% and greater probability of corrosion. However these half-cell potential results illustrate a cluster of readings potential readings higher than the remaining slab, located at grid 195 and 150, where data collection not compromised, ie. covering in repair materials or traffic membrane. This area coincides with corrosion rates by GPT and CEPRA where corrosion rate is greater than $15 \mu\text{A}/\text{cm}^2$ and $1 \mu\text{A}/\text{cm}^2$ respectively. These areas are an indication of a high level of corrosion activity, as per each device's typical corrosion rates. The Four-Point Probe and GPT recorded results for the uncompromised areas that are within typical ranges. However, the electrical resistivity values measured by CEPRA was unable to distinguish between compromised and uncompromised areas, with electrical resistivity values above $25 \text{ k}\Omega\cdot\text{m}$, well beyond typical ranges for varying concrete conditions, indicated an error in the measurement or an inability for CEPRA technology to make an accurate measurement. Four-point probe indicated electrical resistivity lower than $100 \text{ ohms}\cdot\text{cm}$ near grid 195 and 150, indicating high corrosion severity. These readings are in agreement with the location of high corrosion activity by the other techniques.

Since 75% of the top of slab was compromised due to unforeseeable and unmanageable circumstances, destructive testing was not conducted on Slab 11.

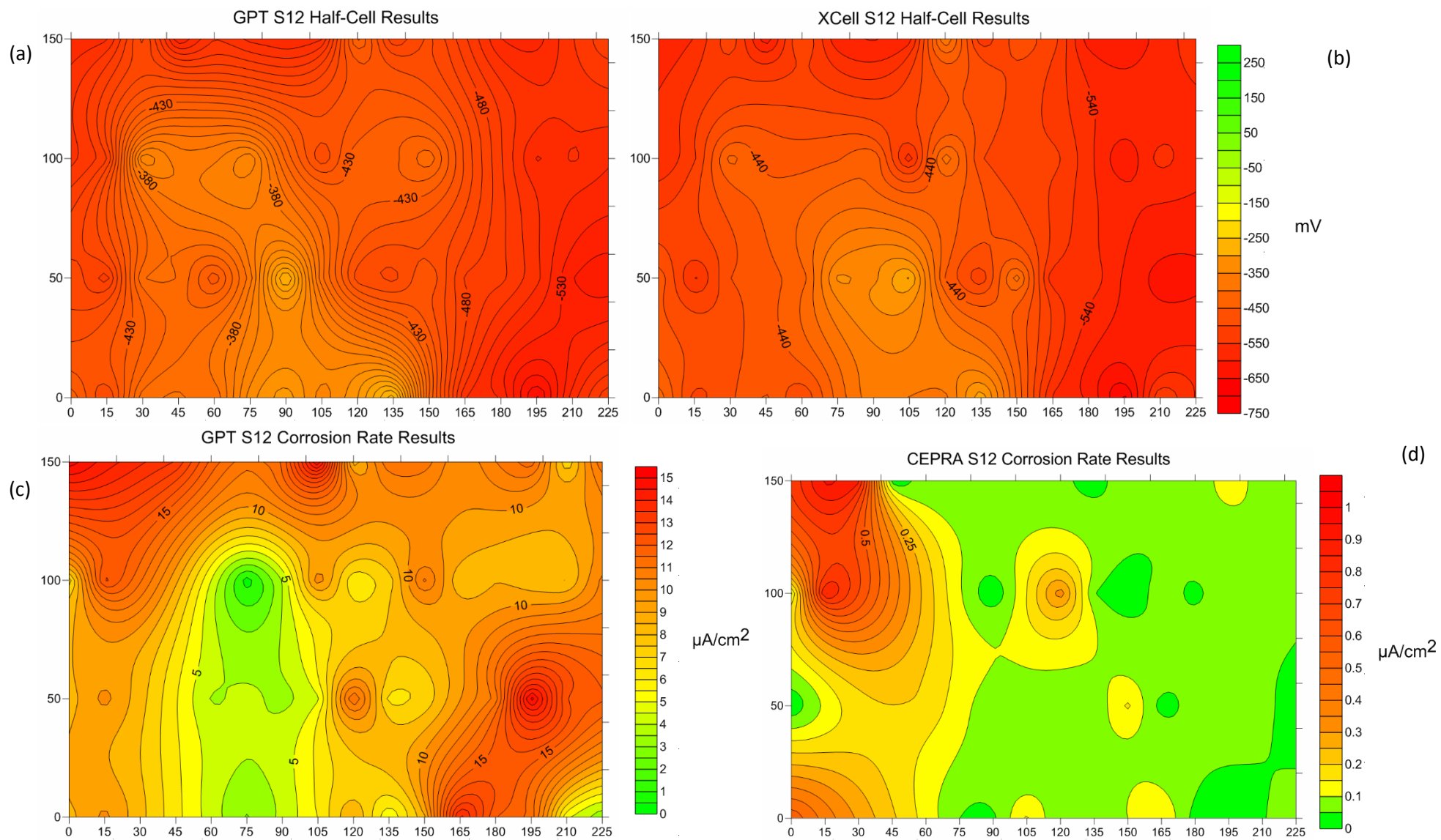


Figure 4.40: S12 Half-Cell and Corrosion Rate Results

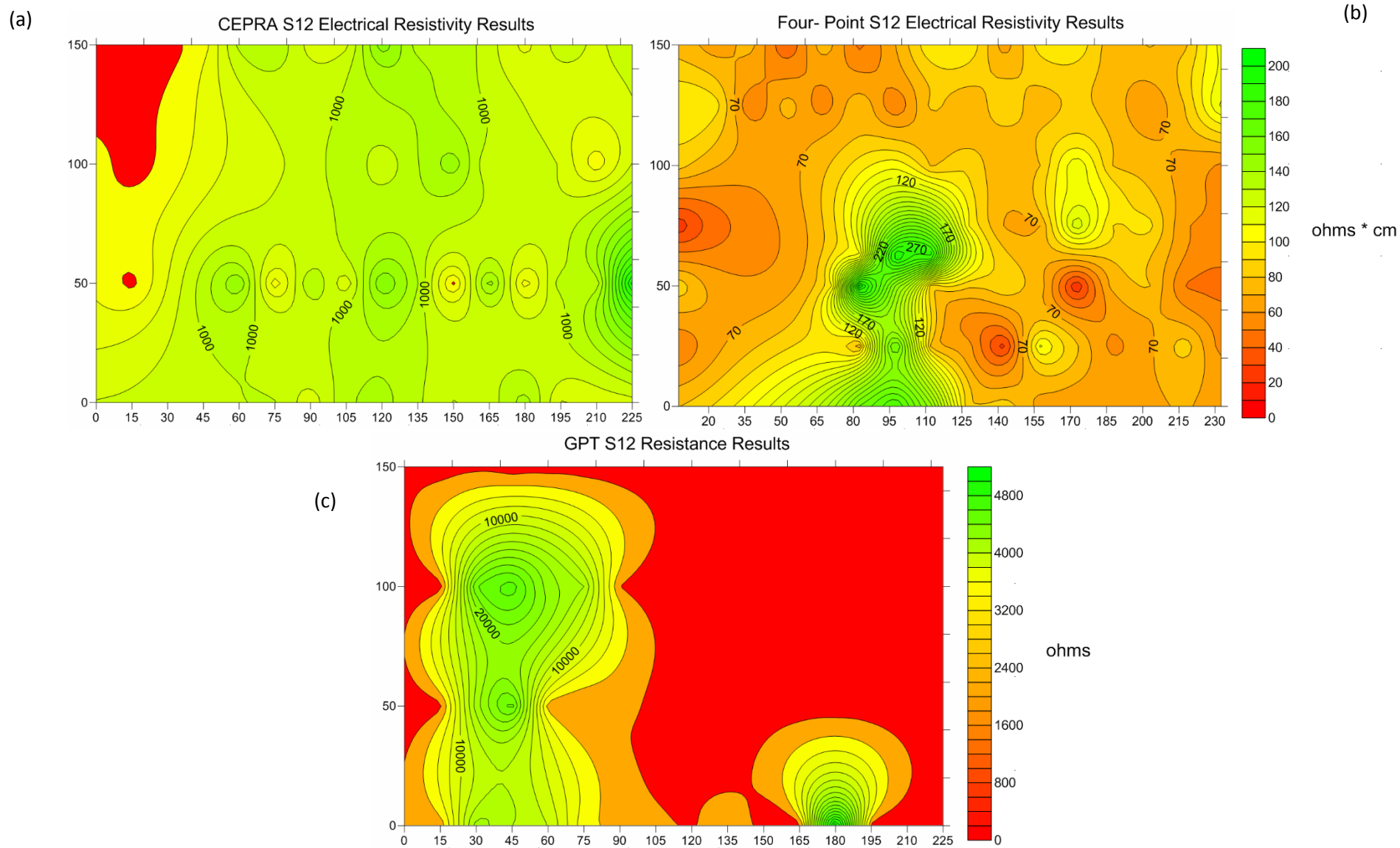


Figure 4.41: S12 Electrical Resistivity and Resistance Results

Figures 4.40 to 4.41 show the contour maps representing the non-destructive results for Slab 12. Similar to Slab 11, approximately 35% of the concrete surface of Slab 12 was also compromised by concrete patch repair and non-concrete decking material. Section 3.3.2 Figure 3.20 is provided in the background for the exact area. This compromised area was representative in the non-destructive testing results by unusually low corrosion rate results and unrealistically high resistance results from GPT within the same area of the slab. In contrast, the CEPRA device recorded realistic corrosion and electrical resistivity results for all portions of the slab. The electrical resistivity results as recorded by CEPRA and Four-Point Wenner Probe are different by one order of magnitude, with CEPRA consistently recording electrical resistivity of 1 k Ω *cm, while the Wenner Probe recorded electrical resistivity closer to 100 Ω *cm. Half-cell potential results consistently hovered within the range of -350 to -450 mV for both GPT and XCell, indicating a 90% and greater probability of corrosion. CEPRA and GPT devices both recorded corrosion rates within the more critical range, seen between grids 0, 0 to grid 45, 150. However, for the remaining portion of the slab surface, corrosion rates differ. CEPRA consistently recorded corrosion rates around 0 μ A/cm² from grid 75,150 to 225, 150, while GPT recorded corrosion rates between 8 to 15 μ A/cm² within the same slab surface.

High corrosion activity, located at gridline 0 and 150, is measured by GPT and CEPRA where corrosion rates are higher than 15 μ A/cm² and 1.0 μ A/cm², respectively. More negative half-cell potential readings by both GPT and CEPRA are also located at gridline 0 and 150. Additionally, electrical resistivity measurements by both Four-Point and CEPRA indicate high risk of corrosion activity, with electrical resistivity below 100 ohms*cm. For gridlines 0 and 150, all devices indicate high level of reinforced concrete deterioration.

Where the surface is compromised, GPT is also to locate and record abnormal measurements, specifically at gridline 15 to 90 and 0 to 100, indicating that the compromised surface of the slab does interfere with the device's ability to record measurements. In contract, CEPRA technology does record measurements for electrical resistivity within this compromised area, greater than 1 Khms*cm. However, electrical resistivity measurements by CEPRA for the entire slab surface are greater than 1 Kohms*cm, indicated CEPRA measurements unable to distinguish concrete surface from non-concrete surface and/or unable to take a reasonable measurement for this slab.

High level of corrosion activity is indicated at gridline 225 and 0 by half-cell potential readings, GPT corrosion rate readings, and Four-Point electrical resistivity readings. In contract, CEPRA, does not indicate this area to be at risk.

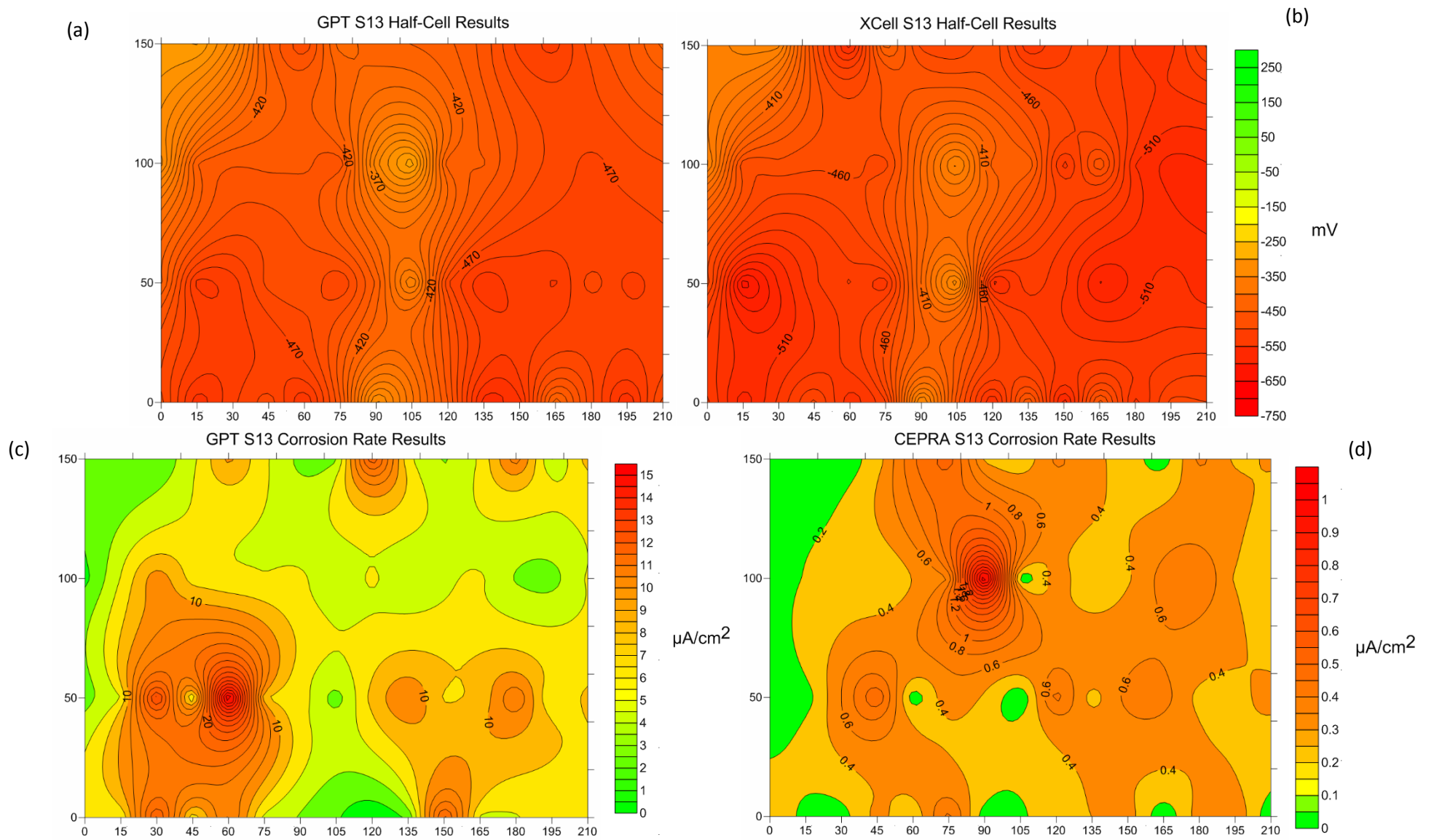
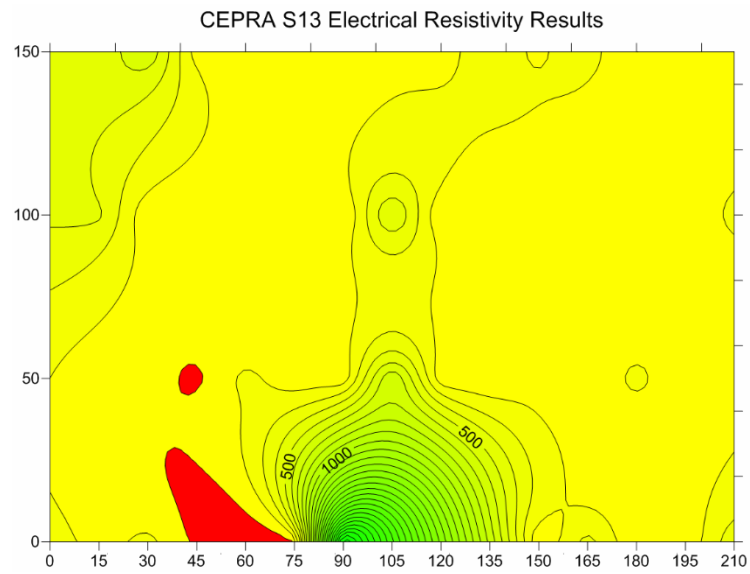
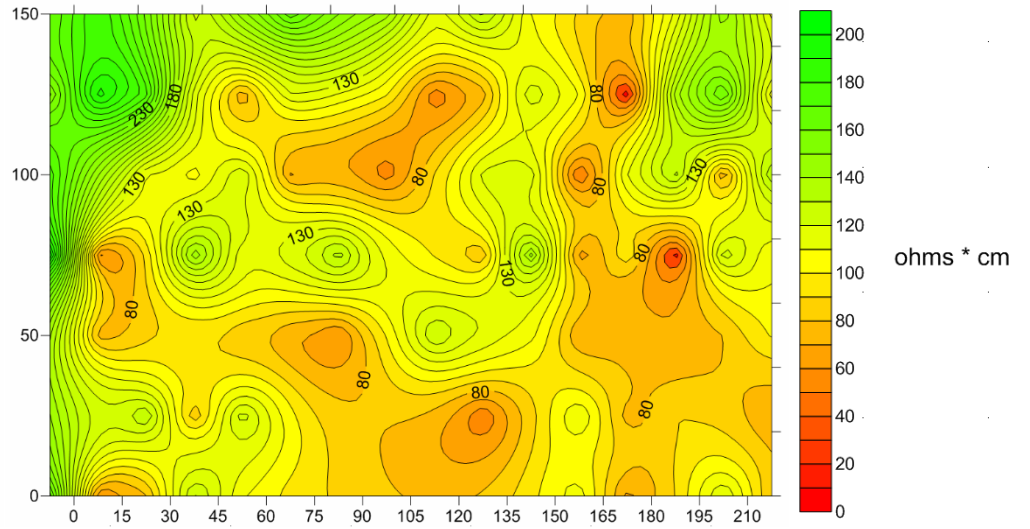


Figure 4.42: S13 Half Cell (a-b) and Corrosion Rate (c-d) Results

(a)

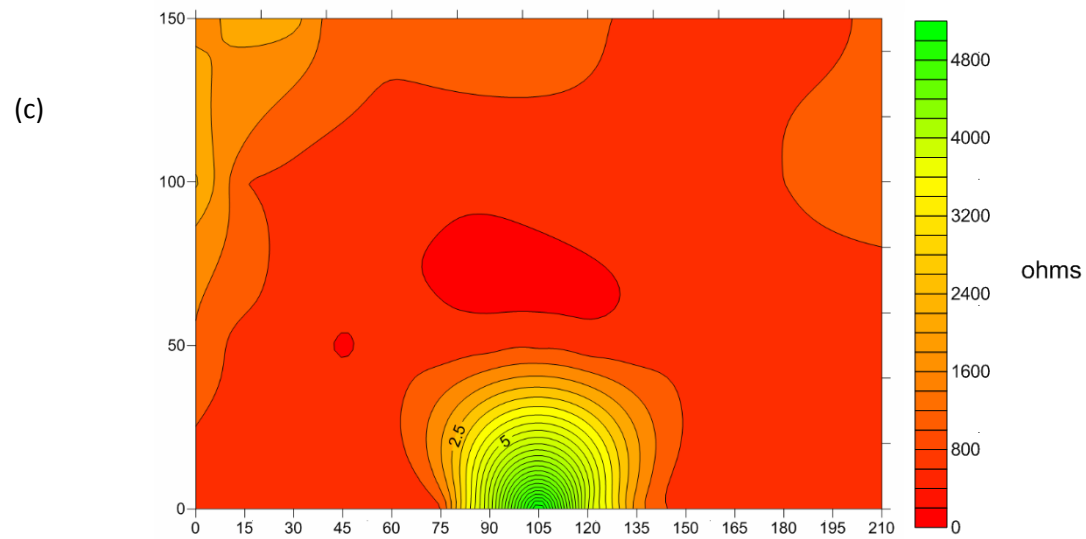


Four-Point S13 Electrical Resistivity Results



(b)

GPT S13 Resistance Results



(c)

Figure 4.43: S13 Electrical Resistivity (a-b) and Resistance (c) Results

Figures 4.42 to 4.43 present the contour maps representing the non-destructive results for Slab 13. Slab 13 has the most favourable slab surface for non-destructive testing, with approximately 10% of the concrete slab surface compromised by concrete patch repair and non-concrete decking material. Refer to shaded areas of contour maps for exact location of compromised areas. This compromised area is more visibly recorded by the CEPRA electrical resistivity and GPT resistance contour plots, seen around grid 105, 25, with data recorded around $1 \text{ k}\Omega\cdot\text{cm}$ and $2 \text{ k}\Omega\cdot\text{cm}$, respectively. The remaining slab surface exhibits consistent results for both CEPRA and GPT. In contrast, the electrical resistivity results as recorded by the Wenner probe recorded results within $40 - 240 \Omega\cdot\text{cm}$, with high variability throughout the slab surface. The corrosion rate results for both the CEPRA and GPT devices displayed high variability throughout the concrete surface with ranges of $0 - 0.8 \mu\text{A}/\text{cm}^2$ and $0 - 20 \mu\text{A}/\text{cm}^2$, respectively. However, the area of the slab surface with relatively high corrosion rates was not consistent for both devices. The half-cell potential results consistently hovered within the range of -350 to -450 mV for both GPT and XCell devices, indicating a 90% and greater probability of corrosion.

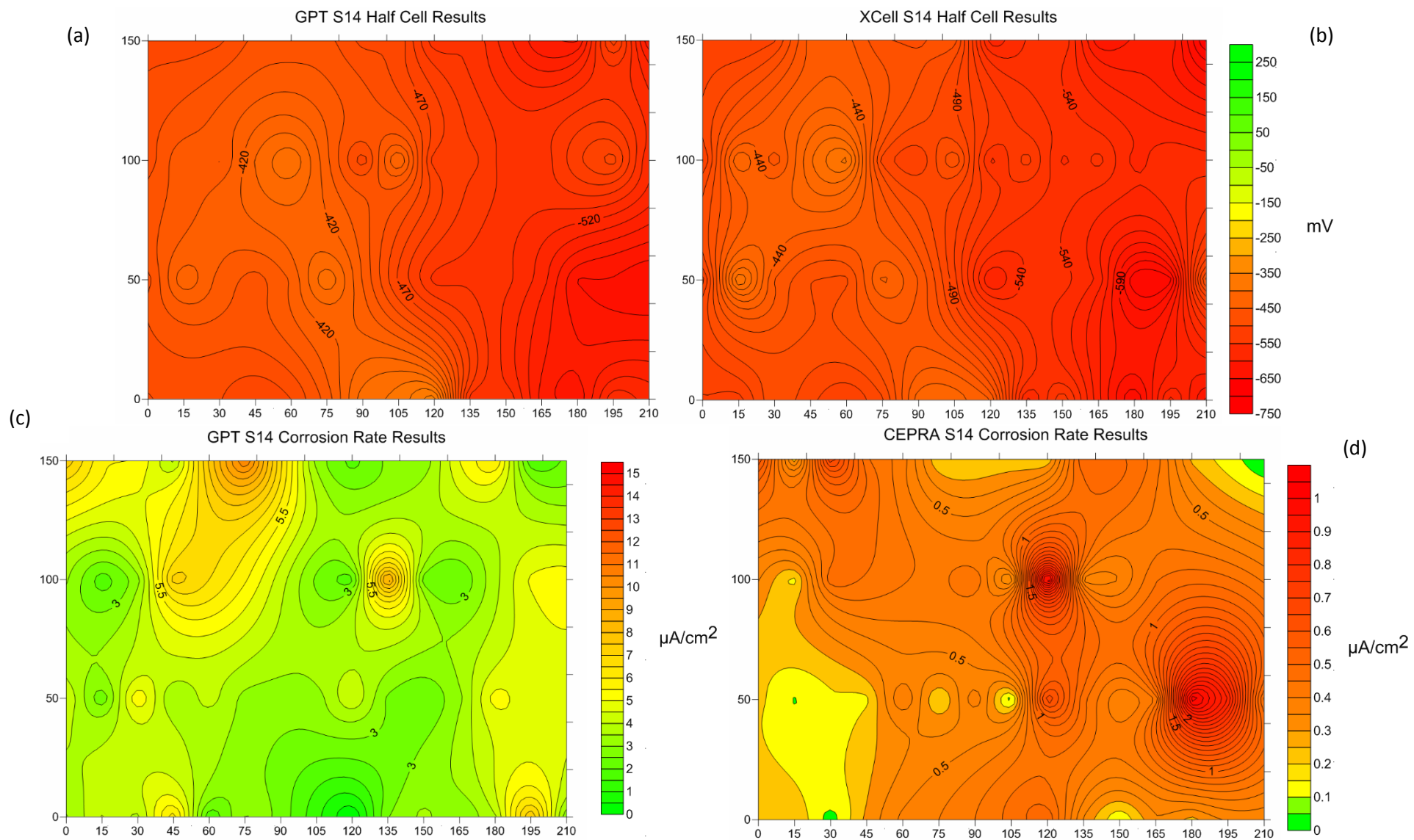


Figure 4.44: S14 Half-Cell (a-b) and Corrosion Rate (c-d) Results

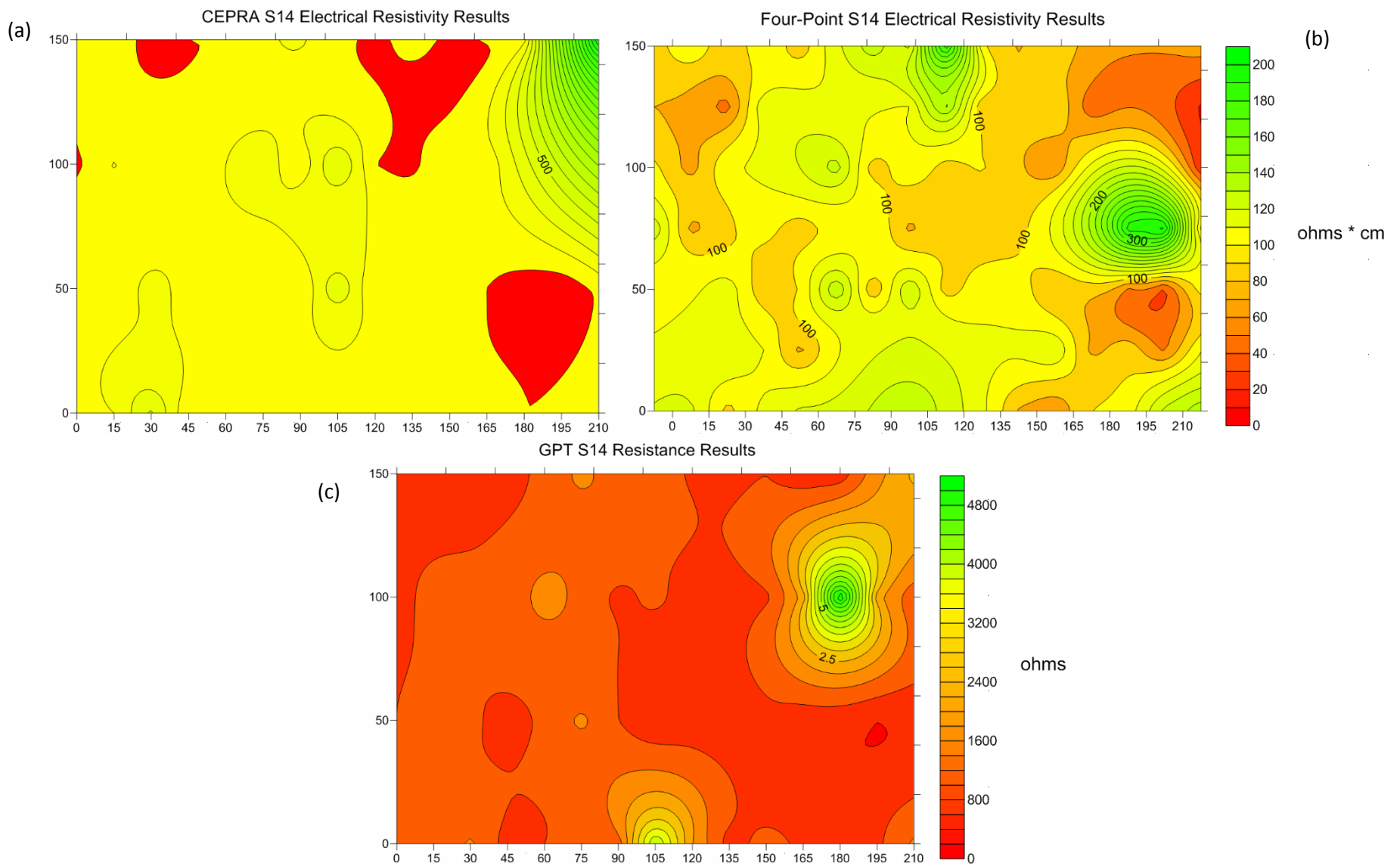


Figure 4.45: S14 Electrical Resistivity (a-b) and Resistance (c) Result

Figures 4.44 to 4.45 show the contour maps representing the non-destructive results for Slab 14. Slab 14 has approximately 25% of the concrete slab surface compromised by concrete patch repair and non-concrete decking material. Refer to background of contour map for exact location of compromised area. This compromised area was recorded by the CEPRA for both corrosion rate and electrical resistivity with $0 \mu\text{A}/\text{cm}^2$ and greater than $1 \text{ k}\Omega\cdot\text{cm}$, respectively. The Wenner Probe was also unable to record results for this compromised area as $0 \text{ k}\Omega\cdot\text{cm}$ is recorded. The electrical resistivity by the Wenner Probe again records variable results for the slab surface, while the CEPRA device records drastically differing results across the slab surface, by one order of magnitude. The corrosion rates by the CEPRA and GPT devices are within $0 - 2 \mu\text{A}/\text{cm}^2$ and $0 - 8 \mu\text{A}/\text{cm}^2$, respectively. However, the areas with higher corrosion rates for CEPRA and GPT are not consistent. The half-cell potential results consistently hover above -420 mV for both GPT and XCell devices, indicating a 90% and greater probability of corrosion.

High levels of corrosion activity indicated at gridline 210 and 0 by both GPT and CEPRA with corrosion rates higher than $6 \mu\text{A}/\text{cm}^2$ and $2 \mu\text{A}/\text{cm}^2$, respectively. Additionally, more negative half-cell potential values are located near gridlines 210 and 0. Although electrical resistivity by CEPRA varies greatly across the slab indicated an error in measurements, CEPRA does indicate high risk of concrete deterioration at gridline 210 and 0 with $0 \text{ ohms}\cdot\text{cm}$ electrical resistivity readings. Four-point probe also picks up this area with electrical resistivity lower than $20 \text{ ohms}\cdot\text{cm}$. Low risk of corrosion activity located at gridline 210 and 150 picked up by CEPRA, Four-Point, and GPT.

The contour maps for Slabs 11, 12, 13, and 14 provide an overall review of the results collected on-site by multiple non-destructive devices: Wenner Probe, XCell, GPT, and CEPRA. The half-cell potential results consistently indicate high probability of corrosion throughout the entire slab surface. The CEPRA and GPT measurements were both affected by the presence of surface obstructions and unable to record clean results. In unobstructed areas, the electrical resistivity results for CEPRA, GPT, and Wenner Probe were recorded. However, results vary greatly between devices. Individually, the CEPRA and GPT devices recorded electrical resistivity results that differ greatly across the slab surface. The Wenner Probe consistently, across all slabs, recorded electrical resistivity results within a low-to-moderate corrosion rate severity range, leading to the impression that Probe was less affected by surface obstructions. The corrosion rate measurements from both CEPRA and GPT typically range from $0 - 2 \mu\text{A}/\text{cm}^2$ and $0 - 8 \mu\text{A}/\text{cm}^2$, respectively, for uncompromised slab surfaces. These ranges both indicate moderate corrosion activity as defined by each individual device. The corrosion rate results were precise across devices and slabs.

The half-cell potential readings were consistently greater than -300 mV across all slabs regardless of compromised slab surface. By the ASTM C876 Standard, this indicates a 90% and greater probability of corrosion. This conclusion by ASTM C876 Standard does not agree with the measurements by GPT, CEPRA, and Four-Point Wenner Probe where these results indicate less of the slab with moderate to high levels of corrosion activity. In conclusion, the industry standard to assess the condition of reinforced concrete structures, ASTM C876, does not distinguish between areas of low to high corrosion activity. Therefore, it is recommended that in-situ condition assessment surveys use a combination of assessment techniques to allow for more meaningful conclusions.

The accuracy of each non-destructive device will be determined by comparison to gravimetric mass loss results, as discussed in Section 4.2.2.

4.2.1.2. Electrical Resistivity

To further the electrical resistivity results presented by the contour maps, the percentage of slab surface area corresponding to electrical resistivity within the low, low-to-moderate, high, and very high corrosion rate severity can be seen in Table 4.5.

Table 4.3: Field Study: Electrical Resistivity Breakdown

Corrosion Rate Severity	Electrical Resistivity ($\Omega \cdot m$)	CEPRA (%)	Four-Point (%)	GPT (%)
low	200	70	0	32
low to moderate	100-200	25	52	70
high	50-100	3	48	1
very high	50	7	0	3

For 70% of the testing locations, the CEPRA technique measured electrical resistivity greater than 200 $\Omega \cdot m$. Additionally, of that 70%, at approximately 55% of those testing locations the measured electrical resistivity was over 1 $k\Omega \cdot m$. Measurements over 1 $k\Omega \cdot m$ were excluded from the analysis as this constitutes a mis-reading (Giatec Scientific Inc., 2016). Furthermore, electrical resistivity by four-point probe indicated that 52% of the slab surface has low to moderate corrosion rate severity while the remaining 48% has high. This contradicts the results by CEPRA and GPT where more slab surface area is within the low to moderate corrosion rate severity. As the percentage of surface area corresponding to

the same level of corrosion rate severity varies greatly by device, a direct comparison of the electrical resistivity by GPT and CEPRA compared to Four-Point at each location highlights the discrepancy for electrical resistivity measurements.

A comparison of electrical resistivity as determined by the Four Point Probe and by the CEPRA and GPT devices is found in Figure 4.46.

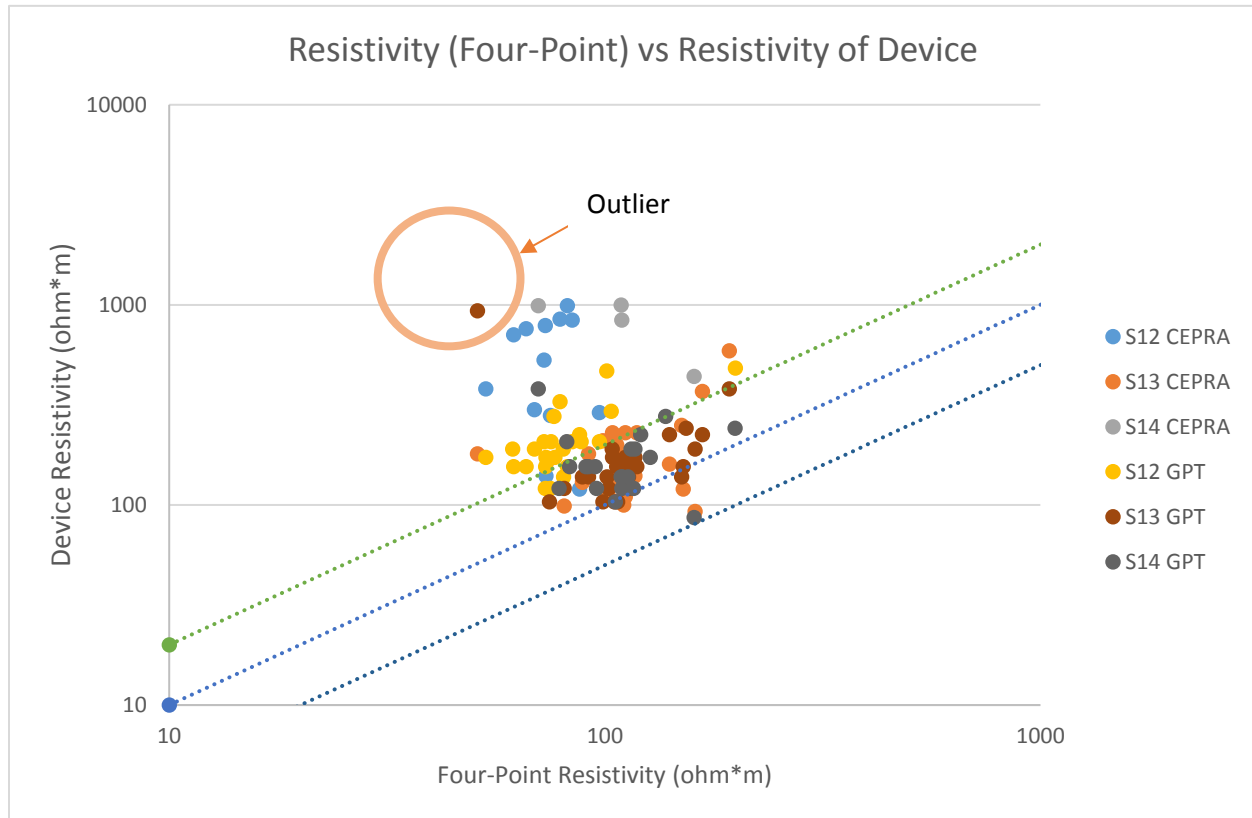


Figure 4.46: Field Study: Resistivity (Four-Point) vs. Resistivity of Device

All devices measure surface electrical resistivity, except for GPT which measures resistance. To allow for comparable data, resistance values are converted to electrical resistivity by using Eq. 2-7, using a geometric factor of $K = (1/0.578)$ (Layssi et al., 2015).

There is a good relationship between electrical resistivity as determined by the Four-Point probe and the GPT device for all slabs, regardless of compromised surface area. S12 by GPT slightly over-estimates electrical resistivity when compares to Four-Point Probe, this may be due to the 35% compromised surface. Additionally, S13 outlier, as noted in Figure 4.46, was removed from analysis. Electrical resistivity measurements taken by the CEPRA equipment for S12 were greatly overestimated over that determined

by the Four-Point probe. It is reasonable to conclude that this overestimation may be due to the fact that 35% of the surface was compromised. In contrast, S13 by CEPRA is in good agreement with that of Four-Point Probe where less than 10% of the surface is compromised. In conclusion, slab surfaces with compromised surfaces result in overestimated electrical resistivity readings. Measurements by GPT are in better agreement with Four-Point Probe than CEPRA.

The relationship between four-point resistivity and the resistivity of that measured by the CEPRA and GPT for S12 is poor. As the surface of S12 contained approximately 35% compromised area, the greatest surface area with slab obstructions (S11 removed from further analysis), this poor relationship can be attributed to the condition of the slab during testing, unlike S13 and S14 where slab obstructions are significantly less than that of S12. The inability for these non-destructive devices to accurately record in-situ measurements when protective membrane or repairs materials are present is a major disadvantage. Most reinforced concrete structures are protected or have been previously repaired.

Factors affecting electrical resistivity are primarily related to the concrete microstructure (w/c, SCMs, and aggregate). It is assumed that the concrete microstructure of the tested concrete is similar for S12, S13, and S14 as these slabs were removed from adjacent locations of the deteriorating bridge. As such, these parameters do not affect the analysis and comparison of S12, S13, and S14 directly. Additionally, the electrical resistivity is affected by environmental factors including temperature and moisture content. These environmental factors were not constant during non-destructive testing where the temperature ranged from a low of 12°C to a high of 23°C and relative humidity ranged from 75-92%. As temperature and electrical resistivity are inversely related (Polder, 2000), it is certain that electrical resistivity measurements were affected by the changing temperature. This analysis does not account for the change in temperature. It is assumed that the change in electrical resistivity is small and the impact negligible.

4.2.2. Destructive Testing

4.2.2.1. *Absorption and Moisture Content*

Concrete cylinders cored from the Cornwall slabs were subjected to testing in accordance to ASTM C642 to determine the absorption, density, and volume of permeable pore space (ASTM C642, 2013). The results from this set of testing can be seen in Table 4.6.

Table 4.4: Cornwall Results for Absorption, Density, and Volume of Permeable Pore Space as per ASTM C642

	S12	S13	S14
Absorption after immersion (%)	5.751	5.548	5.350
Absorption after immersion and boiling (%)	6.744	6.724	6.536
Bulk density, dry (g/cm ³)	2.076	2.021	2.031
Bulk density after immersion (g/cm ³)	2.195	2.133	2.139
Bulk density after immersion and boiling (g/cm ³)	2.216	2.157	2.163
Apparent density (g/cm ³)	2.414	2.339	2.341
Volume of permeable pore space (%)	13.998	13.591	13.272

Using the same concrete cylinders as were used in the testing for density, volume of permeable pore space, and moisture content of the existing concrete at time of in-situ measuring was determined. Where the moisture content, as a percentage, is with respect to the dry weight of the sample. The moisture content was accomplished by removing the concrete specimen from its sealed container, drying it in an oven, and recording the mass after 24 hours of drying. These results can be found in Table 4.7.

Table 4.5: Cornwall Results for Moisture Content on Concrete Samples

	Moisture Content (%)
S12	4.915
S13	4.498
S14	4.694

Unfortunately, this method of determining moisture content at the time of in-situ corrosion rate measurements is inaccurate as the concrete specimens were removed with wet coring. Dry coring was attempted but it was unsuccessful as the concrete broke during coring.

Concrete properties can be strongly correlated to half-cell potential and electrical resistivity measurements. A more appropriate concrete property to determine would be the rate of absorption or sorptivity by ASTM C1585 (2013), which measures the increase in the mass of a specimen resulting from absorption of water as a function of time when only one surface of the specimen is exposed to water (ASTM C1585, 2013). The specimen is conditioned in an environment at a standard relative humidity to induce a consistent moisture condition in the capillary pore system. During testing following ASTM C1585, the exposed surface of the specimen is immersed in water and water ingress of unsaturated concrete is dominated by capillary suction during initial contact with water (ASTM C1585, 2013). Unfortunately, this experimental program did not test the concrete samples to determine rate of absorption. This is a shortcoming of the experimental and is recommended for in future programs of similar nature.

In-situ relative humidity at the concrete surface was measured by CEPRA, ranging from 38-65%. The atmospheric relative humidity as recorded by Environment Canada for Cornwall was 75-92% at the date and time of in-situ measurements. While Environment Canada's historical data is for the city of Cornwall in general, this information provides some level of validation. CEPRA utilizes the relative humidity measured at the surface of the concrete within the algorithm to calculate instantaneous corrosion rate. As such, the difference between relative humidity measured by CEPRA and that by Environment Canada indicates that there may be an error in CEPRA's ability measure relative humidity.

4.2.2.2. Compression, Carbonation, and Chloride Penetration Depth

To conduct rupture testing on the concrete cylinders cored from the Cornwall slabs following CSA A23.2 (2009), the concrete cylinders needed to have a diameter of 95 ± 5 mm and a length of 200 ± 5 mm. Unfortunately, several cylinders broke apart during the coring and/or transportation process. A cylinder meeting the required dimensions was not available for S13. Therefore, two rupture tests were conducted for S12. The compressive strength in MPa along with several measurements as required by the CSA test are found in Table 4.8.

Table 4.6: Field Stage: Compressive Strength Results

Testing Date: September 15, 2016							
	Diameter (mm)	Length (mm)	L/D	Weight (g)	Area (mm ²)	Applied Load (kN)	Comp. Strength (MPa)
S12	95.0	200.0	2.1	3201.0	7.1	253.6	35.8
S12-1	95.0	200.0	2.1	3210.5	7.1	225.3	31.8
S14	95.0	200.0	2.1	3296.0	7.1	263.2	37.2

All cylinders met the length-to-diameter ratio between 1.8 and 2.2, therefore no correction factor was required. All three cylinders ruptured following the Failure Type 3 as seen in Figure 4.47.

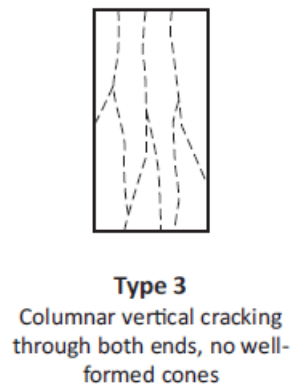


Figure 4.47: Failure Type 3 (A23.1-04/A23.2-04, 2004)

For concrete that was poured in 1956, it is expected that the intended concrete strength ranged from 21 -25 MPa for bridge construction. The compressive testing results indicated concrete has aged well, gradually gaining strength during the lifespan of the structure.

Figure 4.49 shows the split core after compression testing was conducted. These freshly split samples were exposed to a spray of the phenolphthalein indicator (left sample) and silver nitrate spray (right sample).



Figure 4.48: Split Cylinders Tested for Carbonation (left) and Chloride Penetration Depth (right)

Phenolphthalein colours the concrete pink when the pH in the concrete is greater than 8.5, the phenolphthalein test, as shown in Figure 4.48, indicated a drop in pH at approximately 20 mm from the underside of the slab. This indicates that the concrete is still alkaline and not a suitable environment for chloride-induced corrosion at the reinforcement level. The carbonation depth (approximately 20 mm) is not greater than the concrete cover (52 mm, as per original structural drawings) to reinforcement, indicating that the corrosion mechanism of reinforcement is not related to carbonation.

Silver nitrate spray, in contact with chlorides in concrete, will colour the concrete grey-white. As shown in Figure 4.48, chlorides are detected to an approximate depth of 50 mm from the top of the slab for one specimen. The result of this test indicates the corrosion mechanism is related to chloride-induced depassivation of the concrete reinforcement. Chloride content testing was conducted to verify these findings.

4.2.2.3. Chloride Content

The results from the potentiometric titration testing on the concrete powder samples retrieved from the North Channel Bridge slabs are presented here with the intent to observe chloride content at the depth of reinforcement. The total chloride content is given in percent by weight of concrete sample and was calculated according to the ASTM C1152/ C1152 M-04 standard. As discussed previously, powder samples were retrieved at 4-5 mm intervals. As the concrete powder represents a sample for that 4-5 mm interval, results represent an average value with the midpoint depth for each sampled interval. A complete list of all the results obtained during the titration process is found in Appendix G.

A European guideline for existing structures, which shows the corrosion risk given for different ranges of percent chloride either by mass of cement or concrete, is seen in Table 4.9 (Broomfield J. , 2007). These guidelines provide for an indication of the condition of the reinforced concrete slabs at the time when concrete samples were retrieved.

Table 4.7: Corrosion Risk based on Chloride Content (Broomfield J. , 2007)

Chloride by mass of Cement (%)	Chloride by mass of sample (concrete) (%)	Risk
< 0.2	< 0.03	Negligible
0.2-0.4	0.03-0.06	Low
0.4-1.0	0.06-0.14	Moderate
> 1.0	> 0.14	High

The results from the potentiometric titration testing can be seen in Figure 4.49. As can be seen, the risk of corrosion for both S12 and S13 falls within the “high” range, while the risk for S14 falls within the “moderate risk”.

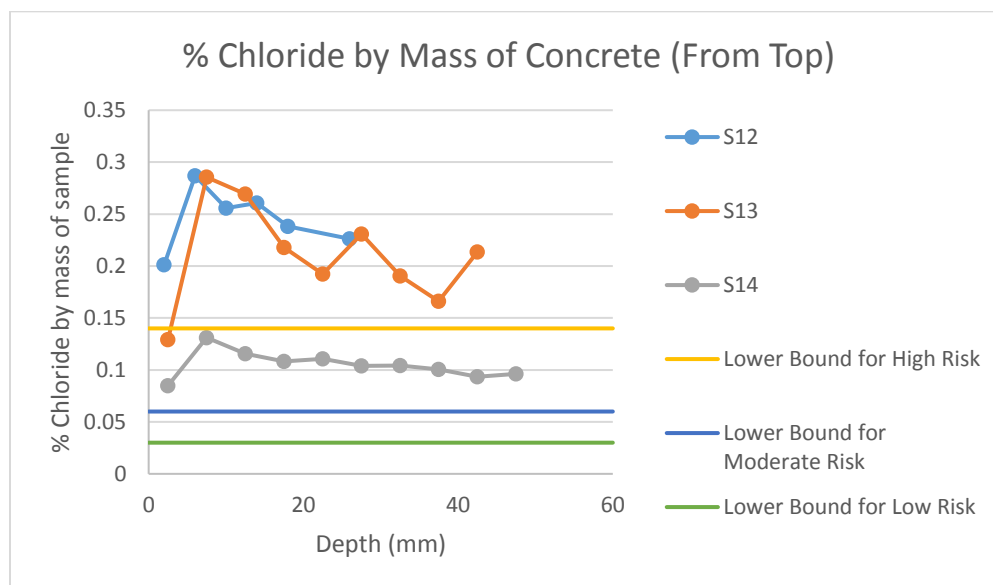


Figure 4.49: Chloride Concentration Profile from Top of Slab

It is expected to observe lower levels of chloride content at the surface of the concrete slabs, approximately 5 mm from the top surface, due to “wash out” effects. As the concrete surface is subjected to wet and dry cycles, de-icing salts are “washed out” from the concrete surface.

In addition, powder samples were also collected from the underside of S13 only as there was enough of a concrete core from such additional testing. These results are seen in Figure 4.50.

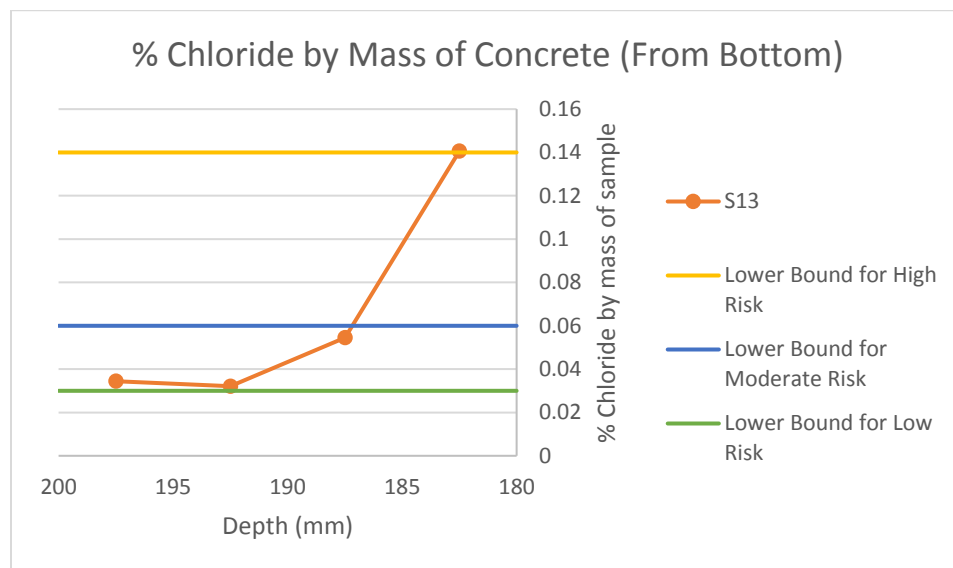


Figure 4.50: Chloride Concentration Profile from Bottom of Slab

All three chloride profiles in Figure 4.50 exhibit a similar trend where the maximum chloride content can be found at a depth of approximately 8 mm from the top of the concrete surface. Following this maximum point, the chloride content decreases as the penetration depth increases. Higher chloride content can be seen from samples collected from slabs S12 and S13, while S14 exhibits approximately 50% the chloride content. However, all three slab samples show a moderate to high risk of corrosion. The underside of S13 also exhibits moderate risk of corrosion, where the maximum chloride content of 0.14 % can be seen at a depth of 185 mm, as illustrated in Figure 4.51. Based on these test results, corrosion of the reinforcement is expected.

4.2.2.4. Gravimetric Mass Loss

The 20M vertical reinforcement retrieved from the North Channel Bridge Slabs S12, S13, and S14 was brought back to the laboratory for gravimetric mass loss testing. Prior to testing, each rebar specimen was visually inspected. It is important to note that not all locations where non-destructive testing measurements were taken along the reinforcement were used for gravimetric testing. Only locations with a good non-destructive measurement and visibly corroded reinforcement were used. Out of these

locations, six reinforcement samples were removed from the analysis due to defects on the reinforcement occurring during construction or removal for testing. Photographs of the top and bottom of the rebars were taken to determine the corroded surface area for each specimen. Refer to Appendix H. Additionally, the length and weight of each reinforcement specimen were recorded. Note that the original weight of the reinforcing bars were not available.

Immediately following specimen processing, gravimetric testing was conducted according to ASTM G1-03 (2011), similar to that conducted in the Laboratory Stage (ASTM G1, 2011). A total of six cycles was needed to return mass loss within 5% difference from the previous cycle. For the majority of the specimens, 80% mass loss occurred during the first cycle. The theoretical mass loss for each specimen was adjusted by the mass loss of four control specimens retrieved from S12, where no trace of corrosion was visibly present. From the results of the gravimetric testing, the corrosion rate based on Faraday's Second Law, Eq. 2-12, was determined and compared to the corrosion rate measured by CEPRA and GPT at the corresponding locations. Adjustments to the gravimetric corrosion rate were made to account for reinforcement surface area with corrosion products. Similar to the Laboratory Study, the *ImageJ* software was utilized. The percentage of surface area with corrosion products and with mortar was determined. Refer to Appendix J for the trend data from gravimetric mass loss testing and the results of *ImageJ*. Additionally, four control reinforcing bars were subjected to the same acid bath cycle to account for the mass loss of the mill scale in the mass loss for the corroded reinforcing bars.

A breakdown of the corrosion activity ranges for each device can be seen in Tables 4.10 and 4.11, these values represent the number of measurements within the corrosion rate range over the total number of measurements taken for the slab. Refer to Tables 2.4 and 2.5 for the typical corrosion rate ranges provided by GalvaPulse and iCOR manuals, respectively (Giatec Scientific Inc., 2016; Germann Instruments, 2011). These ranges are used in Tables 4.10 and 4.11 to gauge the level of corrosion activity.

Table 4.8: Corrosion Activity by iCOR (%)

	Corrosion Rate ($\mu\text{A}/\text{cm}^2$) (Giatec Scientific Inc., 2016)			
	<0.1 (Passive)	0.1-0.3 (Moderate)	0.3-1 (High)	>1 (Severe)
S12	66.67	19.05	14.29	0.00
S13	13.64	13.64	72.73	0.00
S14	12.50	31.25	43.75	12.50

Table 4.9: Corrosion Activity by GalvaPulse (%)

	Corrosion Rate ($\mu\text{A}/\text{cm}^2$) (Germann Instruments, 2011)				
	<0.5 (Passive)	0.5-2 (Negligible)	2 – 5 (Low Corrosion)	5 – 15 (Moderate)	>15 (High)
S12	0.00	0.00	9.52	76.19	14.29
S13	0.00	4.55	18.18	68.18	9.09
S14	0.00	0.00	75.00	25.00	0.00

Based on the corrosion activity ranges by CEPRA, over 60% of S12 testing locations are in a “passive” corrosion state, while between 40-72% of S13 and S14, respectively, have “high” corrosion activity. Corrosion activity ranges by GPT indicate over 65% of testing locations have “moderate” corrosion activity for S12 and S13, while S14 has 75% low corrosion activity. Based on these results, it is anticipated that reinforcement specimens will provide enough corrosion products to conduct a successful mass loss test and allow for reasonable comparison between gravimetric and instantaneous corrosion rate by non-destructive testing.

The actual corrosion rate obtained using the ASTM G1 mass loss procedure compared to the instantaneous corrosion rate as determined by non-destructive devices CEPRA and GPT is seen in Figure 4.52. The two lines besides the equality line indicate the upper and lower limit when considering a tolerance factor of 2; this is commonly used when comparing electrochemical techniques (Luping, 2002).

To account for the mass loss due to non-corrosion based products such as the mill scale, the theoretical weights of each corroded reinforcing bar was adjusted. This was accomplished by subjecting the control reinforcing bars to the same acid bath cycle, recording the final mass, and determining the density of the control specimens based on the measured length, resulting in the weight per linear distance. Based on the measured length for each corroded reinforcing bar, the modified theoretical weight was determined and used to calculate the total mass loss. It is important to note that even though the control bars were selected based on the fact that no corrosion products were visible, there most likely was some amount of corrosion on the control bars. This is a possible source of error.

To address the inconsistent polarization areas used by both non-destructive devices and the length of specimen used for gravimetric testing, the instantaneous corrosion rates were adjusted. CEPRA intends to polarize 95 cm² of lateral surface area. GPT intends to polarize 7 cm of reinforcement length. Gravimetric testing was conducted on reinforcement 10 cm in length. Therefore, all corrosion rates were adjusted to a comparable polarization area. This was accomplished by multiplying the in-situ corrosion rate by the ratio of polarization area as used by each device and lateral surface area of each specimen used to determine gravimetric mass loss. Additionally, the gravimetric corrosion rate was adjusted for the percentage of corroded surface area as determined with *ImageJ* software. The average corroded surface area of the reinforcing specimens is approximately 27 cm², while the minimum and maximum corroded areas are approximately 8 and 58 cm², respectively.

As illustrated in Figure 4.51, the in-situ corrosion rate by GPT S14 compared to the gravimetric corrosion rate fell within the upper and lower 2.0 tolerance factor limit. The comparison of corrosion rates for GPT S13 and S12 falls along the bounds of the upper limit and for some data points, exceeds the upper bounds. This indicated that in-situ measurements by GPT overestimate the instantaneous corrosion rate in comparison to gravimetric mass loss. With an increase in corrosion rate by GPT, the ratio of gravimetric to in-situ measured corrosion rate converges closer to 1. This is illustrated in Figure 4.51 with the slope of the linear regression line moving from above the upper bounds to within the upper and lower 2.0 tolerance for S12, S13, and S14, in that order. The slope of the linear regression trendlines for GPT S12, S13, and S14 are 0.7522, 0.2925, and 0.3226, respectively.

Illustrated in Figure 4.51, the in-situ corrosion rate by CEPRA compared to the gravimetric corrosion rate fell below the bounds of the lower limit tolerance factor for S12, S13, and S14. The slope of the linear regression trendlines for S12, S13, and S14 are 0.0986, 0.0409, and 0.3983 respectively. Both the locations of data points in relation to the tolerance factor and the slope of the linear trendlines that there is a weak

relationship between gravimetric and in-situ measurements by CEPRA. However, the linear regression trendlines are misleading as the data points are scattered. Ultimately, CEPRA drastically underestimates the in-situ instantaneous corrosion rate with respect to the tolerance factor.

The strongest correlation is made by CEPRA for S14 with a coefficient of determination of 0.32. The strongest correlation is made by GPT for S12 with a coefficient of 0.34. Overall, these coefficients of determination show a weak to moderate relationship (Moore, Notz, & Flinher, 2013). For S13, either by CEPRA or GPT, there is little to no correlation. Coefficients of determination are tabulated in Table 4.9.

Table 4.10: Field Study Coefficient of Determination (R^2)

	S12	S13	S14
CEPRA	0.25	0.07	0.32
GPT	0.34	0.14	0.24

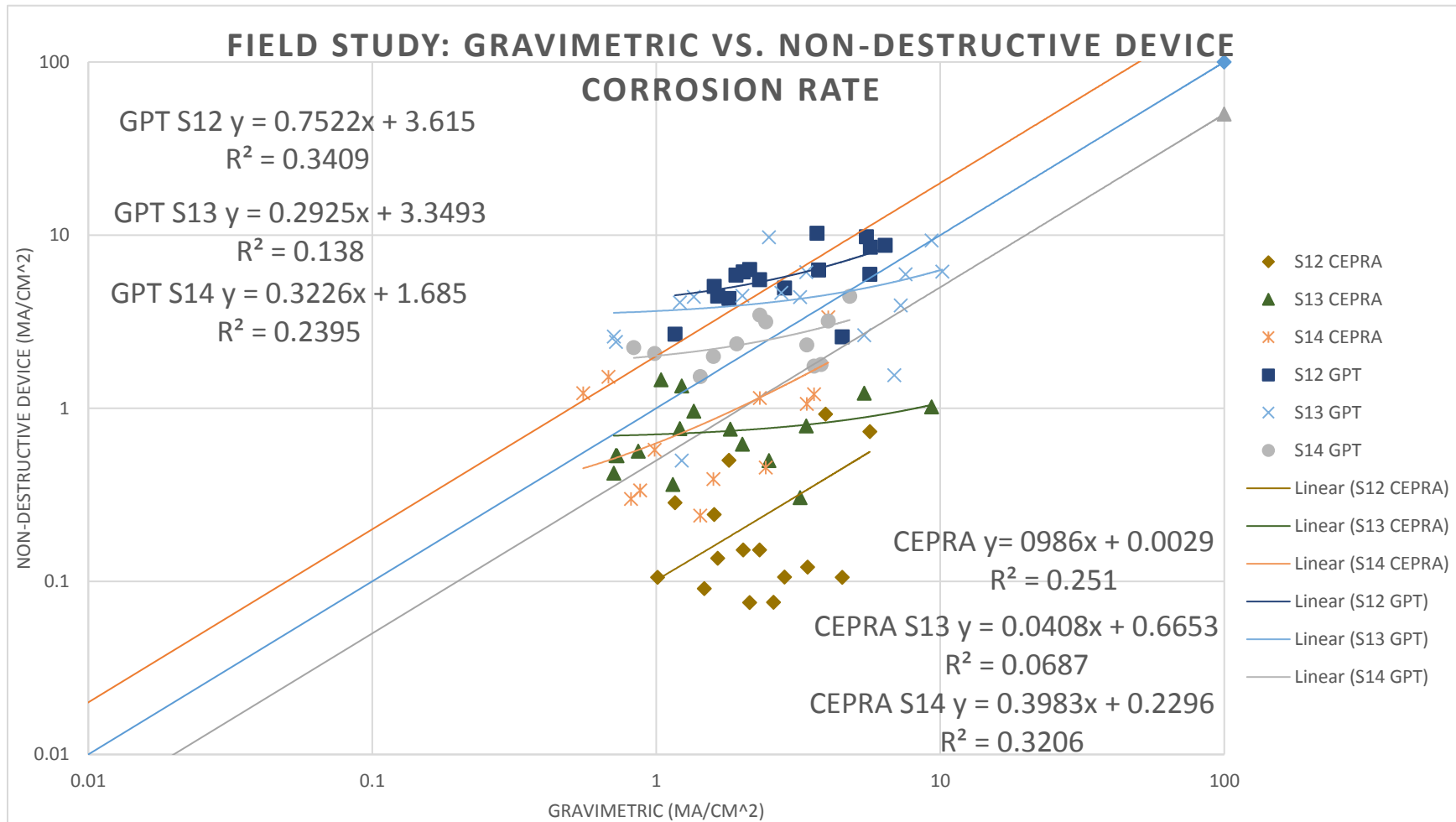


Figure 4.51: Gravimetric Corrosion Rate vs. Instantaneous Corrosion Rate by Non-Destructive Devices

The ratio of gravimetric and instantaneous corrosion rate as an average are presented in Figure 4.52 and 4.53 for CEPRA and GPT, respectively. Additionally, the standard deviations are presented. Outliers falling outside the upper and lower quartile were removed from the analysis. For each slab, approximately 20% of the CEPRA and 25% of the GalvaPulse data points were removed by this process.

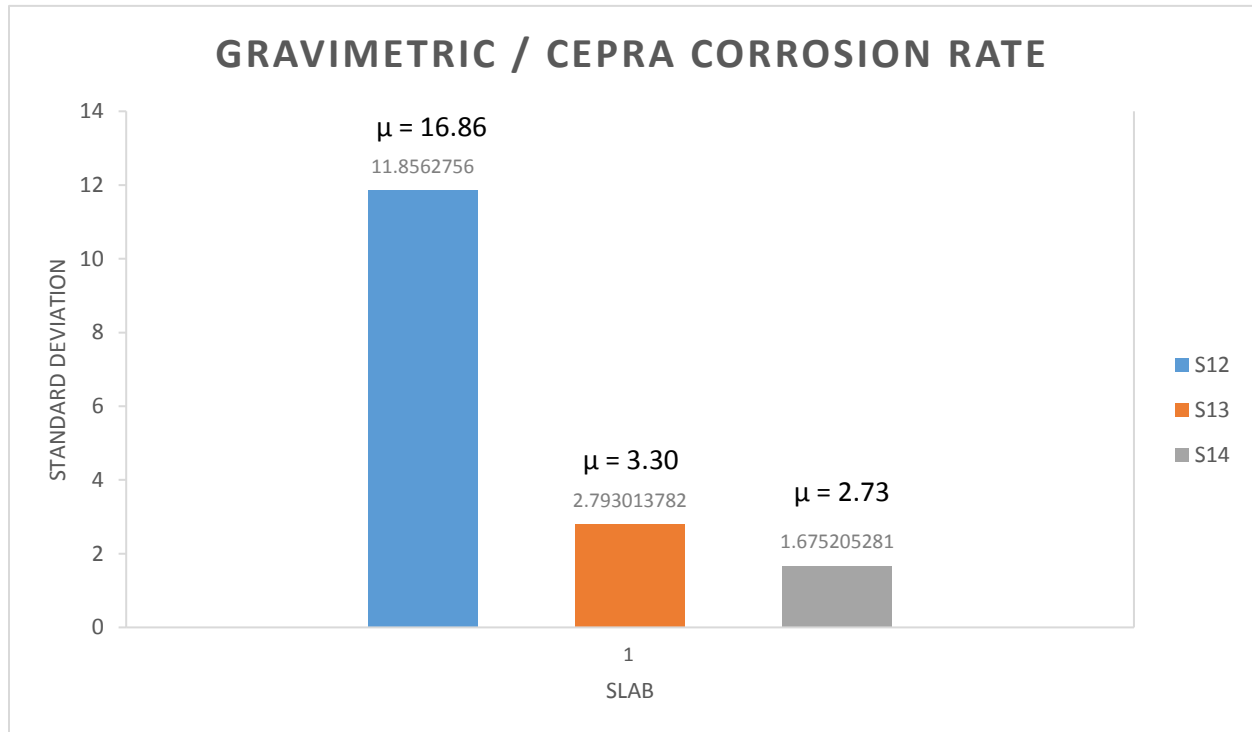


Figure 4.52: Ratio of Gravimetric and CEPRA Instantaneous Corrosion Rate Average and Standard Deviation

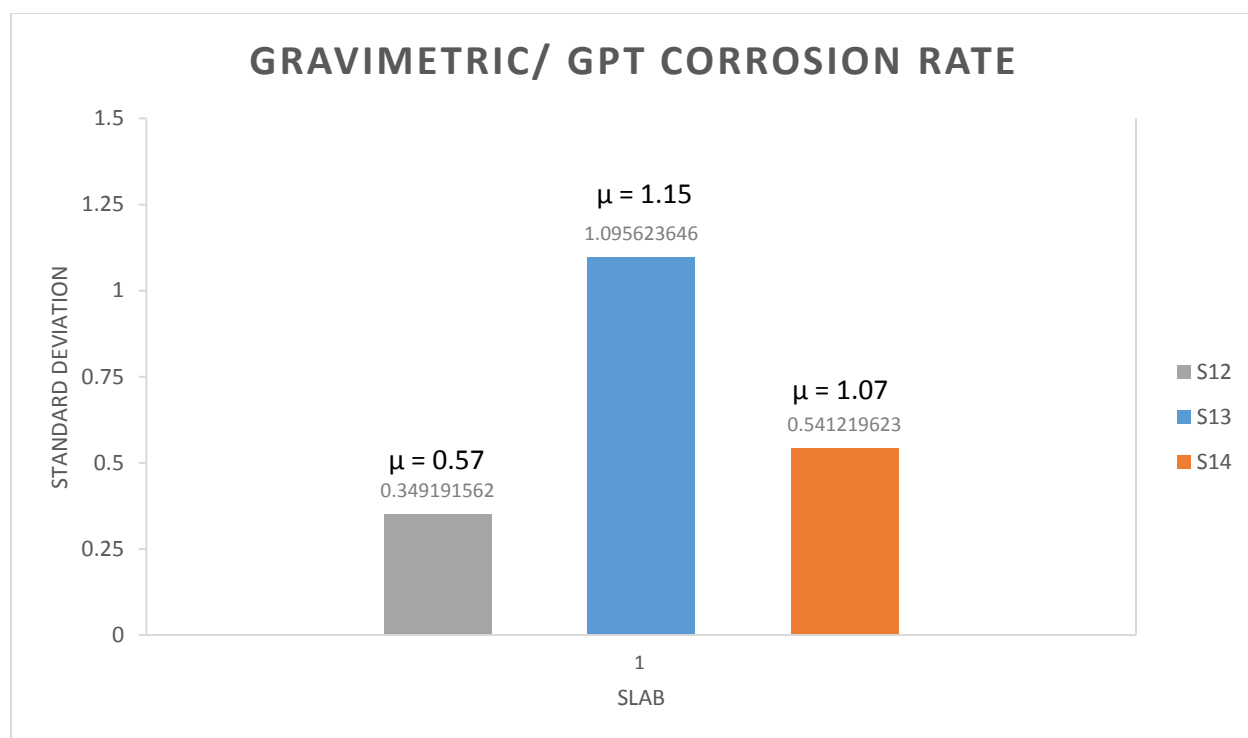


Figure 4.53: Ratio of Gravimetric and GPT Instantaneous Corrosion Rate Average and Standard Deviation

Based on the presented results, the instantaneous corrosion rate as determined by the GPT device is in good agreement with the actual corrosion rate as determined by gravimetric mass loss. The average ratio of gravimetric and instantaneous corrosion rates is 0.57, 1.15, and 1.07 for S12, S13, and S14, respectively. This conclusion is in agreement with findings of Vedalakshmi et al., (2010) who concluded that the use of GPT in the field compared well to mass loss testing. Additionally, the findings of (Frolund, Jensen, & Bassler, 2002) also showed good correlation between the use of GPT in the field and mass loss testing for deteriorating concrete structures. Similar to Frolund, Jensen, & Bassler (2002), the GPT confinement area was comparable to the actual corroding area.

Instantaneous corrosion rate as determined by the CEPRA technology generally underestimated the corrosion rate when compared to the actual corrosion rate. The average ratio of gravimetric and instantaneous corrosion rates is 16.86, 3.30, and 2.73 for S12, S13, and S14, respectively. CEPRA underestimates the instantaneous corrosion rate with ratios greater than GPT for all slabs.

Both non-destructive testing by CEPRA and GPT for the S12 slab presented the greatest deviation from the actual corrosion rate. This deviation is the result of a concrete slab surface with approximately 35% compromised surface area due to concrete patch repair and non-concrete decking material. This

compromised surface results interrupted current and abnormally high concrete resistance leading to unrealistic or great deviations from the actual corrosion rate (Germann Instruments, 2011).

Factors affecting corrosion rate like temperature and relative humidity varied during in-situ testing; this analysis does not address these factors directly. However, testing conditions were sunny with temperatures ranging from 12-23°C and relative humidity ranging from 75-92% for the Cornwall region as per historical data by Environment Canada. The recorded temperature and relative humidity provided for ideal conditions to measure corrosion rate with the deteriorated concrete. Increased temperatures allow for ions within the concrete microstructure to be more mobile (Tuutti, 1982). In-situ measurements were taken from S12 to S14, in that order, starting early in the day and ending around midday. As previously indicated, the temperature slowly increased from 12°C to 23°C. This significant change in environmental conditions may have led to the improving relationship between in-situ and gravimetric corrosion rate for GPT. Additionally, this may explain the CEPRA data points moving closer to the lower bound as S14 measurements were taken midday. Maximum corrosion rates can be found for relative humidity in the atmosphere of approximately 95% (Bertolini L. , Steel Corrosion and Service Life of Reinforced Concrete Structures, 2008). Both temperature and relative humidity provided environmental conditions where the most critical corrosion rate would be determined by non-destructive testing.

4.3. Laboratory and Field Comparison

An overlay of the gravimetric versus NDT corrosion rates for both laboratory and field study results is provided in Figure 4.54 and Figure 4.55 for CEPRA and GPT, respectively. This analysis attempts to find a correlation between laboratory and field study. As previously mentioned, in-situ measurements for the field study were not taken multiple times during varying environment conditions.

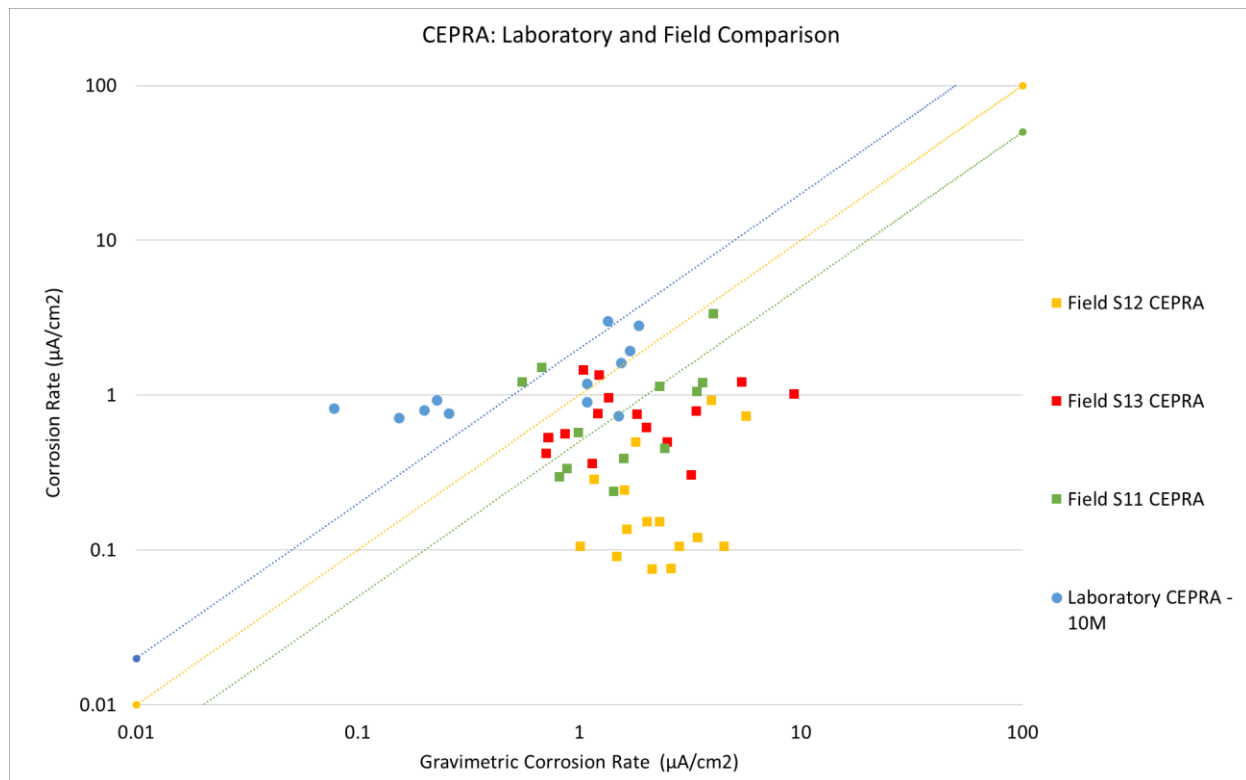
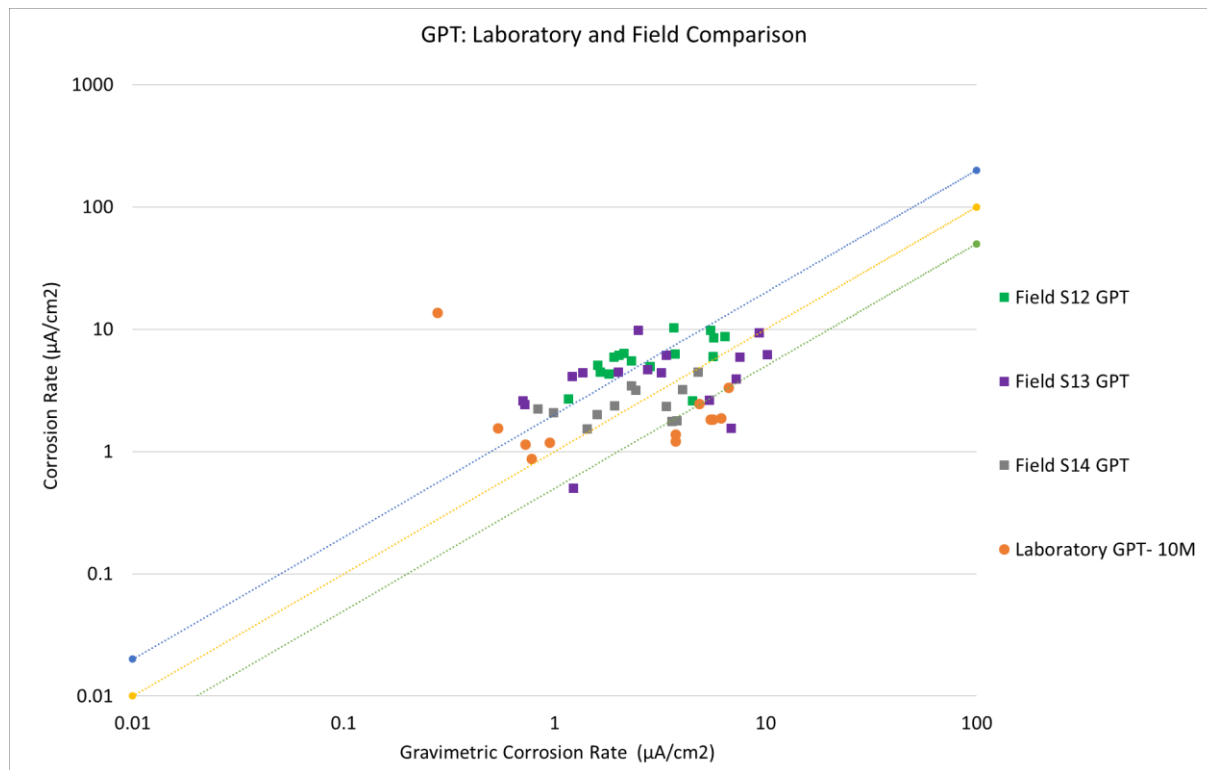


Figure 4.54: Comparison of Laboratory and Field, by CEPRA

The correlation coefficient for the combination of data is 0.11. An important factor to understand, as field NDT occurred once, this data set represent the instantaneous corrosion rate for that in-situ environmental condition. It is preferable to monitor an existing structure by conducting condition assessment surveys multiple times over the span of several seasons. As such, this field study does not accurately represent the changing environmental conditions seen by the Channel Bridge. Therefore, this data set cannot be integrated over the multiple testing periods to allow for the determination of average in-situ corrosion rate, and ultimately compared to the gravimetric corrosion rate, also an average corrosion rate. An additional hindrance to this comparative analysis, the concrete w/cm ratio, concrete cover, reinforcing bar diameter, and chloride content vary across Laboratory and Field Study. Finding common parameters to compare is not possible. As such, the comparative analysis is difficult to quantify, leading to a correlation coefficient of 0.11 which represents little to no strength in relationship.



This gravimetric corrosion rate is plotted against the corrosion rate by NDT. Similarly, the field study by with GPT did not occur multiple times are varying environmental conditions. As such, this comparison is not truly analyzing comparable items. However, it is valuable to note that the comparison by GPT shows a good relationship between gravimetric and instantaneous corrosion rate, 55% data points falling within the tolerance factor with the remaining lining up along the upper tolerance factor bound. The correlation coefficient is for this set of GPT data is 0.4. This is considerable better than the correlation coefficient for CEPRA.

Overall, there is a good relationship between gravimetric corrosion rate and corrosion rate by GPT whereas there is a poorer relationship by CEPRA.

5.0 Concluding Remarks

The purpose of this thesis is to conduct a comprehensive comparative analysis of corrosion rate as determined by non-destructive field-applicable techniques and corrosion rate as determined by destructive gravimetric testing as per ASTM G-1. This is accomplished by a two-part experimental program: (1) Laboratory Study to monitor the deterioration of reinforced concrete slabs subjected to accelerate deterioration in a controlled environment and (2) Field Study based on a condition assessment on the North Channel Bridge, an existing reinforced concrete structure located in Cornwall, ON.

The research presented here utilizes well-known techniques to measure the instantaneous corrosion rate: Galvanostatic Pulse Technique (GPT), as measured by Germann Instrument's GalvaPulse, Linear Polarization Technique (LPR), as measured by James Instrument's Gecor 6, Connectionless Electrical Pulse Response Analysis (CEPRA), as measured by Giatec Scientific's iCOR, and finally, four-point electrical resistivity, as measured by Proceq's Resipod Resistivity Meter.

For both Laboratory and Field Studies, the above-mentioned electrochemical techniques recorded instantaneous corrosion rate. For a comparison to gravimetric corrosion rate, average values were obtained from integrating instantaneous corrosion rate values over time. Reinforcing specimens were removed and subjected to mass loss testing by ASTM G1, resulting in gravimetric corrosion rate data. Comparative analysis on average corrosion rate and gravimetric corrosion rate was conducted. Additionally, several destructive laboratory tests were utilized to provide supporting information on the reinforced concrete: moisture content and absorption, compression, mass loss, carbonation, and chloride content.

Additionally, for both Laboratory and Field Studies, the electrical resistivity was recorded by CEPRA, GPT, LPR, and Four-Point Probe. Comparative analysis was conducted to validate these measurements. The relationship between electrical resistivity and corrosion rate was validated by previous published research.

5.1. Conclusions

While the use of ASTM C876 for the condition assessment of reinforced concrete structures is widely accepted and used in the industry, an overarching theme highlighted in both the field and laboratory studies is that this standard does not accurately represent the condition of the structure. It is

recommended to complement half-cell potential assessments with other non-destructive techniques that measure corrosion rate and/or electrical resistivity such as LPR, GPT, or CEPRA. Additionally, this study has concluded that use of GPT, CEPRA, and electrical resistivity for the assessment of reinforced concrete structures provides for a more accurate assessment. Furthermore, to ensure accurate assessment, the user of these techniques should understand the impacts of in-situ and environmental conditions on corrosion rate and electrical resistivity measurements and adjust the assessment accordingly.

Items specific to each study are presented:

Field Study:

- This experimental study revealed that the instantaneous corrosion rate as determined by GPT is in good agreement with the actual corrosion rate as determined by gravimetric means for non-controlled conditions, whereas CEPRA underestimates the instantaneous corrosion rate.
- The CEPRA technology is able to successfully measure the instantaneous corrosion rate without the need of connection to the reinforcement. However, the instantaneous corrosion rate as determined by the CEPRA technology generally underestimated the instantaneous corrosion rate for field applications as compared to the average corrosion rate obtained from gravimetric mass loss.
- As the initial weight of the reinforcing was not available, there is a strong possibility that gravimetric testing inaccurately represented the mass loss of corrosion products.
- Field study compared the instantaneous corrosion rate, as determined by NDT, with the average corrosion rate by gravimetric mass loss. As the field study measured the corrosion rate once and not several times during various varying environmental conditions, this comparative analysis does not compare equally weighted corrosion rates. As such, this is a shortcoming of the program.
- Individually, each NDT does not provide sufficient information to properly assess the condition of deteriorated reinforced concrete structures. When a combination of half-cell potential, electrical resistivity, and corrosion rate measurements by LPR, GPT, CEPRA, and Wenner Probe are overlaid for comparison, regions of high corrosion activity can be easily determined. For assessment of existing structures, it is recommended that a combination of NDT is used to obtain a more reliable assessment than that of a single NDT.

Laboratory Study:

- This experimental study revealed that the instantaneous corrosion rate as determined by GPT is in good agreement with the actual corrosion rate as determined by gravimetric means for

controlled conditions, whereas LPR underestimates the instantaneous corrosion rate. This generalized conclusion applies to 10M reinforcing bars only.

- Overall, the GPT correlation coefficients for Laboratory Mix #2, 60-mm concrete cover, with both 1.5% and 3% chloride content, reveal a moderate to strong relationship between the corrosion rate measurements provided by GPT and the gravimetric mass loss performed on the rebars.
- The average corrosion rate as determined by the CEPRA technology is in good agreement with corrosion rate as determined by gravimetric mass loss for controlled conditions with 10M reinforcing bars only.
- CEPRA comparative analysis for 1.5% and 3% chloride content indicated a good relationship with data set within the 2.0 tolerance factor.
- Out of all possible presented modifications, it was determined that no modifications to gravimetric corrosion rate by percentage of corroded area on the surface of reinforcing bars or modification to polarization area results in an average corrosion rate ratio of 1.1. As a result, the CEPRA technology measured average corrosion rate accurately. No modification provides improvement in the measurement of average corrosion rate for measurements of 10M reinforcing bars.
- As the initial weight of the reinforcing was not recorded prior to construction of reinforced concrete slabs, there is a strong possibility that gravimetric testing inaccurately represented the mass loss of corrosion products. This conclusion is further strengthened by the larger average ratio of gravimetric to actual corrosion rate for 0% chloride content specimens.
- The CEPRA technology shows a good correlation between electrical resistivity and corrosion rate in comparison to established theoretical relationships previously presented by the research community.

5.2. Future Work

Recommended future work as listed:

- A comprehensive re-evaluation of ASTM G1 for determining the average corrosion rate may be prudent as it was noted that the standard assumes uniform corrosion across the testing specimen and does not address pitting corrosion.
- As all non-destructive devices rely on the user to understand the parameters that may affect the results, either user or environmentally dependant, it is recommended to review the effects of such potential errors.

- Ensure the original weight of the reinforcing bars is measured prior to placement in concrete
- For future field studies, it is recommended to monitor an existing structure by conducting condition assessment surveys multiple times over the span of several seasons to allow for results to be integrated over the testing period and thus determine a more representative average in-situ corrosion rate.
- Destructive testing should determine the rate of absorption allowing for a better understanding of the concrete pore structure and ultimately the concrete's susceptibility to ingress of water and chlorides.
- In order to provide for a comparison between laboratory and field studies, it is recommended that the laboratory study be tailored to the concrete characteristics and geometry of an existing reinforced concrete structure. This would enable common parameters to be analyzed under both controlled and non-controlled conditions.
- Statistical analysis was hindered by limited test results. Scope of future experimental programs should be augmented to allow for more comprehensive statistical analysis.

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Appendix A – Concrete Mix Design

Coarse Aggregate

Following CSA A 23.2-12A “Relative density and absorption of coarse aggregate”

Mass Air Dry “A” = 2904.27 g

Mass SSD “B” = 2907.70 g

Mass Submerged “C” = 1911.21

Over-Dry Relative Density (Apparent Relative Density) = $2904.27 / (2904.27 - 1911.21) = 2.92$

Absorption = $(2907.77 - 2904.27) / 2904.27 * 100 = 0.12\%$

Moisture Content

The moisture content for the coarse aggregate was taken as an average of data collected on the days before mixing.

Mass Over-dry aggregate = 2.59%; 2.09%

Mass stockpile aggregate = 2.61%; 2.10%

Moisture Content = $((2.61 - 2.59) / 2.59) * 100 = 0.91\%$

Moisture Content = $((2.10 - 2.09) / 2.10) * 100 = 0.31\%$

Moisture Content = $(0.91 + 0.31) / 2 = 0.61\%$

Following CSA A23.2-10A “Bulk density of aggregate”

Volume of metal container = 0.0069 m³

Weight of aggregates = 11.0620 kg

Bulk density = $11.0620 \text{ kg} / 0.0069 \text{ m}^3 = 1603.0000 \text{ kg} / \text{m}^3$

Fine Aggregate

Following CSA A23.2-6A “relative density and absorption of fine aggregate”

Mass Air Dry “A” = 494.75 g

Mass Pycnometer filled with water "B" = 1671.70 g

Mass pycnometer filled with water and sand "C" = 1984.70 g

Mass sand "M_f" = 503.98 g

Over-Dry Relative Density (Apparent Relative Density) = $494.75 / (1671.70 + 494.75 - 1984.70) = 2.72$

Absorption = $(503.98 - 494.75) / 494.75 = 1.87 \%$

Moisture Content

Mass Over-dry aggregate = 0.97%

Mass stockpile aggregate = 1.00%

Moisture Content = $((1.00 - 0.97) / 0.97) * 100 = 3.50 \%$

Following CSA A23.2-2A "Sieve analysis of fine and coarse aggregate"

Table A.0.1: Fine aggregate gradation results

Sieve Opening No.	Sieve Opening (mm)	Weight of pan and aggregate (kg)	Weight of pan (kg)	Mass retained on sieve (kg)	Percent Retained (%)	Cumulative percent retained (%)
4	4.75	weight not taken	weight not taken	no sand in sieve	0.00	0.00
8	2.36	492.0	491.5	0.5	0.1	0.1
16	1.18	430.5	427.0	3.5	0.7	0.8
30	0.6	433.5	404.0	29.5	5.9	6.7
50	0.3	597.5	377.0	220.5	44.4	51.2
100	0.15	561.0	351.5	209.5	42.2	93.4
pan		404.5	371.5	33.0		

			Total Weight	496.5	Fineness Modulus	1.5
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Concrete Mix Design – Mix #2

Cement: Holcim GU/Type 10 with a relative density of 3.15

Coarse Aggregate: 20 mm nominal maximum-size gravel, meets CSA grading requirements, with an over-dry relative density of 2.92, an absorption of 0.12%, an over-dry bulk density of 1603 kg/m³, and a moisture content of 0.61%.

Fine Aggregate: oven-dry relative density of 2.72, absorption of 1.87%, moisture content of 3.50%, and a fineness modulus of 2.47

67/33 Slag/ Fly Ash Mix: Material specification were not available. Assumed relative densities of fly ash and slag are 2.6 and 2.9, respectively.

Plasticizer: MasterGlenium® 7500 Full-Range Water-Reducing Admixture. Meets ASTM C 494/C 494 M compliance requirements for Type A and Type F. Dosage used: 280 mL / 100 kg of cement. Assumed relative density of 1.0.

Following “Design and Control of Concrete Mixtures”, adopted from ACI 211.1, the mix design for Mix #2 is as follows (Steve H. Kosmatka, 2002):

Water Content

Table 9.6-Slump Recommendations for slabs: 25-75 mm

Table 9.5- Non-air entrained; 20 mm aggregate; 25-75 mm slump

$$\text{Water demand} = 190.0\text{--}205.0 \text{ kg/m}^3$$

$$\text{Water demand} = 184.5 \text{ kg/m}^3 \text{ (Accounting for 10\% reduction in water demand with plasticier)}$$

Cementing Material Content

$$\text{Cementing Material (w/c} = 0.47) = 184.5/0.4 = 461.25 \text{ kg/m}^3$$

$$\text{Cement (55\%)} = 253.69 \text{ kg/m}^3$$

$$\text{Fly Ash (15\%)} = 69.19 \text{ kg/m}^3$$

$$\text{Slag (30\%)} = 138.38 \text{ kg/m}^3$$

Coarse Aggregate Content

Table 9-4 – bulk volume of coarse aggregate per unit volume of concrete

$$\text{Coarse aggregate (dry)} = 1603 \text{ kg/m}^3 * 0.65 = 1046.46 \text{ kg/m}^3$$

Plasticizer

At 280 ml / 100 kg of cement

$$\text{Plasticizer} = (2.8/1000) * 461.25 = 1.29 \text{ kg/m}^3$$

Fine Aggregate Content

Using the Unit Volume Method

Fine aggregate (dry) = 1-

$$\begin{aligned} & [(184.50/(1.0*1000)) + (253.69/(3.15*1000)) + (69.19/(2.60*1000)) + (138.38/(2.9*1000)) + (1046.76 \\ & / (2.92*1000))] = 0.30 \text{ m}^3 \end{aligned}$$

$$= 0.30 \text{ m}^3 * 2.72 * 1000 = 824.05 \text{ kg/m}^3$$

Water Demand Correction

$$\text{Water demand} = 183.21 - (1046.76 * (0.006 - 0.0012)) - (824.05 * (0.035 - 0.019))$$

$$= 164.65 \text{ kg/m}^3$$

Summary of Actual Concrete Mix Design (adjusted for moisture):

Table A.0.2: Actual Concrete Mix Design Proportions for Mix #2

water	164.65	kg/m ³
cement	253.69	kg/m ³
fly ash	69.19	kg/m ³
slag	138.38	kg/m ³

coarse aggregate (.91%, wet)	1053.14	kg/m ³
fine aggregate (3.5%, wet)	852.89	kg/m ³
plasticizer	1.29	kg/m ³

Total density

2533.22 kg/m³

Actual Concrete Mix Proportions

Mix #2 Casting Date: June 1, 2016

Mix #1 Casting Date: June 2, 2016

Table A.0.3: Concrete Mix Proportions

Mix#1	Control	1.5% NaCl	3% NaCl
V (for 3 slabs+ 3cylinders)=	0.036	0.036	0.036
	Control	1.5% Cl	3.0% Cl
GU 10	9.540	9.540	9.540
20 mm aggregate	38.212	38.212	38.212
sand	35.024	35.024	35.024
water	5.202	5.202	5.202
Chloride	0.000	0.143	0.286
Mix#2	Control	1.5% NaCl	3% NaCl
V (for 3 slabs+ 3cylinders)=	0.036	0.036	0.036
	Control	1.5% Cl	3.0% Cl
GU 10	9.133	9.133	9.133

slag	4.982	4.982	4.982
fly ash	2.491	2.491	2.491
20 mm aggregate	37.913	37.913	37.913
sand	30.704	30.704	30.704
water	5.927	5.927	5.927
plasticizer	0.047	0.047	0.047
chloride	0.000	0.137	0.274

Each concrete mix was separated into three batches so that NaCl could be added to the mix. The sodium chloride was mixed into the water. All mixing and pouring followed CSA A23.2-2C procedures. In addition, concrete compression cylinders were poured so that compression testing could be done at Day 28. These cylinders followed the procedures in CSA A23.2-3C.

Mixing and Casting Visual Documentation with Notes



Figure A.0.1: Mix #2 Mixing

Note: Concrete mix seemed to more workable as the batch went from 0% to 1.5% and 1.5% to 3% chloride content by weight of cement.

Following CSA A23.2-5C, the slump of the concrete was determined. For Mix #1 and Mix #2, the slump values are 2 mm and 3 mm, respectively. The Mix #2 slump value is in agreement with the assumed slump range used in the concrete mix design.



Figure A.0.2: Mix #2 Post- Casting

Note: All slabs and cylinders were allowed 24 hrs to cure before formwork and moulds were removed. During the initial 24 hour curing, all slabs and cylinders were covered.



Figure A.0.3: Formwork Removal

Appendix B - Field Stage Photos



Figure B.0.1: Cornwall Slab 11



Figure B.0.2: Cornwall Slab 12



FigureB.0.3: Cornwall Slab 13



Figure B.0.4: Cornwall Slab 14



Figure B.0.5: Slab 12 Exposed Rebar / Evidence of Green Rust



Figure B.0.6: Slab 13 Exposed Rebar / Removed Rebar



Figure B.0.7: Slab 14 After Rebar Removed

Appendix C – Compressive Strength Testing Data

Following the CSA A23.2-9C standard for “Compressive strength of cylindrical concrete specimens”, compression testing was performed on day 28 to ensure that the concrete mix design produced normal strength concrete. Compression testing was completed on June 29, 2016. Refer to Table C.1.

Table C.0.1: Compressive Strength Testing Data

		Specimen	Diameter (cm)	Length (cm)	L/D	Weight (kg)	Comp. Strength (MPa)	Applied Load (kN)	Type of Failure
Mix #2 w/c = 0.4	0%	1	10	19.5	2.0	3.8145	49.47	388.5	3 or 4
		2	10.1	19.7	2.0	3.857	45.15	354.6	3 or 4
		3	10.2	19.8	1.9	3.8925	48.05	377.4	3 or 4
		AVG					47.56		
	1.50%	1	10.2	19.9	2.0	3.898	41.87	331.8	3 or 4
		2	10	20	2.0	3.934	45.77	359.6	3 or 4
		3	10	20	2.0	3.9205	43.15	338.9	3 or 4
		AVG					43.60		
	3.00%	1	10.1	19.9	2.0	3.901	47.68	369.8	3 or 4
		2	10.2	19.8	1.9	3.8745	50.46	396.3	1
		3	10.1	20	2.0	-	48.73	382.7	3 or 4
		AVG					48.96		
Mix #1 w/c = 0.65	0%	1	10.2	19.8	1.9	3.9005	29.22	229.5	3 or 4
		2	10.1	19.6	1.9	3.8825	31	-	3 or 4
		3	10.2	19.6	1.9	3.852	28.44	223.4	3 or 4
		AVG					29.55		
	1.50%	1	10.2	20	2.0	3.934	31.47	247.2	3 or 4
		2	10.1	20	2.0	3.9255	30.87	243	3 or 4
		3	10.2	19.6	1.9	3.9245	32.55	255.9	3 or 4
		AVG					31.63		

	3.00%	1	10.1	19.6	1.9	3.8165	27.94	219.7	3 or 4
		2	10.1	19.9	2.0	3.8625	29.91	234.9	3 or 4
		3	10.1	19.6	1.9	3.8575	29.87	234.6	3 or 4
	AVG						29.24		

As per the CSA standard, all specimen's length-to-diameter ratio are less than 1.8 so the correction factor does not need to be applied.



Figure C.0.1: Mix #2-28 Day Compression Cylinder Failure



Figure C.0.2: Mix #1-28 Day Rupture Cylinder Failure

Appendix D – Gravimetric Mass Loss Experimental Procedure

Following ASTM G1-03 (2011), all rebar specimens were tested in order to determine the mass loss and subsequently, the average corrosion rate. In order to compare the average corrosion rate as determined by this standard with that of the instantaneous corrosion rates obtained by electrochemical techniques (LPR, GPT, and CEPRA), the instantaneous corrosion rates will be integrated over time (Thomas Frolund, 2002). It is important for the rebar pieces to be cut into small portions or to utilize photo analysis software to determine the percentage of corroded area on the reinforcement.

Utilize the Chemical Procedure (7.2)

As seen in the literature, the most commonly used solution is as follows from Table A1.1, C.3.5:

- 500 mL hydrochloric acid (HCL, sp gr 1.19)
- 3.5 g hexamethylene tetramine
- 500 mL of reagent water (or distilled water)

Materials / Equipment Needed:

- Chemical Fume hood
- 1 L beaker
- Acid Resistant Basin
- Tongs
- Nonmetallic bristle brush
- Stirrer
- Scale
- Glass supports

Procedure:

1. Location of the specimens within the chemical solution should be noted so that identification of the specimen remains throughout the testing.
2. Cut specimens to a representative corroded length
3. Clean specimen mechanically by scrubbing the rebar with the bristle brush. This removes the concrete off the surface
4. Measure the length and weight of the cut rebar

5. Place all specimens in the acid resistant basin and on top of the glass supports. Specimens should not rub against each other.
6. In a 1 L beaker, mix 500 ML of the hydrochloric acid and 500 ML of distilled water. As with any acid, the acid should be added to the water.
7. Stir the solution for one minute
8. Add 3.5 g of hexamethylene tetramine to the solution and stir until the buffer dissolves.
9. Repeat steps 6-8 until enough solution is available to completely submerge all specimens.
10. As per the standard, allow the rebars to soak for a minimum of 10 minutes in the solution until no reaction between the solution and rebars is seen. The average soaking time was 15 minutes. If the specimens are heavy corroded, longer immersion times may be required.
11. Carefully remove the rebars using the tongs and clean the rebars with water using the bristle brush.
12. After cleaning the rebars, allow the rebars to dry.
13. Measure the weight of the specimens after the cleaning cycle is finished.
14. Dispose of the chemical solution.
15. Clean the all equipment.
16. Repeat the whole procedure until the weight of the rebars becomes constant.
17. It might be necessary to repeat the cleaning cycle several times until the weight of the rebar is constant.

If a specimen without corrosion is available, this control specimen should be cleaned following the same procedure. Less cleaning cycles are needed. The point of a control specimen is to acquire the extent of metal loss resulting from the cleaning procedure so that a correction for corrosion mass loss can be used.

The equation to correct using the control specimen is as follows:

$$M = \frac{mf - mi}{mc} \quad \text{Equation D-0-1: ASTM G1 Mass Loss}$$

Where mf is the mass of the corroded rebar after each cleaning cycle, mi is the mass of the corroded bar before cleaning started, and mc is the mass of the control bar of an equivalent length. This is calculated by multiplying the density of the control bar by the length of the corroded bar to be measured. Where the density of the control bar is calculated after cleaning it by measuring its mass and volume.

The mass loss is then plotted as a function of the number of cleaning cycles as illustrated in Figure D.1.

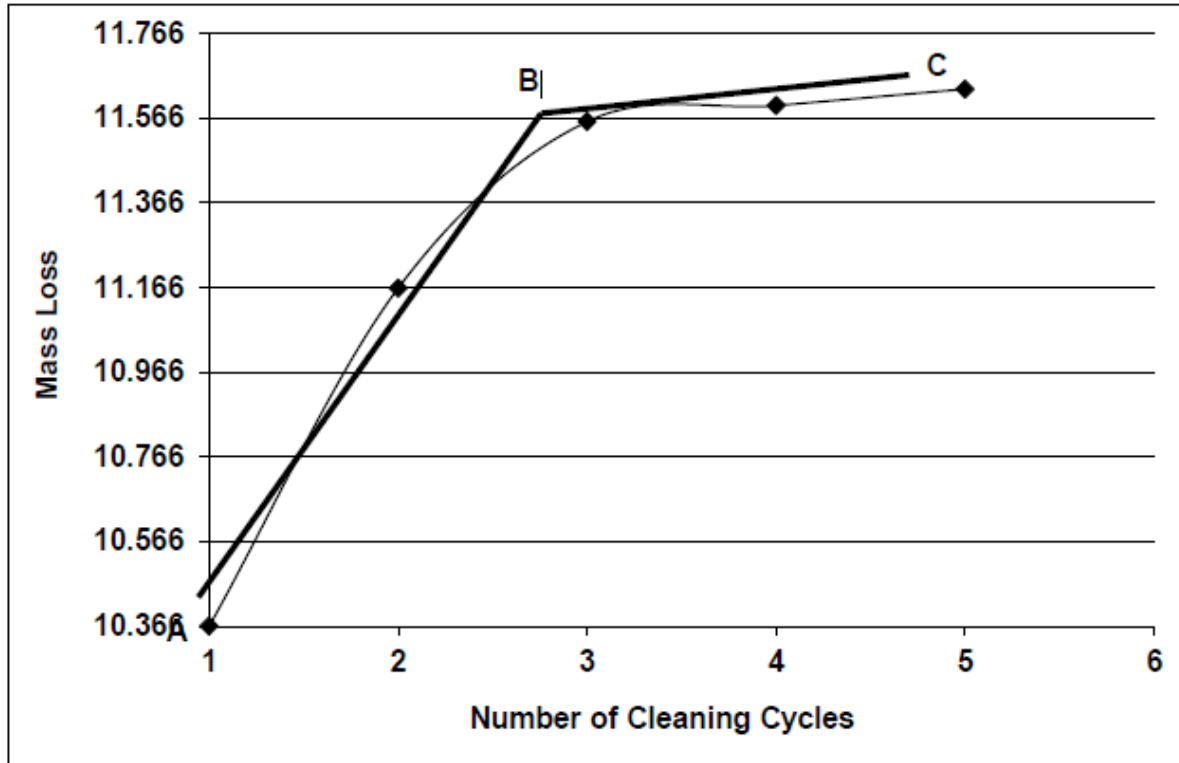


Figure D.0.1: Mass Loss vs Number of Cleaning Cycles (ASTM G1)

Two linear lines are obtained: AB and BC. Line BC represents the corrosion of the metal after removal of corrosion products. Thus, the mass loss due to corrosion will correspond to point B.

If localized pitting is not observed, then the equation for average corrosion rate provided by the standard. Refer to Eq. D-1

$$\text{Average Corrosion Rate} = \frac{(K \times W)}{(A \times T \times D)} \quad \text{Equation D--0-2: ASTM G1 Average}$$

Corrosion Rate

Where K is a constant based on the corrosion rate units desired, T is the time of exposure in hours (or the life of the structure), A is the initial total surface area of the specimen in cm^2 (notes that this is the total surface area of the specimen as when initially exposed to the corrosive environment, do not take into account the changes in total surface area due to pitting, opening of subsurface porosity, etc.), W is the mass loss in g, and D is the density in g/cm^3 . In order to compare, the desired units for corrosion rate will be in micrometers per year or $\mu\text{m}/\text{yr}$, with $K = 8.76 \times 10^7$. Broomfield provided a conversion of $1 \mu\text{m}$ steel section loss per year = $1 \mu\text{A}/\text{cm}^2$ as found in Faraday's law to metal loss (Broomfield J. , 2007).

Unless any information of the steel reinforcement for the existing structure can be found, the density of steel is assumed to be 7.85 g/cm³.

An equation for the average corrosion rate for cylindrical specimens is also provided by the standard. Refer to Eq. D-2 and D-3.

$$\text{Corrosion rate} = \frac{(w \times d)}{(4 \times m \times t)} \quad \text{Equation D-0-3: ASTM G1 Corrosion Rate, Cylindrical Specimen}$$

For $m/w > 50$ or,

$$\text{Corrosion rate} = 100 \times d \frac{\left[1 - \frac{1}{(1000 \times m)}\right]^{0.5}}{2t} \quad \text{Equation D-0-4: ASTM G1 Corrosion Rate, } m/w > 50$$

Where w is the mass loss in mg, m is the specimen mass in g, d is the specimen diameter in mm, and t is the exposure time in years.

The life span of the reinforced concrete structure needs to be determined.

If localized pitting is found, consult ASTM G46.

If corrosion rates are moderate to low, use ASTM G31 equation for amended duration of test.

Appendix E - Absorption Test Procedure

Following ASTM C642 (2013) to determine the density, absorption, and voids in hardened concrete, several concrete samples retrieved from the North Channel Bridge were tested. As specified by the standard, each specimen shall have a volume not less than 350 cm³ or approximately 800 g. The specimens tested were cylindrical in shape with a diameter of 100 mm and a height of 50 mm. With these dimensions, the volume of the actual specimens are more than 350 cm³ and about 840 g. Therefore the specimens meet the requirement.

Procedure:

Oven-Dry Mass

1. Determine the initial mass of the specimens, Z.
2. Place the specimens in an oven at a temperature of 110 ± 5 °C for not less than 24 h.
3. Remove the specimen from the oven and allow to cool in dry air (preferably in a desiccator) to the temperature of 20 – 25 °C and determine the mass.
4. Place the specimen back in the oven for another 24 h drying treatment. Repeat from step 2.
5. If the difference in masses after the first and second drying treatments exceeds 0.5%, return the specimen to the oven for another drying cycle and repeat from step 2-3.
6. The mass after all drying treatments is denoted, A.

Saturated Mass After Immersion

7. Immerse the specimen, after the final drying, cooling, and determination of mass, in water at approximately 21°C for not less than 48 h.
8. Remove the specimen and dry the surfaces with a towel. This state is designated Surface-Dry. Then determine the mass, B.
9. Repeat steps until two successive values of mass of the surface-dried samples at intervals of 24 hrs show an increase in mass of less than 0.5%.

Saturated Mass After Boiling

10. Following the determination of B, place the specimen in a suitable receptacle, covered with tap water, and boil for 5 h.
11. With the specimen still in the tap water, allow to cool by natural loss of heat for not less than 14 h to the final temperature of 20-25 °C.

12. Remove the specimen and dry the surface with a towel. Take the mass of the specimen, designated the soaked, boiled, surface-dried mass, C.

Immersed Apparent Mass

13. Suspend the specimen, after immersion and boiling, by a wire, and determine the apparent mass in water. Designated this apparent mass, D.

The following properties of the specimen can be determined with the following equations:

Absorption after immersion, % = $\left[\frac{B-A}{A} \right] \times 100$ *Equation E-0-1: ASTM C642 Absorption after Immersion*

Absorption after immersion and boiling, % = $\left[\frac{C-A}{A} \right] \times 100$ *Equation E-0-2: ASTM C642 Absorption after immersion and boiling*

Bulk density, dry = $\left[\frac{A}{C-D} \right] * \rho = g_1$ *Equation E-0-3: ASTM C642 Bulk Density*

Bulk density after immersion = $\left[\frac{B}{C-D} \right] * \rho$ *Equation E-0-4: ASTM C642 Bulk density after immersion*

Bulk density after immersion and boiling = $\left[\frac{C}{C-D} \right] * \rho$ *Equation E-0-5: ASTM C642 Bulk density after immersion and boiling*

Apparent density = $\left[\frac{A}{A-D} \right] * \rho = g_2$ *Equation E-0-6: ASTM C642 Apparent Density*

Volume of permeable pore space (voids), % = $\frac{g_2 - g_1}{g_2} \times 100$ or $\frac{C-A}{C-D} \times 100$ *Equation E-0-7: ASTM C642 Volume of permeable pore space*

Where:

A = mass of oven-dries sample in air, g

B = mass of surface-dry sample in air after immersion, g

C = mass of surface-dry sample in air after immersion and boiling, g

D = apparent mass of sample in water after immersion and boiling, g

g_1 = bulk density, dry, Mg/m³

g_2 = apparent density, Mg/m³

$$P = \text{density of water} = \text{Mg/m}^3 = 1\text{g/cm}^3$$

Note that the mass of the specimen in water after immersion and boiling, D , can be determined by subtracting the mass in air after immersion and boiling, C , from the volume of the specimen since the loss of mass in water is equal to mass of displaced water.

Appendix F – Titration Test Procedure

Following ASTM C1152/ C1152M (2012), concrete powder samples were collected, at different penetration depths, from the North Channel Bridge slabs were tested to determine the chloride concentration.

A microprocessor-controlled automatic titrator, the 794 Basic Titrino from Metrohm was used. The ASTM standard calls for a potentiometer titration method known as Volhard Method as silver nitrate is used. Experimental evidence has shown that the chloride contents of samples tested using the Volhard method are higher when compared to the results tested with the Basic Titrino (Deif, 2010). The Basic Titrino is a more superior method because it automatically determines the equivalence point whereas the Volhard method relies on manual determination of the equivalence point. During typical titration, the titrant is fed into the solution while the solution's millivoltmeter reading is taken. As the equivalence point is reached, the millivoltmeter reading will change in larger increments. The Basic Titrino automatically adjusts the titrant feeding in tandem with the millivoltmeter readings so ensure that the equivalence point is exactly reached. In addition, after the equivalence point is reached, the Basic Titrino provides exact values for the concentration in ppm, the amount of titrant used, and the value of the millivoltmeter reached.

Experimental testing has shown that in determining the acid-soluble chloride content of cement and concrete by the potentiometric titration, the filtration step could be eliminated (Climent, Vera, & Estanislao Viqueira, 2004). Therefore, the filtration step as required by the ASTM in 8.2 was eliminated.

The Basic Titrino manual recommends that the Dynamic Equivalence point (DET) is used for chloride content titration. The DET mode allows reagent feeding to have variable volume increments which depends on the slope of the volume of reagent vs. electrical potential graph (Metrohm Ion Analysis, 2002). In addition, the Basic Titrino provides precise titration parameters and operation conditions that are to be programmed into the Basic Titrino before testing. Basic Titrino recommends that the titrant for chloride concentration determination is 0.01 N AgNO_3 for low chloride contents. This solution was prepared prior to testing.

Chemicals Needed:

- 0.05 N AgNO_3
- 50% (1+1) Nitric Acid (prepared with 50% distilled water and 50% partially pure nitric acid)
- 30% hydrogen peroxide

- Methyl orange indicator
- 0.05 N NaCl

Materials Needed:

- 250 mL beakers
- Watch glasses
- Glass stirring rod
- Balance
- Hot plate
- 25 mL, 3 mL, and 2 mL pipets
- Glass dropper

Safety Precautions:

- Fine concrete powder are harmful if inhaled. Masks are recommended.
- Goggles, gloves, and lab coats are required.
- Nitric acid and hydrogen peroxide are harmful if exposed to skin and eyes.

Procedure:

1. Ensure that concrete samples are fine enough to pass through No. 20 sieve
2. Measure approximately 5 g of the concrete powder samples and pour into the 250 mL beaker
3. Add 75 mL of distilled deionized water to the beaker and stir to disperse the powder sample.
4. Slowly add 25 mL of the 50% nitric acid and stir.
5. If the smell of rotten eggs is present, add 3 mL of the 30% hydrogen peroxide.
6. Add three drops of the methyl orange indicator and stir.
7. Cover the beaker with a watch glass and let the solution stand for 1-2 minutes.
8. If a pink colour is not observed or if the solution is not sufficiently acidic (as determined with litmus paper), add drops of 50% nitric acid to the solution until a pink colour is observed or until the solution is sufficiently acidic.
9. Place the beaker on the hot plate and heat until the solution rapidly boils. Allow the sample to boil only for a few seconds.
10. Repeats previous steps with 75 mL of distilled deionized water for the blank determination.

As stated before, the filtration step, ASTM 8.2, was skipped.

11. After the solution has cooled, add 2 mL of 0.05 NaCl to the beaker.
12. Place the beaker on the magnetic stirrer.
13. Place the mV reading probe and the dispensing burette into the solution. Make sure that there is no contact between the probe and burette with the magnet used by the stirrer.
14. Start the automatic titration. Input the amount of solution in the beaker in grams.
15. Wait for the titration to automatically stop.
16. Record the amount of AgNO₃ consumed to reach the equivalence point as this value is needed in the calculation of the chloride content in addition to the amount of AgNO₃ consumed for the blank determination.
17. Clean with probe and burette with distilled water. Dispose of the analyzed solution.

As per the ASTM standard, the percentage of chloride content in the concrete powder sample is calculated according to the following equation:

$$Cl(\%) = \frac{3.545[(V_1 - V_2)N]}{W} \quad \text{Equation F-0-1: Chloride Content}$$

Where V_1 is the volume of consumed 0.05 N AgNO₃ at the equivalence point for the concrete sample titrated in mL, V_2 is the volume of consumed 0.05 AgNO₃ at the the equivalence point for the blank determination in mL, N is the exact normality of the 0.05 AgNO₃, and W is the mass of the concrete powder sample in g. (ASTM G1, 2011)

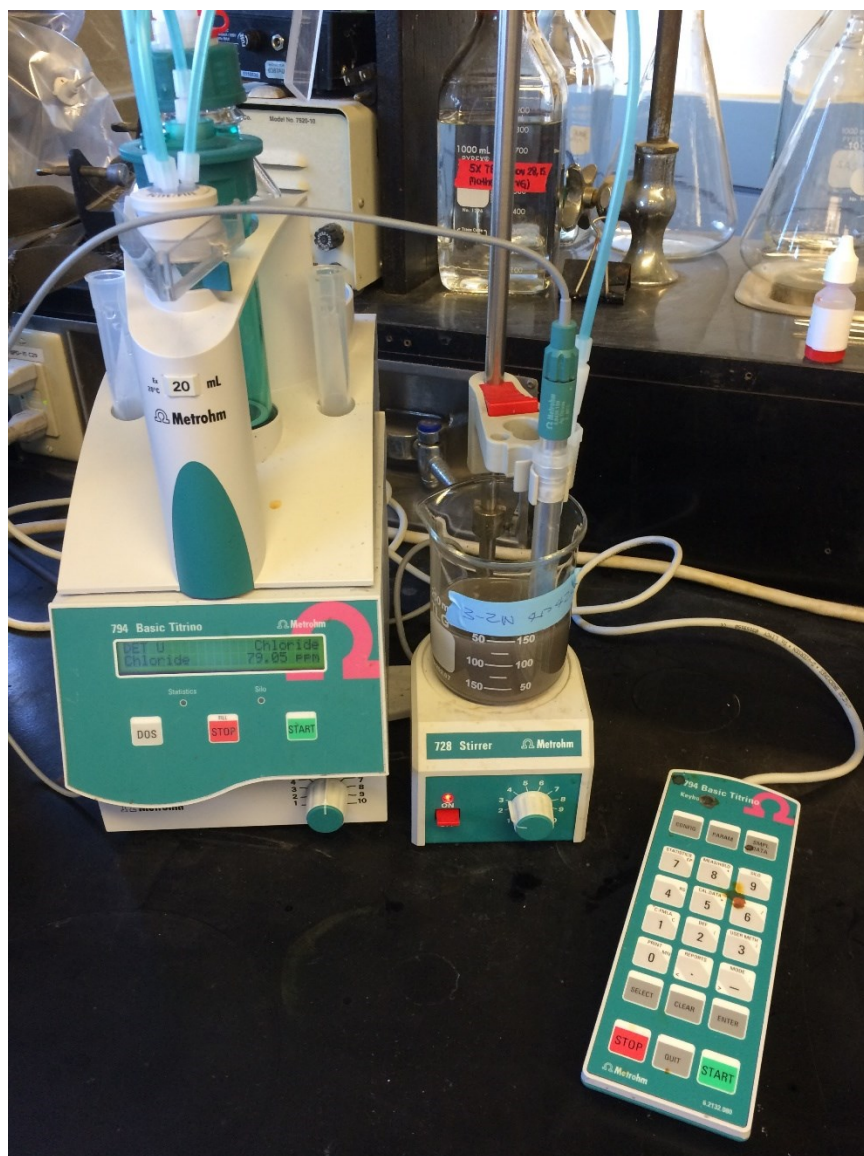


Figure F.0.1: Basic Titrino Titration

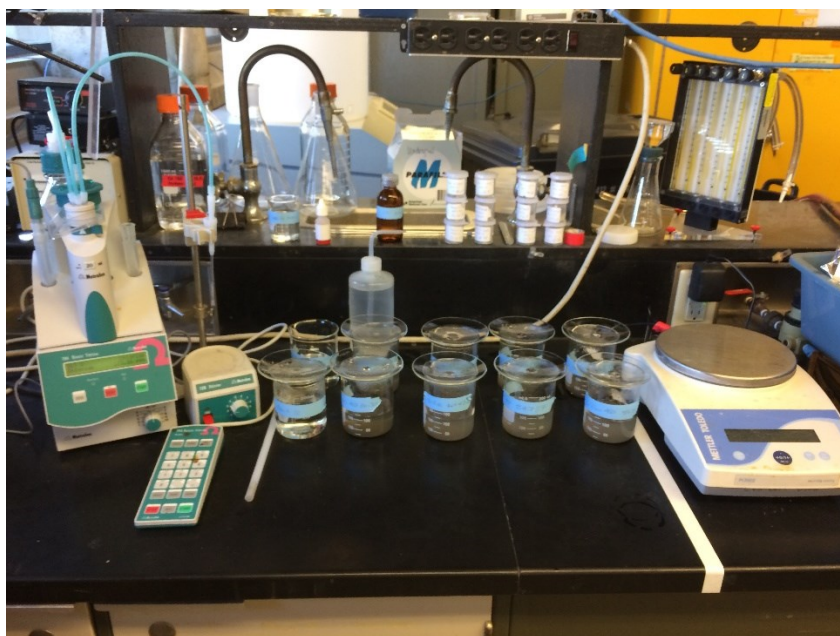


Figure F.0.2: Titration Setup



Figure F.0.3: Samples during heating

Appendix G – Titration Results

Table G.0.1: Titration Results

Depth (mm)	Sample Size (g)	ppm	V1 (mL)	mV	Chloride (%)
S13 from bot					
Blank 1		37.08	2.095	138	
200-195	5.46	55.9	3.157	108	0.0345
195-190	5.33	54.22	3.061	113	0.0321
190-185	5.54	67.36	3.8	113	0.0546
185-180	3.77	90.1	5.085	122	0.1406
S12 from top					
Blank 2		37.14	2.097	137	
0-4	6.02	158.25	8.926	112	0.2012
4-8mm	5.55	195.58	11.076	121	0.2869
8-12mm	5.76	183.67	10.407	105	0.2558
12-16mm	6.55	207.55	11.733	104	0.2609
16-20	5.55	169.35	9.556	104	0.2383
Blank 3		37.01	2.09	143	
20-24	5.13	170.46	9.615	0	Error
24-28	5.06	151.29	8.551	114	0.2262
Blank Average			2.0935		

Depth (mm)	Sample Size (g)	ppm	V1 (mL)	mV	Chloride (%)
S13 from top					
Blank 4		39.06	2.205	142	
0-5	4.95	103.43	5.834	114	0.1293
5-10mm	2.34	106.24	5.992	125	0.2856
10-15mm	2.07	94.03	5.368	128	0.2694
15-20	2.13	85.85	4.842	130	0.2180
20-25	-	-	-	-	-
Blank 5		39.69	2.239	130	
25-30	5.08	137.03	7.731	116	0.1922
30-35	4.77	149.34	8.432	124	0.2308
35-40	4.93	133.21	7.522	117	0.1906
40-45	1.33	61.49	3.469	128	0.1662
45-50	2.92	101.74	5.74	126	0.2135
Blank Average			2.222		
S14 from top					
Blank 6		38.57	2.178	115	
0-5	5.04	84.47	4.601	111	0.0850
5-10mm	4.89	102.45	5.803	121	0.1311
10-15mm	4.94	95.79	5.405	109	0.1155
15-20	5	95.8	5.235	121	0.1081
20-25	2.43	64.19	3.702	133	0.1107
Blank 7		38.82	2.192	140	
25-30	5	90.72	5.121	127	0.1041
30-35	5.13	91.03	5.205	121	0.1043
35-40	5.02	89.27	5.034	119	0.1006
40-45	5.02	85.61	4.83	126	0.0934

45-50	2.13	59.17	3.34	143	0.0961
Blank Average			2.185		

Appendix H- Field Study Reinforcement Photos



Figure 0.1: S12 x0B



Figure 0.2: S12 x0T



Figure 0.3: S12 x4B



Figure H.4: S12 x4T



Figure 0.5: S12 x6B



Figure H.6: S12 x6T



Figure 0.7: S12 x8B



Figure H.8: S12 x8T



Figure 0.9: S12 x10B



Figure 0.10: S12 x10T



Figure 0.11: S12 x12B



Figure 0.12: S12 x12T



Figure 0.13: S12 x14B



Figure 0.14: S12 x14T



Figure 0.15: S13 x0B



Figure 0.16: S13 x12B



Figure 0.17: S13 x14B



Figure 0.18: S13 x14T



Figure 0.19: S13 x13T



Figure 0.20: S13 x2B



Figure 0.21: S13 x2T



Figure 0.22: S13 x4B



Figure 0.23: S13 x4T



Figure 0.24: S13 x6B



Figure 0.25: S13 x6T



Figure 0.26: S13 x8B



Figure 0.27: S13 x8T



Figure 0.28: S13 x10B



Figure 0.29: S13 x10T



Figure 0.30: S13 x12T



Figure 0.31: S14 x1B



Figure 0.32: S14 x1T



Figure 0.33: S14 x3B



Figure 0.34: S14 x3T



Figure 0.35: S14 x5B



Figure 0.36: S14 x5T



Figure 0.37: S14 x7B



Figure 0.38: S14 x7T



Figure 0.39: S14 x9B



Figure 0.40: S14 x9T



Figure 0.41: S14 x11B



Figure 0.42: S14 x11T

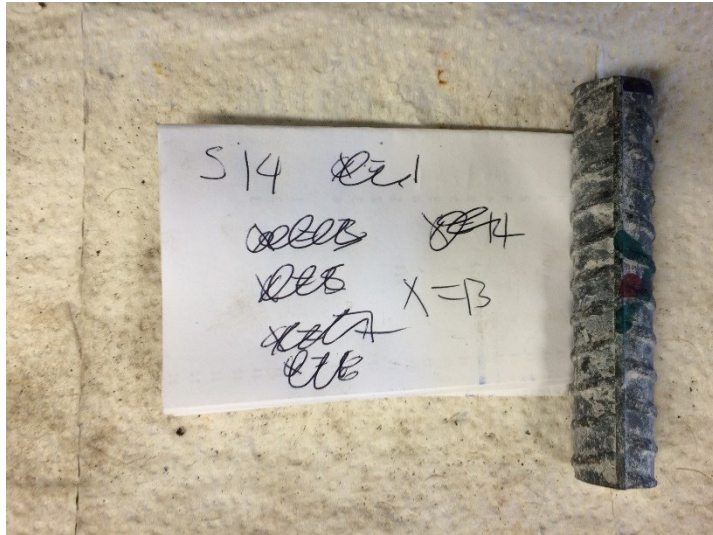


Figure 0.43: S14 x13B

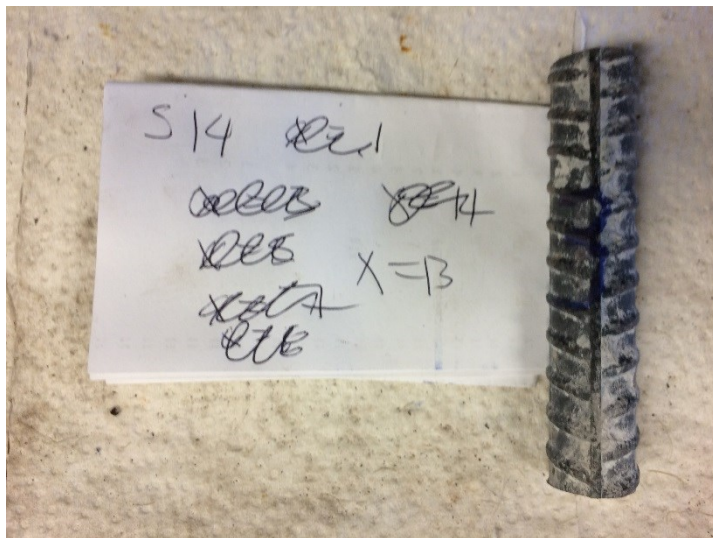


Figure 0.44: S14 x13T

Appendix I - Laboratory Reinforcement Photos

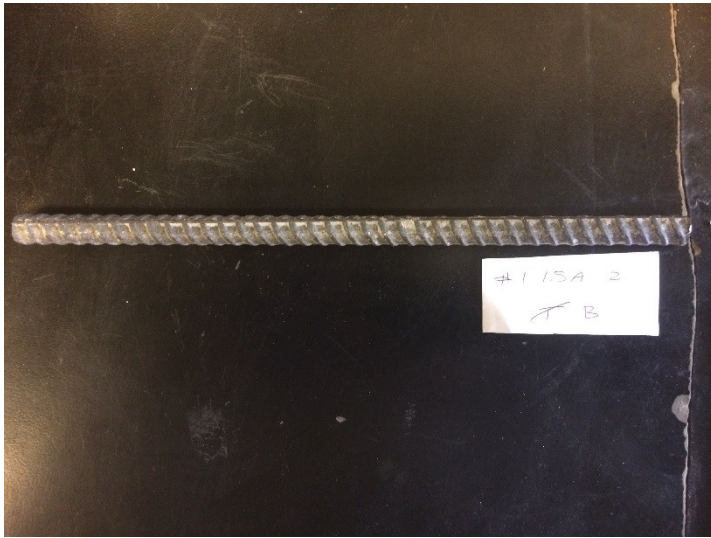


Figure 0.1: #1 1.5A 2B

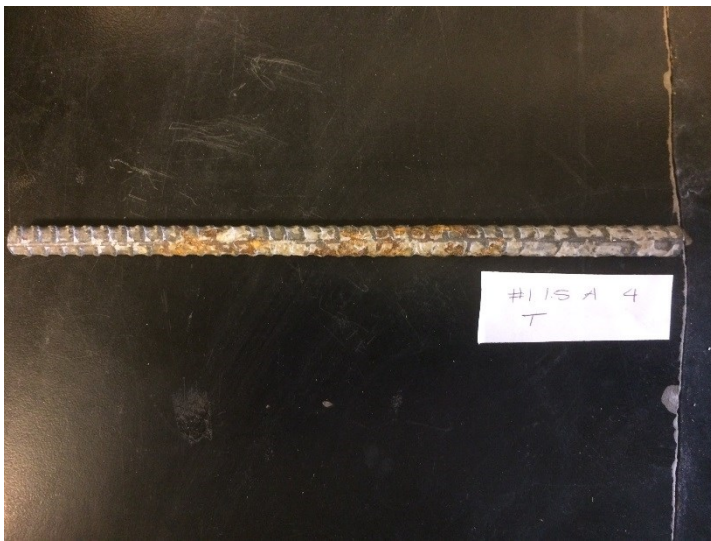


Figure 0.2: #1 1.5A 4T

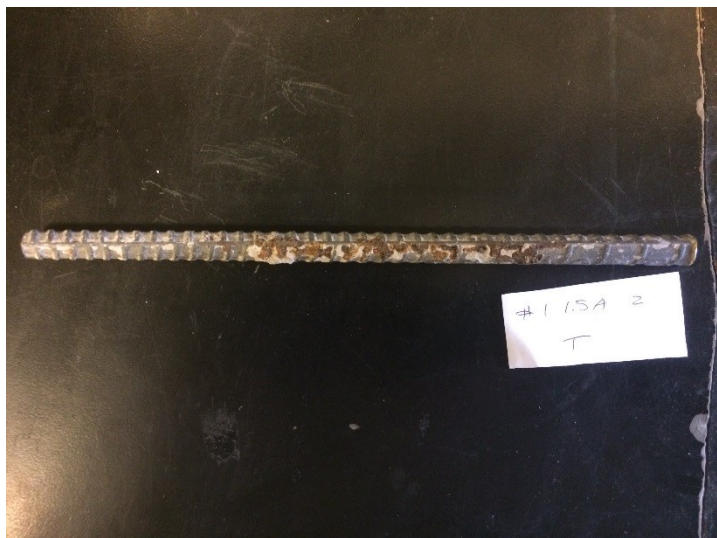


Figure 0.3: #1 1.5A 2T



Figure 0.4: #1 1.5A 2T



Figure 0.5: #1 1.5A 4B



Figure 0.6: #1 1.5B 2B



Figure 0.7: #1 1.5B 2T



Figure 0.8: #1 1.5B 4T

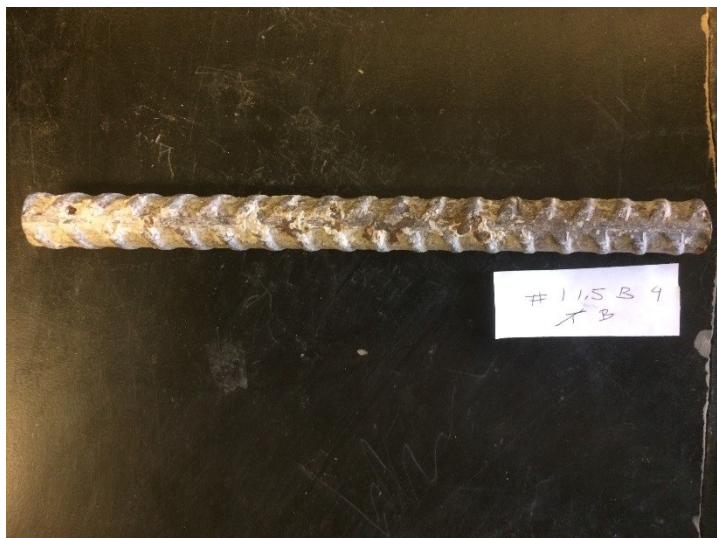


Figure 0.9: # 1 1.5B 4B

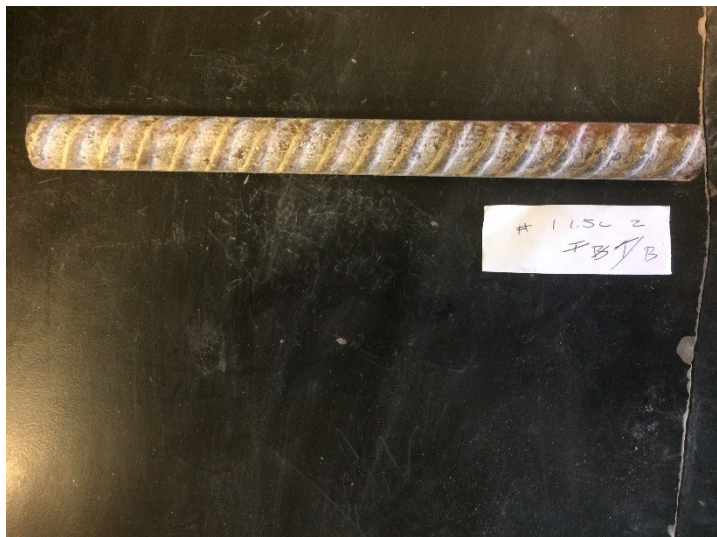


Figure 0.10: # 1 1.5C 2B

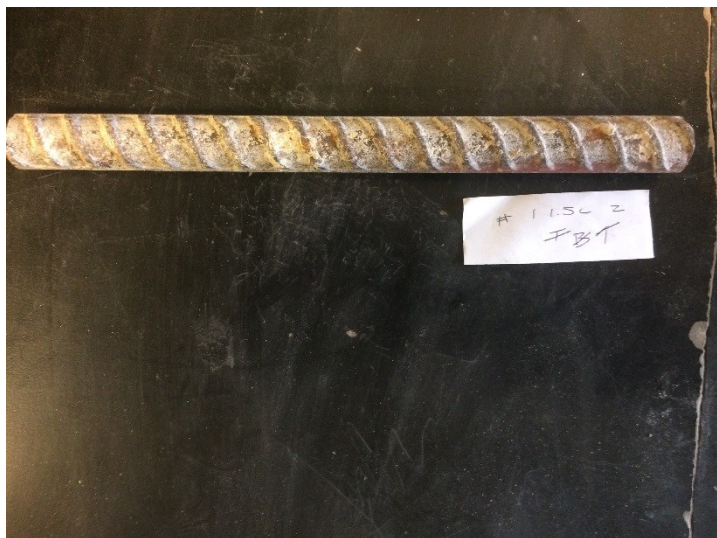


Figure 0.11: #1 1.5C 2T



Figure 0.12: #1 1.5C 2T



Figure 0.13: #1 1.5C 4B



Figure 0.14: # 1 1.5C 4T

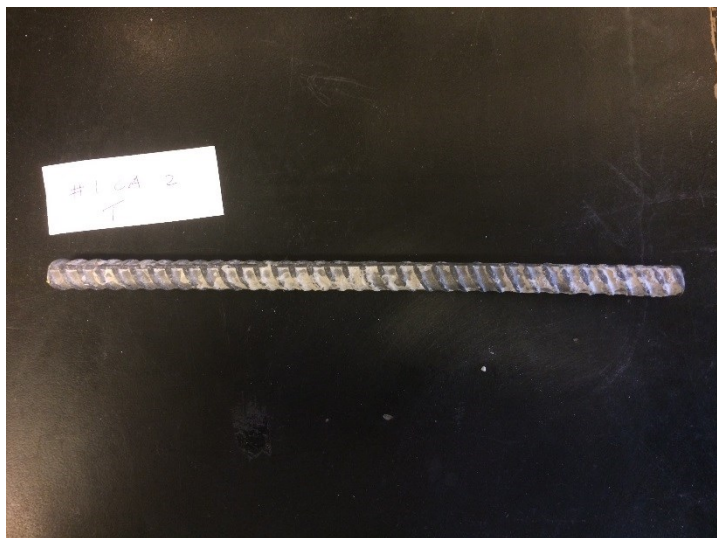


Figure 0.15: #1 CA 2T



Figure 0.16: #1 CA 4B



Figure 0.17: #1 CA 2T

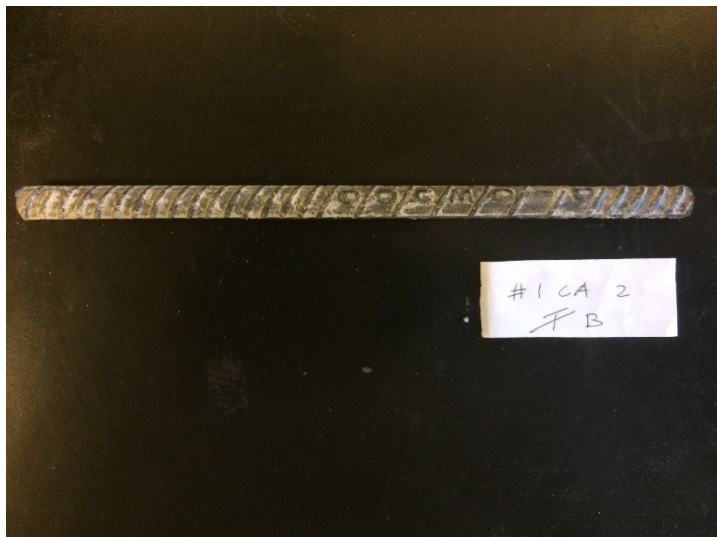


Figure 0.18: #1 CA 2B

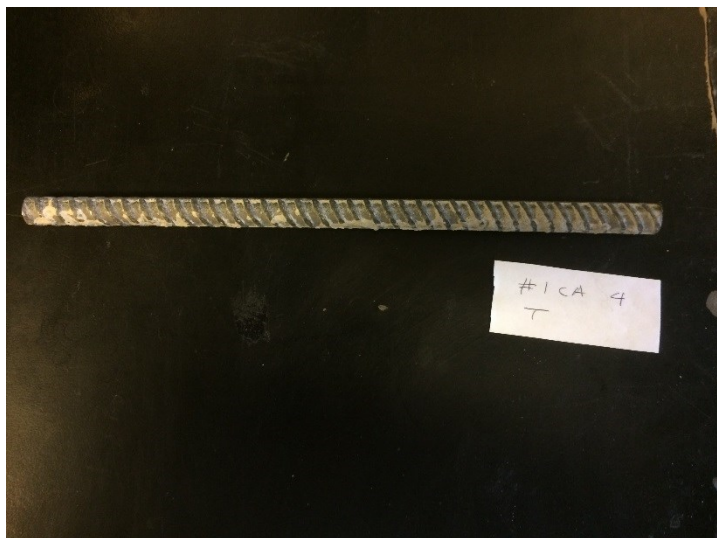


Figure 0.19: #1 CA 4T

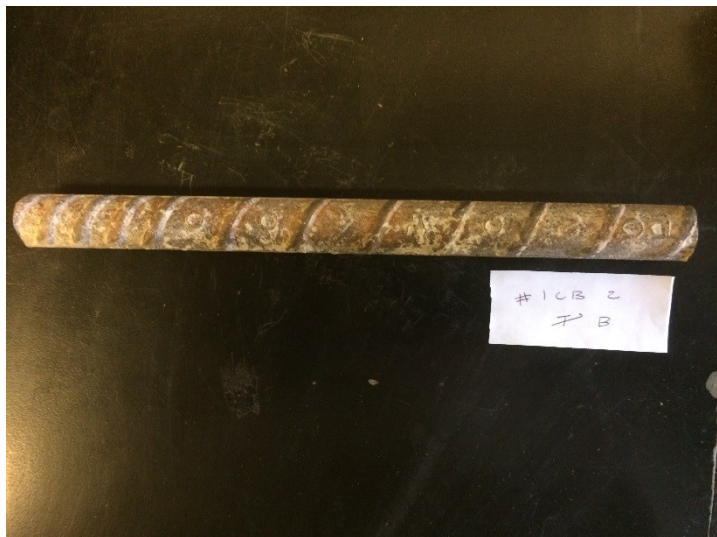


Figure 0.20: #1 CB 2B



Figure 0.21: #1 CB 2T

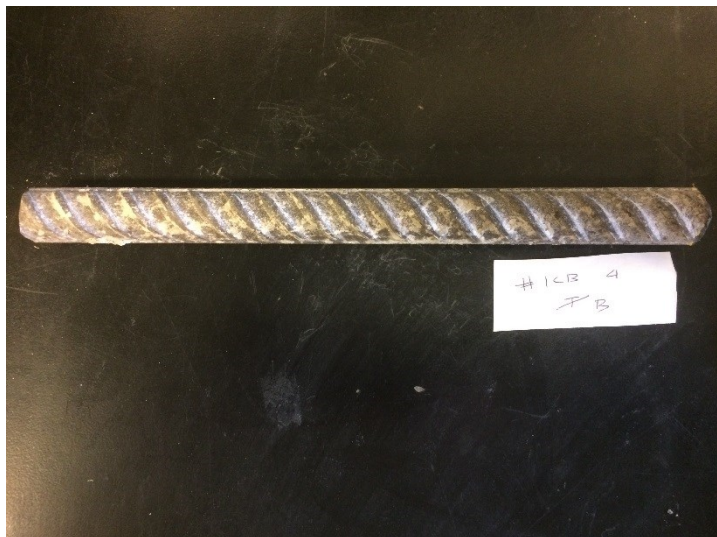


Figure 0.22# 1CB 5B



Figure 0.23: #1 CB 4T



Figure 0.24: #1 CC 2B

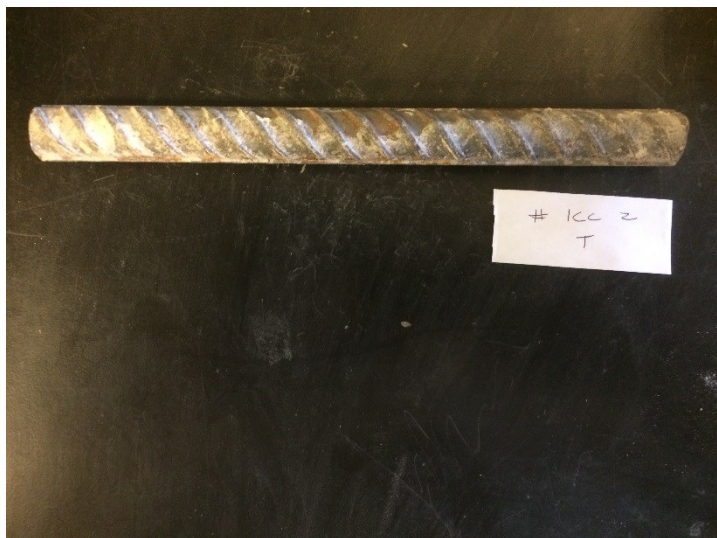


Figure 0.25: #1 CC 2T



Figure 0.26: #1 CC 4B



Figure 0.27: #1 CC 4T



Figure 0.28: #2 1.5B 2B



Figure 0.29: #2 1.5B 2T

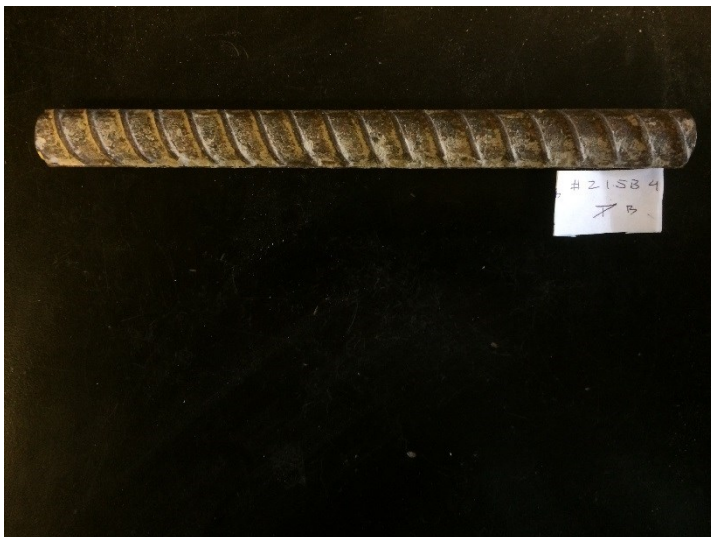


Figure 0.30: #2 1.5B 4B



Figure 0.31: #2 1.5B 4B



Figure 0.32: #2 1.5B 4T



Figure 0.33: #2 1.5C 2B



Figure 0.34: #2 1.5C 2B



Figure 0.35: #2 1.5C 2T



Figure 0.36: #2 1.5 C 2T



Figure 0.37: #2 1.5C 4B



Figure 0.38: #2 1.5C 4B

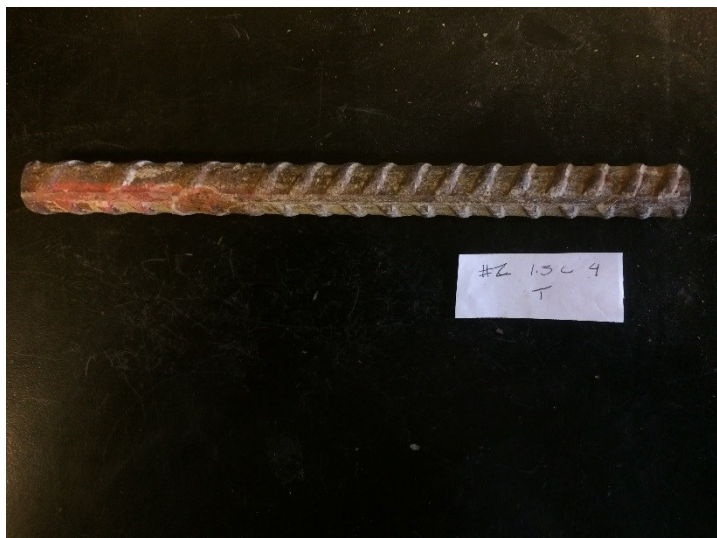


Figure 0.39: #2 1.5C 4T



Figure 0.40: #2 1.5C 4T

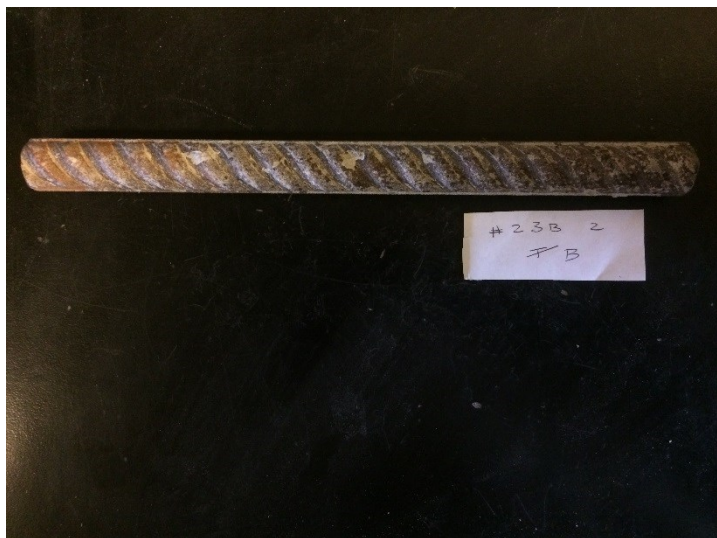


Figure 0.41: #2 3B 2B

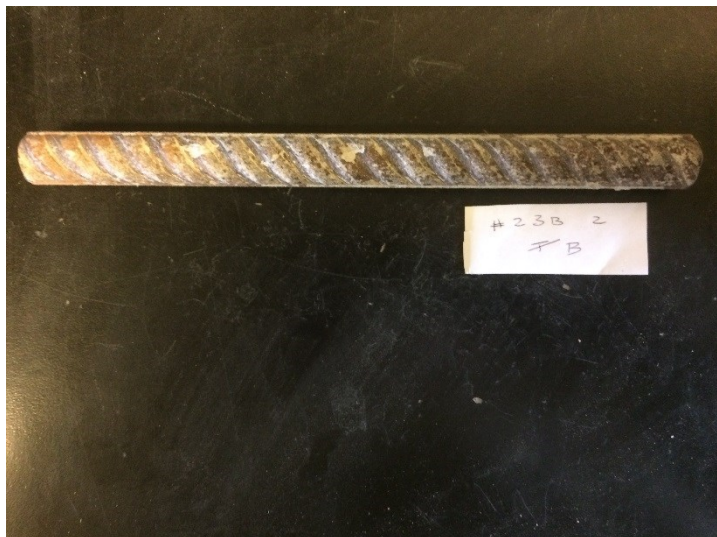


Figure 0.42: #2 3B 2B



Figure 0.43: #2 3B 2T



Figure 0.44: #2 3B 2T



Figure 0.45: #2 3B 4T



Figure 0.46: #2 3B 4T

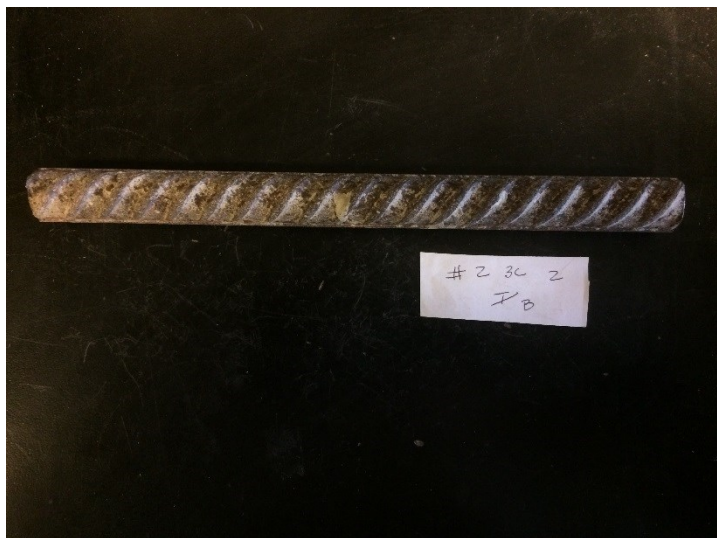


Figure 0.47: #2 3C 2B

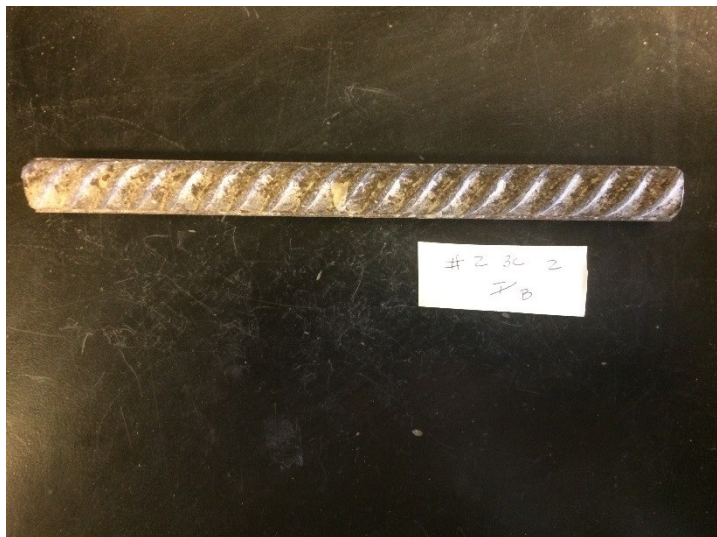


Figure 0.48: #2 3C 2B



Figure 0.49: #2 3C 2T



Figure 0.50: #2 3C 2T



Figure 0.51: #2 3C 4B



Figure 0.52: #2 3C 4B

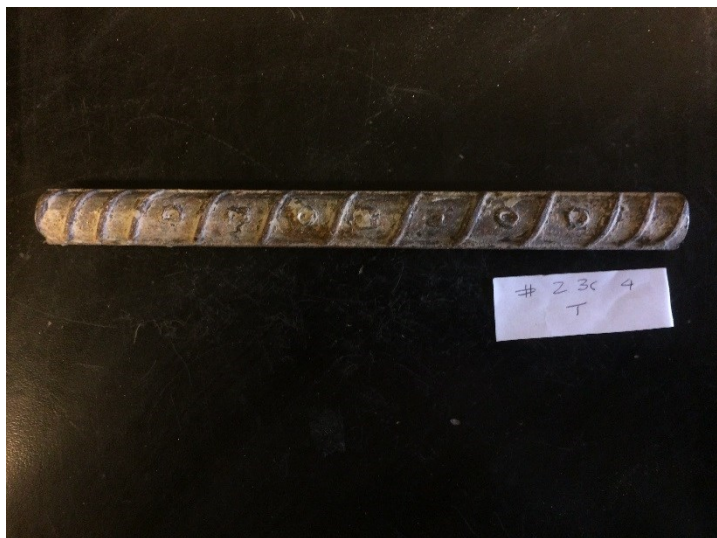


Figure 0.53: #2 3C 4T

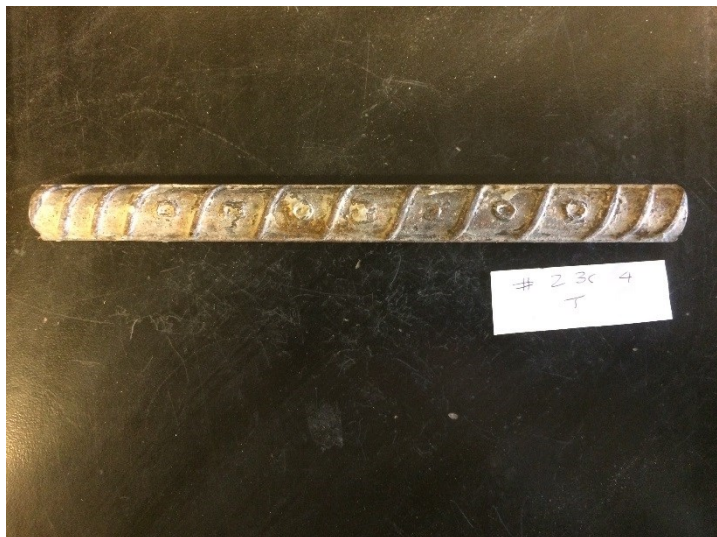


Figure 0.54: #2 3C 4T



Figure 0.55: #2 CA 2B



Figure 0.56: # 2 CA 2T

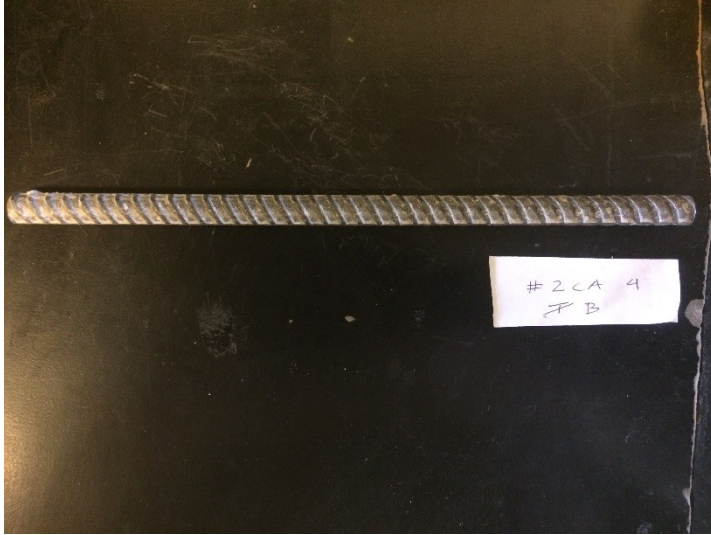


Figure 0.57: #2 CA 4B

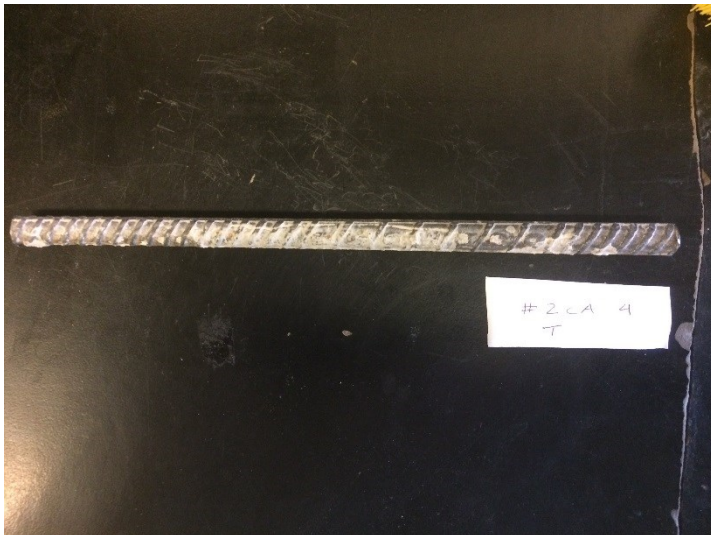


Figure 0.58: #2 CA 4T



Figure 0.59: #2 CB 2B



Figure 0.60: #2 CB 2T

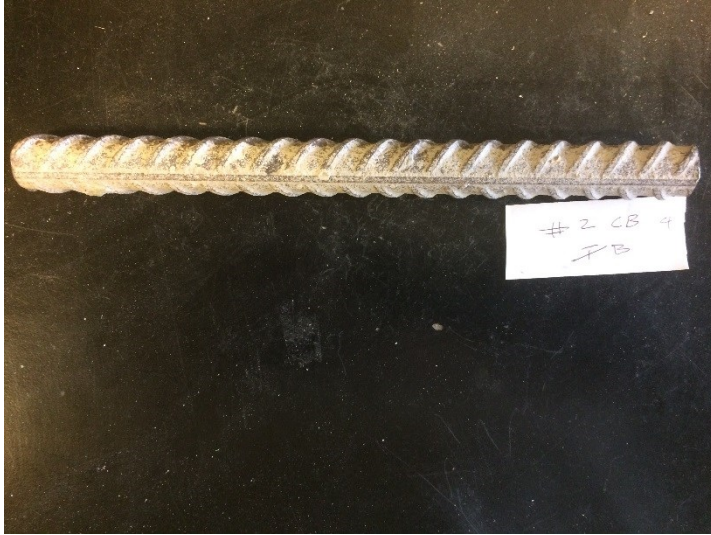


Figure 0.61: #2 CB 4B



Figure 0.62: #2 CB 4T



Figure 0.63: #2 CC 2B



Figure 0.64: #2 CC 2T



Figure 0.65: #2 CC 2T



Figure 0.66: #2 CC 4B



Figure 0.67: #2 CC 4B

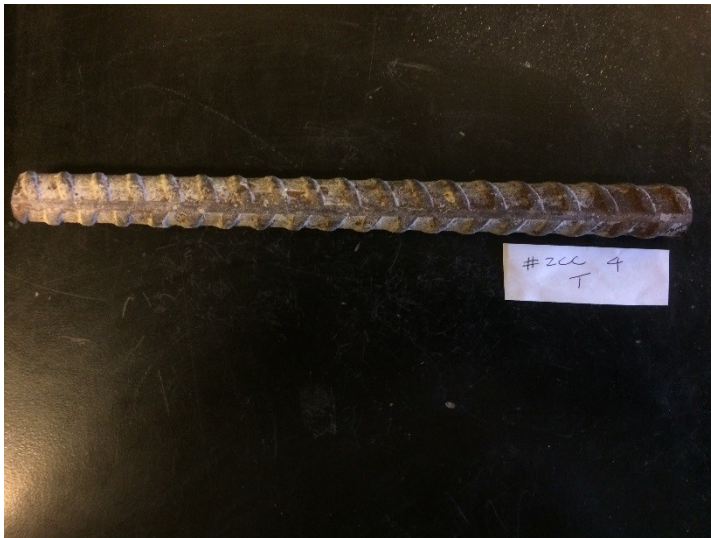


Figure 0.68: #2CC 4T



Figure 0.69: #2 CC 4T

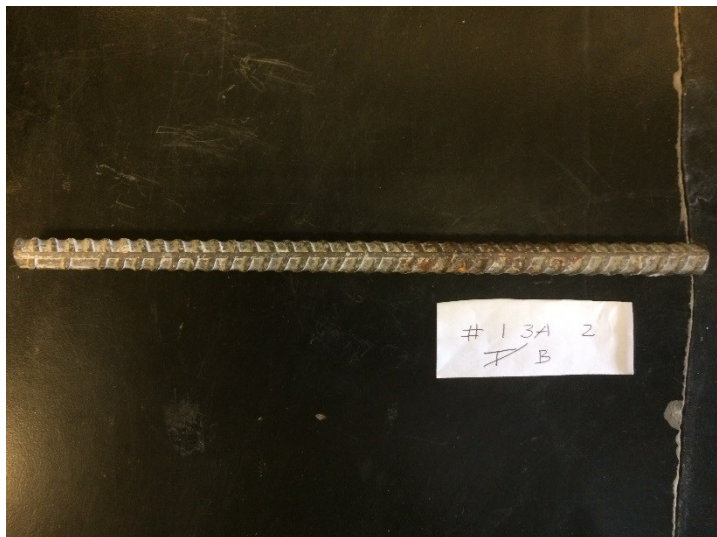


Figure 0.70: #1 3A 2B

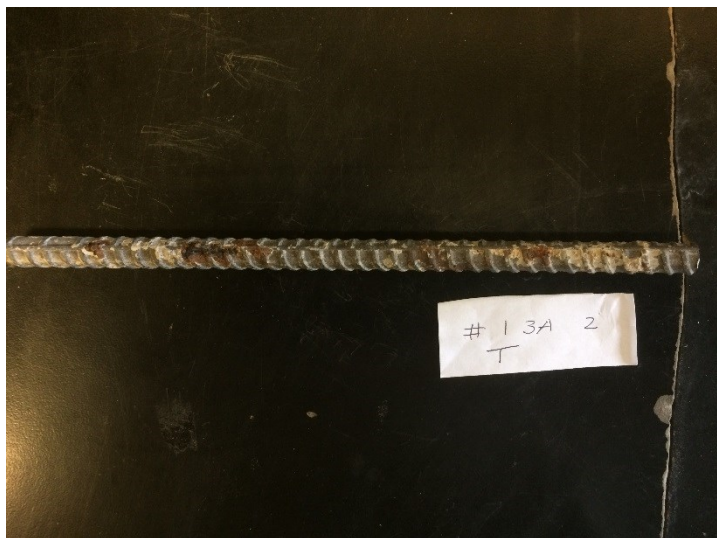


Figure 0.71: #1 3A 2T



Figure 0.72: #1 3A 4B



Figure 0.73: #1 3A 4T

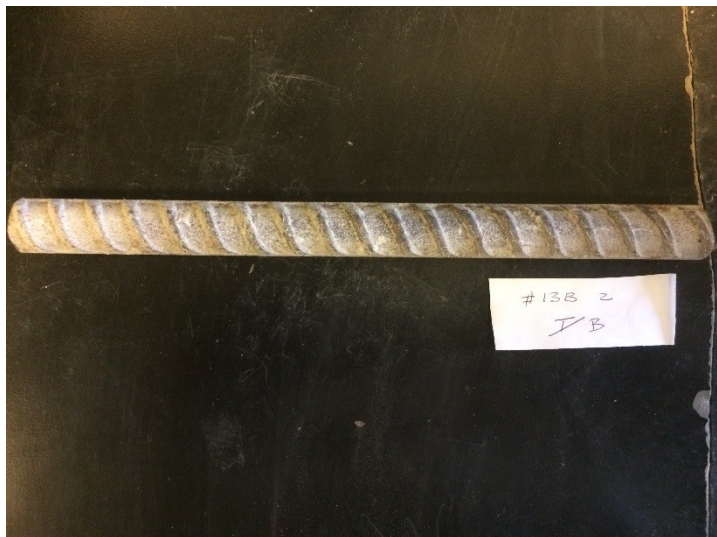


Figure 0.74: #1 3B 2B



Figure 0.75: # 1 3B 2T



Figure 0.76: #1 3B 4B

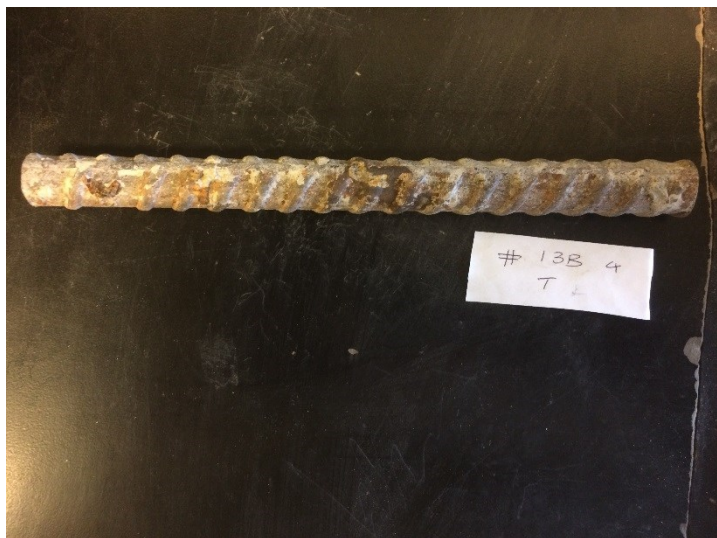


Figure 0.77: #1 3B 4T



Figure 0.78: #1 3B 4T

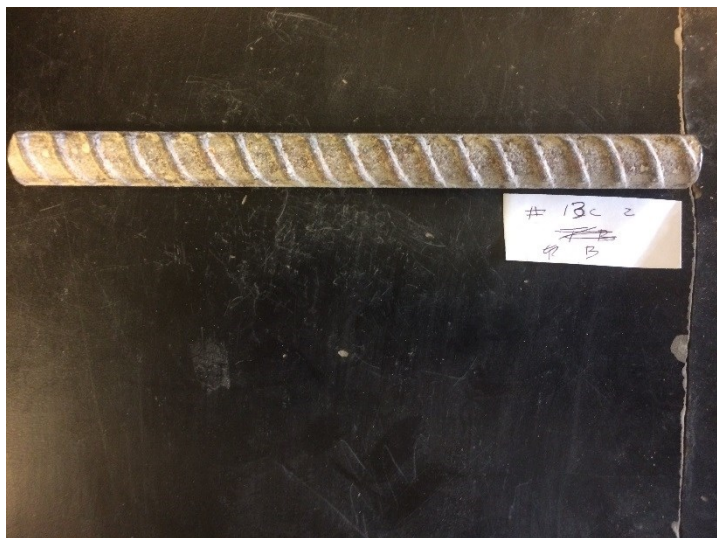


Figure 0.79: #1 3C 2B



Figure 0.80: #1 CC 2B



Figure 0.81: #1 3C 2T



Figure 0.82: #1 CC 2T

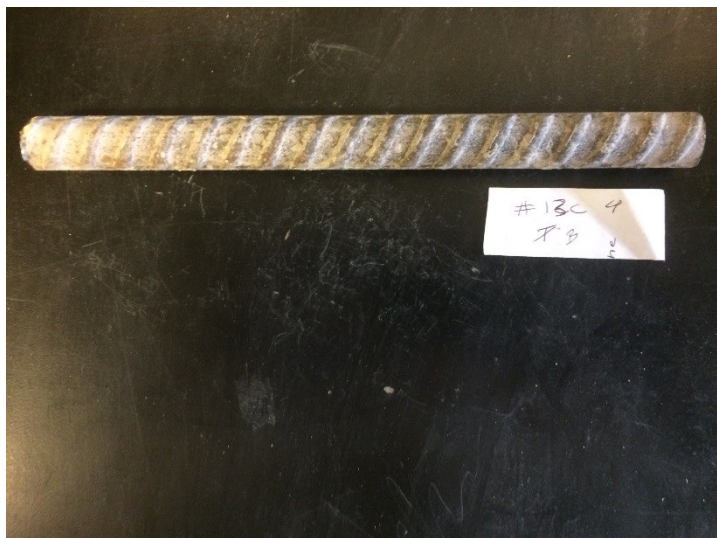


Figure 0.83: #1 3C 4B

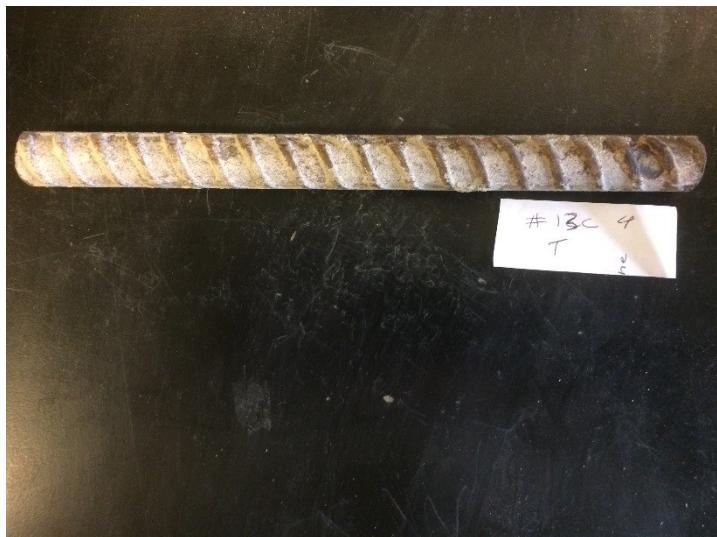


Figure 0.84: #1 3C 4T



Figure 0.85: #2 1.5A 4B

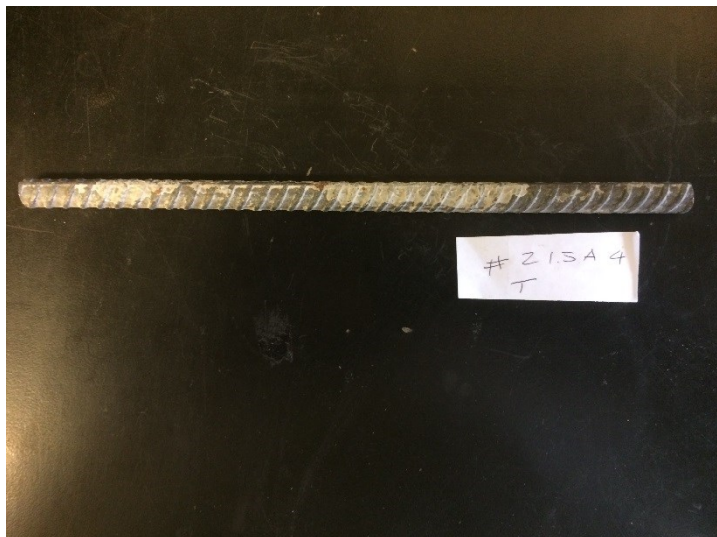


Figure 0.86: #2 1.5A 4T

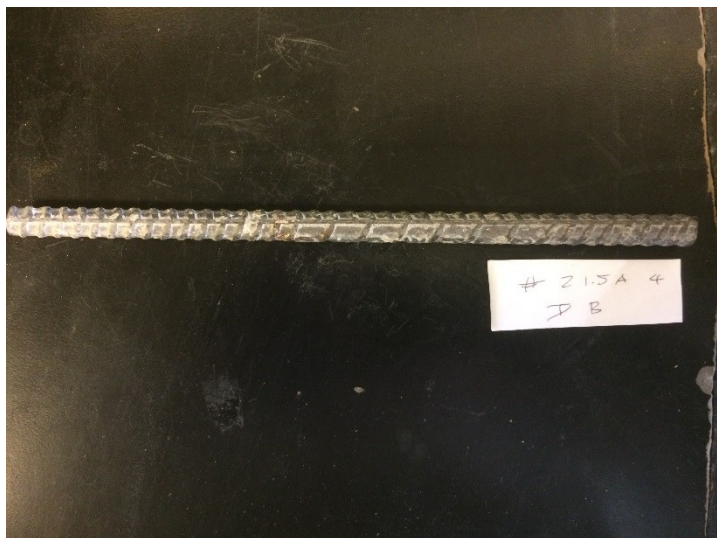


Figure 0.87: #2 1.5A 4B



Figure 0.88: #2 1.5A 4T

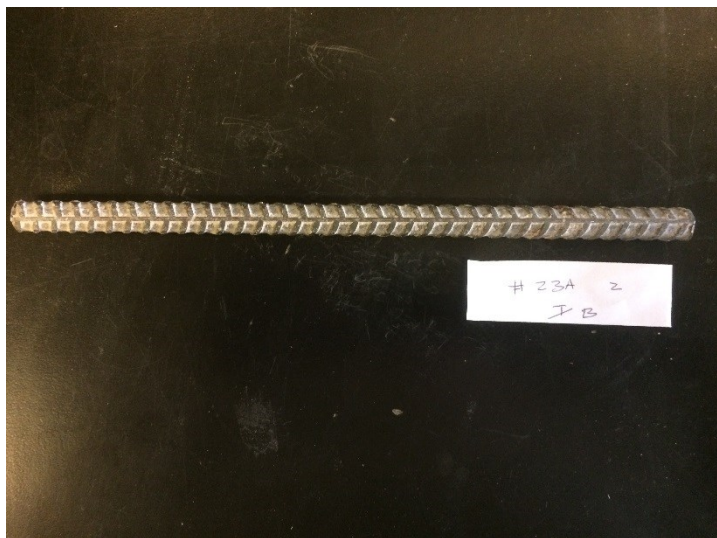


Figure 0.89: #2 3A 2B



Figure 0.90: #2 3A 2T



Figure 0.91: #2 3A 4B



Figure 0.92: #2 3A 4T

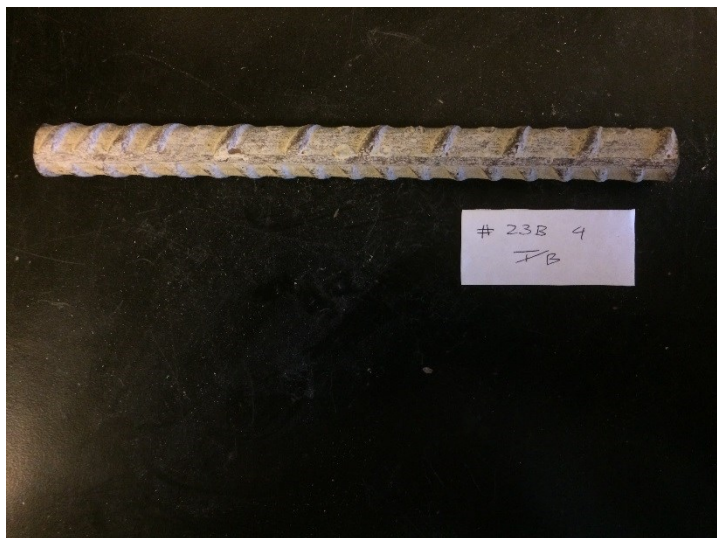


Figure 0.93: #2 3B 4B

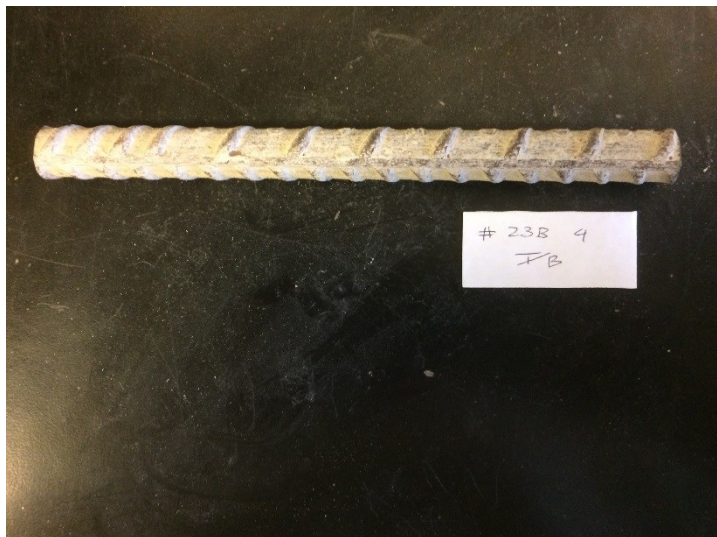


Figure 0.94: #2 3B 4B

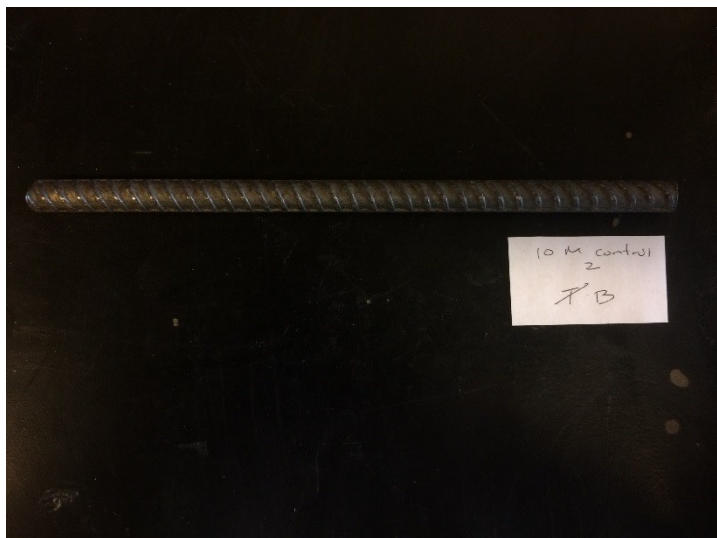


Figure 0.95: 10M Control 2B



Figure 0.96: 10M Control 2B



Figure 0.97: 10M Control 1B

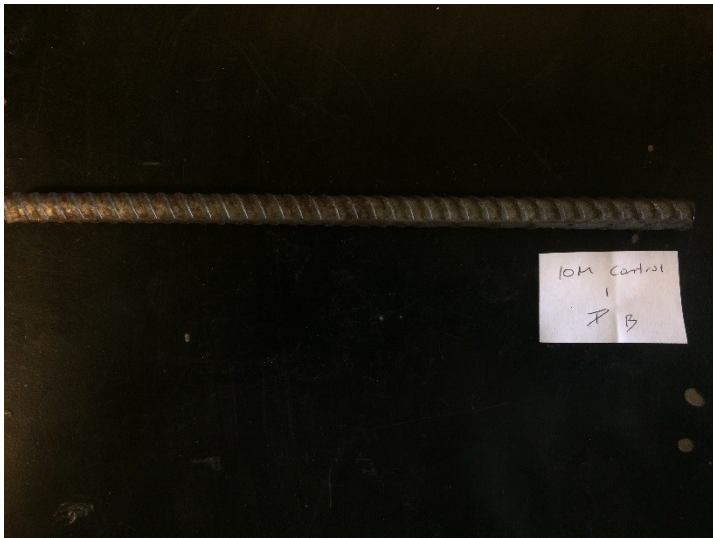


Figure 0.98: 10M Control 1B

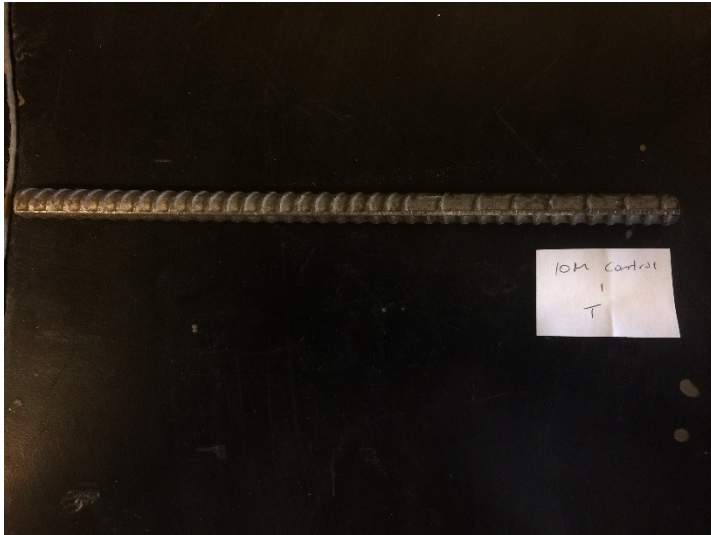


Figure 0.99: 10M Control 1T

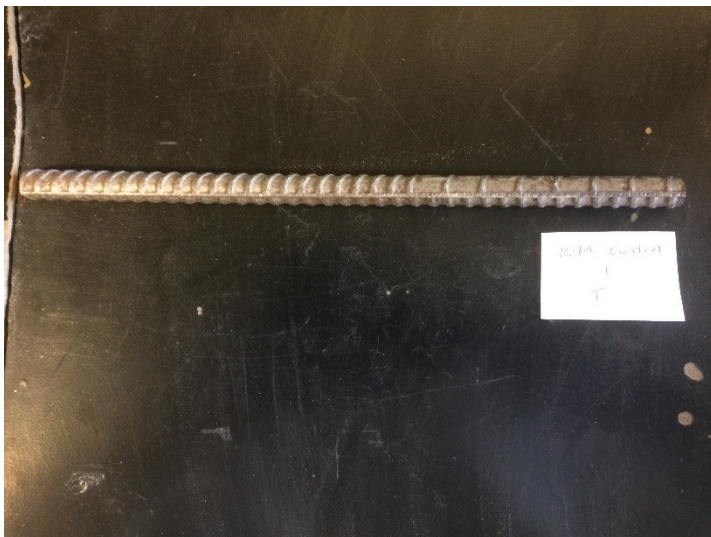


Figure 0.100: 10M Control 1T

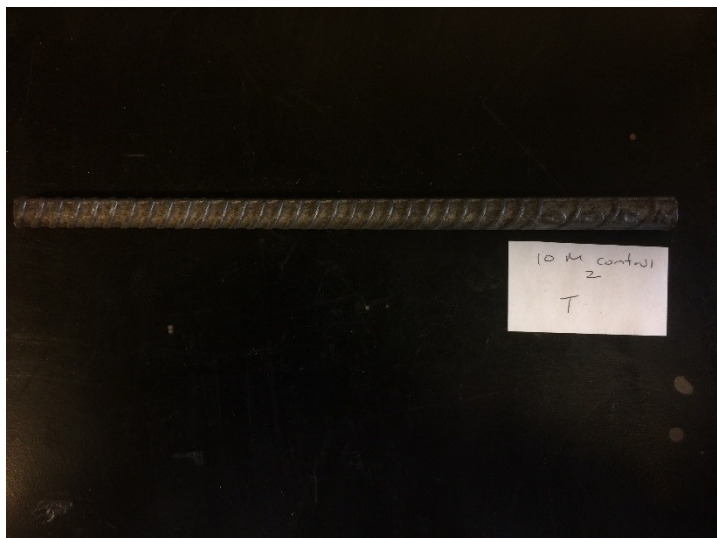


Figure 0.101: 10M Control 2T

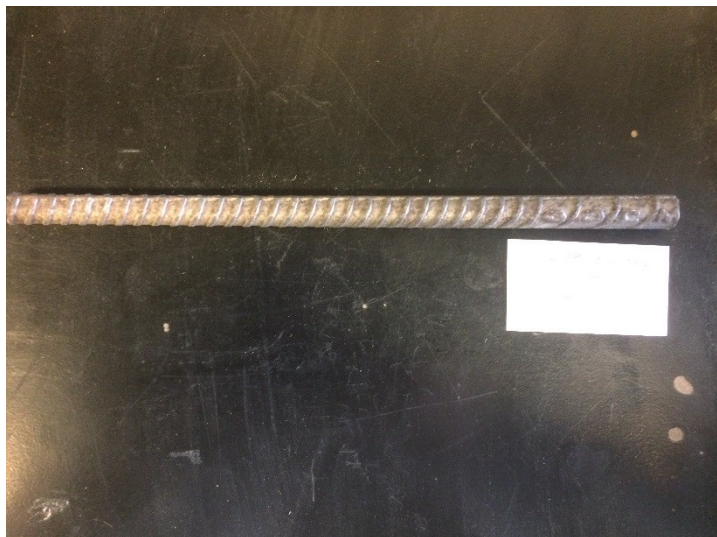


Figure 0.102: 10M Control 2T

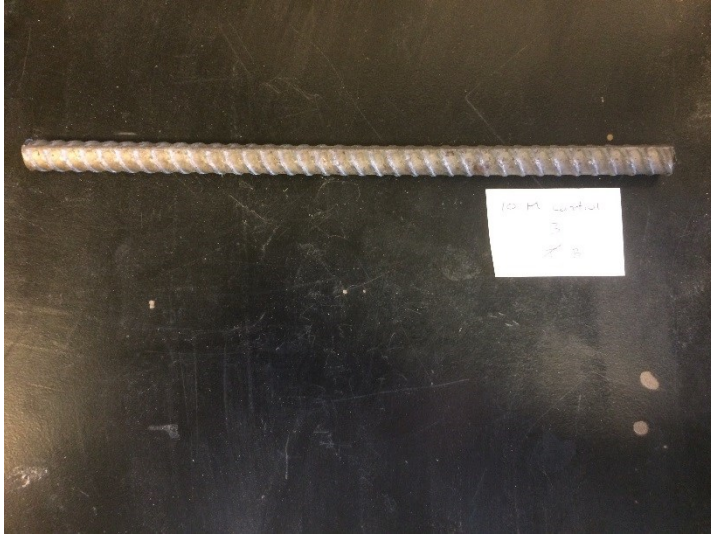


Figure 0.103: 10M Control 3B

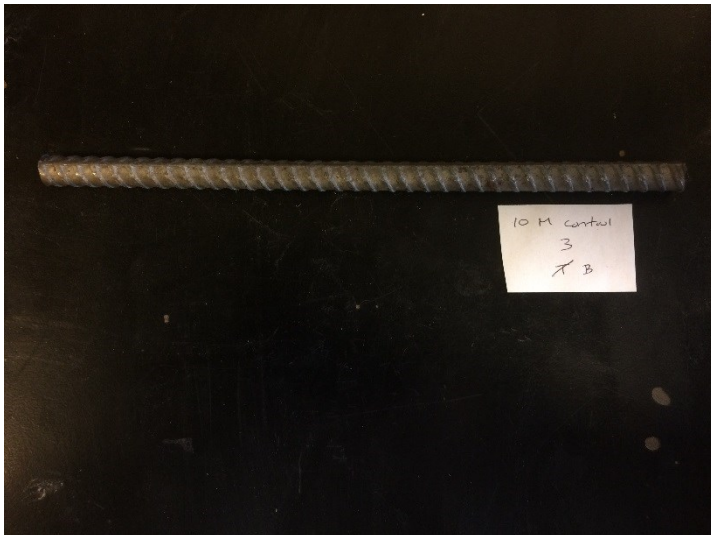


Figure 0.104: 10M Control 3B

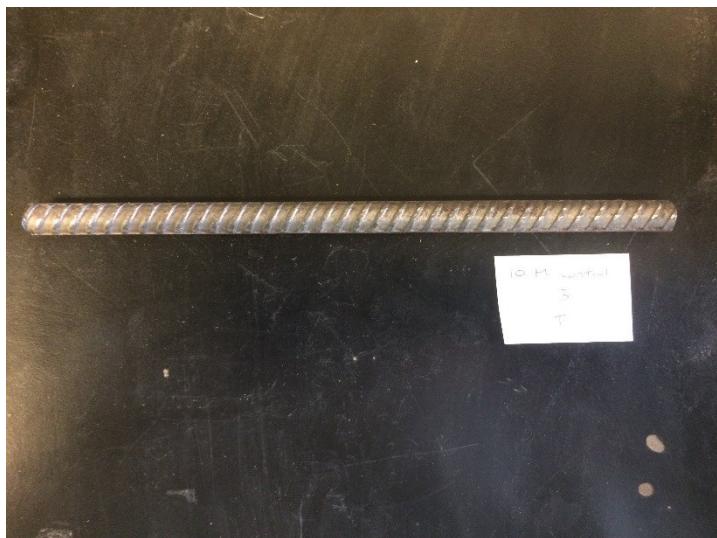


Figure 0.105: 10M Control 3T

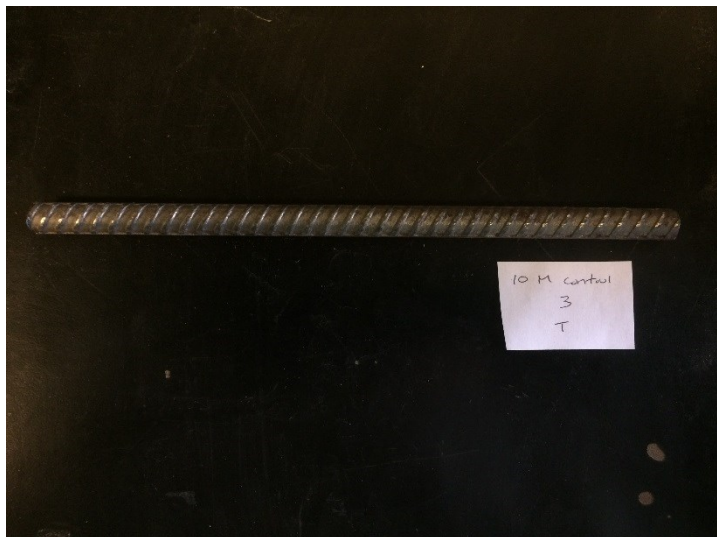


Figure 0.106: 10M Control 3T



Figure 0.107: 20M Control B

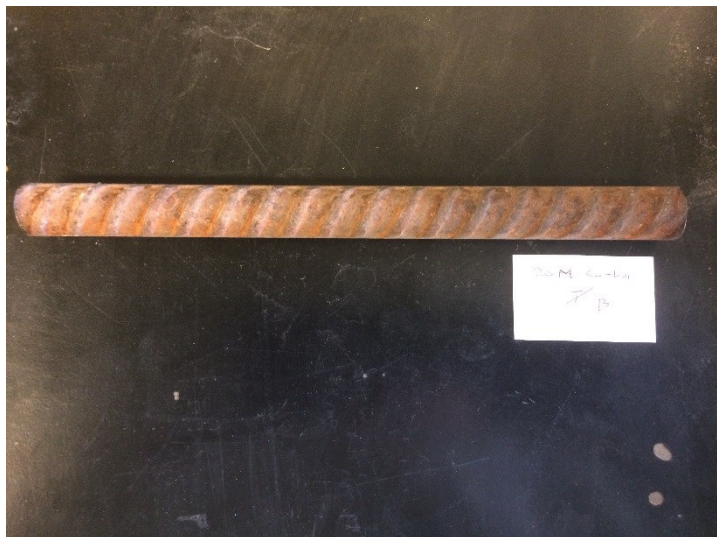


Figure 0.108: 20M Control 2B

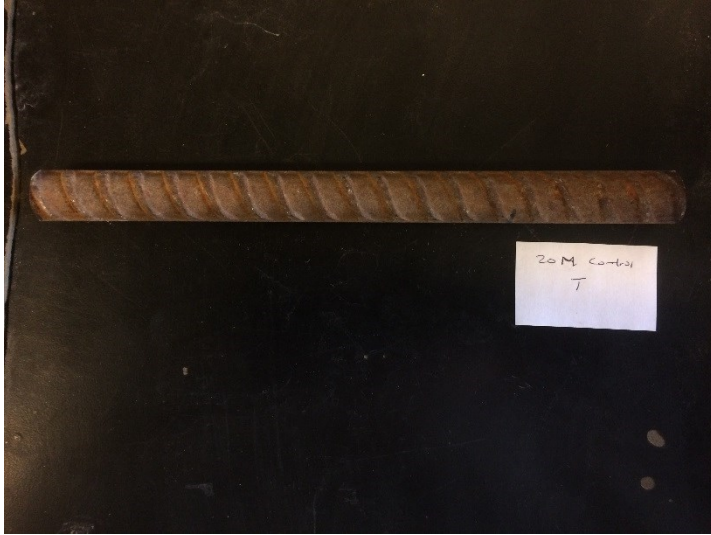


Figure 0.109: 20M Control T



Figure 0.110: 20M Control 2T

Table J.0.1: Laboratory Study Gravimetric Results

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Table J.O.2: Field Study: Gravimetric Results

Testing Date: September 12-13, 2016													
Rebar Density: 7.85 g/cm ³													
K= 87600000 um/y													
Lateral S.A.= Exposure Period (hrs): 1387584000	2*pi*L*r												
Rebar	Grid Location x	Grid Location y	Length (mm)	Initial W (g)	Weight after cycle 1	Weight after cycle 2	Weight after cycle 3	Weight after cycle 4	Weight after cycle 5	Weight after cycle 6	Final mass loss (g) "B"	Lateral Surface Area (cm ²)	Corroded Area (%)
Control 1			103.16	239.37	237.67	237.64	237.62	237.54	237.51	237.48			
Control 2			101.48	234.88	233.07	233.03	233.03	232.95	232.9	232.89			
Control 3			101.62	235.04	233.54	233.42	233.41	233.32	233.29	233.28			
Control 4			101.45	236.14	234.62	234.58	234.57	234.53	234.49	234.43			
S12	0	150	100.75	100.75	235.5	228.02	226.55	226.31	226.27	226.22	10.805236	61.68923	31.595
	0	100	101.21	101.21	235.5	226.57	223.52	222.46	221.69	221.52	15.353168	61.97088	59.195
	0	50	101.84	101.84	235.5	228.71	224.4	222.89	221.9	221.78	16.214503	62.35663	70.37
	0	0	102.11	102.11	235.5	229.42	227.14	226.79	226.64	226.6	13.480339	62.52195	42.97
	60	150	101.38	101.38	235.5	230.19	225.21	223.18	221.47	221.02	14.494848	62.07497	80.87
	60	50	101.8	101.8	235.5	230.82	225.41	222.71	220.97	220.55	15.742149	62.33214	77.365
	90	150	100.56	100.56	235.5	228.46	225	223.54	222.78	222.53	12.506052	61.57289	35.315
	90	100	99.76	99.76	235.5	217.29	215.12	214.16	213.76	213.69	20.281437	61.08305	61.69
	90	50	101.41	101.41	235.5	228.8	227.5	227.01	226.89	226.83	11.529547	62.09334	22.81
	120	150	101.02	101.02	235.5	227.55	223.14	221.7	220.82	220.59	15.477948	61.85455	77.805

	120	100	100.89	100.89	235.5	224.79	220.68	219.38	28.69	218.39	17.517157	61.77495	67.305
	120	50	102.73	102.73	235.5	224.55	220.18	218.13	216.93	216.51	22.732769	62.90158	75.715
	120	0	101.15	101.15	235.5	213.97	209.61	206.46	203.98	202.89	30.023240	61.93415	69.435
	150	150	100.9	100.9	235.5	228.06	226.51	226.31	226.21	226.14	11.172167	61.78107	20.54
	150	100	101.39	101.39	235.5	229.02	227.07	226.84	226.76	226.69	11.784956	62.0811	29.665
	150	50	100.58	100.58	235.5	227.14	223.8	223.1	222.81	222.75	13.399829	61.58513	49.625
	150	0	100.42	100.42	235.5	227.31	225.61	225.47	225.4	225.33	10.933708	61.48717	40.265
	180	150	100.72	100.72	235.5	229.39	227.56	227.44	227.42	227.37	9.6690806	61.67086	33.03
	180	100	100.04	100.04	235.5	227.65	225.74	225.62	225.57	225.51	9.2941624	61.25449	26.795
	180	50	100.97	100.97	235.5	227.83	226.35	226.3	226.24	226.18	11.442333	61.82393	22.43
	180	0	102.55	102.55	235.5	231.92	230.05	230.03	229.99	229.93	11.449281	62.79137	30.74
	210	150	100.9	100.9	235.5	230.81	229.03	229	228.99	228.88	8.5741151	61.78107	32.435
	210	100	100.51	100.51	235.5	229.75	227.97	227.94	227.87	227.83	8.7286874	61.54227	41.675
	210	50	101.11	101.11	235.5	230.53	228.77	228.76	228.69	228.67	9.3306397	61.90965	38.3
	210	0	100.82	100.82	235.5	229.17	227.45	227.41	227.39	227.32	9.9796945	61.73209	35.24
S13	0	0	100.68	100.68	235.5	224.71	221.65	221.37	221.26	221.18	15.55454	61.64636	60.18
	30	150	100.8	100.8	235.5	234.4	232.62	232.59	232.55	232.47	4.746172	61.71984	27.26
	30	100	100.59	100.59	235.5	234.32	232.81	232.75	232.67	232.64	4.270775	61.59126	34.295
	30	50	102.09	102.09	235.5	235.38	233.95	233.96	233.91	233.88	6.440797	62.50971	16.16
	30	0	100.62	100.62	235.5	233.98	232.53	232.51	232.49	232.45	4.407018	61.60963	25.58
	60	150	101.53	101.53	235.5	230.97	227.54	226.55	226.14	225.99	13.70901	62.16682	63.915
	60	100	101.13	101.13	235.5	233.92	232.16	231.95	231.87	231.84	6.073137	61.9219	94.625
	60	50	101.31	101.31	235.5	234.42	231.91	231.34	231.01	230.77	6.86488	62.03211	42.96
	60	0	101.15	101.15	235.5	233.54	229.08	227.69	227.09	227.7	9.621845	61.93415	63.235
	90	150	100.23	100.23	235.5	230.57	227	225.65	224.97	224.42	9.588224	61.37083	61.71
	90	100	100.96	100.96	235.5	234.48	233.12	233.04	232.99	232.41	4.69412	61.81781	19.49

	90	50	100	100	235.5	232.63	230.76	230.48	230.36	230.34	4.850507	61.23	36.585
	90	0	101.01	101.01	235.5	233.64	231.94	231.78	231.66	231.63	5.987381	61.84842	29.34
	120	150	101.44	101.44	235.5	233.51	232.24	232.2	232.1	232.05	6.646391	62.11171	12.295
	120	100	101.74	101.74	235.5	222.48	219.27	217.87	217.32	216.83	20.90553	62.2954	74.68
	120	50	100.55	100.55	235.5	231.4	229.66	229.37	229.19	228.99	7.210002	61.56677	35.305
	120	0	99.81	99.81	235.5	222.68	218.25	216.15	215.42	214.87	17.75574	61.11366	75.475
	150	150	101.58	101.58	235.5	232.49	229.1	228.35	228.17	228.02	10.45501	62.19743	70.27
	150	100	101.32	101.32	235.5	232.79	231	230.98	230.91	230.85	7.602371	62.03824	26.265
	150	50	100.34	100.34	235.5	231.45	229.51	229.44	229.4	229.28	6.801747	61.43818	27.975
	150	0	101.04	101.04	235.5	228.82	225.72	224.99	224.76	224.61	12.55368	61.86679	44.115
	180	150	99.1	99.1	235.5	228.25	226.56	226.55	226.47	226.42	6.81571	60.67893	13.225
	180	100	100.15	100.15	235.5	231.61	229.38	229.79	229.64	229.55	6.264287	61.32185	21.795
	180	50	101.15	101.15	235.5	233.46	231.63	231.58	231.52	231.43	6.582092	61.93415	20.445
	180	0	100.27	100.27	235.5	218.05	212.76	209.32	208.5	207.43	24.86963	61.39532	91.475
	210	150	100.62	100.62	235.5	232.69	231.04	230.85	230.85	230.79	5.985496	61.60963	22.53
	210	100	100.2	100.2	235.5	232.13	230.35	230.24	230.24	230.21	5.6536	61.35246	32.56
	210	50	101	101	235.5	234.53	232.82	232.74	232.74	232.68	5.05131	61.8423	19.28
	210	0	100.31	100.31	235.5	232.93	231.21	231.15	231.15	231.07	5.027725	61.41981	25.53
S14	15	150	100.38	100.38	235.5	232.16	230.14	229.91	229.77	229.67	6.350282	61.46267	43.575
	15	100	101.75	101.75	235.5	235.92	234	233.78	233.69	233.59	5.709797	62.30153	29.155
	15	50	101.11	101.11	235.5	234.33	232.75	232.68	232.64	232.59	5.376542	61.90965	27.275
	15	0	101.02	101.02	235.5	233.15	230.24	229.73	229.5	229.26	7.846781	61.85455	44.965
	45	150	101.24	101.24	235.5	236.44	232.94	232.49	232.32	232.2	5.672868	61.98925	64.465
	45	50	101.39	101.39	235.5	235.06	233.55	233.48	233.42	233.4	5.24362	62.0811	31.26
	45	0	100.55	100.55	235.5	233.26	231.76	231.7	231.66	231.63	5.051077	61.56677	19.815
	75	150	102.28	102.28	235.5	237.03	234.89	234.67	234.54	234.51	6.082194	62.62604	48.955

	75	100	101.72	101.72	235.5	236.64	235.22	235.13	235	234.99	4.354807	62.28316	32.205
	105	150	101.46	101.46	235.5	230.97	225.69	223.64	222.22	221.76	14.28773	62.12396	74.905
	105	100	101.81	101.81	235.5	223.5	220.29	218.93	217.83	217.4	20.11063	62.33826	59.52
	105	50	101.48	101.48	235.5	234.64	230.29	228.86	228.01	227.81	9.438186	62.1362	60.91
	135	150	100.71	100.71	235.5	234.39	232.52	232.29	232.18	232.12	4.75147	61.66473	40.98
	135	100	100.18	100.18	235.5	232.41	230.15	229.97	229.68	229.61	5.872077	61.34021	35.79
	135	50	101.59	101.59	235.5	233.58	232.38	232.33	232.29	232.23	6.870201	62.20356	23.185
	135	0	100.85	100.85	235.5	233.61	232.1	232	231.95	231.91	5.432993	61.75046	32.65
	165	150	101.35	101.35	235.5	236.08	234.53	234.45	234.44	234.42	4.169415	62.05661	26.14
	165	100	100.12	100.12	235.5	232.47	231.07	230.99	230.96	230.95	4.745645	61.30348	23.42
	165	50	101.05	101.05	235.5	234.16	232.7	232.64	232.6	232.57	5.2986	61.87292	25.07
	165	0	102.04	102.04	235.5	237.07	235.53	235.49	235.46	235.3	4.795295	62.47909	27.885
	195	50	100.02	100.02	235.5	231.62	230.02	229.95	229.95	229.92	5.547195	61.24225	29.385