University of Nevada, Reno

Assessment of In-situ Corrosion Conditions at Nevada Mechanically Stabilized Earth Wall Sites: Using Electrochemical Soil Characteristics and Linear Polarization Resistance

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Abstract

The inability of soil to provide sufficient tensile strength presents challenges for soils being used as a structural building material. However, it is possible to improve the structural performance with the inclusion of a reinforcing system. The development of these systems has been a major advancement of the civil engineering practice. Mechanically stabilized earth (MSE) wall systems typically consist of a: concrete facing panel, specified backfill, reinforcing elements, and the retained fill. The interaction of the backfill with the reinforcements, and the reinforcements with the facing panels, produces a system that when properly designed, can be a cost effective engineering solution. In Nevada there are over 150 MSE walls that have been constructed using metallic reinforcements (Thornley 2009). Corrosion of metallic elements a naturally occurring electrochemical process is irreversible an inevitable. The rate of metal loss (corrosion) is a function of the environmental conditions and metal type. For MSE walls key parameters include the backfill's: salt content, organic content, saturation level, as well as the metal type of the reinforcements.

Nevada has two previous corrosion investigations, an extensive site investigation at I-515/ Flamingo Rd. and a statistical analysis of as-built soil records along with a preliminary investigation for I-15/ Cheyenne Blvd. These studies form the foundation for this investigation of in-situ corrosion conditions. Seven MSE wall sites were investigated using electrochemical backfill characterization and linear polarization resistance (LPR) corrosion rate monitoring. Evaluation of electrochemical backfill characteristics has resulted in the discovery of six sites that fail current NDOT/ AASHTO MSE wall backfill requirements. The in-situ soil

samples collected and analyzed more than doubled the available data used to describe the corrosiveness of the backfill.

Linear polarization resistance corrosion rates were obtained for more than 200 different elements. These data suggest that despite the aggressive nature of the backfill, most elements are preforming well and are below the anticipated rates. However, several elements were discovered with corrosion rates in excess of five times the design model. The use of the LPR corrosion monitoring has concluded that the conditions at I-15/ and Cheyenne Blvd. are equivalent to or worse than the conditions evaluated in 2004 at the I-515/ Flamingo Rd. complex. The discoveries at Flamingo Rd. led to remediation of the largest wall at the complex.

Through the use of electrochemical backfill characteristics and LPR corrosion rates, the seven sites investigated have been ranked. The rankings are dependent on several factors such as backfill electrochemical conditions and comparison of corrosion rates data with design models. This study has confirmed that observations of conditions along the exterior of the wall are not sufficient when determining the condition of the soil reinforcements. Routine corrosion monitoring is required to monitor the depletion of the soil reinforcements and should be incorporated into a Long-term Corrosion Monitoring and Asset Management Plan (LCMAMP). It is anticipated that a program will be integrated into Nevada’s current asset management systems. The development and implementation of LCMAMP, directly reflects the federal initiative for systematic detailed evaluation of critical assets, MAP-21.

Dedication

This is dedicated to Bubbee and Zaydee who had faith in me even when I did not.

Acknowledgment

Special thanks to my wife Heather who has supportive of my educational adventures, without her continued support and none of this would be possible. I would like to thank my advisor Dr. Raj Siddharthan who has developed my passion for geotechnical engineering through countless colorful conversations and long hours. Dr. Ken Fishman, for his expertise, direction and the nearly boundless amount of information. Special thanks to Hairy Thompson for his expertise in statistical analysis and gentle guidance. I would like to also thank the dedication and support from NDOT personal as well as Barbra Luke and her group from UNLV. Finally, I would like to thank my children, Eyan, Hope and Isaac. They are constant reminders of humility that keeps me grounded and provide constant motivation for me to strive for success.

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Chapter 1 Introduction

The inability of soil to provide sufficient tensile strength presents challenges for use as a structural building material. However, it is possible to improve the structural performance with the inclusion of a reinforcing system. The development of these systems has been a major advancement of the civil engineering practice. Soil reinforcements are designed to stabilize a failure surface by providing restraining forces, which resist the movement of the soil along a potential failure surface. These resisting tensile forces ensures internal stability. This advancement has led to the development of modular retaining wall systems. These systems rely on mechanical reinforcements to stabilize the backfilled soils, thus the name mechanically stabilized earth (MSE) wall. The ability of MSE walls to provide a cost effective and reliable solution is a result of their capability to retain an earth mass behind a wall while supporting a variety of loading conditions. As a result of this, MSE wall construction is popular with many transportation departments. Mechanically stabilized earth wall systems typically consist of: concrete facing panel, specified backfill, reinforcing elements, and the retained fill (**Fig. 1-1**). The interaction of the backfill with the reinforcements, and the reinforcements with the facing panels, produces a system that when properly designed, often results in a cost-effective retaining system.

In Nevada there are over 150 MSE walls that have been constructed using metallic reinforcements (Thornley 2009). The ability of these reinforcements to provide the required tensile strength to stabilize the failure wedge is fundamental. It is well documented that metal corrodes when buried in soil. This naturally occurring electrochemical process is irreversible and inevitable. The rate of metal loss (corrosion) is a function of the environmental conditions and metal type. For

MSE walls key parameters include the backfill's: salt content, organic content, saturation level, as well as the metal type of the reinforcements.

To account for the section loss that will occur due to corrosion (metal loss), the required cross sectional area is increased with a sacrificial thickness (Elias 2009; Fishman and Withiam 2011). This additional non-structural thickness ensures that at the required tensile capacity of the reinforcement is not affected during its design life. Determining of the required sacrificial thickness to prevent premature failure of the systems is based on published metal loss models. These design models are dependent on the quantification the corrosiveness of environment (i.e. MSE wall backfill) in which the reinforcements are placed. To ensure the validity of these design models, MSE backfill has controlling electrochemical specifications imposed. These specifications are implemented as a set of pass/ fail tests. These test are intended to manage the rate of corrosion (i.e. retard the rate of metal loss) by limiting the electrolytic potential of the MSE backfill. Several metal loss models have been developed to predict the metal loss of corrosion for MSE walls, the most common being the American Association of State Highway and Transportation Officials (AASHTO) metal loss model (Elias 2009; Fishman and Withiam 2011; NDOT 2001; Romanoff 1957).

The application of a corrosion design model requires the assumptions used during its development be applicable for field conditions. If not, the application of the model is questionable. Nevada has two documented sites where the anticipated corrosion rate (i.e. metal loss model) drastically underestimated the in-situ field conditions, thus questioning the applicability of the design assumptions. They are I-515/ Flamingo Rd. and I-15/ Cheyenne Blvd., both in Las Vegas. The discoveries of elevated corrosion at both sites occurred by accident, while the sites were under construction for other improvements (Fishman 2004; Thornley 2009).

The discovery at I-515/ Flamingo Rd. occurred during the excavation of the foundation for a new sound wall. During the excavation for the wall's foundation the upper layers of the MSE wall soil reinforcements were exposed. These reinforcements showed obvious signs of aggressive corrosion. The Flamingo Rd. site subsequently became the focus of a 2005 report, prepared by McMahon & Mann Consulting Engineers (MMCE) (Fishman 2004). The report concentrated on evaluating corrosion rates, characterizing existing electrochemical soil properties, metallurgical testing and the impact of accelerated corrosion on the service life for the three MSE walls at the site.

In similar manner, corrosion concerns were identified at the I-15/ Cheyenne Blvd. site during the demolition of a MSE wall for a lane-widening project. During this activity several MSE wall reinforcements were removed and were available for direct observation. The observations indicated advanced metal loss and aggressive backfill conditions. A report prepared by the Universities of Nevada, Reno and Las Vegas (UNR and UNLV, respectively) attempted to address the corrosion problem along with a statistical analysis of MSE backfills for all walls in the state's inventory. The report also included investigation of the methods used for characterization/ specification of the backfill (Thornley 2009). At both sites the discoveries of the elevated corrosion was accidental and led the Nevada Department of Transportation (NDOT) to be concerned, and to question how extensive the corrosion damage is.

1.1 Development of Current Study

The discovery of excessive corrosion at I-515/ Flamingo Road initiated a joint decision between the Federal Highway Administrations (FHWA) and NDOT to provide remediation for the largest wall at the site and to perform a rigorous site investigation. The remediation of the wall included the construction of a cast-in-place tie-back wall, in front of the existing MSE wall. This was done concurrently with the McMahon & Mann investigation, led by Dr. Kenneth Fishman. The investigation was multifaceted, focusing on the backfill environment, reinforcement system, and structural capacity. To accomplish this, eleven monitoring stations were constructed at which time soil samples were also collected for testing of the electrochemical characteristics. These stations were then used to monitor the in-situ uniform corrosion rate of the reinforcements using the Linear Polarization Resistance (LPR) technique. The exposed reinforcing layers and exhumed sections were measured for section loss, and reinforcement samples underwent metallurgical testing. The site's significance is due in part to its level of documentation, costly mitigation and surprise discovery. Although not as well documented, the I-15/ Cheyenne Blvd. site is significant. The discovery at Cheyenne Blvd. confirmed that the issues of excessive corrosion was not isolated, to the Flamingo Rd. site. Thus it was a catalyst for NDOT to examine and instrument other sites in its MSE wall inventory.

Concern regarding the extent and identification of other sites with potential for aggressive corrosion was the foundation of the initial University of Nevada, Reno study (Phase I). Initiated after the Flamingo Rd. discovery, it was focused on identifying other sites that have the potential for aggressive corrosion through securitization of as-built records. During this process the discovery of the I-15/ Cheyenne Blvd. elevated corrosion confirmed that Flamingo Rd. was not an isolated situation. The report consisted of the following: a statistical analysis of backfill records to identify other sites that may be experiencing advanced metal loss, preliminary evaluation of the I-15/ Cheyenne Blvd. corrosion situation and re-evaluation of Flamingo Rd. complex. The report was completed in 2009.

As with Flamingo Rd., the discovery of excessive corrosion at I-15 and Cheyenne Blvd. was accidental. However, unlike Flamingo Rd. this discovery did not result in site mitigation. The Cheyenne Blvd. discovery occurred while a MSE wall was being removed, at which time samples of the soil reinforcements were gathered allowing for section loss measurements and backfill samples collection. The metal loss measurements were used to estimate the uniform corrosion levels throughout the site. Soil samples were collected and tested for electrochemical properties. This information was incorporated into larger database and subsequently used to identify seven sites in the state that have a higher likelihood of experiencing similar conditions.

NDOT tasked UNR and UNLV with conducting field investigations (Phase II) to assess in-situ corrosion rates at the sites that were identified in the 2009 report. This was achieved through the installation of 38 corrosion monitoring stations and collection of 74 backfill samples. The corrosion monitoring stations were used to collect uniform (idealized) corrosion rates using the LPR method. This information in conjunction with electrochemical soil testing results have been used to provide recommendations to the NDOT for further studies and allows for the creation of a Long-term Corrosion Monitoring and Asset Management Program (LCMAMP). It is anticipated that LCMAMP will be combined with Nevada's current asset management systems. The development of this program and implementation of long-term corrosion monitoring is a direct link to MAP-21, a federal law which emphasizes systematic detailed evaluation of critical assets (MAP-21 2012).

1.2 Project Information

This project is a continuation of the corrosion evaluation work that was conducted by the Universities of Nevada, Reno and Las Vegas (Phase I) and addresses NDOT's desire to identify and quantify corrosion related issues within the state's MSE wall inventory. The intention of the project is to quantify the corrosion levels at the sites that were identified during the first phase. The scope of the project is provided below.

1.2.1 Scope of the Phase II Investigation

To achieve the goals of establishing a long-term corrosion monitoring and asset management program and providing a quantitative evaluation of the corrosion rates, the following major tasks were undertaken in Phase II study:

1. Gather a new set of corrosion measurements from the I-515 and Flamingo Rd. complex using the monitoring stations that were installed previously and provide analysis of the progression.

2. Conduct site specific electrochemical soil studies at the seven sites that were identified as possibly problematic in the Phase I study.

3. Install nonstructural (e.g. steel and galvanized steel) coupons at the sites identified in Phase I and gather an initial set of corrosion rate measurements using the LPR method. LPR measurements included both the steel and galvanized coupons, as well as, in-service reinforcing elements.

4. Develop and refine service life estimates using the corrosion rates obtained with the LPR method.

5. Prepare a priority list for future and continued corrosion monitoring.

6. Suggest remediation, if necessary for problematic sites.

As it is observed this plan is intended to be thorough and systematic in its approach. This is to ensure that the MSE wall corrosion investigation is addressed in a comprehensive manner.

1.2.2 Organization of Report

This report is divided into seven chapters. Chapters 1 and 2 are intended to provide context of this research and supply the necessary background information to put the data presented into perspective. The information provided is not intended to create proficiency but, familiarize the reader with the materials contained in the following chapters. Many of the topics that are introduced in Chapter 2 are expanded upon in relevant chapters.

Chapter 3 provides details on the electrochemical soil testing and specifications as they pertain to MSE wall corrosion. This is done with a review of the current testing parameters and test standards and their influence on the corrosion process. The testing results and analysis of this phase of research are then discussed. In-situ corrosion rates are presented in Chapter 4. This includes discussion of metal types, site specific corrosion rates, and corrosion severity ratios. Available metal loss models are presented for comparison with the field-measured data. Application of this data presented in Chapter 5 in which cumulative distribution functions are derived. Conclusions are presented in Chapter 6, on an individual site by site basis for backfill conditions, in-situ corrosion rates, and then subsequently in a combined dataset. Finally recommendations are provided in Chapter 7 after the case for each of the recommendations has been developed.

Chapter 2 Background

In order to better understand the current investigation of mechanically stabilized earth (MSE) wall corrosion in Nevada, background information is presented. It is not the intention to provide complete understanding, but rather to give context to the current methodologies. Underground corrosion studies have been conducted in a variety of environments and conditions depending on the interested agency. While not all the studies are directly applicable to MSE walls, they all had an influence on the state of practice and provided insights to the mechanisms present during underground corrosion. A majority of the studies that have direct relevance are the result of a state's department of transportation reacting to the discovery of a corrosion issue. These studies have been documented in reports from many states including but not limited to: Idaho, Florida, Kentucky, Nevada, Utah, and California (Armour, et. al. 2004; Sagüés, Alberto A. et. al. 1998; Fishman 2005; Salazar and Hilfiker 2005; Beckham et. al 2005; Billings 2011; Jackura et. al 1987).

2.1 Underground Corrosion

The history of underground corrosion literature demonstrates the longevity and complexity of the problem. Initially concerns regarding underground corrosion were primarily of the interest of the oil and gas industry due to the extensive networks of subterranean pipes. Driven by the fear of lost profits and environmental impact if failure were to occur, studies were conducted to understand the phenomena. While, there is still great interest from the petroleum industry, the inclusion of reinforcing steel in geotechnical engineering applications (e.g. MSE walls) has once again elevated the concern and brought underground corrosion into the limelight.

The influence of underground corrosion on infrastructure is vast, including the deterioration of pile foundations, the loss of tensile capacity of rock-bolts and soil reinforcements. The corrosion process is characterized by the disassociation of metallic ions, which overtime results in a reduction of cross-sectional area. Accounting for the loss of cross-sectional area over time is critical for the safety of the structure and the public. With the technical advancement and acceptance of MSE walls, the necessity to understand and quantify underground corrosion is vital. Corrosion (i.e. metal loss) directly affects the structural capacity of the system. If the corrosion process is not properly accounted for during the design, premature failure of individual reinforcements or reinforcing systems can lead to local and global wall failures.

2.1.1 National Bureau of Standard Circular 579

The National Bureau of Standards (now the National Institute of Standards and Technology) conducted an extensive investigation on underground corrosion (Romanoff 1957). This publication is generally seen as the foundation and preeminent study on underground corrosion. The study was conducted from 1910-1955 throughout the United States and encompassed a variety of soils and metals, many of which are not applicable to typical MSE walls. This study concluded the process of underground corrosion is predominantly an electrochemical process. The study also identified soil characteristics which have significant influence on this process. As a result of the study a metal loss model was developed using periodic maximum pit depth measurements. The predictive metal loss model is a function of exposure time and metal type, given by:

$$x = kt^n \qquad \textbf{(2-1)}$$

- x = maximum pit depth [mm]

- *k* and *n* are metal type constants
- *t* = exposure time [years]

The values of k and n are empirically derived for various metal types. This equation reflects the observation that corrosion rates attenuate over time due to passivation, and is modeled with *n* being less than unity (Romanoff 1957; Fishman et al. 2011; Elias et al. 2009).

2.1.2 Underground Galvanic Corrosion Fundamentals

Corrosion can be defined as the process of a material, usually a metal, deteriorating due to a reaction with its environment. This process is irreversible and inevitable. The environmental conditions surrounding the metallic object have been shown to have a significant effect on the process (Decker et. al. 2008; Elias 2009; Ghods et. al 2013; Padilla 2013; Romanoff 1957). Environmental conditions have the ability to accelerate or retard the rate of corrosion by orders of magnitude. Underground corrosion naturally occurs as a degenerative electrochemical process which requires the following conditions: (i) an electrolyte to allow flow of ions, (ii) electric potential to stimulate ion transfer, and (iii) the presence of oxygen to allow an oxidation/ reduction process to occur (other electrically active species are possible). In the case of MSE walls, due to the construction sequence and requirement of a free draining backfill, oxygen typically is not a limiting reactant. The soil acts as an electrolyte, consisting of salt ions and moisture, and electric potential is provided by micro-imperfections along the reinforcement's surface. This creates an electrochemical corrosion circuit. The corrosion process is manageable through controlling of any one of the three contributors (**Fig. 2-1**).

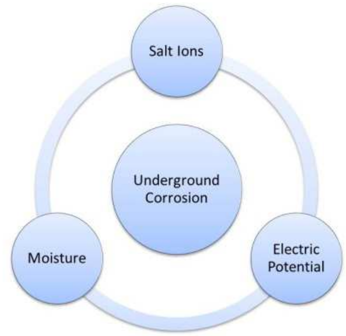

Fig. 2-1: Interdependent relationship required to create a corrosion circuit

Control or elimination of the electric potential is beyond any reasonable effort given the scale of most infrastructure projects. Microscopic imperfections along the elements surface, which are naturally occurring, provide sufficient electric potential to produce corrosion. The elimination of these imperfections is nearly impossible and the additional cost for protective coating, typically do not justify their inclusion. However, limiting the electrolytic potential of the soil can be accomplished through the use of a “select” backfill. The electrolyte results from the dissolution of naturally occurring salts into the moisture of the backfill soils. Moisture and salts are naturally present in all soils. Additionally, water is required to achieve soil compaction and generally is used for dust control during construction. Thus, the emphasis should be placed on controlling the salt content. This is accomplished through a series of pass/fail specifications (e.g. AASHTO 2012 and Nevada silver book 2001) that are intended to limit the maximum amount of the salt ions. These specifications include limits on: Chlorides, Sulfates, and minimum soil resistivity (an indirect measure of total dissolved salts). The objective of these specifications is to reduce the electrolytic capability of the backfill and thus manage the corrosion process.

Along with controlling the electrochemical corrosion environment, materials that are less susceptible to the electrochemical process (i.e. less reactive with environmental conditions) are often used. The customary method for MSE wall construction is the use of galvanized metallic reinforcements. The galvanization process applies a thin layer of zinc on the exterior of the base steel. The zinc layer then acts as a sacrificial anode. Upon reacting with the environment the zinc reacts to create zinc oxide, which eventually transforms into zinc carbonate (Padilla et. al. 2013). This presents itself as a dull gray/ white layer on the surface of the object **Fig. 2-2.** This layer then allows passivation of the base steel and hinders the progress of corrosion.

The process of galvanic corrosion is complex involving interactions with the backfill soils and the galvanized reinforcement. The process consists of both electrochemical and physical interactions that form a symbiotic process that is best described by three distinct phases. These phases are illustrated in the **Fig. 2-3** (Padilla et. al. 2013).

Stage I is characterized by elevated corrosion rates as a zinc oxide layer is formed from interaction with the air. In Stage II, the corrosion rate is reduced as a layer of zinc carbonate forms along the surface, resulting in a new alloy which allows passivation of the base steel. Stage II lasts until substantial portions of the zinc are consumed and the base carbon steel is exposed. Finally in Stage III, the amount of red rust present rapidly increases as the base steel is being consumed. This stage is characterized by a uniform corrosion rate, which matches that of an untreated steel element. Despite there being patches of zinc remaining along the surface, the zinc no longer acts as a sacrificial anode and the consumption of the base carbon steel is the dominant process. (Ghods et. al. 2013; Padilla et. al. 2013)

2.2 MSE Wall Corrosion Literature

To provide context and understanding of MSE wall corrosion a partial review of the available literature is provided. Underground corrosion is a very prevalent concern and has had numerous studies conducted to understand the governing mechanisms. There are numerous examples where MSE walls have performed successfully. However, there are a few well-documented field cases where the effects of corrosion have led to premature failure of the reinforcement system (Armour and Pfister 2004; Blight and Dane 1989). In each of these cases poor backfill conditions (i.e. more aggressive soils) or wall conditions (i.e. poor construction methods) allowed the effects of corrosion to propagate into a failure. To understand the current practice of MSE wall backfill specification, a review of the specifications and reference materials are provided. This is supplemented with information on each of the governing electrochemical soil parameters, and acknowledgement of several other soil characteristics that are influential in the corrosion process.

Due to the many uncertainties present, monitoring of the corrosion phenomena is important and recommended (Elias et. al. 2009; Fishman 2007). The development of corrosion monitoring programs is reviewed. This includes the instrumentation of the site, the construction of sacrificial coupons and connection to the in-service reinforcements. To understand the technique used for monitoring, fundamentals of the galvanized corrosion process are discussed along with an introduction to the Linear Polarization Resistance (LPR) technique.

After reviewing the development of MSE wall corrosion specifications and monitoring, a brief examination of the Nevada corrosion investigations is presented. This is intended to provide justification for the continued interest and the need for this research effort. Two Nevada case histories demonstrate the potential corrosion issue within the state and serve as the basis for the current study. To understand the scope of the current study, a review of the first phase is presented. The first phase is responsible for the selection of the seven sites that are the focus of evaluation for this study (Phase II).

2.2.2 Federal Highway Administration Publications

Starting in 1989 the Federal Highway Administration (FHWA) initiated a series of studies and they resulted in guidelines and manuals which are intended to characterize the long-term performance of soil reinforcements. The original study was a five year effort to: consolidate and summarize the available data, provide a method for evaluation of both metallic and geosynthetic reinforcements and, develop a test/ monitoring method for corrosion rates (Elias 1990). This report is the foundation for the current practice of in-situ MSE wall corrosion rate monitoring, and resulted in the development of the FHWA's monitoring equipment. The report has been updated and revised in the two proceeding publications.

2.2.2.A FHWA-RD-89-18B (1990): Durability/ Corrosion of Soil Reinforced Structures

This can be viewed as the start of a directed interest by the FHWA to address the corrosion related issues associated with MSE walls. The first task was to characterize and evaluate the backfill parameters that control the corrosion process. Nine different governing parameters were identified: soil resistivity, moisture content, soluble salts, pH, oxygen-reduction potential, soil compaction, oxygen transfer, organic materials and soluble iron content (Elias 1990). Many of the parameters are interrelated. Among the above nine parameters, five have had specifications placed on them in an effort to control the aggressiveness of the MSE backfill. Each of the five parameters that have been specified have a more thorough discussion in later text.

Another major task undertaken in the 1990 publication is the development of a method to investigate and evaluate the corrosion process. This resulted in the development of a field capable LPR corrosion monitoring equipment, PR 4500 by CC Technologies. This equipment was designed for use in the field, and allowed the user to measure instantaneous, in-situ, uniform (idealized) corrosion rates on a corrosion susceptible element. Along with the development of the equipment, a framework for a monitoring program was established, including a monitoring schedule. The program identified the need to provide sacrificial samples (coupons) to be used for comparison with instrumented in-service elements. This program and equipment was intended to provide a framework for the long-term monitoring of MSE wall soil reinforcements, to ensure that the reinforcements would provide the required tensile capacity for the system.

2.2.2.B FHWA-NHI-00-044 (2000): Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes

In 2000 an update and revision to the 1990 FHWA report was completed. This report was again intended for use as reference by practicing engineers involved in the design of MSE walls and reinforced slopes. The new manual elaborates on current metal loss models. This includes the segmented linear model that is used by the American Association of State Highway and

Transportation Officials (AASHTO) and provides updates to the Romanoff equation (**Eq. 2-1**).

Justification of the Romanoff equation with the current MSE wall practice, is provided. Romanoff derived the equation from analysis of maximum pit depths, however current practices assumes MSE wall corrosion is uniform over the element's surface. The justification is accomplished through an explanation of the applicability of the equation and notes on the necessity for conservatism in modeling. It is again acknowledged that the development of the Romanoff equation (**Eq. 2-1**) is based on a large range of soil conditions and may not completely characterize the MSE wall backfill conditions at a given site. Numeric values of *k* and *n* are updated for both galvanized and plain steel elements (Elias 2000).

2.2.2.C FHWA-NHI-09-087 (2009): Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes

This is the latest revision in the series of FHWA manuals. In a similar manner of the preceding versions, this manual compiles the information available and provides specifications, procedures and monitoring guidelines for reinforced soil structures. Knowledge gained in the decade between publications are highlighted. The manual is the current reference manual (Elias et al. 2009) for engineers involved with the design of reinforced soil slopes or MSE walls.

2.2.3 NCHRP Report 675 LRFD Metal Loss and Service-Life Strength Reduction Factors for Metal-Reinforced Systems

The 2011 National Cooperative Highway Research Program (NCHRP) published report 675 (Fishman and Withiam 2011), which is a recent effort to understand and characterize the effects of MSE wall corrosion. This report focuses on the two most common types of metallic reinforcements and systematically address all the characteristic parameters associated with the corrosion process for each element type. It addresses the progress of the practice of civil engineering by promoting the consideration of MSE wall corrosion from an allowable stress design (ASD) to the current methodologies of load reduction factor design (LRFD). To accomplish this task a national database was constructed to determine and calibrate the strength reduction factors. In the process of accomplishing this task several metal loss models were evaluated against the complied database.

Monitoring of corrosion rates was also addressed in this report, comparing several of the different methods available to measure the in-situ corrosion rates. As part of this a close evaluation of LPR method is provided and the equipment developed under the FHWA is compared with a commercially available product (e.g. Gamry Instruments). The corrosion rates that were measured using these equipment were also compared to physical metal loss measurements. Together, these data provide a method to monitor performance of the reinforcements and justification of the LPR method.

2.2.4 Caltrans-Interim Design Procedures

The California Department of Transportation (Caltrans) has a network of sites for investigation of MSE wall corrosion. Beginning in 1979 Caltrans has installed sacrificial reinforcements into various MSE walls, with the intention of their removal for corrosion investigations. Caltrans conducted a statewide corrosion investigation that included 14 MSE walls, during which a verity of backfill conditions were encountered (Thornley 2009). These findings led Caltrans to develop corrosion design models that incorporates backfill conditions that differ from that of AASHTO (Jackura et. al. 1987). The specifications used by Caltrans include:

resistivity values as low as 2,000 ohm-cm, pH between 5.5 and 10.0, sulfates less than 500 ppm and chlorides less than 250 ppm. In conjunction with the loosen backfill requirements, an associated metal loss model was presented. The model includes the assumption that the zinc coating last only 10 years (as opposed to 16 years under AASHTO), a design life of 50 years (instead of 75 or 100 years), as well as constants for use in a metal loss model (**Eq. 2-2**), values for C and K are provided in **Table 2-1.**

$$X(\mu m) = \left(t_f - C(yrs)\right) \times K\,\frac{\mu m}{yr} \qquad \textbf{(2-2)}$$

Where:

- X – metal loss per side [μm]
- t_f – design life
- C – the time to zinc depletion
- K – corrosion rate of base steel

The designer was required to select the appropriate metal loss model constants based on the available source material (Fishman 2012).

In January 2014, Caltrans released an update that modified the AASHTO corrosion requirements to allow for less restrictive backfills. The Caltrans modified metal loss model, specifies: the loss of galvanization occurs at 10 years and base steel depletion occurs at 1.1 mil/ year (28 μm/yr). The updates also clarifies that all permanent retaining walls should be designed for a 75-year service life. The Caltrans method requires 1,820 μm of sacrificial thickness per side compared with the 708 μm that AASHTO requires, which is an increase of 157%.

2.3 Electrochemical Soil Parameters

Starting with the 1910 (Romanoff) NBS study, an effort has been placed on understanding the mechanisms that control underground corrosion. In the case of mechanically stabilized earth structures, a greater emphasis has been placed on controlling the aggressiveness of the environment in which the critical tensile reinforcements are placed. This is to ensure that the end of the design life there is adequate tensile capacity. While the underground corrosion process is complex and there are numerous parameters that affect the rate of this process, five key electrochemical soil parameters have been identified, subsequently each of these has had a limits placed on them. The series of pass/ fail tests are intended to create a backfill condition that can be classified as moderately aggressive according the NACE classification (**Table 2-2**) and in doing so allows for the prediction of the metal loss and corrosion rate. The following sections are intended to provide an introduction to each of these parameters.

2.3.1 Minimum Soil Resistivity

Minimum soil resistivity is readily accepted as a critical predictor of the corrosiveness of the soil (Elias 1990; Elias 2000; Elias et al 2009; Fishman 2011; Padila et. al. 2013). While there is a general trend that resistivity and corrosion rates are negatively correlated, soil resistivity alone cannot adequately predict corrosion rates. Resistivity is a measure of how strongly a material opposes the flow of current. Materials with low resistivity values present little resistance to the flow of current, thus do little to impede the disassociation of metallic ions. Originally developed by the agricultural community as a measure of the total dissolved salt content of a soil, resistivity is an indirect measure of electrolytic potential of the soils. There are several different test methods for determining this value; however, for many of the test the intention is to determine the minimum value. The minimum value was chosen as a benchmark due to its repeatability and consistency. Soil resistivity is very sensitive to moisture content and the use of a minimum value, which normally occurs around the saturation limit, allows for a consistent benchmark. While the minimum value typically occurs around full saturation, the AASHTO test method (T-288) allows for a sample to proceed pass this point in order to achieve a stable minimum value. The minimum value is conservative and is usually obtained at a condition that ideally does not occur under normal operation (i.e. full saturation or beyond) of a MSE wall. The test targets the minimum value and therefore, generally does not represent in-situ conditions surrounding the metallic soil reinforcements.

Controlling electrolytic potential is the primary purpose of the specification. The limit has been placed at a minimum of 3,000 ohm-cm. The rationale behind this limit is based on both practical and scientific reasoning. While minimum soil resistivity does not predict corrosion rates (Elias 2009; Fishman 2009; Ghods et. al. 2013; Thornley 2009), soils which are below this limit tend to demonstrate considerably more scatter in the data, with many values above the current design models. As a practical limit, this value (3,000 ohm-cm) ensures that backfill materials sources are readily available throughout the United States (Elias 1990), while still providing control of the electrolytic potential of the backfill. Backfill descriptive classifications are based on the soil resistivity as described in NCHRP Report 20-50, 1978, which resulted in classifications from very corrosive to non-corrosive (**Table 2-2**).

2.3.2 Chloride Content

Chloride is one of many salts that are present in most fill materials. It is typically very soluble and thus is easily extracted in a laboratory setting. Solubility, while beneficial for laboratory testing, is detrimental to corrosive performance of the MSE wall reinforcements. The electrolytic potential and capacity is directly proportional to dissolved salts. Thus, chlorides play a significant role in the aggressiveness of the backfill.

2.3.3 Sulfate Content

Sulfates require a greater effort for extraction and (Elias et. al. 2009) soluble sulfate may not characterize the complete sulfur content of the backfill soils. However, sulfates and sulfides can cause severe deterioration of the reinforcements, and have been proven to impede the passivation of the base carbon steel. Thus, the presence of sulfate may negate the benefits of from the galvanization process. When dissolved sulfates increase the electrolytic potential of the backfill and increase the aggressive potential of the backfill.

2.3.4 pH

Another important indicator of a soils aggressiveness is pH (Elias 2009; Romanoff 1957). It represents the concentration of hydrogen ions, and can be seen as an intensity indicator of the soil solution. A major factor that influences backfill pH values is the concentration of dissolved salts. As the amount of salt increases, the pH will also increase and the soil trends to be more basic. Soils with pH values at either extreme end ($pH < 4.0$ or $pH>10.0$) tend to be very aggressive and accelerate the electrochemical corrosion process. This should be expected as these values are also associated with strong electrolytes. Therefore, limits are placed to contain the backfill pH to levels near a neutral pH of 7.0.

2.3.5 Organic Content

Soils with high levels of organic material tend to support microbial growth. This then presents the opportunity for organic acids to form. These acids can produce pitting type corrosion. The organisms also have the ability to create pockets of anaerobic conditions which promote the development of sulfate-reducing bacteria that can initiate severe macro-cell pitting corrosion (Elias et. al. 2009). Organic content of the soil may increase over the life of the MSE wall, as a result of materials infiltrating the backfill. A common source of this is fertilizers.

2.3.6 Other Influential Soil Characteristics

The process of underground galvanic corrosion is a very complex phenomenon, in which many measureable and un-measureable factors influence the process. While the current specifications are intended to characterize the corrosive potential of the soil, there are several external influences that can play a significant role in either retarding or accelerating the deterioration of the reinforcement. These include stray electrical current, differential aeration, moisture content and backfill soil temperature.

Each of the aforementioned has been proven to influence the corrosion process. Stray electrical currents which originate from an external source were originally thought to be the source of all underground corrosion (Romanoff 1957). Mitigation of this phenomena usually occurs by limiting the number of utilities that are in close vicinity to the MSE wall backfill. The corrosion process is an oxidation-reduction reaction such that the lack of oxygen present in the soils can provide a limit to the corrosion process. Differential aeration usually occurs near the concrete facing of the MSE walls where variable compaction and the use of differing soil gradation (e.g., drainage layer) usually exist. This creates zones where the corrosion process is accelerated along the soil reinforcement and results in macro cell corrosion (Ghods et. al. 2013; Padilla et. al. 2013). As is the case with most electrochemical process, the rate of the reaction is sensitive to the environmental temperature (Escalante 1988).

2.4 Corrosion Monitoring

Verification of design assumptions and performance evaluations are fundamental to civil design projects. Validation of the design assumptions, related to MSE wall backfill soil specifications and metal loss models is important. This allows the owner to be confident that the structure will provide the required serviceability and safety. Starting in the early 1900s the United Sates government started to pay special attention to underground corrosion. The results from the 45 year study on underground corrosion (1910 NBS study) forms the foundation for underground corrosion evaluation and monitoring. In the 1990s the government sponsored the development of the Polarization Resistance Monitor PR 4500. Designed and developed for the FHWA with specific application to MSE walls, the equipment was intended to be used to collect instantaneous in-situ uniform corrosion rate measurements for the purpose of MSE Wall performance evaluation. The recommendation for monitoring corrosion is reiterated in literature and emphasized by the few documented cases of poorly performing MSE walls due to greater than anticipated corrosion rates (Armour 2004; Fishman 2005).

Corrosion is an electrochemical process; therefore an associate current flow is essential for the process. If an accurate assessment of this current can be made and the element's surface area known, then a direct calculation of the corrosion rate is possible. The resistance measured on the element is inversely proportional to the corrosion rate. This concept is at the heart of Linear Polarization

Resistance (LPR) monitoring technique recommended by the FHWA (Elias 1990; Fishman and Withiam 2011; Fishman 2005). The method has been implemented in the current study and past site investigations with the state of Nevada.

Long-term monitoring of in-situ corrosion rates is needed to capture and characterize the various stages of the galvanic corrosion process. It is recommended that frequent measurements be made in the infant stages (i.e. less than 2-years of service) of the MSE wall life. Then as the age of the wall increases and the corrosion process stabilizes, the frequency of measurements can be reduced (e.g. annually or bi-annually). This information can then be incorporated into a MSE wall Long-term Corrosion Monitoring and Asset Management Program (LCMAMP), and used to determine/ warn of impending failures.

2.4.1 Installation of Monitoring Stations

In order to monitor in-situ corrosion rates, corrosion monitoring stations are located along the face of the MSE wall. These stations consist of one or more of the following: connection to in-service reinforcements, sacrificial carbon steel coupon, and sacrificial galvanized steel coupon. The monitoring station installation spans the height of the wall, and should allow the instrumentation to be localized near ground level for ease of data acquisition. During installation of monitoring station backfill samples may be collected. Initially this is an intrusive process requiring access to the backfill soils and reinforcements, typically achieved with the use of a core trough the concrete facing panel. Once access is gained, soil samples can be collected and monitoring elements can be installed. After creation of the monitoring stations corrosion rate data collection is non-destructive, allowing for the long-term performance of instrumented elements to be collected with relative ease. During the installation of the stations it is imperative that care be taken so that connections to the elements do not influence the corrosion process or measurements.

The arrangement of stations is intended to obtain spatially distributed measurements throughout the entire wall such that it can be reasonably assumed that the samples collected/monitored are representative of the wall site. Due to the small number of samples collected emphasis is placed on locations; that show probable signs of distress, near drainage inlets or locations of greatest wall height. This is done with the intention of capturing poor performing in-service elements, thus resulting in a “selective random” sampling of elements.

To develop NDOT’s LCMAMP, and keep data organized, a consistent naming convention and wiring scheme was used. Station names consisted of the site name, wall number, and station identification letter. Site names are assigned based on the construction contract name and therefore may not best describe the physical location. This was done for consistency with NDOTs database and the previous phase of study (Thornley 2009). Wall numbers are obtained from the construction documents; station identification is alphabetical starting on the left of the wall face as determined from a person on site facing the MSE wall face. This convention further continued for the identification of corrosion rate observations, in which the individual testing conditions and spatial relationship are also included.

The naming convention used is based on the recommendation provided in FHWA publication number FHWA-NHI-00-044, and includes the working and reference elements for the LPR. When multiple elements of the same type are located at a single station, they were numbered starting at the top of the wall and increasing as elevation decreases. When elements are located at the same elevation they are identified as either left or right. For the purposes of LPR testing and filenames, site locations need to be abbreviated but still provide enough information to clearly identify the site. The naming convention is as follows: ***state-date-site-wall-station-working element-reference element.*** For example; **NV** (Nevada) -**06-12-13** (Date) –**I515LVB** (I-515/ Las Vegas Blvd.) -**W1** (Wall 1) -**SB** (Station B) -**GC2** (Galvanized Coupon, 2nd from top of wall) -**EL** (Existing reinforcing, left side).

Along with a naming convention, a wire and knotting convention was also implemented so that various elements can be identified from their wire termination at the junction box. The color convention follows national standard with red wire for in-service elements, white wire for galvanized coupons and black wire for carbon steel coupons. When multiple elements of the same metal type are at a monitoring station, the individual wires are knotted starting at the base of the wall and increasing with each successive layer. Therefore the bottom most elements will have no knots.

2.4.1.A In-Service Soil Reinforcements

Monitoring of in-service reinforcements allows for evaluation of the progression of corrosion. If the in-situ corrosion rates are known then comparison with the design model (anticipated rate of metal loss) allows for an estimation of the corrosion severity at the site. Monitoring of in-service reinforcements is critical for performance evaluation as corrosion of the base steel directly affects the tensile capacity of the reinforcements, and the internal stability of the MSE wall.

Connection to the in-service reinforcements requires a permanent connection, which does not influence the corrosion rate measurements. This is accomplished with the use of a three component system. This system includes a mechanical connection, encapsulation of the connection with a non-conducting hard epoxy, and a water proof sealant. Connection to the reinforcements is made approximately six inches behind the back face of the concrete facing panels. This method is used for all in-service elements.

2.4.1.B Sacrificial Coupons

Installations of sacrificial coupons allow corrosion rates to be gathered. These corrosion rates can then provide insight to: (i) the aggressiveness of the backfill soils, (ii) the effects of time on corrosion rates, and (iii) allow for comparison with in-service reinforcements. The coupons are installed as sets, consisting of a carbon (plain) steel and galvanized sample such that they are consistent with the original soil reinforcements. The coupons are installed with a known condition, thus providing bracketing scenarios for the in-service reinforcements. The galvanized coupon represents the infant stage of the reinforcements, while the carbon steel represents the reinforcements post zinc consumption. This information can provide insight to which corrosion stage is in progress, when used with corrosion rates form in-service reinforcements (**Fig. 2-3**). In-situ corrosion rates, obtained in a periodic manner, from the coupons provide insight to the early stages of the MSE walls life, thus allowing for a more complete picture of the MSE wall corrosion process. The same three component protections system is implemented for the coupons.

2.4.2 Linear Polarization Resistance (LPR)

Linear Polarization Resistance (LPR) measurements involve measuring the change in potential along the surface of an electrically isolated element, due to an impressed current. This is accomplished through the use of three electrodes, a working, reference, and counter electrode. The reference electrode is a copper/ copper sulfate half-cell, which maintains a stable reference. This serves as the baseline for the monitoring the subsequent changes in the potential of the working electrode. The working electrode (i.e. the element being monitored) is perturbed from the free-field potential and the current required to polarize the samples is recorded and used in the calculation of the in-situ instantaneous uniform corrosion rate. The resistance to the potential is calculated from the impressed current and the associated polarization potential. The resistance is the sum of the soil's resistance and the resistance across the interface between the metal element and the backfill. An idealized LPR circuit is provided in **Fig. 2-4**.

2.5 Overview of Previous NDOT Corrosion Studies

Nevada is among several other states that have implemented an evolving corrosion monitoring program (Armour, et. al. 2004; Sagüés, Alberto A. et. al. 1998; Fishman 2005; Salazar and Hilfiker 2005; Beckham et. al 2005; Billings 2011). The development of this program resulted from the discovery of elevated corrosion on MSE wall soil reinforcements. The implementation of the corrosion monitoring program is viewed as a part of the NDOT's performance asset management system, which is intended to monitor the health of the MSE walls in the state's inventory.

In 2004 sound walls were constructed along the I-515 corridor in Las Vegas. During the excavation for the walls foundation, the upper layers of the MSE wall's soil reinforcements were exposed, which demonstrated greater than expected metal loss. This occurred at the I-515/Flamingo Road complex. The discovery of elevated corrosion resulted in remediation for the largest wall at the site. The wall, originally constructed with precast concrete facing panels and steel welded wire fabric, was retrofitted with a cast-in-place tie-back wall (Fishman 2005; Thornley 2009). This experience lead the NDOT to pose the question, "What extent is corrosion an issue within the state and what other sites might be experiencing similar conditions?”

In 2009, a lane widening project at the I-15/Cheyenne Blvd. complex resulted in demolition of an existing MSE wall. The MSE wall reinforcements removed, again showed signs of excessive corrosion (Thornley 2009). This occurred while the Universities of Nevada, Reno and Las Vegas (UNR and UNLV, respectively), were evaluating backfill testing records (Phase I) to identify other sites with potential of advanced corrosion. The discovery of a corrosion issue at I-15/ Cheyenne Blvd. was then incorporated into the current university study. As a result of Phase I, eleven sites across the state were recommended for further evaluation.

2.5.1 Consultant’s Report: Corrosion Evaluation of MSE Walls at I-515/Flamingo Rd. Las Vegas, Nevada

During the construction of new sound walls along the I-515 corridor in Las Vegas, the upper layers of the soil reinforcements for Wall 1 at the I-515/Flamingo Rd. complex were exposed. The cold drawn steel, welded wire fabric, which were not galvanized, demonstrated greater than anticipate metal loss. As a result, NDOT and the FHWA decided to provide remediation of the largest wall with the greatest consequence from failure at the site, Wall 1. This wall, originally constructed with precast concrete facing panels and steel welded wire fabric, was retrofitted with a cast-in-place tie-back wall (Fishman 2005). The NDOT contracted with McMahon and Mann Consulting Engineers (MMCE) to conduct a site investigation and performance evaluation regarding the impact and extent of the advanced corrosion.

The study conducted by MMCE involved the use of: test pits, access to reinforcements through the wall face, physical metal loss measurements, LPR data collection/analysis, electrochemical soil testing, and metallurgic testing. The report concluded that backfill soils were considerably more aggressive than the original specification allowed. This resulted from: elevated salt contents, low resistivity, high moisture content and, differential aeration (Fishman 2005). Due to the aggressive nature of the backfill material and the elevated corrosion rates present, it was concluded that substantial structural capacity was lost, and the structure could not be relied upon to achieve the intended service life.

2.5.2 Use of Statistical Method to Study Corrosion Aggressiveness at Nevada Mechanically Stabilized Earth Wall Sites (Phase I)

In 2009 construction at I-15 and Cheyenne Blvd in Las Vegas included the demolition of a MSE wall to make room for a lane widening project. The debris of the MSE wall included the removed soil reinforcements, initial visual inspection of the reinforcements indicated that the corrosion rates/ metal loss at the site were greater than anticipated (Thornley 2009). This discovery occurred when UNR and UNLV were reevaluating the I-515/Flamingo Rd. site. This included scrutiny of the electrochemical laboratory test methods used for characterization of the backfill. Upon discovery of the issues at the Cheyenne Blvd. complex a limited site evaluation was included in their study.

The aggressiveness of the backfill soils was under scrutiny after the I-515/Flamingo Rd. discovery. The historical electrochemical soil test results were examined to identify what sites, if any, could be experiencing similar conditions as the Flamingo Rd. complex. This study focused on: reevaluation of the Flamingo

Rd. complex, evaluation of the electrochemical testing methods used by the NDOT and, evaluation of the Cheyenne Blvd. complex

The reexamination of the Flamingo Rd. complex confirmed the previous conclusion that the site had sustained significant impact to its design life. The UNR study focused heavily on the direct metal loss measurements and the idealized uniform corrosion rates based on this data, while the MMCE report placed emphasis on the LPR corrosion rate measurements. While there were differences on the impact of the metal loss, both studies agreed that a problem existed. This background was then used to evaluate the Cheyenne Blvd. complex. Samples of the exposed reinforcements were gathered and metal loss measurements were used to estimate the corrosion rates at the site. Based on the metal loss measurements the idealized uniform corrosion rate for the Cheyenne complex indicated that it was experiencing similar corrosion levels as the Flamingo Rd. site (Thornley 2009). Along with the metal loss measurements the backfill soils were sampled for testing.

The electrochemical testing methods used by NDOT were evaluated against the AASHTO recommended test method. Prior to the results of this study the NDOT was using an in-house test method, Nevada T235B, for minimum soil resistivity, while AASHTO recommended the use of T-288 (Thornley 2009). A statistical comparison of the methods was conducted using soils which had both test methods preformed. The analysis resulted in discovery that the Nevada test method over predicted the minimum soil resistivity. The results indicated that the Nevada test method underestimates the corrosive aggressiveness of the soils. On average there is 31% difference between the two methods (i.e. Nevada T235 was

≈ 31% higher than AASHTO T288). The following correlation between the two methods was developed (**Eq. 2-3**):

$$\rho_A = 0.895\rho_N^{0.963} \quad \textbf{(2-3)}$$

where:

- ρ_A is the AASHTO T-288 equivalent minimum soil resistivity [ohm-cm]
- ρ_N is the Nevada T235B minimum soil resistivity [ohm-cm]

This correlation enabled the comparison of soil resistivity at a site using the AASHTO T-288 method as the basis.

The NDOT MSE wall inventory was scrutinized to identify wall sites which may have a high likelihood for aggressive metal loss. Eleven (11) sites were identified, which included both Flamingo Rd. and Cheyenne Blvd. that had the potential for aggressive backfill conditions. Along with identifying sites for future evaluation, it was recommended to adopt the AASHTO T-288 for determining minimum soil resistivity. The NDOT has adopted many of the recommendations provided. These recommendations led to the initiation of the current study.

2.6 Phase II Corrosion Investigation

The results of the Phase I provided a list of prioritized Nevada MSE wall locations for further investigation (**Table 2-3**). These sites along with the two previously investigated sites (I-515/Flamingo Rd. and I-15/Cheyenne Blvd.) form the foundation of the Phase II: MSE wall corrosion investigations. During the planning and permitting phase of the fieldwork sites were removed from the list due to difficulties in accessing the sites. The finalized set of MSE wall sites for investigation is provided in **Table 2-4**. Under the this current investigation eight of the sites were chosen for investigation, along with I-15/ Cheyenne Blvd and collection of a second set of LPR data from I-515/Flamingo Rd

Chapter 3 Assessment of MSE Wall Backfill Soils

3.1 As-Built MSE Wall Backfill Soil Testing

The sites included in this Phase II study span more than 25 years of Nevada MSE wall construction. Over that time the backfill specifications have changed resulting in a variety of backfill conditions. Starting with the 1986 edition of the Standard Specifications for Road and Bridge Construction (NDOT Silver Book) limits have been placed on the backfill used for MSE walls. From the time that specifications were initiated to the present there has been a series of revisions. **Table 3-1** provides a summary of these changes. While NDOT has a long history of using internal testing methods (Thornley 2009, NDOT 1968; NDOT 1976; NDOT 1986; NDOT 1996; NDOT 2001), specifications for each of the parameters has closely resembled national standards. Due to changes in specifications, it is possible that fill materials that passed at the time of construction may not meet current physiochemical constraints.

3.1.1 MSE Wall Backfill Soil Assessment Using As-Built Soil Testing Records

Construction documents for the walls under consideration for Phase II were obtained from the NDOT records and were evaluated for MSE backfill testing results **Table 3-2**. This table includes all available testing results (both, passing and failing) for: chlorides, sulfates, minimum soil resistivity and pH. Organic content which is specified under current AASHTO standards (AASHTO 2012) did not have a formal specification from the NDOT. The testing results for samples that failed are also presented; these allow a qualitative understanding of backfill selection process. The pass or fail condition is based on construction era criteria and testing methods. Throughout this study a single set of requirements specified in NDOT 2001, which are identical to the current (2012) AASHTO standards were used to assess the pass/fail criteria (**Table 3-3).**

It is also important that test methods are consistent. Based on the work by Mr. John Thornley during the Phase I MSE wall corrosion study, a correlation equation (**Eq. 3-1**) was developed between the two different minimum soil resistivity methods. This was used to convert the test results from Nevada test method T235 to an equivalent AASHTO T-288 value for minimum soil resistivity. The other testing results (i.e. chlorides, sulfates, and pH) do not have any documented discrepancies. Soil resistivity is a critical indicator of soil corrosivity and therefore, for Phase II it is used as the primary electrochemical characteristic to describe the aggressiveness of the backfill. As-Built minimum soil resistivity test results were converted prior to analysis and evaluation using the correlation given in **Eq. 3-1** (Thornley 2009):

$$\rho_A = 0.895\rho_N^{0.963} \qquad \textbf{(3-1)}$$

where:

ρ_A is AASHTO T288 Equivalent resistivity [ohm-cm]

ρ_N is Nevada T235B resistivity [ohm-cm]

This use of the conversion is important due to a roughly 30% difference between testing methods. **Table 3-4** provides the As-Built soil testing results that were used for evaluation in this study. The data in **Table 3-4** is populated from the As-Built MSE backfill samples that passed specifications. The table includes the converted minimum soil resistivity (AASHTO equivalent) values. As noted above, the highlighted pass or fail criteria has been updated to reflect the current requirements (NDOT 2001).

It is assumed that if a material passed requirements at the time of testing only then it was used during construction. There are no records indicating where a particular source of fill was used during the construction of a MSE wall site. Therefore, assessment of the condition of a site needs to be conducted on a site wide basis, without distinguishing different walls or location within the complex. Backfill conditions are assessed using descriptors of the central tendency of the testing results. Mean values are the primary descriptor used for classification and acceptance/ rejection determinations in this investigation. The minimum and maximum values supply the range and provide an indication of the variation within each sample set. The As-Built classification (pass/ fail) and range characteristics are provided in **Table 3-5**. Based on current MSE wall backfill specifications and using mean values, only a single site (I-15 and Cheyenne Blvd.) meets specifications. Of the 29 soil records (**Table 3-4**) only 4 (14%) samples meet current physiochemical backfill requirements.

3.1.2 Site Specific Descriptive Statistics of the As-Built Backfill Testing Records

When the construction documents provided four or more testing results, descriptive statistics were evaluated. Four sites provided sufficient data and were examined in greater detail in the following sections. Confidence intervals were generated for the mean values using a significance factor of 5% (α=0.05 or 95% confidence level). The reported confidence regions have been adjusted for lower limits. The lower level of the interval is truncated when it would result in a non-positive value. Therefore, the mean may not be centered in the reported range. The coefficient of variation (COV) value is used to gage the relative precision of the results. A threshold of 15% (COV $\leq$15%) is used to indicate reliability and precision of the statistical descriptor, while values greater than this suggest the data maybe insufficient for a robust characterization of the backfill parameter.

3.1.2.A ALT. US 95 and ALT. US 50

The descriptive statistics for ALT. US 95 and ALT. US 50 are provided in **Table 3-6**. Both the chloride and minimum resistivity values (COV = 0.67 and 0.33, respectively) indicate insufficient precision in the test result. Thus, may not adequately characterize the backfill conditions. As observed in the confidence range values, the upper limit of the range (best-case scenario) for the site would marginally pass the 3,000 ohm-cm limit for soil resistivity. It is interesting to note that low resistivity values are associated with low salt content (sulfates and chlorides) for this complex.

3.1.2.B I-15 and Cheyenne Blvd.

The descriptive statistics for I-15 and Cheyenne Blvd. are provided in **Table 3-7**. Significant scatter is present in the minimum resistivity values (COV = 0.39), indicating imprecision in the testing results. This dataset's ability to characterize the backfill conditions is insufficient for producing a statistically robust representative minimum resistivity value. It is worth mentioning that I-15 and Cheyenne is the only site in which the mean values pass current (NDOT 2001/ AASHTO 2012) MSE wall backfill specification. However, the confidence interval does indicate the possibility that the mean value for resistivity could be less than specification.

3.1.2.C I-515 and Charleston Blvd.

The descriptive statistics for I-515 and Charleston Blvd. are provided in **Table 3-8**. The chloride, sulfate and minimum resistivity values (COV = 0.55, 0.65 and 0.33, respectfully) indicate considerable scatter within the data. Thus, the data may not adequately characterize the backfill conditions. The low resistivity value is associated with elevated salt content as expected. This site fails to meet specification for chlorides, sulfates, and minimum soil resistivity.

3.1.2.D SR 160 and Jones Blvd.

The descriptive statistics for SR 160 and Jones Blvd. are provided in **Table 3-9**. Based on the coefficient of variation there is no significant scatter within the testing results. Indicating that the precision of the testing regiment is adequate in producing representative values for characterization of the backfill. This site fails to meet specification for minimum soil resistivity.

3.2 Phase II MSE Wall Backfill Testing

During the process of instrumenting the MSE walls for corrosion rate monitoring, soil samples were collected and tested to characterize the in-situ MSE wall backfill conditions. The tests were done using the AASHTO specified test methods. Characterization of the backfill soil condition is one of the most reliable tools to assess the aggressiveness and potential for corrosive degradation of the reinforcing elements. The backfill test results in conjunction with the LPR data, are the backbone of the Long-term Corrosion Monitoring and Asset Management Program.

3.2.1 Phase II Backfill Soil Assessment

Seventy-two (72) soil samples (**Table 3-10)** were collected and tested from the seven sites investigated in the Phase II project. This is a significant increase (148%) in data that can be used to characterize backfill conditions at each site. The entire dataset was scrutinized in terms of the minimum, maximum, mean values, and pass/ fail assessment. Of the 72 samples tested only 16 (22%) meet the requirements placed on MSE backfill. This information is presented in **Table 3-**

11. Related to mean values only a single site (I-15 and Cheyenne Blvd.) satisfies all of the electrochemical requirements for MSE wall backfills.

The data obtained under this project more than doubles the available soil test data. The As-Built soil testing records used significantly less samples to characterize an entire site, which may include walls that were not included in the current project (Phase II). The current project collected and analyzed a greater number of samples at the test locations. The post construction backfill samples have detailed location information, which allows for greater refinement in condition assessments. Backfill conditions can be compared spatially within a wall and within an entire complex. This ability is not possible with the As-Built records, thus the Phase II data can be used to identify locations of backfill anomalies, and as an important reference document for future investigations.

3.2.2 Site Specific Descriptive Statistics for Phase II Soil Testing

Descriptive statistics were produced for each of the test parameters, along with confidence intervals for the mean value, using a significance factor of 5% (α=0.05 or 95% confidence interval). As was done with the As-Built backfill analysis, the confidence regions have been adjusted for the physical lower limits, and the COV values is used to evaluate each dataset for precisions. Using Minitab® standard box-plots were created for visual interpretation of the backfill parameters. The box-plots are used to compare conditions within a specified location and for site to site comparison. Comprised of quartile data box-plots provide a simple graphical tool to describe the data. The sparse dataset used in the backfill analysis may prove insufficient in developing robust conclusions about the characterization of the backfill. However, the level of detail achieved under the current investigation is a significant improvement and allows for a more refined determination of the physiochemical conditions of the backfill.

3.2.2.A ALT. US 95 and ALT. US 50

The descriptive statistics for ALT. US 95 and ALT. US 50 are provided in **Table 3-12**. There is significant scatter (COV > 15%) in the results for: chlorides, sulfates, resistivity and organic content (COV = 1.20, 0.37, 0.24 and 0.18, respectively). The largest imprecision is present in the chloride content. Despite the large statistical deviation, the results from the chloride testing indicate relatively low concentrations. Though the range of resistivity test results (460 to 860 ohm-cm) is narrow, the COV value indicates scatter. This site fails specification for: sulfates, minimum soil resistivity, and organic content. Based on the resistivity values, the backfill is considered to be corrosive based on **Table 2-2.**

3.2.2.B US 395 and Huffaker Lane

The descriptive statistics for US 395 and Huffaker Lane are provided in **Table 3-13.** There is significant scatter in the test results for: chlorides, sulfates, resistivity and organic content (COV = 2.04, 1.38, 0.56 and 0.31, respectively), with the largest imprecision present in the salt ions (i.e. chlorides and sulfates). The descriptive statistics for this site are highly influenced from the test result from, Wall 1-Station A-Panel 1 (**Table 3-10**). This location provides an extreme value for sulfates, chlorides and resistivity. This may be the result of a local pocket of more aggressive soils. This location cannot be inferred as an isolated case. Even with this test result removed from the dataset, the site fails for minimum soil resistivity and organic content and is considered to be very corrosive based **Table 2-2** and soil resistivity test results.

Using the location information the backfill conditions of the individual walls is assessed and presented in **Table 3-14**. This table does not provide the detailed descriptive statistics used in the site description due to the lack of data. Using the mean value and the range, an adequate classification may be achieved. It is concluded that both Walls 1 and 2 are moderately corrosive (**Table 2-2**). To evaluate and compare the conditions between the two walls, box-plots were created for each of the five requirements: chlorides (**Fig. 3-1**), sulfates (**Fig. 3-2**), resistivity (**Fig. 3-3**), pH (**Fig. 3-4**), and organic content (**Fig. 3-5**). The overlapping regions suggest consistency between the different walls. The lack of fingers on the individual box-plot figures is a result of the small dataset used.

3.2.2.C I-15 and Cheyenne Blvd.

The descriptive statistics for I-15 and Cheyenne Blvd. are provided in **Table 3-15**. There is significant scatter in the test results for: chlorides, sulfates, resistivity and organic content (COV = 2.50, 0.97, 0.64 and 0.28, respectively). The highest levels of imprecision based on COV are present in the salt ions. The test results for chlorides, sulfates and minimum soil resistivity have a large range of values. Consequently, the Cheyenne Blvd. complex has the largest statistical imprecision of the sites investigated (i.e. largest average COV values). The large COV values suggest that the backfill soils are not robustly characterized through the testing. A global classification of this site is statistically poorly supported, and more detailed categorization is recommended. Unlike the As-Built records in which the test result cannot be assigned to a location, detailed location information is available for the current dataset. This additional level of information can be used to subdivide the complex, and identify areas of concern allowing a more detail investigation of the backfill conditions.

Using the location information the backfill conditions of the individual walls is assessed using **Table 3-16**. This table does not provide the descriptive statistics details used in the site description due to the sparse data available. However, using the mean values and the range an acceptable classification may be achieved. It is concluded that Walls 1 and 4 fail specifications whereas, Walls 2, 5, and 6 meet electrochemical criteria. Interestingly, Walls 1 and 4 are the largest walls at the site. The remaining un-sampled walls should be cautiously classified. The imprecision of the testing results and the two failing walls suggest such an approach is warranted. Even though, the overall data (mean values) does pass soil specification, the level of precision and multiple samples below requirements support conservative interpretation. The imprecision of the test results produces wider confidence intervals. The confidence intervals suggest the possibility that the mean value could fail the physiochemical requirements. A conservative classification of moderately corrosive to corrosive (**Table 2-2**) is recommend based on minimum soil resistivity values.

To evaluate and compare the conditions between the five walls, box-plots were created for each of the five parameters: chlorides (**Fig. 3-6**), sulfates (**Fig. 3-7**), resistivity (**Fig. 3-8**), pH (**Fig. 3-9**), and organic content (**Fig. 3-10**). The resistivity (**Fig. 3-8**) assessment of the entire complex is heavily influenced by the results from Walls 5 and 6, as the higher levels recorded on these walls strongly influence the mean value for the site. The box-plot data from Walls 1 and 4 show the data is weighted on the lower end of the test results. This is observed in the unbalanced individual figures. These unbalanced data result in under predicting the corrosive potential of the site. Sulfate results (**Fig. 3-7**) demonstrate the large variation within the reported values. Again Walls 1 and 4 exhibits significant spread in the reported values. The pH of the site is narrowly constrained (8.7 – 9.4), yet **Figure 3-9** implies that a single value would be inadequate in describing the backfill.

3.2.2.D I-15 and Lamb Blvd.

The descriptive statistics for I-15 and lamb Blvd. are provided in **Table 3-17**. There is significant scatter in the test results for sulfates, resistivity and organic content (COV = 0.35, 0.28 and 0.37, respectively). The chloride content of the backfill soils was at or the below the testing threshold of 10 mg/kg (ppm) for all samples while, the sulfates ranged from 100 to 260 mg/kg. The data implies that while a significant portion (3 of 7, 43%) (**Table 3-10**) of the samples exceed the sulfate limit, the total salt content is moderately low. The mean value for minimum soil resistivity classifies the site as corrosive (**Table 2-2**). With the exception of minimum soil resistivity, the mean values of the electrochemical constituents meet backfill requirements.

The backfill conditions of the individual walls can be assessed using **Table 18**. This table does not provide the detailed descriptive statistics used in the site description due to the lack of data for each the wall. It is concluded that backfill for Wall 1 is corrosive while Walls 2, 3 and 4 are moderately corrosive (**Table 2-2**). Walls 1 and 2 pass all requirements other than resistivity.

To evaluate and compare the conditions between different walls, box-plots were created for each of the five requirements: chlorides (**Fig. 3-11**), sulfates (**Fig. 3-12**), resistivity (**Fig. 3-13**), pH (**Fig. 3-14**), and organic content (**Fig. 3-15**). The overlapping regions indicate consistency between the different walls. The lack of fingers on the individual box-plot figures is a result of sparse data that was available. The resistivity figures (**Fig. 3-13**) depict relative consistency between the various walls, with the single sample from Wall 4 being slightly lower than the other three walls. In general, there appears to be good consistency between the different walls; no single wall is obviously identified as unique based on the backfill testing results. While the overall data indicates scatter (COV > 15%), the box-plot figures suggest that the scatter is consistently prevalent throughout the complex.

3.2.2.E I-515 and Charleston Blvd.

The descriptive statistics for I-515 and Charleston Blvd. are provided in **Table 3-19**. There is significant scatter in the test results for: chlorides, sulfates, resistivity and organic content (COV = 1.17, 0.40, 0.75, and 0.51, respectively). The large COV value of the sulfates is misleading, as all the test results form a relatively narrow range (10-90 mg/kg). The COV value is influenced by the results of Wall 3-Station C (**Table 3-10**). The COV value for the resistivity results is influenced by the result from Wall 4-Station B (**Table 3-10**), the only sample passing specification. This site is classified as corrosive (**Table 2-2**), based on the mean resistivity value. This site fails specification for sulfate and minimum soil resistivity.

Table 3-20 presents the range and mean value for each of the test results on a per wall basis. It is concluded that Wall 3 backfill is corrosive, and approaching the very corrosive classification (**Table 2-2**) with a mean resistivity value of 760 ohm-cm. Wall 4 is on the threshold of corrosive and moderately corrosive. Due the large scatter suggested by the elevated COV values, and the belief that resistivity is a critical indicator of corrosiveness, it is recommended to adopt the conservative classifications of very corrosive and corrosive for Walls 3 and 4, respectfully.

Comparing conditions between the two walls, box-plots were created for each of the five backfill requirements: chlorides (**Fig. 3-16**), sulfates (**Fig. 3-17**), resistivity (**Fig. 3-18**), pH (**Fig. 3-19**), and organic content (**Fig. 3-20**). The lack of fingers on the individual box-plot figures results from the sparse dataset used. The box-plot figures provide indication that there is dissimilarity between the backfill characteristics of Walls 3 and 4. The organic content (**Fig. 3-20**) is the only figure to have clear overlapping regions, which indicates constituency between locations. The box-plot figures in conjunction with the large COV values suggest that the backfill characteristics are not robustly characterized.

3.2.2.F I-515 and Las Vegas Blvd.

The descriptive statistics for I-515 and Las Vegas Blvd. are provided in **Table 3-21**. There is significant scatter in the sulfate (COV = 1.04) data. The three samples obtained from this site show consistency amongst the physiochemical parameters, with the exception of sulfate content which ranges from 41 to 1,700 mg/kg. All of the chloride test results were at or below the reporting threshold of 10 mg/kg. All three samples exceed the 3,000 ohm-cm minimum specification for soil resistivity. This is noteworthy due the fact that this the only site in which this occurs and, that the inclusion of this site in the Phase II study is a result of its age and lack of As-Built testing results for resistivity. This site fails backfill requirements for both sulfates and organic content, and is classified as mildly corrosive (**Table 2-2**).

***3.2.2.G* SR 160 and Jones Rd.**

The descriptive statistics for SR 160 and Jones Rd are provided in **Table 3-22**. There are significant levels of imprecision in the results for: chlorides, sulfates, resistivity, and organic content (COV = 0.54, 0.39, 0.61 and 0.25, respectfully).

The largest imprecision is in the resistivity data, and is reinforced by the large range of reported values. This is the single largest subset of the Phase II soil testing data. Twenty-four (24) soil samples were tested which, represents approximately a third of all the Phase II soil testing. All of the samples were obtained from Wall 1, the remaining five (5) walls were not included in this investigation. The exclusion of the other walls is due to the size of the selected wall and recent construction.

Both chloride and organic content have COV values above the benchmark value of 15% for imprecision. This is counter intuitive based on the relatively narrow range in testing results. The entire range of values for both chloride and organic content are within the specification. The mean sulfate content, including entire confidence interval, is below the requirement of a maximum 200 ppm (mg/kg). However, 7 of the 24 (29%) samples collected exceed the maximum sulfate limit. The vast majority (92%) of the samples are below the minimum soil resistivity requirements (3,000 ohm-cm); a single sample collected exceeds the specifications (Wall 1- Station G- Top sample). This site has low resistivity values despite the low salt content. This site fails specification for minimum soil resistivity and is classified as moderately corrosive (**Table 2-2**).

3.3 Comparison of As-Built and Phase II Soil Testing

With the additional sampling and testing from Phase II, comparisons of the datasets is possible. This includes comparison of mean and extreme values for: sulfates, chlorides, pH and minimum soil resistivity. Organic content is excluded due to a lack of As-Built testing results. During the construction of MSE wall complex, it is common practice to sample backfill material at multiple source locations. Once the requirements are met, the fill material is approved for use at the site. During MSE Wall construction backfill is placed and compacted in lifts.

There is no documentation that reveals where the fill material was used within the complex. It is possible for a single MSE wall site to be stratified with material from different source locations, thus with different physiochemical composition.

Without the detailed information on where a source material was used and the limited number of samples, the applicable comparison between datasets can be made based only on statistical descriptions of the data. Extreme values and descriptors of the central tendency of the information is compared to establish and identify consistency and irregularities. This analysis can be undertaken only on a site-wide basis for a site. The comparison between the two datasets is done using the descriptive statistics, box-plot figures for visualization of the dispersion of data and when available, Mann-Whitney testing was done to evaluate compatibility between sample sets.

3.3.1 Descriptive Statistics

Table 3-23 is provides the fundamental descriptive statistics for each site and the datasets. Using this table, a qualitative comparison of the datasets can be conducted. This identified 17 of the 28 (61%) data pairs that show noticeable variation. This includes all the sulfate results with the exception of the SR 160/ Jones Rd. wall; raising concern of the accuracy and sensitivity of the historic sulfate test methods. Through conversation with NDOT personal it was noted that there as a period of transition, when NDOT was changing from an internal test procedure to the more accepted AASHTO test method for sulfate testing. The zero value test results for the Alt. US 95 and Alt. US 50 site, are possibly placeholders on the As-Built record logs. This was not further investigated due to emphasis on resistivity testing. Even with the exclusion of the Alt. US 95 and Alt. US 50 site, five of the seven (71%) of the sites report no sulfates present in the As-Built testing records. In contrast, the current dataset indicates levels as high as 1,700 mg/kg.

The chloride testing for US 395/Huffaker Lane, I-515/Charleston Blvd. and SR 160/ Jones Rd. is of interest, as the reported levels appear to have changed significantly in many cases. This may be the result of the natural variation of the material, as evident from the large variations (COV values as large as 204%). However, even with the large scattering of the data, the test results from Phase II tend to be in a narrow range.

The variation within the salt content testing results can help to explain the differences in the resistivity and pH results. The general, trend of the Phase II data is that the soil resistivity encompasses a larger range of values and is on average lower than As-Built values. The extreme values expand the range of values for both the upper and lower limits. Even with the extreme resistivity values, the backfill conditions sampled never achieve the classification of non-corrosive (**Table 2-2**). Due to a few high values, the mean resistivity value is increased, which results in the backfill corrosiveness being under predicted.

Most notably are the test results from Alt. US 95/ Alt. US 50. The As-Built resistivity values have a mean resistivity of 2,418 ohm-cm. This fails specification but, is still classified as moderately corrosive. However, the Phase II data has a mean value of 618 ohm-cm and is classified as very corrosive. This stark reduction in the mean value raises concern. The Alt. US 95/ Alt. US 50 site was visited post data analysis, in hopes to identify any obvious reasons (e.g., drainage inlets, visual corrosion residues etc.) to explain the differences between the dataset. The site visit did not yield any obvious explanation for the reduced resistivity values.

The pH values of I-15/Lamb Blvd., I-515/Charleston Blvd. and SR 160/Jones Rd. have increased by over a full pH unit. This signifies that the backfill conditions are more alkaline than was previously reported. Both sets of backfill data have pH values concentrated within a fairly narrow range and it lies within the specification. However, the difference between the two time periods is noteworthy. The increased alkalinity of the backfill conditions could be attributed to increased salt content. An increase in salts would explain the increase in the pH values and the reduction of the resistivity values.

3.3.2 Global Comparison of Backfill Parameters

Box-plots are used to provide a visual inspection of the backfill parameters and the variation between the two testing periods. The distance between the mean (circle and cross) and median (solid line) values indicated on the figures provide an estimation of the variability with the dataset. The influence of local extreme values is observed in the separation between the mean and median values. Box-plot figures were prepared for: chlorides, sulfates, resistivity and pH, based on the total sample population for each complex and shown in **Fig. 3-21**, **Fig. 3-22**, **Fig. 3-23**, and **Fig. 3-24**, respectively.

3.3.2.A Chloride

Figure 3-21 displays the variation of the chloride content within each site and between datasets. Due to the limited data available for each site, many of the individual box-plots are missing fingers/ boxes. The vast majority of all test results are below the specification of 200 ppm. The appearance of a cluster of outliers for SR 160/Jones Rd. is due to 19 of the 24 (79%) being at a level of 11 ppm or less and the other five values around 20 ppm (**Table 3-10**). When plotted the five values appear as irregularities.

3.3.2.B Sulfates

Figure 3-22 presents the sulfate values for each site and between testing periods. Due to the sparse data available for each site, many of the individual box-plots are missing fingers/ boxes. It is obvious that I-515/ Charleston Blvd. has elevated sulfate levels. The Phase II results for I-515/ Las Vegas Blvd. has a large variation in results. In general, the mean (circle and cross) is relatively close to the median value (center bar); this suggests minimal effects of the test variations on the central tendency of the data.

3.3.2.C Resistivity

Figure 3-23 displays the variation of the minimum soil resistivity content within each site and between datasets. Many of the individual box-plots are missing fingers/ boxes. It is observed that the vast majority (10 of 13 (77%)) of the testing results are below the specification of 3,000 ohm-cm. I-15/Cheyenne Blvd. is of special interest, as this site has the largest range of values for both the As-Built and Phase II resistivity data. Examining the Phase II box-plot for I-15/ Cheyenne Blvd., it is observed that the mean value is above the specification while the median is below. This is due to most of the data being below specification with a few large values are influencing the mean value. This results in the mean being above specification.

3.3.2.D pH

Figure 3-24 displays the variation of the pH content within each site and between testing periods. Due to the sparse data for each site, many of the individual box-plots are missing fingers/ boxes. All test data passes specification and is consistent between NDOT districts. The compact nature of all the figures demonstrates consistency between the soils used within a complex and throughout the sites under consideration. The generalized observation is that Nevada soils are mildly alkaline and consistent within a source.

3.3.3 Mann-Whitney Testing

Mann-Whitney testing is a non-parametric test used to determine if two datasets are statistically different from each other. The null hypothesis for this testing method is that the two sample sets are of the same larger population based on median values. Therefore, rejection of the null hypothesis suggest (but does not confirm) that the two sets are from different populations. Mann-Whitney testing is based on the following assumptions: the samples are independent of each other, ordinal, and the distributions of each group are similar (i.e. same shape) but normality is not required. When the As-Built records provide four or more results for resistivity testing, Mann-Whitney testing was performed using a significance level of 0.05 (95% confidence). P-Values below the significance level indicate that the null hypothesis should be rejected.

Four sites provided sufficient data for analysis and they are: I-15 and Cheyenne Blvd, Alt. US 95 and Alt. US 50 (**Fig. 3-25**), I-515 and Charleston Blvd. (**Fig. 3-26**) and SR 160 and Jones Rd. (**Fig. 3-27**). I-15/ Cheyenne Blvd. is examined independently, due to the availability of Phase I data. **Table 3-24** summarizes the results from of the Mann-Whitney testing. It can be concluded that I-515/Charleston Blvd. soil resistivity values are of the same population (P-value > 0.05). The testing indicates that the null hypothesis for Alt. US 95 and Alt. US 50 and SR 160/ Jones Rd. should be rejected. This does not confirm that samples collected a various periods are from different sources, but rather indicates that more testing is required to determine if the difference with the datasets is a result of natural variation or some other source.

3.3.3 I-15 and Cheyenne Boulevard

I -15 and Cheyenne Blvd. is examined in greater detail due to the advantage of a third set of soil testing records, available as a result of the Phase I research. As part of the previous investigation four samples were collected and tested for electrochemical backfill characteristics. All three datasets are presented in **Table 3-25**. Backfill testing results fail the requirements for minimum resistivity (minimum of 3,000 ohm-cm); in addition large scatter is present among the results. During Phase I investigation, ANOVA testing (Thornley 2009) was preformed between the As-Built and Phase I resistivity values. This concluded that the two datasets were not of the same population. ANOVA testing is a popular robust sample population hypothesis testing method. However, it is based on the assumption that the datasets being compared are normally distributed. Due to the sparse data available, (four data points for each) this assumption may not be applicable. A more appropriate test method is a non-parametric test that does not rely on the normality assumption.

Mann-Whitney testing was done between each of the three datasets. Using a significance value of 0.05, the null hypothesis that the data pairs are from the same populations was tested. **Table 3-26** summarizes the results and parameters of the Mann-Whitney testing. Details of each test are also provided to support the conclusion that all three sampling periods are from the same general population. The differences between each of the datasets are not statistically significant.

When the As-Built and Phase I (**Fig. 3-28**) datasets were tested, a P-value of 0.0606 was computed. This is a weak conformation of the null hypothesis, but does not allow its rejection. This does not support the previous conclusion that the resistivity found in the As-Built records are inconsistent with the material found at the site. The current dataset was compared with the two previous sampling periods (As-Built (**Fig. 3-29**) and Phase I (**Fig. 3-30**)) and it shows a more robust conformation of the null hypotheses. This indicates that there is consistency between the testing results and that the apparent differences between datasets is not statistically significant.

3.4 Phase II Physiochemical Relationships

To confirm the relationship between minimum soil resistivity and the total salt content, the relationships were plotted. Resistivity is an indirect measure of the conductivity of the backfill, and the electrolytic potential. This potential is directly affected by the total salt content, and it is expected that resistivity is negatively correlated to the salt content.

3.4.1 Resistivity and Salt Ions

When the minimum resistivity values were plotted against the chloride content (**Fig. 3-31**) there is no distinct relationship is observed. It is interesting to note that the Phase II data is clustered on the low end of reported chlorides, while the resistivity spans a larger range of values. This observation is based on the relationship presented in **Fig. 3-31**. It appears that minimum soil resistivity values are not described by the chloride content. Similar behavior is seen between the sulfate content and resistivity values (**Fig. 3-32**). Minimum resistivity appears to have a weak relationship with sulfate content; this is more evident in the Phase II data than the As-Built dataset. The association of low resistivity with high sulfates is observed, however a strong relationship does not exist.

The relationship between the salt ions and resistivity was further investigated using the theoretical minimum resistivity value. The National Association of Corrosion Engineers (NACE) conducted controlled laboratory testing in which the conductivity of a solution was measures as salt content was systematical increased. This produced a relationship that can be used to estimate the resistivity value of a backfill based on the salt content. This cannot provide a design value but rather it is an approximation. This figure is reproduced in **Fig. 3-33**, along with Phase II data and the specification limits super-imposed.

The data is **Fig. 3-33** has two distinct clusters one at the 10 ppm level and the other approximately at 150 ppm. In both cases it is observed that the reported resistivity values are lower than the theoretical relationship. A majority of the data is below the resistivity values of 3,000 ohm-cm. Based on the figure it is observed that resistivity values are not strongly correlated with either the chloride or sulfate content. It is observed that while the general trend between the sulfates and resistivity is as expected (i.e. negative trend line slope), the relationship between chlorides and resistivity is not. This confirms the previous observations, made in this study, that resistivity is may not be characterized by a single salt ion in MSE wall backfill.

3.4.2 Backfill Testing Comparison Figures

Site to site comparison for physiochemical backfill characteristics could be undertaken in a qualitative manner. This information can be used to determine how each set of backfill data compares to the other sites in the study. Each of the five (AASHTO) soil specifications is presented in **Fig. 3-34** to **Fig. 3-38**. Qualitative criteria for ranking of the severity of each set of results is based on how the data compares with the specification, the dispersion of data within a site, and the number of samples.

Chlorides are compared (**Fig. 3-34**) and ranked based on the established qualitative criteria. I-15/Cheyenne Blvd. is ranked as the most severe, based on the large range as well as the largest reported values. While SR 160/Jones Rd., receives the lowest rank due to the large number of data, clustered well below the limit. The sulfate content (**Fig. 3-35**) results in I-515/ Las Vegas Blvd. getting the worst ranking and US 395/Huffaker Lane receives the best. The resistivity values are compared using **Fig. 3-36**. Alt.US 95/ Alt. US 50 has the lowest cluster of resistivity values and as such receives the worst ranking. I-515/Las Vegas Blvd. is the only site which does not have a sampling below specification and thus is assigned the top ranking. Due to the large scatter and median value being below requirements, I-15/Cheyenne Blvd. is ranked as the second most sever site. The results from the pH (**Fig. 3-37**) show all data pass specification. Ranking is based on the proximity to a pH level of 9.2, which in this study corresponds to the most severe corrosion rates. However, due to the narrow range of values all the sites could be considered equivalent in regards to pH. Finally, organic content was ranked using **Fig. 3-38**, US 395/ Huffaker received the worst ranking while I-15/Cheyenne receives the best. It is noteworthy that that even if a site performs poorly in regards to one specification it may perform well in another. It should be emphasized that there is significant scatter in the soil testing results.

3.5 Phase II Global Evaluation and Conclusions

A simple initial approach to assess the overall severity may be undertaken using the “total score” by summing all the individual rankings. The range for the “total score” is 5 to 35, higher values indicating increased severity. The ranking of the various MSE wall backfill parameters is summarized in **Table 3-27**. When considering the total score, it is observed that US 395/ Huffaker Lane, Alt. US 50/ Alt. US 95 and, I-15/ Cheyenne Blvd. receive similar overall severity ranking (25, 27, and 25, respectfully). While the method of ranking the sites is highly subjective it does allow a reasonable assessment of how each site compares within the current investigation and can serve as a starting point to create a priority list for future investigations and testing.

Based on the statistical analysis and of the Phase II data, it can be concluded that the MSE backfills are not completely characterized using the limited number of samples required during the construction process. This is evident in the large scatter and COV values that are observed as well as confirmed with the Mann-Whitney testing. In general the conditions of the seven sites are unacceptable according to current MSE wall backfill requirements. Only I-15 and Cheyenne Blvd. would pass backfill requirements based on mean values, but as it has been demonstrated this is due to a few extreme values affecting the mean value. If the median value were used then all seven sites fail requirements. Based on the backfill characteristics a more aggressive and corrosive environment exists at the sites. This will negatively affect the soil reinforcements, and potentially reduce the service life of the MSE wall structures.

Chapter 4 In-situ Corrosion Rates

4.1 Corrosion Rates from Linear Polarization Resistance (LPR) Measurements

Stern and Geary (1957) defined polarization resistance (PR) as the change in potential divided by the change in applied current ($d\varepsilon/di_{app}$), and proved that for small changes (±20mV) from the free field corrosion potential (ε) the corrosion current density (i_{corr}) is inversely proportional to the polarization resistance. This is relationship is given by:

$$R_p = \left[\frac{d\epsilon}{di_{app}}\right]_{\epsilon \to 0} = \left[\frac{\Delta\epsilon}{\Delta i_{app}}\right]_{\epsilon \to 0} = \frac{\beta_a \beta_c}{2.3 \times i_{corr}(\beta_a + \beta_c)} = \frac{B}{i_{corr}} \quad \textbf{(4.1)}$$

Where

- R_p = the polarization resistance normalized with the monitored elements surface area, such that R_p=PR x A_s (PR is the measured polarization resistance [ohm], and A_s is the element's contact area with backfill [cm^2]) [ohm-cm^2]
- ε = free field potential [volts]
- i_{app} = is the applied current density [amperes/cm^2]
- β_a = the anodic Tafel constant
- β_c = the cathodic Tafel constant
- i_{corr} = the corrosion current density [amperes/cm^2]
- *B* = an environmental constant [volts]

The corrosion current density (i_{corr}) is the current which results from the disassociation of the metallic ions, in the absence of an external source.

The LPR method involves changing the surface potential of the monitored element, by ±20mV in increments of 5mV from the free field potential with a potentiostat. During which time the applied current is measured using an ammeter. This allows the creation of a potential versus current (E vs i) plot. Polarization resistance (R_p) is then estimated as the slope of the linear best-fit line (i.e., $R_p = \varepsilon / i_{corr}$). If the surface area of the element being measured is known, the corrosion current density (i_{corr}) can be calculated using (**Eq. 4-1**). This is then used to determine the corrosion rate by means of **Eq. 4-2**. The corrosion rate (CR) is estimated as follows using Faraday's law:

$$CR\left[\frac{\mu m}{yr}\right] = (3.27 \times 10^6) \times \frac{i_{corr} \times w}{\rho \times n} \quad \textbf{(4-2)}$$

Where:

- w = atomic weight [g/mole]
- n = Valence (number of electrons transferred)
- ρ = density [g/cm^3]
- i_{corr} = corrosion current density [mA/cm^2]

During the LPR measuring process the measured resistance incorporates the resistance of the surrounding soil and the resistance of the interface (i.e. sample elements contact area with the backfill) to direct current flow (Polarization Monitoring 1999). To correct for the backfill's influence an estimate of its resistance is required. The backfill soil's resistance is estimated by using a high frequency AC signal, at the end of the LPR measurement sequence. The AC signal acts as a short and bypasses the capacitance effect of the elements surface (**Fig. 2-4**). This permits a measurement of the soil resistance (R_s) independent of the resistance of the monitored element. The soil resistance is then subtracted from the polarization resistance (**Eq. 4-3**) and the corrected resistance (R'_p) is used to calculate the corrosion rate.

$$R_p' = PR - R_s \tag{4-3}$$

- R_p' = Corrected Polarization Resistance
- PR = Total Polarization Resistance
- R_s = Soil Resistance

It has been demonstrated that the effects from uncorrected polarization resistances can be substantial (Elias et. al. 2009; Polarization Monitoring 1999; Fishman and Withiam 2011). A general rule of practice is that when the ratio of soil resistance to corrected polarization is greater than four, the accuracy of the results may be questionable (Lawson 1993 Fishman and Withiam 2011).

4.1.1 Assumptions of the LPR Method and Phase II Corrosion Rate Analysis

The LPR method calculates an instantaneous uniform, idealized corrosion rate based on measured values and user defined input parameters. The corrosion along an MSE wall soil reinforcement is not uniform nor are the corrosion rates constant with respect to time. However, the LPR method has been shown to be useful and appropriate (Elias et. al. 2009; Fishman and Withiam 2011; Fishman 2005, Thornley 2009). The LPR method requires several site/ element specific input parameters that are either assumed or measured. With a set of measurements that are taken over a period of time, the LPR method provides an acceptable method for evaluating reinforcement condition.

4.2 Metal Loss Models

During the design process, an additional sacrificial thickness is added to the cross-sectional area of the soil reinforcements to account for the metal loss that occurs due to corrosion. The determination of the sacrificial thickness is based on one of several metal loss models. The following sections briefly describe the models relevant to this study.

4.2.1 Darbin-Romanoff Model

As noted in Chapter 3, the Darbin-Romanoff model was developed as a result of the U. S. National Bureau of Standards (NBS) 45 year study. Romanoff (1957) investigated the underground corrosion phenomena using a broad network of buried metals under a variety of soil conditions. In general, it was observed through metal loss measurements that corrosion was more aggressive in the early years and attenuated as time progressed. Romanoff proposed a power model to predict the metal loss at some time (*t*) after burial:

$$X = Kt^{n} \tag{4.4}$$

where *K* and *n* are constants that describe the soil condition. Attenuation of corrosion rates is accounted for though the use of *n* being less than unity. The development of Romanoff model was based on a very extensive survey of materials (greater than 36,000) and locations (more than 120) (Romanoff 1957). However, approximately only ten percent of the cases represent the typical backfill conditions of MSE walls. Darbin et al. (1988) attest to this limitation with a 20 year study addressing the physiochemical backfill parameters that most directly affect the electrochemical corrosion process in reinforced soils. The study included the refinement of *K* an *n* constants that reflect the unique conditions associated with a MSE backfill.

4.2.2 American Association of State Highway Transportation Officials (AASHTO)

The AASHTO model is the current design model for galvanized steel reinforcements. This is a segmented linear model that reflects three distinct stages (tri-linear) of the corrosion process. This model is only applicable when the backfill meets the required specifications of **Table 3-3** (AASHTO 2012). The backfill requirements are intended to create a mildly-corrosive environment by controlling the electrolytic capacity. The first segment of the model represents the aggressive consumption of the zinc surface. This is followed by the second stage, characterized by a reduced corrosion rate due to passivation of the zinc layer occurs and lasts until the zinc layer has been consumed. The final stage represents the consumption of base steel and the consequential reduction in the tensile capacity of the reinforcements. This is summarized in **Table 4-1** and presented in **Fig. 4-1**. Gladstone et al. (2006) showed the AASHTO metal loss model is conservative, and acts as an upper limit for corrosion rates.

Although construction with plain steel (un-galvanized) soil reinforcements is rare, it can still be an acceptable method if the consumption of the steel is properly accounted for during the design and construction process. The design model for this case is a bi-linear metal loss model. The first stage is characterized as an aggressive equalization between the steel and the environment, in which the readily accessible metallic ions are consumed. The second stage is characterized by a constant corrosion rate until complete consumption of the base steel. The plain steel model is characterized by (**Table 4-1**) this description of metal loss is based on the study of the NBS data from the Stuttgart University (Elias 1990):

- 45 μm/yr for the first 2 years

- 12 μm/yr until complete consumption of steel

These models are presented together in **Fig. 4-1.**

4.2.3 AASHTO 75-Year Equivalent Metal Loss (AEML)

The AASHTO segmented linear metal loss model reflects the physical variations of the corrosion rate. In practice a uniform corrosion rate could also be used. The AASHTO 75-year equivalent metal loss model is comprised of a single uniform corrosion rate which results in the same total consumption of metal, i.e. zinc and base steel. It is developed with the intention of it being used for comparison with LPR rates. It allows for a qualitative way to estimate the impact of a discrete corrosion rate measurements.

The total metal loss, assuming a design life of 75 years, is calculated using the AASHTO metal loss model this is then divided by the design life for the equivalent uniform corrosion rate (Annual Equivalent Metal Loss – AEML):

$$AEML\left[\frac{\mu m}{yr.}\right] = \frac{X_t}{t} \qquad \textbf{(4.5)}$$

where:

- X_t =Total metal loss per side
- t = time [yr]

It may be noted that the total metal loss for galvanized steel at the end of a 75-year life is 794 μm (**Fig. 4.1**). Therefore, AEML is given by:

$$AEML\left[\frac{\mu m}{yr}\right] = \frac{794\mu m}{75yr} = 10.6\,\mu m/yr \qquad \textbf{(4.5)}$$

The AEML rate is a useful way to estimate the severity of a discrete corrosion rate and comparing it to a (continuous) long-term uniform rate. The measured corrosion rate can be compared with a theoretical equivalent value, which then gives and qualitative interpretation of the severity of the measured corrosion rate.

4.3 Corrosion Severity Ratio (CSR)

The corrosion severity ratio (CSR) is defined as the measured corrosion rate divided by the theoretical corrosion rate. Phase II LPR data (measured corrosion rates) and metal type and age appropriate AASHTO metal loss rates (theoretical corrosion rates, given by **Table 4-1**) are used for these calculations. The CSR is an indicator of the severity and accommodates the differing element conditions (e.g. metal type and age), thus allowing comparison of corrosion severity with a larger dataset, consisting of differing conditions. A CSR value of one indicates that the measured corrosion rate is at the design level and a value greater than one indicates a corrosion rate in excess of the design model. It is assumed that the current backfill conditions meet the requirements of the AASHTO model, which was a requirement during construction.

The AASHTO metal loss model is metal type and time dependent. Due to these constraints, comparison of corrosion levels between different MSE structures is complicated. By normalizing the in-situ LPR corrosion rate with the appropriate model (i.e. CSR), the imposed dependencies are removed and comparison of corrosion severity between walls of differing age, metal type and backfill conditions is possible. The AASHTO design model provides an upper envelope for the expected corrosion rates. Gladstone et al. (2006) confirmed this hypothesis, using data from several corrosion monitoring programs throughout the United Sates. The findings indicate that when the MSE backfill meets/ exceeds electrochemical specifications the measured corrosion rates are below the anticipated levels. Based on this information, a CSR value at or approaching one is concerning. For Phase II, it was decided to categorize a CSR value of greater than 0.80 as “concern,” while 1.0 and above as “problem.”

4.4 2012 Flamingo Rd.

The original MSE wall corrosion study conducted in Nevada, was the result of advanced metal loss at I-515/ Flamingo Rd. in Las Vegas. McMahon and Mann Consulting Engineers (MMCE) conducted a site evaluation which focused on performance and corrosion evaluation of the three MSE walls at the 1-515/ Flamingo Rd. complex. This study’s data consisted of: electrochemical backfill testing, direct metal loss measurements, LPR corrosion rate monitoring and metallurgical testing. The findings of the report suggest that the elevated corrosion resulted from (i) development of macro-cells due to differential compaction occurred near the wall face, and (ii) increased moisture content for soils in close proximity to drainage inlets. It was also discovered that (iii) backfill conditions were very corrosive (**Table 2-2**) due to low resistivity (median ≈1,000 ohm-cm) and elevated sulfate content (median 660 ppm). Electrochemical soil characteristics and corrosion rates were determined to be randomly distributed throughout the entire complex (Fishman 2005). It was concluded that all three walls at the complex were experiencing similar conditions.

A joint decision between NDOT and FHWA officials was made to provide remediation for the largest wall at the site, Wall 1. This was accomplished through the construction of a cast-in-place concrete tie-back wall which was constructed in front of Wall 1. No remediation was undertaken for Walls 2 and 3.

4.4.1 MMCE In-situ LPR Corrosion Rates (I-515/Flamingo Rd. 2004)

LPR corrosion rates were obtained in March and August of 2004; these data are reproduced and combined with the appropriate backfill testing results in **Table 4-2**. In general, the corrosion rates from March are slightly higher than those of August and Phase II analysis treats these as a single dataset. The MMCE LPR dataset (2004) and Phase II (2012) LPR data were obtained using different environmental constants. During 2004 the state of practice was to use B≈0.050 V for galvanized steel and B≈0.035 V for plain steel (**Eq. 4.1**) (Fishman 2009), while current state of practice is B≈0.035 V for galvanized steel and B≈0.026 V for plain steel (Fishman and Withiam 2011). The transition between the two sets of constants results in approximately a 26% reduction of calculated corrosion rates.

To facilitate uniform comparison of data between the MMCE and Phase II LPR data, the reported corrosion (MMCE LPR) rates have been adjusted to reflect the current environmental constants. The modified and expanded MMCE dataset used for the Phase II project is presented in **Table 4-3** and the descriptive statistics are presented in **Table 4-4.** The mean value is estimated between 5.02 – 8.60 μm/yr, which is below the design levels. The corrosion rates demonstrate considerable scatter. This observation is supported by a large coefficient of variation (COV = 142%). The maximum observed (≈ 60 μm/yr) corrosion rates are nearly five times the design metal loss rate of 12 μm/yr.

4.4.2 Phase II In-situ LPR Corrosion Rates (I-515/Flamingo Rd. 2012)

Phase II LPR corrosion rates are a part of a long-term corrosion monitoring and asset management program. Development of this program includes periodically revisiting previously studied sites. LPR corrosion rates were collected during September of 2012 for the I-515/Flamingo Rd. complex as part of the Phase II investigation. Between the monitoring periods (2004 for MMCE investigation and

2012 for Phase II) the site remediation program had been completed. A cast-in place tie-back wall now exists in front the original Wall 1. Due to construction of the new wall, access to several of the original in-service reinforcements was lost in addition to several of the element identification labels. The eight years between corrosion monitoring allowed the sacrificial coupons installed as part of the 2004 MMCE investigation to mature.

The Phase II dataset includes all elements that could be monitored. This includes: elements without identification, plain steel coupons and galvanized steel coupons (**Table 4-5**). For comparison with the previously reported MMCE dataset only the in-service elements are examined these are combined with the appropriate backfill conditions in **Table 4-6**. The original LPR corrosion rates were adjusted for the update in environmental constants. No new soils testing was conducted, therefore all electrochemical characteristics assignments are based on proximity to 2004 sampling locations.

4.4.2.A Phase II In-Service Soil Reinforcements

Thirty-five (35) of the 45 (78%) in-service soil reinforcements re-monitored (**Table 4-7**). These data produced two measurements in which the polarization resistance to soil ratio (i.e. PR/Rs) exceeded 4.0. The large ratio suggests that the calculated corrosion rate may be significantly less than the actual surface corrosion rate (Polarization Resistance 1999). Removing these two measurements from the Phase II, in-service dataset increases the mean corrosion rate by approximately six percent, while reducing the coefficient of variation by nearly five percent. This allows for both a more conservative and better description of the in-situ corrosion rates.

The descriptive statistics from the Phase II in-service reinforcement's data (**Table 4-8**) indicate that there has been no significant change in the condition of the I-515/Flamingo Rd. complex. In general, the corrosion rates appear to be randomly distributed throughout the site. This conclusion is supported by the large COV (195%) value. The data indicates that mean corrosion rate may be lower (lower bound of confidence interval) than MMCE data, 2.77 vs. 5.02 μm/yr, respectfully. This is not surprising, since corrosion rates have been documented to attenuate over time (Romanoff 1957; Elias 1990). However, the upper bound of the mean confidence interval and maximum observed corrosion rate remain essentially unchanged. The consistency observed between the two time periods supports the design metal loss models, which postulate that after two years the rate of metal loss remains nearly constant.

A histogram of the corrosion rates (**Fig. 4-2**) for both the MMCE and Phase II in-service LPR data, demonstrates a similar distribution. The bin counts are misleading due to the limited Phase II data. Nevertheless, the general shape is consistent between these datasets. The majority of corrosion rates observed are between 3 and 6μm/yr. Both datasets demonstrate a positive skewness with a long trailing tail.

4.4.2.B Phase II Sacrificial Coupons

During the MMCE corrosion investigation sacrificial coupons were placed in the backfill. The original construction of the Flamingo Rd. complex used plain steel (un-galvanized) welded wire fabric; therefore, emphasis was placed on the installation of plain steel coupons. There were a limited number of galvanized coupons installed along with the plain steel coupons. **Table 4-9** presents the Phase II LPR data for all the coupons that were installed during the MMCE investigation.

The plain steel coupons can be used to estimate the corrosion levels experienced in the early years of the I-515/Flamingo Rd. walls. While the galvanized samples can be used to estimate the performance of galvanized reinforcements.

The coupon corrosion levels appear to be randomly distributed throughout the complex. The LPR descriptive statistics for these elements are presented in **Table 4-10**. This information reveals that the steel coupons have a mean corrosion rate of ≈ 5.0 μm/yr with a maximum value of ≈ 19.0 μm/yr. The mean value is less than half of the long term design metal loss rate of 12.0 μm/yr. The maximum value is nearly 60% greater than anticipated. Similar behavior was observed in the 2004 MMCE report for in-service reinforcements. While a majority of the coupons are performing adequately, a few are well above the design model. In contrast, the performance of the galvanized steel coupons appears to be excellent. The galvanized coupons have a mean value of ≈ 2.0 μm/yr with a maximum observation of ≈ 3.3 μm/yr. Both of these values are below AASHTO design rate of 4.0 μm/yr. It appears that galvanized steel could perform well, in the backfill conditions of I-515/ Flamingo Rd.

4.4.3 Corrosion Severity Ratio (CSR)

The use of a corrosion severity ratio (CSR) allows for elements of differing age, metal type and environmental conditions to be compared. This is accomplished by normalizing the observed corrosion rates with the appropriate metal loss model. As was stated earlier in this chapter a CSR value of greater than or equal to 0.80 indicates “concern,” while a value greater than 1.0 represents “problem.” CSR values above 1.0 suggest that the metal losses will reduce the tensile capacity prematurely, and failure may occur before the end of its intended design life. I-515 and Flamingo Rd. data was converted to CSR values, using the bi-linear plain steel metal loss model, which predicts metal loss to occur at a rate of 12 μm/yr for elements beyond 2-years of age. Galvanized coupons were converted to CSR using the AASHTO Galvanized Metal Loss model.

4.4.3.A MMCE Data

Corrosion severity Ratio (CSR) statistics for the 2004 MMCE data are presented in **Table 4-11**. The mean value of 0.57 indicates that in general, the reinforcements are performing as anticipated and the corrosion levels are below the design levels. The maximum value of 4.72 recognizes there are areas with very severe corrosion. Fourteen percent (14%) of the data has a CSR value that is above the concern threshold of 0.80. Six percent (6%) of the total dataset has a CSR value > 1.0. This is interpreted as approximately six percent of the soil reinforcements are losing cross-sectional area in excess of the design conditions. This increases the probability that these elements will fail to provide the required tensile capacity necessary to maintain internal stability before the end of its intended design life.

4.4.3.B Phase II Data

The in-service reinforcement CSR data for Phase II is provided in **Table 4-12**. This subset of the Phase II data is provided for a direct comparison between the MMCE and Phase II datasets. The Phase II in-service elements have a mean CSR value of 0.53, with a maximum value of 5.01. This agrees well with the information derived from the MMCE data. The Phase II data indicates that the majority of the measured corrosion levels are below the design values with a limited number experiencing elevated metal loss. The Phase II calculations used a different set of environmental constants, which are in line with current state of practice. As with the MMCE data, the Phase II CSR calculations indicate that six percent of the elements are above a CSR of 1.0. There is slightly more scatter in the Phase II data, as indicated by the increased COV value (157% vs. 142%, respectfully).

The data from the coupons installed during the initial MMCE investigation are separated by metal type (**Table 4-13**). However once the corrosion rates are converted to CSR values this is unnecessary. In general, the galvanized coupons are performing better than the plain steel. This is observed in number of elements at or near the critical value of 1.0. The galvanized coupons have a slightly larger mean value (0.51 vs. 0.41), yet are more closely grouped (COV = 63% vs. 113%). The most significant difference, however, is there are 4 of the 14 (17%) plain steel coupons in with CSR > 1.0, while there are no galvanized samples experiencing this rate of deterioration.

The use of CSR allows the combination of data, through the normalization of the corrosion rates. The Phase II in-service and coupon CSR data are combined and the descriptive statistics are provided in **Table 4-14**. The combination of these data does not significantly change the values that are calculated from each of the smaller datasets. The most significant change is that the total Phase II dataset indicates that about ten percent of the elements are experiencing corrosion levels above the design values. In other words as much as 10% of all metallic elements at the I-515/Flamingo Rd. site could fail prematurely due to excessive metal loss.

4.4.3.C I-515 and Flamingo Rd. Lifetime CSR

The long-term LPR corrosion rate monitoring and subsequent CSR evaluation presents an overall picture of the corrosion severity. The CSR values obtained from the MMCE and Phase II studies are combined and presented in **Table 4-15.** The long-term CSR representation is very similar to the individual CSR datasets, which indicates that the corrosion levels experienced at this site have stabilized. The 95% confidence interval for the mean CSR value is between 0.42 and 0.64, demonstrating that mean corrosion levels are below the design values. However, at least eight percent (8%) of the data are at or above the design model (i.e. CSR ≥ 1.0). This implies that approximately eight percent of the elements may be experiencing metal loss in excess of the design considerations. The MSE walls at this site have an increased likelihood of premature structural deficiency.

4.5 In-Service Galvanized Soil Reinforcements

A total of 29 in-service soil reinforcements were instrumented and monitored during the spring of 2013. These elements were distributed through five of the seven sites included for Phase II. In addition to the 29 in-service elements, eight galvanized steel coupons from I-515 and Flamingo Rd. are also included in this dataset. The distribution of elements is provided in **Fig. 4-3.** In-service elements for the Phase II investigation are defined as, galvanized steel which has been in the backfill for more than two years. This definition allows the inclusion of galvanized steel coupons from I-515/Flamingo Rd., which were installed during the MMCE site investigation in 2004. The definition is partly upon the LPR environmental input parameters for galvanized samples. The galvanized elements use LPR input parameters that are an average of the zinc and plain steel parameters.

Precise locations of in-service soil reinforcements could not be identified for I-515 and Charleston Blvd. Though the soil reinforcements for I-15 and Lamb Blvd. were instrumented, electrical isolation of the reinforcements was not apparent. The readings obtained from these in-service elements were not included. The remaining five sites were monitored for in-situ corrosion evaluation and LPR corrosion rates were obtained. Due to the various walls ages, Stage II and Stage III corrosion was anticipated, as designated by the segmented AASHTO metal loss model.

4.5.1 In-Service Galvanized Soil Reinforcements Corrosion Rates With Respect to MSE Wall Backfill Specifications

In-situ corrosion rates are presented against the measured electrochemical soil parameters. The corrosion rate data presented was acquired from the five sites with in-service reinforcements that were able to be monitored using the LPR method. Data was gathered during the spring of 2013.

4.5.1.A In-service LPR Corrosion Rates and Chloride Content

In-service LPR corrosion rates are presented against the chloride results in **Fig. 4-4**. The majority 33 of 37 (89%) of the corrosion rate are for soil that meet the 100 ppm maximum limit. It is anticipated that corrosion rates will increase as the salt ion content is increased. The data obtained does not support this assumption. However, the LPR corrosion rates are instantaneous, and may not reflect the long-term conditions. Using all the data available there is no appreciable trend. When the data obtain from I-15 and Cheyenne Blvd. is not considered, a more stable relationship is seen. Although the reduced dataset appears more stable, no strong correlation is observed. Based on this it is concluded that the corrosion rates for in-service soil reinforcements are not predicted by the chloride content.

4.5.1.B In-service LPR Corrosion Rates and Sulfate Content

In-service LPR corrosion rates are presented alongside the sulfate results in **Fig. 4-5**. Due to the large range of values reported the information is separated into additional figures **Fig. 4-6** and **Fig. 4-7**, so that the data is more readable.

Slightly more than half (54%) of the corrosion rates are obtained from soils that meet the 200 ppm maximum requirement. There is large scatter present in the data. Based on all the data it appears that corrosion levels are significantly increased when the sulfate content is near or above the 200 ppm specification.

4.5.1.C In-service LPR Corrosion Rates and Minimum Soil Resistivity

Corrosion rates from the in-service galvanized elements are presented against soil resistivity in **Fig. 4-8**, **Fig. 4-9** and **Fig. 4-10**. Resistivity is often accepted as a critical indicator of the aggressiveness of the backfill environment. It is understood that resistivity alone cannot predict specific corrosion levels or rates. The established relationship states that the rate of metal loss increases as the minimum soil resistivity decreases. The data presented in **Fig. 4-8** supports this expectation. All of the very high corrosion rates occur at resistivity values below 3,000 ohm-cm, the AASHTO minimum requirement. Only 9 of the 37 (24%) observation are obtained from elements which are in backfills that meet this minimum specification (**Fig. 4-9**). Literature suggests that corrosion rates for elements in soils that are below this limit experience an increase in scatter (**Fig. 4-10**). When the soil exceeds the minimum required of 3,000 ohm-cm, the measured corrosion rates are considerably more stable. A corrosion rate of 10 μm/yr provides an adequate upper bound for these observations, which have a mean value of approximately 2 μm/yr. When the backfill is below the minimum soil specification (< 3,000 ohm-cm) the corrosion rate data are significantly more scattered. The expected trend of increased metal loss with decreasing resistivity appears to prevail. These data points indicate that the corrosion levels have a broad generalized trend. The mean corrosion level is ≈ 8 μm/yr, which is more than four times the rate associated with elements in backfill conditions meeting minimum soil resistivity specifications. This information supports the assumption that the corrosion rates are pseudo-random, exhibiting a predictable trend but not a specific value.

Corrosion rates obtained from I-15 and Cheyenne Blvd. are significantly greater than any other site. These demonstrate the presence of aggressive corrosion and suggest advanced metal loss at the complex. However, not all of the in-situ in-service reinforcement corrosion rates gathered from the Cheyenne Blvd. site are concerning.

4.5.1.D In-service LPR Corrosion Rates and pH

The in-service elements corrosion rates are presented against the backfill soils pH in **Fig. 4-11.** There is a single outlying data point from Jones Rd. which has been omitted from the figure. The data is closely grouped around a pH level of 9.0. All of the elevated corrosion rates are associated with a single site, I-15/ Cheyenne Blvd. Corrosion rates do not appear to be characterized by the backfill pH. There is no discernable pattern in the data in the relatively narrow range of pH observed during Phase II.

4.5.1.E In-service LPR Corrosion Rates and Organic Content

There is no identifiable trend present in the data. Corrosion rates appear to be unaffected by the level of organic matter. Metal loss rates are believed to have poor correlation with organic content. This assumption is supported by the I-15/ Cheyenne Blvd. data. Here high corrosion rates are associated with very little organic content.

4.5.2 In-Service Galvanized Soil Reinforcements Corrosion Rates With Respect to MSE Wall Location

In order to identify the MSE wall sites of greater interest, the in-situ LPR corrosion rate data from each site is presented together. The six sites that have in-service elements represent MSE walls that are either in Stage II or Stage III of the AASHTO metal loss model (galvanized passivation, 4.0 μm/yr or base steel consumption, 12.0 μm/yr, respectfully). The age dependency is not obvious in the data. However, the data does allow for locations of obvious concern to be identified and allows for a quantitative comparison of the Phase II LPR data. The observation places emphasis on the mean corrosion rate as well as the central tendency of the data. It is understood that concentration on the central corrosion rate information omits the extreme data, thus may underestimate the severity of the corrosion problem.

The Phase II corrosion rate data is grouped by site and wall number in **Fig. 4-13**. This figure clearly identifies I-15 and Cheyenne Blvd. Walls 1 and 4 as interesting. While the majority of the data is sub 10.0 μm/yr range, these two walls at the Cheyenne Blvd. complex are clearly above this threshold. The non-symmetric individual box-plots indicate that the distribution of corrosion rate data is not uniform. The box-plot for Cheyenne Wall 1 demonstrates that the majority of the data is above the 30.0 μm/yr range (median bar) with a few data in the sub 10.0 μm/yr range (lower quartile finger). In contrast to this is the corrosion rates for Wall 4. Wall 4's median corrosion rates is approximately 13.0 μm/yr, with a few extreme values creating wide ranges for 3rd and 4th quartiles. According to the AASHTO metal loss model, the expected corrosion rate for I-15 /Cheyenne Blvd. is 4.0 μm/yr, interpreted based on its age of 15 years at the time of mointoring. **Figure 4-13** clearly indicates metal loss rates in excess of the AASHTO model.

The mean corrosion rates and the 95% confidence intervals are calculated and presented in **Table 4-16**. The coefficient of variation can be used as an indicator of the stability for the mean and associated confidence interval. As noted previously, a value of 15% (COV) is used as threshold to indicate stability and precision. It is demonstrated that the galvanized in-service corrosion rates for I-15/Cheyenne Bld. Wall 5, I-515/Flamingo Rd. Walls 1 and 3, and US 395/Huffaker Lane Wall 1 are stable/ nearly stable with COV values less than 20%. This stability of the data suggests that the limited number of test results can sufficiently describe the corrosion rates of the wall structure. In contrast to this level of precision, the data from: Alt. US 50/Alt. US 95, I-15/Cheyenne Blvd. Wall 4, I-515/ Flamingo Rd. Wall 2 and SR 160/ Jones Rd. reveal substantial imprecision of the statistics. This implies that while the measurements describe the conditions of the individual elements they do not capture a complete description of the corrosion rates. Due to the limited number of LPR data. Interpretations about a site's performance should be based on the site-wide information.

Combining the individual wall data into a larger dataset for each site the in-service LPR corrosion rate data is presented in **Fig. 4-14** and **Table 4-17**. As before when these data were presented, the I-15/Cheyenne Blvd. site stands out. It is observed that the majority of in-service LPR corrosion rate data for this site are in the sub 15.0 μm/yr range. This is demonstrated by the compact first and second quartile data. The information contained in the lower half (minimum to median) of the data spans a range of 12.5 μm/yr while the upper half of the data has a range of 43.9 μm/yr (median to maximum). Noteworthy observations can be made with the Alt. US-50 and Alt. US-95 data. For this site the lower 50% of data is very compact, and nearly negligible. While, the upper half of the data has a range of nearly 8.0 μm/yr. The dispersion of corrosion rates with the other sites is lost due to the scale required to accommodate the Cheyenne Blvd. complex.

The descriptive statistics for each of the site wide LPR corrosion rates (**Table 4-17**) reveal that the data obtained is unstable based on a minimum COV value of 23%. This can be interpreted as an indication that the data obtained is insufficient to provide a stable statistical description of the corrosion rates. However, the data obtained from: I-515/Flamingo Rd., I-515/Las Vegas Blvd., SR 160/ Jones Rd., and US-395/ Huffaker Lane indicate that the variation within the data is minimal. The standard deviations near 1.0 μm/yr for the previously mentioned sites suggest that while the data is not statistically stable, the application of the data is meaningful. This then suggests that the data obtained can be used reliably to assess the corrosive conditions of each of the above sites.

In contrast to the reasonably stable corrosion rates associated with the aforementioned sites, the statistics for both Alt. US-50/ Alt. US-95 and I-15/ Cheyenne Blvd. present a differing situation. The COV and standard deviation for both exhibit non-uniformity and lack of central tendency of the LPR corrosion rate data. This behavior is also observed in the previous box-plot figures (**Fig. 4-13** and **Fig. 4-14**). The statistics for Alt. US-50/ Alt. US-95 are slightly misleading, due to a single corrosion rate record having substantial influence. Three of the four calculated corrosion rates are below 0.8 μm/yr, with the fourth observation at ≈8.0 μm/yr. The limited data allows the mean and central tendency statistics to be significantly influenced by the single data point, resulting in a lack of robustness. A better descriptor for this data would be the median value of 0.7 μm/yr. The mean value inadvertently allows for a more conservative analysis, because of its ability to be influenced by a few extreme values.

The data from I-15 and Cheyenne Blvd. has similar behavior as the Alt. US-50/ Alt. US-95 wall, except the data is influenced by three low corrosion rates. Of the ten observations, three would be considered in the low range, while four are considered very high. In closer examination reveals that two of the three low metal loss values are associated with a single monitoring station. The I-15/ Cheyenne Blvd. data leads to the conclusion that the data obtained from the LPR monitoring cannot adequately predict a rate of metal loss, but confirms the suspicion of elevated corrosion levels. It should also be noted that the limited data collected does not capture the maximum or minimum corrosion levels experienced throughout the entire complex.

4.5.3 In-Service Galvanized Soil Reinforcements Corrosion Severity Ratio

The corrosion severity was calculated for all in-service galvanized elements using the AASHTO metal loss model (**Table 4-18**). Even when data was near the transitional periods, strict adherence to the model was used. This allows for consistency with the design assumptions and service life predictions. In general, the mean CSR values indicate that the galvanized metal elements are performing sufficiently to achieve the long-term performance goals. There are only two sites that have CSR values greater than 1.0 with an additional site above the 0.8 concern level.

The maximum CSR value of ≈2.0 for Alt. US-50/ Alt. US-95 is the only value for the site that is above the 1.0 threshold. However, it should be noted only one element of the four produced a CSR value greater than 1.0. This implies that there are significant regions of elevated corrosion within the structure and that the corrosion rates are randomly distributed throughout the site. Based on the available data, the 95% confidence interval for the mean CSR is between 0.0 and 1.6.

The data for I-15/Cheyenne Blvd. reveals that the site is experiencing extremely high corrosion levels. The mean value of ≈5.0, (95% confidence range (1.9 – 8.3)) and a COV value of greater than 100% confirm the suspicion of a corrosion problem. There are ten CSR values for the Cheyenne Blvd. site, of which seven are above 1.0. This implies that nearly 70% of the galvanize in-service soil reinforcements are experiencing metal loss in excess of the design rate of those elements with the mean rate of metal loss being approximately five times greater. The excessive metal loss can lead to premature reduction of tensile capacity and possible wall failures. The magnitude of the I-15/Cheyenne Blvd. CSR values is clearly seen in **Fig. 4-15**, which reveals the scale of the corrosion severity.

4.5.4 Galvanized In-Service Reinforcements Combined Evaluation (All Sites)

All the galvanized in-service soil reinforcement LPR corrosion rate data were combined for a generalized statewide assessment (**Table 4-19**). The combined data has a mean uniform corrosion rate of approximately 6.7 μm/ yr which corresponds to roughly two-thirds of the 75-year equivalent metal loss rate, and is approximately 160% of the stage II corrosion rate (i.e. 4 μm/ yr). The large COV value (200%) implies that there is significant scatter within the dataset. The large COV also indicates that statistically the mean value is unstable and not a robust indicator of the central value. The confidence range (95%) places the mean value between 2.4 and 11.0 μm/ yr, supporting the observations that a global value is unacceptable for corrosion assessment and predictions. As was stated previously, I-15/ Cheyenne Blvd. has substantially high corrosion rates and they do not fit well with the remaining sites (e.g. **Fig. 4-15**). Descriptive statistics were recalculated for the state LPR corrosion information, with the Cheyenne Blvd. information removed from the dataset. This results in a considerably decrease in all values. The mean value reduces to approximately 2 μm/ yr a reduction of over 400%. Similar reductions are observed for the other statistical descriptors. This phenomenon supports the observation that unique conditions and excessive corrosion is present with the Cheyenne Blvd. complex.

4.6 Phase II Zinc Elements

Galvanized steel coupons, which have been installed for less than two years, are assumed to have the corrosion process controlled by the zinc coating applied during the galvanization process. During which the underground galvanic corrosion process is characterized by the rapid consumption of the loose zinc ions (Stage I, **Fig. 2-3**) (Padilla et. al. 2013; Ghods et. al. 2013). After two years the galvanized reinforcements have been partially passivated through the participation of zinc oxide along the surface. During this transition stage (Stage II, **Fig. 2-3**) the metal is neither zinc nor the base carbon steel. After the initial two years the corrosion process changes slightly. These differing conditions are reflected in the choice of environmental constant and material specific constants for the LPR calculations. During the infant years (less than 2-years of service) the environmental and material specific input parameters are based on zinc whereas galvanized elements are modeled as an average between zinc and steel. All initially galvanized elements that have been in service for greater than two years are modeled as galvanized.

The use of sacrificial galvanized coupons allows simulation of these early stages of underground galvanic corrosion. These are then used in conjunction with in-situ reinforcement monitoring to provide a more complete account of the soil reinforcement deterioration. A total of 75 zinc elements (i.e. galvanized coupons less than two years old) were monitored. The distribution of the samples is provided (**Fig. 4-16)**. This bar graph clearly illustrates the influence that both I-15/ Cheyenne Blvd. and SR 160/ Jones Rd., have on the dataset.

4.6.1 Corrosion Rates of Zinc Elements with Respect to MSE Wall Soil Conditions

In-situ corrosion rates for the zinc elements is presented against the set of electrochemical soil specifications. The corrosion rate data presented was acquired during the summer of 2013 from the seven sites where sacrificial galvanized coupons were installed as part of the Phase II Nevada corrosion investigation. Due to complications in connecting to in-service galvanized soil reinforcements for I-15/Lamb Blvd. and I-515/Charleston Blvd, the galvanized coupons that were installed at these sites provide the only indication on the performance of the galvanized in-service soil reinforcements.

4.6.1.A Zinc Elements LPR Corrosion Rates and Chloride Content

Zinc element LPR corrosion rates are presented against chloride content in **Fig. 4-17**. The majority 73 of 75 (97%) of the data was collected with soils that meet the 100 ppm maximum limit. The assumed relationship between chloride content and backfill aggressiveness is the rate of metal loss will increase along with the salt (chloride) content. The data obtained does not support this assumption. Based on this information it is concluded that the corrosion rates for zinc elements are not predicted by the chloride content. Furthermore, corrosion rates are randomly distributed given a specified chloride content. The LPR corrosion rates are instantaneous measurement and may not reflect the long-term conditions. Sequential monitoring is needed to establish robust evaluations of corrosion conditions.

4.6.1.B Zinc Element LPR Corrosion Rates and Sulfate Content

Zinc element LPR corrosion rates are presented against the sulfate testing results in **Fig. 4-18**. Forty-eight (48) of the 75 (64%) are obtained from soils that meet the maximum limit of 200 ppm. There is significant scatter in the data. The trend of corrosion data appears to be proportional to the sulfate content. The data becomes increasingly sparse for sulfate values greater than 300 ppm. No appreciable trend is observed form the figure. This informations suggest that the rate of metal loss is not predicted by the sulfate content alone and that corrosion rates are random for a given sulfate content.

4.6.1.C Zinc Element LPR Corrosion Rates and Minimum Soil Resistivity

Corrosion rates from zinc elements are presented against soil resistivity in **Figures 4-19** (total spectrum) and **4-20** (ρ < 3,000 ohm-cm). Resistivity is generally accepted as a critical indicator of the backfill corrosiveness. The rate of metal loss is predicted to increase as the minimum soil resistivity decrease. While there is no robust predictive models that use resistivity to predict corrosion rates, it has been demonstrated that at low resistivity values the corrosion rate increase and become increasing scattered (Elais et. al 2009; Fishman and Whitham 2011; Padilla et. al 2013; Romanoff 1957). The data presented in **Fig. 4-19** supports these observations. There is a noticeable trend in the data, with higher corrosion rates occurring at resistivity values below 3,000 ohm-cm. However, only 18 of the 75 (24%) observation are obtained from elements installed in backfills that meet the AASHTO requirement. The trend in the data is less obvious when only data below 3,000 ohm-cm (**Fig. 4-20**) is examined. When the backfill exceeds the specification, the recorded corrosion rates are considerably more stable. For resistivity values greater than 3,000 ohm-cm, an upper envelope of approximately 4 μm/yr appears to adequately capture the metal loss rates. These data points have a mean value of approximately 0.1 μm/yr, well below the AASHTO metal loss model value of 15 μm/yr.

When the backfill is below the minimum soil specification, the corrosion rates are significantly more scattered. Data near 1,000 ohm-cm indicates an increase in metal loss with the data becoming progressively more scattered as the resistivity value decreased. The LPR data suggest that the corrosion levels are broadly generalized by the resistivity results. No definitive correlation is observed. The information supports the hypothesis that the corrosion rates are pseudo-random having and predictable trend but, not specific value.

4.6.1.D Zinc Element LPR Corrosion Rates and pH

Zinc element corrosion rates are presented against the backfill soil pH in **Fig. 4-21**. There is a single outlying data point from SR 160/ Jones Rd, while the rest of the data is grouped around a pH level of 9.0. The highest corrosion levels are associated with a pH value near 9.2

4.6.1.E Zinc Element LPR Corrosion Rates and Organic Content

The largest corrosion rates (**Fig. 4-22**) are associated with very low organic content. Only 13 of the 75 (17%) of the data is obtained from soils that exceed the maximum value of 1.0% organic content. The data appears to be less scattered when the backfill soil meets specification.

4.6.2 Zinc Elements Corrosion Rates with Respect to Phase II MSE Wall Location

In order to identify MSE wall sites that deserve additional attention, in-situ LPR corrosion rate data from each site is presented together. Zinc elements represent the initial stage of the galvanized corrosion process and the first segment of the AASHTO metal loss model (corrosion rate 4.0 μm/yr, t < 2-years). The data does allow for the identification of locations of obvious concern and a quantitative comparison of the Phase II LPR data. This information is the only metal type specific corrosion rate data for both I-15/Lamb Blvd. and I-515/Charleston Blvd.

The following information places emphasis on the mean corrosion rate as well as the central tendency of the data. It is understood that concentration on the central corrosion rate information omits the extreme data, thus may underestimate the severity of the corrosion. However, due to the sparse data it provides an adequate measure of a site's condition.

The zinc corrosion rates are grouped by site and then to specific walls and presented in **Fig. 4-23**. There is no standout location based on this information. Lack of fingers on the figures is indicative of the sparse data that was used, and therefore, the data has been combined for further examination on a site wide basis (**Fig. 4-24**). Using this information an indication of the distribution within each site can be estimated. In general, a few higher corrosion rates expand the range of corrosion. This phenomenon is seen by the unsymmetrical nature of the box-plots.

The mean corrosion rates and the 95% confidence intervals are presented in **Table 4-20**. The coefficient of variation is used as an indicator of the stability for the mean and associated confidence interval. None of the data suggest stability. All COV values are greater than 15% with two site having values above 100%. A maximum corrosion rate of 13.5 μm/yr occurs at I-15/Lamb Blvd. This is a local maximum and is still below the design corrosion rate (15 μm/yr). This suggests that despite backfill conditions, the zinc metal loss is below the AASHTO galvanized metal loss model.

4.6.3 Zinc Elements Corrosion Severity Ratio

The corrosion severity was calculated for all zinc elements using the AASHTO metal loss model (**Table 4-21, Fig. 4-25**). The CSR values indicate that the zinc elements are performing sufficiently to achieve the long-term performance goals. From **Fig. 4-25** it is observed that both I-15/Lamb Blvd. and I-515/Charleston Blvd. have the largest overall CSR values, with the remaining five sites experiencing similar behavior. The mean CSR value for the five similar sites is ≈ 0.1; in other words the LPR corrosion rates are approximately 10% of the design value. I-15/Lamb Blvd. has a mean value of 0.4 with a maximum of 0.9. Although this is above the norm, it is below the design levels.

4.6.4 Zinc Elements Combined Evaluation

All the zinc element data were combined (**Table 4-22**) for a generalized statewide assessment. The combined data has a mean uniform corrosion rate of 1.9 μm/ yr, which corresponds to roughly fifteen percent of the design metal loss rates. This information indicates during the first two years after installation the consumption of the zinc coating is slower than anticipated. This suggests that the benefits of the galvanization process could exceed the 16-years assumed under the AASHTO design method.

4.7 Phase II Plain Steel Elements

Plain steel elements are available at the seven sites of Phase II and at I-515/ Flamingo Rd. Along with the steel coupons from the seven Phase II sites, in-service elements and coupons are available at the I-515/ Flamingo Rd. site. The plain steel coupons represent the long-term conditions of the soil reinforcements.

This is the third stage of the galvanized metal loss process (according to AASHTO metal loss model, t > 16-years), which is characterized by the consumption of the base steel. In this third stage metal loss leads to the depletion of the structural cross-sectional area of the soil reinforcements. The LPR data of 72 plain steel coupons installed/ monitored during 2013 as well as 60 plain steel soil reinforcements from 1-515/Flamingo Rd. are used in the analysis. The distribution of the data is presented in **Fig. 4-26.**

4.7.1 Corrosion Rates of Plain Steel Coupons and MSE Wall Backfill Specifications

In-situ corrosion rates for the Phase II sacrificial plain steel coupons are presented against the electrochemical soil specifications. Focus in the following sections is placed on the Phase II data. The information obtained from I-515/Flamingo Rd. is not included. The data is presented for each of the Phase II sites.

4.7.1.A Plain Steel Coupons LPR Corrosion Rates and Chloride Content

LPR corrosion rates for plain steel coupon are presented against the chloride results in **Fig. 4-27**. Nearly all (71 of 72, 99%) of the corrosion rates are for soils that meet the 100 ppm maximum limit. There is no appreciable predictable trend in these data. However, generally accepted trend is for corrosion rates to increase with an increased levels of salts. Based on this information it is concluded that the corrosion rates for plain steel elements are randomly distributed with respect to chloride content.

4.7.1.B Plain Steel Coupons LPR Corrosion Rates and Sulfate Content

Plain steel element LPR corrosion rates are presented against the sulfate testing results in **Fig. 4-28**. Forty-eight of the 72 (67%) of the corrosion rates are obtained from soils that meet the 200 ppm maximum requirement. Large scatter is present in both the corrosion rate and sulfate content data. From these data it appears that the corrosion rates increase substantially around the 200 ppm limit. There is no appreciable predictable trend, and therefore it is concluded that the corrosion rates for plain steel coupons are randomly distributed with respect to sulfate content.

4.7.1.C Plain Steel Coupons LPR Corrosion Rates and Minimum Soil Resistivity

Corrosion rates from plain steel coupons are presented against soil resistivity in **Fig. 4-29.** When the backfill is below the minimum soil resistivity specification (min. of 3,000 ohm-cm), the corrosion rates are significantly more scattered. These data points indicate that the corrosion levels have a broad generalized trend, and they support the assumption that the corrosion rates have predictable trend but not specific value.

4.7.1.D Plain Steel Coupons LPR Corrosion Rates and pH

Corrosion rates for plain steel coupons are presented against the backfill soil pH in **Fig. 4-30**. All of the soils pass pH requirements. There is no discernible trend in the data. Corrosion rates are randomly distributed throughout the range of pH values for Phase II plain steel coupons.

4.7.1.E Plain Steel Coupon LPR Corrosion Rates and Organic Content

Corrosion rates for plain steel coupons are compared against the organic content of the backfill. The data in **Fig. 4-31** demonstrates substantial scatter. There is no discernible trend in the data.

4.7.2 Plain Steel Elements Corrosion Rates with Respect to MSE Wall Location

In order to identify locations of greater concern, the in-situ LPR corrosion rate data from each site has been combined (**Fig. 4-32**). Plain steel coupons represent the final stage of galvanic corrosion, which is characterized by the consumption of the base carbon steel. According to the AASHTO model, the steel corrosion is expected to be very aggressive (e.g. corrosion rate of 45.0 μm/yr) for the first 2-years, after which it stabilizes at 12.0 μm/yr. The plain steel second corrosion stage is synonymous with the third stage galvanized corrosion. In either case, the consumption of steel resulting in a reduction of the cross-sectional area. Plain steel coupons that have been installed for more than two years provide clues to the long-term rate of metal loss. Included in this analysis are the 60 LPR corrosion rates obtained from the I-515/Flamingo Rd. complex. The analysis places emphasis on the mean corrosion rate as well as the central tendency of the data. Concentration on the central corrosion rate information omits the extreme data, thus may underestimate the severity of the corrosion and does not account for localized conditions. However, due to the sparse data available, the central values are expected to provide an adequate measure at a site.

The plain steel corrosion rates are grouped by site, then wall number and are presented in **Fig. 4-32**. No site is identifiable as unique, and there is some uniformity throughout the dataset. Lack of fingers on individual box-plot figures is a result of the sparse data that was available. Therefore, the data has been combined to get a site-wide dataset (**Fig. 4-33**). This information provides an indication of the corrosion rate distribution within each site. In general, a few high corrosion rates expand the range of corrosion, signifying that the mean value is unstable and susceptible to influence of extreme values. This behavior also indicates that dispersion (i.e. COV) can be used to quantify the stability of the mean. This phenomenon is seen from the un-symmetrical nature of the box-plots. With the exception of the single very high corrosion rate (≈60 μm/yr) at the Flamingo Rd. site, all of the sites are experiencing similar conditions for plain steel elements.

The mean corrosion rates and the 95% confidence intervals are calculated and presented in **Table 4-23**. The coefficient of variation is used as an indicator of the stability for the mean and associated confidence interval. None of the data suggest stability. The scatter in the LPR corrosion rates is significant and is supported by the high coefficient of variation value. A maximum corrosion rate of 28.2 μm/yr for the plain steel coupons occurs at US-395/ Huffaker Lane, while a maximum corrosion rate of 60.2 μm/yr was observed for the in-service steel reinforcements at I-515/Flamingo Rd. All of the calculated corrosion rates for plain steel coupons are below the Stage I design metal loss rate of 45.0 μm/yr. The scatter in the data suggests that the corrosion rates are generally unpredictable. The local maxima of the Flamingo Rd. site is significantly above the Stage II design rate of 12.0 μm/yr. There are a total of six data points (10%) which are above the design rate at the I-515/Flamingo Rd. site. This suggest that approximately 10% of the MSE wall reinforcements are undergoing metal loss in excess of the design conditions. Therefore, the probability of a premature reinforcement/ wall failure exists.

4.7.3 Plain Steel Elements Corrosion Severity Ratio

The corrosion severity ratio was calculated for all plain steel elements (plain steel coupons and in-service reinforcements) using the bi-linear carbon steel metal loss model (Stuttgart metal loss model, **Table 4-24, Fig. 4-34**). The CSR values indicate that the plain steel elements are performing adequately and are below the long-term performance constraints. From **Fig. 4-34,** it is observed that I-515/Flamingo Rd. has the largest CSR values. The reaming seven sites are experiencing similar behavior and are below the concern level of 0.80. The mean CSR value for the Phase II sites is approximately 0.1. In other words the LPR corrosion rates are roughly 10% of the design value. While the mean CSR value for I-515/Flamingo Rd. is 0.50, the maximum value is nearly 5.0. This data suggests that the electrochemical backfill conditions at I-515/ Flamingo Rd. are the most reactive in regards to plain steel depletion. It is hypothesized that for the seven Phase II sites, the corrosion of the base steel is at or below the design level.

Chapter 5 Application of Phase II LPR Corrosion Rate Data

5.1 Assumptions and definitions

For clarity the following terms are defined as they are used with the application of the Phase II LPR data.

Design Life: The intended useful life, in which minimum or no impact to structural capacity is anticipated. This is assumed to be 75-years for all the sites included in this site.

Reinforcement Failure: The point when the metal loss is more than the design anticipates. For the AASHTO metal loss model this is 794μm (zinc coating and sacrificial carbon steel). This is a functional failure which may or may not lead to a structural failure of the reinforcement. This is the point after which metal loss impacts the service life of the reinforcements and MSE wall structure.

5.2 Application of the AASHTO Metal Loss Model with Phase II LPR Corrosion Rate Data

The AASHTO metal loss model is the national standard for galvanized MSE wall soil reinforcements. The model is based on both physical observations and practical considerations. AASHTO guidelines place requirements on the backfill so that only "mildly corrosive" environment exists throughout the design life (**Table 2-2**). By restricting the physiochemical/ electrochemical characteristics of the select granular backfill material in such a way that the electrolytic potential of the soils is limited. This is accomplished using a series of pass/fail backfill specifications (see Chapter 3 for greater detail). Under these conditions, the AASHTO metal loss model is applicable. Even though a brief summary was provided previously, some important aspects are reviewed with specific attention given to statistical interpretation of the corrosion phenomenon.

The AASHTO metal loss model reflects the three stages of galvanic corrosion. These stages are represented in the segmented linear metal loss model in which a finite time range for each stage is assumed and they are associated with a corresponding uniform corrosion rate. The AASHTO metal loss model provides an upper envelope for the anticipated corrosion rates (Gladstone et. al. 2006). This model does not take into account a transitional period as the stages of galvanized corrosion proceeds, but rather assumes a finite time period after which the corrosion rate abruptly changes. The rate of metal loss for the base steel is consistent for both the galvanized and plain steel models. Thus, the real impact of this model is the establishment of a restrictive electrochemical backfill and the prescribed method for the zinc consumption along a galvanized element. Inherent to this model is the dependency for metal type, backfill characteristics, and age of the element. In the analysis of Phase II LPR data, the inherent characteristics and dependencies are accounted for through the use (conversion to) of Corrosion Severity Ratio (CSR). LPR data is then normalized with the appropriate metal loss model.

The use of the CSR allows for comparison between various corrosion environments and this then can be used to directly compare the corrosion conditions from one element to that of another in a different environment (i.e. corrosion rates from different corrosion stages, different metal types, backfill conditions that do not meet AASHTO requirements).

5.3 Application of the AASHTO 75-year Equivalent Metal Loss Model (AEML)

The AASHTO segmented linear metal loss model reflects the physical variations of the galvanized corrosion process. This can be simplified into a single uniform corrosion rate for the design life. The AASHTO 75-year equivalent metal loss model (AEML) is a single uniform corrosion rate that results in the same total consumption of metal. This includes the consumption of the zinc layer and subsequent base steel. Using the assumption that a single corrosion rate can be used to describe the galvanized metal loss, it can be extended to incorporate the LPR data.

Although the AEML rate does not have a physical meaning it is a simpler and useful way to estimate the severity of a discrete corrosion rate measurement. Using the assumption that a single corrosion rate can describe the metal loss is, the AEML concept is extended to the LPR data. Under this ideology the severity of the measured LPR corrosion rate is assumed to be a constant for the life of the element. This corrosion rate will then be converted to an AELM-CSR value and used to predict the metal loss and impact of the corrosion rates on the service life of the elements at a site.

5.3.1 AEML and Service Life impact

The mean AEML-CSR value for each site is used as a multiplier to the AEML base corrosion rate (10.6μm/yr, refer to Chapter 4 for details). The sacrificial thickness is then divided by this new rate to estimate a new design life based on the AEML-CSR corrosion rate. This provides a hypothetical service life of the MSE wall soil reinforcements. The descriptive statistics for the AEML-CSR using all elements at a site are compiled (**Table 5-1**) and used to calculate the equivalent 75-year corrosion rate (**Table 5-2**). It should be emphasized that the LPR values measured during the Phase II field investigation used in this evaluations represent an instantaneous uniform corrosion rate. A major influencing factor is the LPR measurements from the recently installed coupons. This reveals that the mean 75-year equivalent corrosion rate for all the sites is below the AEML baseline. Indicating that based on central tendency of the data, most of the reinforcements will achieve the desired design life. However, careful consideration must be placed on the maximum values. Due to the sparse data used to characterize each site, there is a certain likelihood that the true maximum values were not recorded. Six of the seven sites have AEML CSR maxima at or above 1.0. This suggests that while the majority of the elements are experiencing acceptable levels of corrosion a few elements are experiencing aggressive conditions.

Using the mean 75-equivalent corrosion rate the service life of the MSE wall sites has been estimated. This estimate reflects the time required for complete consumption of the sacrificial metal thickness. The service life is calculated using **Eq. 5-1** as follows:

$$Service\ Life_{AEML-CSR} = \frac{Total\ Sacrifical\ Metal}{AEML-CSR \times AEML} \qquad \textbf{(5-1)}$$

$$Service\ Life_{AEML-CSR} = \frac{794\mu m}{0.61 \times 10.6\ ^{\mu m}/_{yr}}$$

$$Service\ Life_{AEML-CSR} = 122.6\ yr$$

After which time the structural capacity of the reinforcements is diminished, through the reduction of structural cross-section. This is a deterministic approach based on the use of mean values, where the assumption is that a single corrosion rate can be used to describe galvanic corrosion (e.g. three stage) rate process. The data shown in **Table 5-3** is very optimistic with a minimum service life of nearly 123 years (I-15 and Lamb Blvd.). While these values imply superior performance of the reinforcements, the field-measured electrochemical characteristics of the backfill suggest that the opposite could occur. The performance expectations calculated using the AEML-CSR are heavily influenced by the recently installed coupons, and should be revaluated with subsequent LPR measurements.

5.4 Probabilistic Approach to Corrosion Rate Predictions

Due to the sparse data available at each site, the LPR data was combined into a single comprehensive dataset. Two such datasets examined in the following sections are: data from galvanized/ zinc elements and the complete dataset (inclusion of all metal types, age, and backfill conditions). A cumulative distribution function (CDF) is created for each of the datasets.

The CDF curves (**Fig. 5-1** and **5-2**) can be used to predict the likelihood of a corrosion rate; however they do not predict the distribution of such rates. The CDF value is based on all data available and is not applicable to small localized area, i.e. corrosion stations.

5.4.1 CDF for Galvanized and Zinc Elements

The galvanized and zinc elements are analyzed independently of the plain steel elements. These data are combined to create a larger dataset that neglects the backfill conditions and age (corrosion stage). This larger dataset was then separated based on the pass/fail criteria for minimum soil resistivity. The total dataset contains 110 LPR corrosion rates, while the failing minimum resistivity dataset contains 83 data points. All three datasets have been overlaid on a CDF. **Figure 5-1** demonstrates the ability of the backfill conditions to control the corrosion rate.

Based on this figure a probability table was created (**Table 5-4**). The relationship between the rate of metal loss and backfill conditions is evident. There is approximately a 70% reduction in the corrosion rate between the passing specification and failing datasets. The closeness of the all elements and failing specification CDF is due to the majority of (83 of 110, 75%) the data is in soils that fail specification. This complete dataset represents all three stages of galvanized corrosion and therefore it is analogues to the AEML corrosion rate. Based on this analogy there is up to a 7% chance that the corrosion rates will exceed the 75-year equivalent metal loss rate.

5.4.1 CDF for All Element Types

Using all the data obtained from Phase II LPR testing a pair of CDF curves was created for all element types. In order to combine all the data, the LPR corrosion rates were converted to CSR values, based on the AASHTO metal loss model and the AASHTO equivalent metal loss model. This method allows the severity of the corrosion to be predicted. Once established, this value can be used to modify the appropriate corrosion rate models based on the MSE wall conditions (i.e. metal type and age). The curves presented in **Fig. 5-2** are similar for both high and low probabilities.

The AASHTO CSR curve predicts that 95% of all elements will be at a CSR value of 1.0 or lower. This establishes that approximately 5% of all elements are experiencing metal loss in excess of the design metal loss. If the 0.80 CSR values is used as the limiting value, the probability increases to 7% of the elements. A CSR value of 0.80 is used to indicate elements of concern, based on the assumption that the AASHTO metal loss model is conservative therefore any corrosion rate approaching the design level should be a concern. The AASHTO-CSR curve indicates that there is a small percentage (2%) of elements that will experience metal loss in excess of four times (CSR ≥ 4.0) the design levels. The maximum CSR of more than 14.0, illustrates that very aggressive metal loss is possible on a few elements.

The AEML-CSR curve is more conservative for CSR values less than about 2.2. Based on the equivalent metal loss rate approximately 9% of all elements are experiencing metal loss above the 75-year uniform rate. Between CSR values of 0.6 and 1.0 there is approximately a 4% difference between the two curves. A key difference between the two curves is the maximum value that is predicted, 14.0 and 5.5 (AASHTO and AEML, respectfully). The implication is that while the AEML CDF predicts a greater number of elements experiencing aggressive metal loss, the AAHSTO CDF indicates the probability of substantially higher metal loss.

Chapter 6 Conclusion

The conclusions presented below are based upon the data obtained under the Phase II investigation. The following three distinct groups of data were utilized: electrochemical soil characteristics, LPR corrosion rates, and overall site characteristics which combines the soils and metal loss data. This allows for each of the two datasets to have an independent conclusion and then a generalized observation based on all available data.

6.1 Electrochemical Soil Characteristics

Literature reveals that there is a strong documented relationship between electrochemical soil conditions and the associated metal loss. The use of backfill characteristics to predict and control corrosion is the underlying objective of the MSE wall backfill specifications.

6.1.1 Phase II MSE Wall Backfill Testing

A total of 72 MSE wall backfill samples were collected and tested for the five electrochemical specifications. This data was then compared with the As-Built soil testing records. The Phase II data more than doubled the previous soil data and provides a better characterization of the MSE wall backfill conditions. Based on the Phase II backfill testing, the severity of the sites have been ranked relative to individual electrochemical requirement, see Chapter 3 for details. These rankings are now combined to produce an overall (or composite) rating in **Table 6-1.** This table concludes that Alt. US-50/ Alt. US-95 has the most severe overall backfill conditions. When severity ratings were tied, LPR data was considered to break the tie.

6.1.1.A Chlorides

In general, the chloride testing results indicate very little concern. Only two of the 72 (3%) samples exceed the 100 ppm maximum limit. The majority of the soils tested are below 25 ppm. Chloride content is a minimal concern for Nevada MSE wall backfills.

6.1.1.B Sulfate

Sulfate testing results indicate some concern. Twenty-five (25) of the 72 (35%) samples exceed the 200 ppm maximum limit. Three sites have a mean value above specification (Alt. US-50/ Alt. US-95 (236 ppm), I-15/Charleston Blvd (653 ppm), I-515/Las Vegas Blvd. (804 ppm)). All but US-395 and Huffaker Lane had at least a single sample failing requirements. Sulfate content is of moderate concern for Nevada MSE wall backfills.

6.1.1.C Minimum Soil Resistivity

Soil resistivity is often a critical indicator of the corrosiveness of the backfill. Only I-515/ Las Vegas Blvd. did not have a sample failing specification for soil resistivity. Of the 72 samples tested 55 (76%) failed specification. The mean value for five of the seven sites is below specification. The maximum value of 7,430 ohm-cm is low when compared with national data. There are more samples with values below 1,000 ohm-cm than there are above 3,000 ohm-cm AASHTO requirement. Soil resistivity is a primary concern for Nevada MSE wall backfills.

6.1.1.D pH

The pH testing results indicate no concern. All of the samples are with the required range for pH. In general, the soils are mildly alkaline with a pH value near 9.2. The pH is a minimal concern for Nevada MSE wall backfills.

6.1.1.E Organic Content

Organic content of the sites in NDOT District 2 may be of concern. Nine of the ten samples that exceeded the 1.0% maximum value occurred in District 2. Alt. US-50/ Alt. US-95 is problematic as it is a relatively new site that failed organic content limit (mean value 1.2%). Due to the age of the walls at US-395/ Huffaker Lane the excessive organic content is not surprising, however it does fail to meet current AASHTO backfill specifications. In general, organic content for NDOT District 2 is problematic while it is of minimal concern for District 1 MSE wall backfills.

6.1.2 Phase I and Phase II Comparison

The Phase II electrochemical testing results confirm the predictions from Phase I. Each of the seven sites tested had at least 25% of the test data failing a specification. As a result of the Phase I effort, a regression correlation for the resistivity value was created, and selection of sites of interest was based on several factors. The Phase II test data supports the site identification process and raises concern regarding the other electrochemical tests. It is unclear at this time if the variation in soil characteristics is the result of natural randomness, the limited data used during conditions or time dependent phenomena.

During Phase I it was suggested that the backfill soils for I-15 and Cheyenne Blvd. were statistically different based on ANOVA hypothesis testing. With the creation of a third and much larger dataset (Phase II) for this site, statistical significance was once again tested using the non-parametric Mann-Whitney hypothesis test. Phase II refutes the previous conclusion of dataset's independence. It is possible that all three datasets are from the same population. However, even though the Phase II investigation was more comprehensive, the statistical variation (as indicated from high COV values in excess of 15%) suggest that the characterization of backfill still may not be adequate.

6.2 LPR Corrosion Rates

The LPR data presented in Chapters 4 and 5 is the basis for the conclusion presented in the following text. All conclusions are based on the Phase II LPR data independent of the backfill soil conditions. Emphasis is placed on in-service reinforcements when available.

6.2.1 I-15 and Flamingo Blvd.

The Phase II data collected during the revisit of I-15/Flamingo Rd (**Fig. 6-1**) supports the previous LPR data and conclusions (2004 MMCE report). While measurements for identical elements have changed, the overall site conditions is consistent. The Phase II LPR corrosion rate data profile compares well with the MMCE LPR corrosion rate profile (**Fig. 4-2**). The data supports the metal loss model stabilizing after an initial time period. The mean corrosion confidence range (95%) is 2.8 to 9.3 μm/yr which compares well with the MMCE dataset. In each dataset there are a few elements which have aggressive metal loss. Approximately 6% of the elements have a CSR value above 1.0. This value is consistent for both datasets and suggests that up to 6% of the soil reinforcements will fail service life requirements prematurely due to excessive metal loss.

6.2.2 LPR Data of Phase II Sites

The LPR data represents an instantaneous uniform corrosion rate. This can be used to estimate the rate of metal loss and the severity of the underground galvanized corrosion process. With the development of periodic dataset, more refined conclusion can be achieved. The current Phase II conclusions are based on site and/or wall datasets for each of the seven sites investigated. Due to the limited number of data, it cannot be confirmed that excessive corrosion is not present. However, based on existing information it is concluded that even if present, it is limited in extent.

6.2.2.A US-395 and Huffaker Lane

The LPR data for US-395 and Huffaker Lane (**Fig. 6-2**) obtained from both walls is considered as a single dataset, thus it represents the entire complex. Based on the Phase II LPR corrosion rates, the MSE wall soil reinforcements are experiencing metal loss at a rate less than predicted with the design models. All 13 LPR corrosion rates are below the AASHTO design metal loss rates. A peak CSR value of 0.6 supports the observation that the soil reinforcements are performing sufficiently. All of these data indicates that the wall should achieve the expected service life. Due to the limited number of data it may not be confirmed that excessive corrosion is not present. However based on the existing information it is concluded even if present, it is limited in extent.

6.2.2.B Alt. US-50 and Alt. US-95

The limited data for Alt. US-50/ Alt. US-95 (**Fig. 6-3**) indicates that there is potential for aggressive corrosion. The AASHTO metal loss model predicts an upper bound corrosion rate of 4.0 μm/yr, the confidence interval (95%) for the mean estimates the upper bound of the mean to be ≈ 6.0 μm/yr, indicating a potential metal loss at 150% of design level. Therefore the service life would be affected. It is concluded that the MSE wall at Alt. US-50/ Alt. US-95 has the potential for higher than anticipated corrosion, which could result in a reduction of the service life.

6.2.2.C I-15 and Cheyenne Blvd.

Due to scattered and severe corrosion levels calculated during the LPR process (**Fig. 6-4**), I-15 and Cheyenne Blvd. MSE wall complex has had walls individually scrutinized. This allows separation of walls that are performing sufficiently from those that are not. Out of the five walls that were instrumented for in-situ LPR corrosion rates, three (Walls 1, 4, and 5) have in-service soil reinforcements available for monitoring. Walls 2 and 6 (**Fig. 6-5** and **Fig. 6-6,** respectfully) only have sacrificial coupons available for monitoring corrosion rates. These data from Walls 2 and 6 indicate that the corrosion conditions are acceptable with a maximum corrosion rate of ≈ 6.0 μm/yr. In both walls, the zinc elements (galvanized coupons) have a higher corrosion rate than the corresponding plain steel coupons. This suggests that the backfill conditions are accelerating the depletion of zinc.

The corrosion condition of Wall 5 (**Fig. 6-7**) appears to follow expected trend. With plain steel experiencing a higher corrosion rate than either the in-service galvanized reinforcement or the zinc element. All the LPR corrosion rates are below the corresponding design levels, indicating that the walls corrosive condition is below the design model expectations and no negative impact to the service life is expected.

Wall 4 has all three element types for LPR evaluation (**Fig. 6-8**). The galvanized in-service reinforcements have sustainably higher corrosion rates than either of the other element types. The corrosion rates of zinc elements are the lowest as is expected. The mean corrosion rate of the in-service reinforcements is ≈ 21 μm/yr, which corresponds to a CSR value of 5.25 or 525% of the design level. This will have a negative impact of the service life, leading to the possible premature failure of the soil reinforcement.

Wall 1 has all three element types for LPR evaluation (**Fig. 6-9**). The galvanized in-service reinforcements have sustainably higher corrosion rates than either of the other element types. The corrosion rates zinc elements are the lowest as is expected. The mean corrosion rates monitored from the plain steel coupons is approximately five times that of the zinc elements. This supports the assumption that the zinc coating is beneficial and behaving in an expected manner. In contrast to this are the corrosion rates of the in-service reinforcements. The mean corrosion rate for in-service reinforcements is ≈ 30 μm/yr or a CSR value of 7.5. This very aggressive metal loss will have a negative influence on the service life of the reinforcements.

Based on the limited number of number (10) of in-service LPR corrosion rates the site-wide mean corrosion rate is ≈ 20 μm/yr, which is five times the design level. Consequence of high corrosion rates were observed and photographed during the instrumentation process (**Figures 6-10** through **6-15**) at the site. The photographs and field observations support the conclusion of aggressive metal loss of the in-service soil reinforcements. Heavy scaling and buildup of corrosion deposits suggest that the noticeably excessive corrosion rates were not an anomaly; they have occurred and are expected to continue. During the Phase I investigation metal loss measurements were conducted on a set of reinforcements removed during the demolition of a MSE wall for a lane widening project (Thornley 2009). The Phase I estimated metal loss rates correspond relatively well with the Phase II LPR data. This provides a 5-year time span of documented elevated metal loss for the I-15 and Cheyenne Blvd. Based on the entirety of data available for the Cheyenne Blvd. Complex, there is significant advanced metal loss which will impact the service life of the MSE walls. The excessive metal loss will reduce the cross-sectional area of the reinforcements leading to premature reduction of tensile capacity and possible structural failure of the MSE walls.

6.2.2.D I-15 and Lamb Blvd.

There are no in-service elements available for LPR analysis for I-15/ Lamb Blvd., however, there are some photographs of the in-service elements (**Fig. 6-16** and **6-17**). Corrosion rate evaluation is based on the sacrificial coupons installed in each of the four walls. Data from this site is evaluated as a single set **Fig. 6-18** due to the sparse data. All the LPR data available indicates that the site's corrosion levels are below the design levels. A single CSR value of 0.9 was calculated for a zinc element, this indicates that concern for the corrosiveness is appropriate. However the levels are below the anticipated rate of metal loss. Based on the coupon LPR data it is concluded that I-15 and Lamb Blvd. at this time is performing sufficiently to not have an impact on the service life of the MSE walls.

6.2.2.E I-515 and Charleston Blvd.

The LPR data for I-515/ Charleston Blvd (**Fig. 6-19**) was considered as a single dataset, representing the entire complex. Based on the Phase II LPR corrosion rates for coupons only, the MSE wall soil reinforcements are experiencing metal loss at a rate less than predicted by the design models. There are no in-service elements available for this site. All 12 LPR corrosion rates are below the design metal loss rates. A peak CSR value of 0.7 supports the notion that the soil reinforcements are performing sufficiently. All of these data indicates that the wall should achieve the expected service life.

6.2.2.F I-515 and Las Vegas Blvd.

The LPR data for I-515/ Las Vegas Blvd (**Fig. 6-20**) was considered as a single dataset, representing the entire complex. Based on the Phase II LPR corrosion rates for all the elements, the MSE wall soil reinforcements are experiencing metal loss at a rate less than predicted with the design models. All 8 LPR corrosion rates are below the design metal loss rates. This means that the wall should achieve the expected service life.

6.2.2.G SR 160 and Jones Rd.

LPR corrosion rate data for SR 160 and Jones Rd. (**Fig. 6-21**) indicate that the wall is experiencing corrosion rates below the design levels. This wall has a total of 64 LPR corrosion rate measurements. The peak CSR value of 0.6 indicates that all the elements at this time are undergoing metal loss below the design model. Based on all the LPR data it is concluded at this time that the Jones Rd. MSE wall will achieve the intended service life. This site is very large and contains multiple walls. The conditions at the other walls may not be reflected in the LPR data from the MSE wall that was instrumented. It should be noted that due to the size of the complex elevated corrosion rates are possible, despite the findings from Wall 1. The extent of the elevated corrosion cannot be hypothesized for the other walls.

6.3 General Comments on the Extent and Condition of Phase II MSE Wall Corrosion

The Phase II data presents two distinctive pictures (1) the majority (58 of 72, 81%) of backfill conditions examined indicate potential to be very corrosive, and (2) most elements (201 of 213, 94%) have LPR corrosion rates below the design levels. There is ample evidence that relates backfill conditions to corrosive potential, yet the dataset indicates this relationship may not be as strong as described in literature. Conclusions about Nevada's MSE wall inventory, must consider both backfill conditions and in-situ corrosion rates. The following summarizes several key findings and observations that result from the multifaceted Phase II investigation.

The data indicates that I-15 and Cheyenne Blvd. requires careful examination. Supported with the scattered electrochemical soil testing, the Phase I findings, and elevated LPR corrosion rate measurements. It is a fair assumption that the limited number of LPR measurements may not have captured the extreme corrosion rates for the complex. Corrosion rates exceeding 500% of the design rate were observed at the site. Assuming that the corrosion severity is constant throughout the life of the MSE wall, a CSR value of 2.0 reduces the service life to 42-year, while a CSR value of 5.0 results in a service life of only 22-years. The I-15 and Cheyenne Blvd. complex was built in 1998 making it approximately 16-year old.

The MSE wall at Alt. US-50/ Alt. US-95 had 1 of 18 (6%) corrosion rates which produced a CSR value greater than 1.0, while the backfill conditions are considered the most severe of the seven sites investigated (**Table 6-1**).This suggest that the majority of soil reinforcements are experiencing metal loss below those of the design model, despite the backfill conditions. Similar trends are present at other Phase II sites suggestive of corrosion rates that are accounted for using the AASHTO metal loss model, despite the backfill conditions not meeting the requirements. These observations support the belief that the current AASHTO design model is conservative, based on an upper bound corrosion rate. The conservatism inherent to the model reflects the inability to accurately predict corrosion rates. The lack of a robust correlation between backfill electrochemical characteristics and metal loss rates, rationalizes the need to be conservative in estimating corrosion rates. As these directly affects the structural capacity of the soil reinforcements and safety of the structure.

The rankings presented in **Table 6-1** are based on: mean chloride value, mean sulfate value, minimum resistivity value, mean resistivity value, resistivity COV, and mean organic content. Severity of the mean value is established on its deviation from the respective limit. The minimum reported resistivity value and resistivity COV are included in the rankings. The inclusion of the additional resistivity criteria, is intended to emphasis the reliance of resistivity to predict backfill corrosiveness. Due to the narrow range of reported, pH values for all the sites were considered equivalent and therefore not included in the severity rankings.

The severity rankings for Phase II corrosion rates is presented in T**able 6-2**. Severity ranking of corrosion is based on two key aspects, the mean corrosion rate and their coefficient of variation (COV). Severity rankings were assigned based on the magnitude of mean corrosion rate and COV. This allows consideration of both elevated corrosion and the scatter within the measurements. When data was unavailable (e.g. in-service reinforcements for I-15/ Lamb Blvd.) a ranking of three (3) was assigned, a conservative mid-range value. In total six different corrosion descriptors were used to establish the severity rankings presented in **Table 6-2**.

Inetstate-15 and Cheyenne Boulevard is identified as the most severe site based on the corrosion criteria. With I-15 and Lamb Blvd. and I-515 and Charleston Blvd. becoming in second and third, respectfully. These site receive and increased severity based on the lack of in-service reinforcement LPR data. The data reveals that most of the sites (5 of 7, 71%) have similar corrosion severity. The MSE wall at I-515 and Las Vegas Blvd. stands out from the other sites, as the data indicates that this locations has minimal concern regarding elevated corrosion.

Combining both the backfill characteristics and LPR corrosion rate data a composite severity ranking was established. This total ranking is based on 12 different criteria, six from electrochemical backfill characteristics and six LPR corrosion rate descriptors, resulting in the **Table 6-3**.

Table 6-3: Phase II MSE Wall Composite Corrosion Condition Severity Ranking

Site	Soil Severity Rank	Corrosion Severity Rank	Composite Rank	**Phase II MSE Wall Corrosion Condition Severity Rank[1]**
I 15 and Cheyenne	2	1	3	**1**
ALT US-50 and ALT US-95	1	6	7	**2**
I 515 and Charleston	4	3	7	**3**
I 15 and Lamb Blvd.	6	2	8	**5**
US 395 and Huffaker Lane	3	5	8	**4**
SR 160 and Jones Rd.	5	4	9	**6**
I 515 and Las Vegas Blvd.	7	7	14	**7**

[1] 1 = Most severe conditions

The Phase II investigation has identified I-15/ Cheyenne Blvd. as the site with most severe conditions. The identification of I-15 and Cheyenne Blvd. is due to, scattered backfill conditions and the aggressive rate of metal loss that was observed. Along with identification of the issues at I-15/ Cheyenne Blvd., Alt. US-50/ Alt. US-95 is recognized as potentially problematic. The backfill conditions at this site were ranked as the most severe, while the LPR corrosion rate data suggest that the site is performing sufficiently. The well documented relationship between backfill conditions and the aggressiveness of metal loss should be considered. In contrast to the severe conditions identified the conditions at I-515 and Las Vegas Blvd, are reassuring.   Ranked as the least severe for both backfill and corrosion rates, I-515 and Las Vegas Blvd. demonstrates sustainably less severe conditions. The remaining four sites all indicate relatively similar severity.

Chapter 7 Recommendations

The use of mechanically stabilized earth walls is an integral part of modern infrastructure. The capacity to stabilized and retain a soil mass with minimal footprint is the primary function of MSE walls. This ability is achieved through the use of a system of soil reinforcements, typically metallic. This system stabilizes the failure wedge by providing tensile capacity to the system. The tensile capacity of the metallic soil reinforcements are directly proportional to their cross-sectional area. During the design process the use of established metal loss models compensates for the metal consumption due to corrosion. These corrosion models are intended to ensure that at the end of the design life adequate tensile capacity remains, ensuring the safety of the systems.

Mechanically stabilized earth walls require periodic inspection and performance evaluation, as is common practice for vital structures. These inspections allow for the routine monitoring and evaluation, ensuring public safety. According the US Congress Bill known as MAP-21 (Moving Ahead for Progress in the 21st Century Act) “Performance management will transform the Federal-aid highway program and provide a means to the most efficient investment of Federal transportation funds by refocusing on national transportation goals, increasing the accountability and transparency of the Federal-aid highway program, and improving project decision-making.” [§1203; 23 USC 150(a)] (Map-21 2012). Corrosion monitoring, electrochemical backfill characterization and performance assessments which have been undertaken during this investigation are important elements of the vision of MAP-21.

Corrosion monitoring can only be achieved with access to the backfill soils and soil reinforcements. Exterior observation of MSE walls are not sufficient when determining the condition of the soil reinforcements. This is shown by the two accidental discoveries of advanced corrosion on I-515/Flamingo Rd and I-15/Cheyenne Blvd. Neither of the sites showed obvious signs of distress or elevated corrosion, yet when investigated, corrosion levels were observed in excess of 500% of the design level (i.e. CSR > 5.0). The deteriorated conditions observed at I-515/ Flamingo Rd. led to the costly emergency remediation of the site with a cast-in-place tie-back retaining wall. The I-515/Flamingo Rd. complex was constructed with plain steel (i.e. un-galvanized) reinforcement while the I-15/Cheyenne Blvd. site used galvanized steel soil reinforcements. These sites confirm that advanced corrosion is present within Nevada's inventory of MSE walls and involves a variety of metal types.

These discoveries led to an investigation and evaluation of Nevada's MSE wall inventory. Phase I research (2009) resulted in identification of the ineffectiveness of the Nevada test method (Nevada T235B) for minimum soil resistivity. (Thornley 2009) the Nevada test method overestimated the soil resistivity value and thus, underestimated the corrosiveness of the backfill soils. The study established a correlation that can convert results from the two test methods, thus allowing the historical resistivity test results to be converted to values consistent with the current AASHTO test method. This was used, in part, to identify a series of sites that had a greater probability for aggressive backfill conditions and subsequent advanced metal loss. These sites have been evaluated for both electrochemical backfill characteristics and in-situ corrosion rates under the current project (Phase II). The results of this evaluation have been presented in the preceding chapters. These results verify many of the assertions made in

Phase I and confirm the presence of corrosive backfills and elevated metal loss rates.

It is recommended to establish a comprehensive Long-term Corrosion Monitoring and Asset Management Program (LCMAMP). This initiative should include the following:

1) Continued LPR Corrosion rate monitoring of Nevada MSE walls.
2) Expansion of NDOT's corrosion and asset management plan.
3) Improved MSE backfill electrochemical characterization database.
4) Performance evaluation of unremediated walls at I-15/Flamingo Rd.
5) Rigorous site investigation for I-15 and Cheyenne Blvd.
6) Investigation of localized MSE wall failures as a result of advanced corrosion

The basis behind the development for each of the recommendations is presented in this report. The lack of robust correlation between the backfill conditions and LPR corrosion rate data solidifies the need for all corrosion evaluations be multi-faceted, employing the use of both backfill evaluation and in-situ corrosion observations.

7.1 Continued LPR Corrosion Rate Monitoring of Nevada MSE Walls

The use of polarization resistance to estimate uniform corrosion rates is an accepted method to assess the in-situ condition of MSE wall reinforcements. A limitation of the LPR method is the measurements reflect instantaneous conditions. Without a series of periodic measurements, the method fails to establish long-term conditions. The measurement reflects a discrete point in the life cycle of the element. Therefore, it is recommended that the current corrosion monitoring stations be observed annually for a minimum of five consecutive years, then every other year thereafter. Monitoring should occur at roughly the same time each year to minimize the influence of seasonal variations. The initial monitoring scheme is intended to capture the transition between the first and second galvanic corrosion stages and establish the galvanized rate of metal loss. The subsequent monitoring frequency will provide long-term data so that a robust trend can be established. The information obtained thus far establishes the foundation for LCMAMP, while the subsequent measurements allow for the development of robust dataset to support decisions on the safety and stability of the MSE walls.

7.2 Expansion of NDOT's Corrosion and Asset Management Plan

The LCMAMP activities should include the following: additional MSE walls, which were not considered at the sites that were evaluated under Phase II, and additional in-service reinforcements for the currently instrumented sites. All new corrosion monitoring stations should have emphasis on monitoring of in-service reinforcements. Installation and monitoring of coupons allow for evaluation of the historical conditions for the MSE wall reinforcements and provide limited data on the current in-situ situation. Access and instrumentation of in-service elements are recommended by coring through the MSE wall concrete facing panels with 6 to 8 inch diameter holes. This allows adequate room for sampling the backfill, connection to in-service reinforcements, and installation of sacrificial coupons. This process will reduce the total cores required while allowing for better data acquisition. The Phase II study omitted several MSE walls as a result of accessibility complications. The following walls should have priority: two additional walls at I-15/ Cheyenne Blvd., four walls at Alt. US-50 and Alt. US-95, and five walls at SR 160/ Jones Rd.

The observations from this study hypothesize that there are other sites that may be experiencing elevated metal loss. Priority for existing wall sites that are included into the LCMAMP should be based on the recommendations provided by Phase I. It is also recommended that all new MSE walls have corrosion monitoring stations established at the time of construction. This study has confirmed the suspicion that there are elevated metal loss rates and excessive corrosiveness in MSE wall backfills. If these conditions are not accounted for during the design process, the excessive corrosiveness will jeopardize the service life of these MSE walls, increasing the possibility for MSE walls not to achieve the design life. Identification is vital in order to maintain confidence in the safety and stability of the MSE wall inventory.

Inclusion of new monitoring stations at MSE walls during the construction process is seen as proactively addressing the concerns that have been documented. While it is the belief that new sites will adhere to the MSE backfill specifications and design models, the ease of instrumentation and performance evaluations warrant their inclusion as part of the LCMAMP.

7.3 Improved MSE Backfill Electrochemical Characterization Database

The electrochemical backfill characteristics have a strong influence on the corrosive potential. While each of the specified constituents cannot independently predict the aggressiveness of corrosion, the relationships do have predictable trends. Improving the characterization of the backfill will assist in understanding / minimizing the corrosion severity. Investigation into the source of the discrepancies between As-Built and Phase II tests of the electrochemical characteristics should be undertaken. It is recommended that source materials be tested according to the current state of practice, with additional samples collected on the MSE wall site during construction. These additional site-based samples help to improve the correlation with future corrosion monitoring efforts. The additional testing also increases the statistical robustness of the results and can improve the characterization of the backfill.

7.4 Performance Evaluation of Non-Remediated Walls at I-15/Flamingo Rd.

Unlike Wall1 at I-515 and Flamingo Rd, Walls 2 and 3 did not receive remediation in 2004. This study through the use of LPR corrosion rate measurements has confirmed that the documented conditions have not significantly improved at the Flamingo Road complex. Wall 1 had a cast-in-place tie-back wall constructed to provide stability and maintain long-term operational safety. Performance evaluation of the two remaining walls should be included in the long-term stability assessment.

7.5 Rigorous Site Investigation for I-15 and Cheyenne Blvd.

This study has confirmed the aggressive corrosiveness at the I-15 and Cheyenne Blvd. complex. In 2009, elevated corrosion rates were estimated based on thickness measurements from samples of reinforcements that were removed. Backfill samples were obtained from on-site stockpiles and tested for electrochemical characteristics. The soil testing indicated that the backfill conditions did not meet AASHTO specifications. The backfill testing results support the observations of aggressive metal loss due to corrosion. Through the use of additional backfill testing under Phase II, it has been confirmed that there are areas within the backfill that do not meet AASHTO or Nevada backfill standards. It has been demonstrated that the backfill conditions are extremely variable both within the site and within a single wall.

Linear polarization resistance corrosion rates indicated areas of extreme metal loss. Corrosion severity ratios in excess of 10.0 (CSR > 10.0) for in-service reinforcements were observed. These extreme cases are supported with photographic evidence (**Figures 6-19** through **6-24)**. The corrosion data (i.e. LPR corrosion rates and physiochemical/ electrochemical backfill characteristics) indicate that the conditions at I-15/Cheyenne Blvd. site are equivalent to or worse than the condition discovered in the 2004 at I-515/Flamingo Rd. Therefore, it is recommended that a rigorous examination of the I-15 and Cheyenne Blvd be undertaken.

The recommended investigation should be on a scale comparable to the 2004 MMCE I-515/ Flamingo Rd. study. Similar to the 2004 decision the study will consider the simultaneous development of a site remediation plan. The study must place emphasis on the in-service reinforcements and should attempt to quantify the extent and distribution of the corrosion rates based on a statistical and probabilistic framework. This is in-line with the transition to LRFD design methodologies. The new investigation should be conducted on all MSE walls at the complex.

7.6 Investigation of MSE Wall Failures Due to Elevated Corrosion

This study has confirmed that there are MSE walls in Nevada's inventory that are experiencing aggressive corrosion. This has been confirmed through the use of LPR corrosion rate monitoring. Locations were discovered with corrosion levels in excess of 500% of the design rates. At the same time, the corrosion monitoring program identified areas where the in-situ corrosion was significantly below the design rates. The differing corrosion conditions were often observed within a single wall, demonstrating that the corrosion phenomenon is neither uniform nor consistent. The apparent random distribution of corrosion rates is further compounded with the discovery that the electrochemical backfill characteristics are also exhibiting similar uneven distributions. The unexpected combination of conditions can lead to possible premature failure of the MSE wall soil reinforcements.

Current practice is that once it is determined that a corrosion problem exists and operational safety is jeopardized site remediation should be undertaken on the entire wall (Armour 2004, Blight and Dane 1989, Fishman 2004). This is a very expensive and conservative approach. The approach of globalizing failure in such a manner is quite common (Bourgeois et. al. 2013). Phase II has determined that in contrast to the uniform globalized conditions assumed in most failures assessment and design procedures, corrosiveness (e.g. electrochemical backfill characteristics and corrosion rate profile) of the backfill is not uniform. The conditions observed during the Phase II investigation have demonstrated that conditions can vary within a single corrosion monitoring station, MSE wall, and MSE wall complex. MSE walls are designed to be flexible structures. This allows for “engineering intuition” to assume that the MSE wall’s system of reinforcements is capable of load redistribution in the event that limited number of individual reinforcements lose their ability to provide sufficient tensile capacity. This intuition is supported by the Idaho MSE wall failure (Blight and Dane 1989), in which a localized cavity formed when several concrete facing panels fell out of place. The entire structure did not fail, although it was deemed to be unusable until remediation was completed. This event provides some concrete evidence that corrosion related failures are not globalized, but instead are localized. However, there is no way to identify in a simplified way to what extent this type of failure has on performance of the entire structure.

Therefore it is recommended that advanced numerical investigation of localized MSE failures be conducted. This should include identification of critical areas within a MSE wall and quantification on the extent of the reduction in tensile capacity of the reinforcements required for failure. As part of the MSE wall modeling effort, robust statistical models that characterize the distribution of corrosion rates and backfill parameters should be integrated. These activities will improve the applicability and usefulness of the numerical investigation in predicting the safety of MSE walls. NDOT is uniquely positioned to provide in-situ corrosion conditions (environmental and metal loss rates) at documented problematic locations to the engineering community. By combining both a comprehensive field and laboratory investigation and thorough numerical modeling of the current problematic MSE complexes (e.g. I-15/ Cheyenne Blvd. and Alt. US 50/ Alt. US 95) better understanding of current situation and development of a robust estimation of the consequence of corrosion in MSE walls can be achieved.

References

AASHTO (1973). Standard Specifications for Highway Bridges, 11th Edition. American Association of State Highway and Transportation Officials, Washington D.C.

AASHTO (1989). Standard Specifications for Highway Bridges, 14th Edition. American Association of State Highway and Transportation Officials, Washington D.C.

AASHTO (1992). Standard Specifications for Highway Bridges, 15th Edition. American Association of State Highway and Transportation Officials, Washington D.C.

AASHTO (1996). Standard Specifications for Highway Bridges, 16th Edition. American Association of State Highway and Transportation Officials, Washington D.C.

AASHTO (1996). Standard Specifications for Highway Bridges, 16th Edition. American Association of State Highway and Transportation Officials, Washington D.C.

AASHTO (2002). Standard Specifications for Highway Bridges, 17th Edition. American Association of State Highway and Transportation Officials, Washington D.C.

AASHTO (2007). LRFD Bridge Design Specifications, 4th Edition. American Association of State Highway and Transportation Officials, Washington D.C.

AMSE 2006. Reduced Zinc Loss Rate for Design of MSE Structures. Association of Metallically Stabilized Earth, McLean, Virginia, 2006

Armour, T., Bickford, J., and Pfister, T. (2004). Repair of Failing MSE Railroad Bridge Abutment. GeoSupport 2004: Drilled Shafts, Micropiling, Deep Mixing, Remedial Methods, and Specialty Foundation Systems, ASCE, Orlando, FL., 380-394.

ASM International. Metals Handbook, 9th Edition. Vol. 13. Metals Park, Ohio, 1987.

Beckham. Tony L., Leicheng Sun and Tommy C. Hopkins. Corrosion Evaluation of Mechanically Stabilized Earth Walls. KTC-05-28/SPR 239-02-1F. University of Kentucky. 2005.

Billings, Daniel A. Assessing Corrosion of MSE Wall Reinforcements for I-15, Salt Lake City, UT. Brigham Young University. 2011

Blight, G.E. and M.S.W. Dane. “Deterioration of A Wall Complex Constructed of Reinforced Earth”. Geotechnique. Vol. 39.1. March 1989.

Bourgeois, Emmanuel, Alain Corfdir, and Truong-Lih Chau. Analysis of Long-Term Deformations of MSE Walls Based on Various Corrosion Scenarios. Soils and Foundations. 53(2) pp 259-271. 2013

C.C. Technologies. Polarization Resistance Monitoring PR 4500 Operation Manual. C.C. Technologies Systems, Inc. 1999.

Darbin, M., J.M. Jailloux, and J. Montuelle. Durability of Reinforced Earth Structures: The Results of a Long-term Study Conducted on Galvanized Steel. Proceedings – Institution of Civil Engineers. Part 1 Design and Construction, Vol. 84. 1988.

Decker, Jeramy B., Kyle M. Rollins, and Jared C. Ellsworth, Corrosion Rate Evaluation and Prediction for Piles Base on Long-Term Field Performance. Journal of Geotechnical and Geoenvironmental Engineering. Vol. 34, No. 3, pp. 341-351, 2008

Elias, Victor. Durability/Corrosion of Soil Reinforced Structures. FHWA-RD-89-186, Washington D.C., 1990.

Elias, Victor. Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes. FHWA-NHI-00-044, Washington D.C., 2000.

Elias, Victor, Kenneth. L. Fishman, Barry. R. Christopher & Ryan. R. Berg. Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes. FHWA-NHI-09-087, Washington D.C., 2009.

Elias, Victor, Barry R. Christopher, and Ryan R. Berg, Mechanically Stabilized Earth Walls and Reinforced Slopes Design and Construction Guidelines. FHWA-NHI-00-043, Washington D.C., 2001.

Fishman, Kenneth L. Consultant's Report: Corrosion Evaluation of MSE Walls I-515/Flamingo Road Las Vegas, Nevada. McMahon & Mann Consulting Engineers, P.C. May 2005.

Fishman, Kenneth L., James L. Withiam. LRFD Metal Loss and Service-Life Reduction Factors for Metal-Reinforced Systesms. NCHRP 675. 2011

Gerber, Travis M. and Colin R. Cummins. Modeling and Analysis to Quantify MSE Wall Behavior and Preformance. UT-10.03. Brigham Young University. 2009.

Ghods, Pouria, Akram Alfantazi, FNACE, and Victor Padilla. Steel Corrosion in Mechanically Stabilized Earth Walls. NACE International, Vol. 52, No. 11, 2013

Gladstone, R.A., Anderson, P.L., Fishman, K.L, and Withiam, J.L., 2006, *Durability of Galvanized Soil Reinforcements: More Than 30 Years of Experience with Mechanically Stabilized Earth.* Transportation Research Record, Journal of the Transportation Research Board, No. 1975, Transportation Research Board of the National Academics, Washington, D.C., pp. 49-59

Jones, Denny L. Principles and Prevention of Corrosion, 2nd Edition. Princeton Hall: Upper Saddle River, 1996.

Lawson, K. M., Thompson, N.G., Islam, M., and Schofield, M.J., Monitoring Corrosion of Reinforced Soil Structures. British Journal of NDT, British Institute of Nondestructive Testing Liverpool, England, 35(6),1993

Moving Ahead for Progress in the 21st Century Act (MAP-21), Pub. L. No. 112-141, § 119, 126 STAT. 432-437, 2012

NCHRP 24-28, LRFD Metal Loss and Service-Life Strength Reduction Factors for Metal Reinforced Systems in Geotechnical Applications, Phase 1 Interim Report. April 2007.

NCHP-50. Durability of Drainage Pipes. TRB. Washington D.C. 1978

NDOT (1968), Standard Specifications for Road and Bridge Construction. Nevada Department of Transportation, Carson City, Nevada, 1968.

NDOT (1976), Standard Specifications for Road and Bridge Construction. Nevada Department of Transportation, Carson City, Nevada, 1976.

NDOT (1986), Standard Specifications for Road and Bridge Construction. Nevada Department of Transportation, Carson City, Nevada, 1986.

NDOT (1996), Standard Specifications for Road and Bridge Construction. Nevada Department of Transportation, Carson City, Nevada, 1996.

NDOT (2001), Standard Specifications for Road and Bridge Construction. Nevada Department of Transportation, Carson City, Nevada, 2001.

Padilla, Victor, Pouria Ghods and Akram Alfantazi. Practical model for Three-Stage Corrosion behavior of Galvanized Steel Reinforcements in Soil. Corrosion Vol. 69, No. 5, 2013

"Polarization Resistance Monitor PR 4500 Operation Manual". Version 2.0. CC Technologies Sytemes, Inc., 1999

Raeburn, Christopher L. M. Murat Monkul, and Marvin R. Piles. Evaluation of Corrosion of Metallic Reinforcements and connections in MSE Retaining Walls, Project 643. FHWA-OR-RD-08-10, Washington D.C. 2008.

Romanoff, Melvin. Underground Corrosion. National Bureau of Standards Circular 579. Houston: National Association of Corrosion Engineers, 1957

Scully, J.C. The Fundamentals of Corrosion, 3rd Edition. Oxford: Pergamon, 1990.

Sagüés, Alberto A., Juan Rossi, Randall J. Scott, José A. Peña. Influence of Corrosive Inundation on the Corrosion Rates of Galvanized Tie Strips in Mechanically Stabilized Earth Walls. University of South Florida. Florida Department of Transportation. Report WPI 0510686. 1998

Thornley, John D. Use of Statistical Methods to Study Corrosion aggressiveness at Nevada Mechanically Stabilized Earth Wall Sites. University of Nevada Reno. 2009.

Yajim, Ayako, Hui Wang, Homero Castaneda, & Robert Liang. Application of Cluster Analysis for Soil Corrosivity Assessment. Transportation Research Board. Washington D.C. 2014

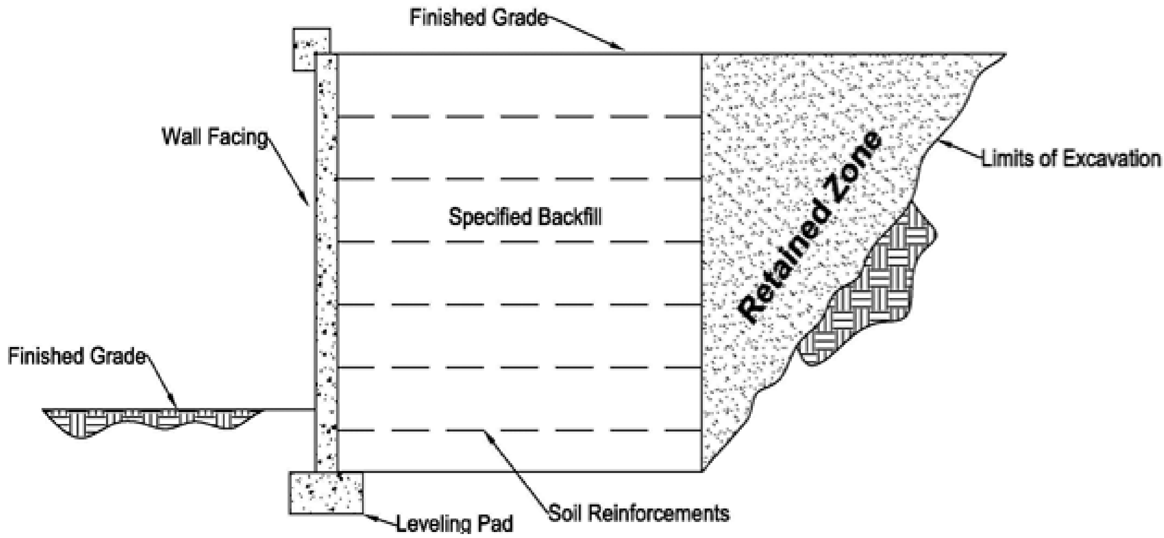

Figure 1-1: Typical elements of a mechanically stabilized earth (MSE) wall system

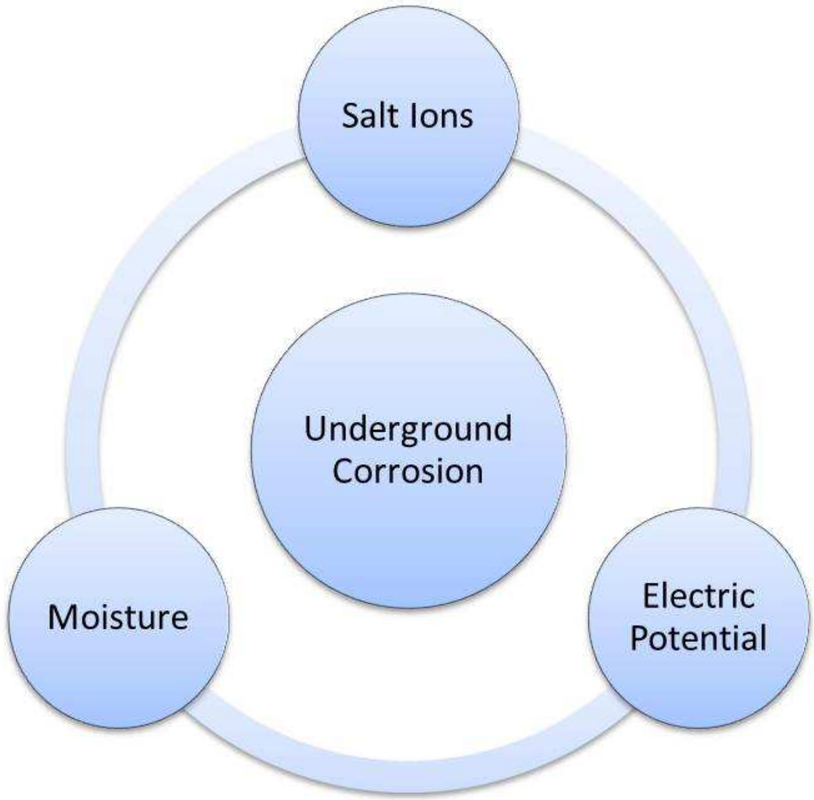

Figure 0-1: Underground corrosion interdependency

Figure 0-2: In-service galvanized reinforcement, a layer zinc carbonate can be seen along the surface. (ALT US 95 and ALT US 50, W1-SB-E1)

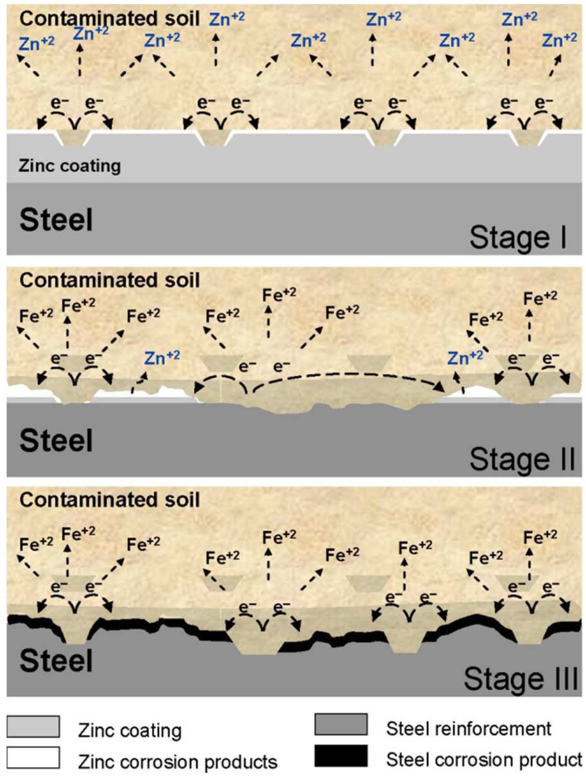

Figure 0-3: Schematic showing the three phases of galvanic corrosion (Padilla et. al. 2013)

Table 0-1: Caltrans interim design metal loss coefficients (Jackurea et. al, 1987)

Fill Type	K [µm/ yr]	C [years]
Neutral & Alkaline	28	10
Acidic	33	10
Corrosive	71	6
Select Granular	13	30

Notes:

Neutral and alkaline: minimum resistivity > 1,000 ohm-cm and pH>7

Acidic: minimum resistivity > 1,000 ohm-cm and pH < 7

Corrosive: minimum resistivity < 1,000 ohm-cm

Select granular soils are clean, free draining gravels with less than 5% fines and minimum resistivity > 1,000 ohm-cm

Table 0-2: Effects of resistivity on soil corrosiveness, (NCHRP, 1978)

Aggressiveness	Resistivity [Ω-cm]
Very Corrosive	<700
Corrosive	700 to 2,000
Moderately Corrosive	2,000 to 5,000
Mildly Corrosive	5,000 to 10,000
Non Corrosive	>10,000

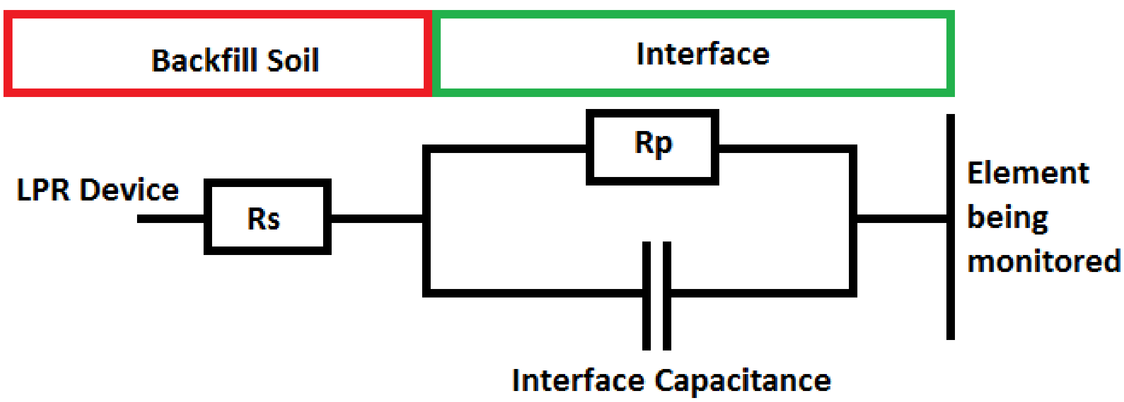

Figure 0-4: Idealized LPR monitoring circuit

Table 0-3: Prioritized list of MSE wall sites for further investigation, as provided by Phase I

Ranking	Contract	Year of Contract	District	Soil Reinforcement Type	Avg. Resistivity [ohm-cm]	Avg. Chloride [ppm]	Avg. Sulfate [ppm]
1	1918	1982	1	T.S. galv.	1777	133	550
2	1916	1981(?)	1	T.S. galv.	No Data	No Data	No Data
3	2202	1987	1	WWF galv.	2671	60	275
4	2203	1987	2	WWF galv.	2884	30	40
5	3324	2007	1	T.S. galv.	2100	78	104
6	3189	2003	1	WWF galv.	2348	50	0
7	3237	2004	2	Unknown	2418	70	0
8	3003	2000	1	Unknown	2630	75	0
9	3215	2005	1	T.S. galv.	2869	44	154
10	3290	2006	1	WWF galv.	2901	55	57

Table 0-4: MSE wall locations for investigation during Phase II as suggested by Phase I

Contract No.	Year of Contract	City/Area	NDOT District	Intersection	Reinforcement Type
2066	1985	Las Vegas/Central	1	I-515 and Flamingo Rd.	WWF1
2853	1998	Las Vegas / North LV	1	I-15 and Cheyenne	GS
1916	1981?	Las Vegas / Central	1	I-515 and Las Vegas Blvd.	GS
1918	1982	Las Vegas / Central	1	I-515 and Charleston	GS
3189	2003	Las Vegas / Nellis AFB	1	I-15 and Lamb Blvd.	Bar Mat
3324	2007	Las Vegas / Enterprise	1	SR 160 and Jones Rd.	GS
3237	2004	Fernley	2	Alt US 95 and Alt US 50	GS
2203	1987	Reno	2	US 395 and Huffaker Lane	WWF

[1] Not galvanized

WWF - Welded Wire Fabric

GS - Galvanized Strips

Table 2-1: Summary of the MSE wall specifications that have been used within the state of Nevada

AASHTO and NDOT Historical Electrochemical Specification*					
Specification	Pre-1986[1]	1986	1992	1996-Present	
	NDOT	NDOT 1986 EDITION	AASHTO 15TH Edition[2]	NDOT 1996 & 2001 Editions	AASHTO 16th, 17th, & LRFD Editions
pH	6.4 to 9.5	5 to 10	5 to 10	5 to 10	5 to 10
Resistivity [ohm-cm]	1,000 min.	3,000 min.	3,000 min.	3,000 min.	3,000 min.
Chlorides [ppm]	500 max.	200 max.	50 max.	100 max.	100 max.
Sulfates [ppm]	2,000 max.	1,000 max.	500 max.	200 max.	200 max.

*Respective Standard specifications for Highway Bridges (AASHTO) and Standard Specifications for Road and Bridge Construction (NDOT Silver Book)

[1] There are no references to retaining walls in NDOT edition before 1986, these requirements were found in the material test data sheet (Contract 1918, July 1982)

[2] There are no references to MSE walls prior to this AASHTO edition

Table 2-2: Soil testing records obtained from the construction documents for the seven sites included in Phase II

Site	Sample Location	Chloride [ppm]	Sulfate [ppm]	Resistivity [Ω-cm]	pH	Pass/Fail	Date
US 395 and Huffaker Lane	Helms Rock & Sand	40	0	4587	8.2	Pass	1988
Alternate US 95 and US 50	Gopher Pit (Fernley) Sample B	70	0	3067	9.2	Pass	2005
Alternate US 95 and US 50	Gopher Pit (Fernley) Sample C	70	0	3003	9.2	Pass	2005
Alternate US 95 and US 50	Gopher Pit (Fernley) Sample A	70	0	3448	9.1	Pass	2005
Alternate US 95 and US 50	Gopher Pit (Fernley)	-	-	5780	9.0	Pass	2005
Alternate US 95 and US 50	*Gopher Pit (Fernley)*	-	-	*2778*	*9.1*	*Fail*	*2005*
Alternate US 95 and US 50	*Gopher Pit*	-	-	*2702*	*9.2*	*Fail*	*2005*
Alternate US 95 and US 50	*LY 23-01*	-	-	*1815*	*9.0*	*Fail*	*2005*
Alternate US 95 and US 50	*Paiute Pit*	*40*	*250*	*3175*	*6.6*	*Fail*	*2005*
Alternate US 95 and US 50	*Black Mountain Pit*	-	-	*1610*	*8.5*	*Fail*	*2005*
I 15 and Cheyenne Blvd.	Chem Star Apex Pit Stockpile #3	70	0	9009	8.3	Pass	1998
I 15 and Cheyenne Blvd.	Chem Star Apex Pit Stockpile #2	70	0	9709	8.2	Pass	1998
I 15 and Cheyenne Blvd.	Frehner Sloan Pit Stockpile #1	90	0	3472	8.5	Pass	1998
I 15 and Cheyenne Blvd.	Frehner Sloan Pit Stockpile #3	90	0	6173	8.5	Pass	1998
I 15 and Cheyenne Blvd.	*5th St Plant - Frehner Const*	*1100*	*800*	*382*	*11.8*	*Fail*	*1998*
I 15 and Cheyenne Blvd.	*Frehner Rd. Millings Stockpile*	*190*	*500*	*494*	*8.1*	*Fail*	*1998*
I 15 and Cheyenne Blvd.	*L.V.P.Apex Stockpile #1*	*2200*	*1000*	*426*	*8.6*	*Fail*	*1998*
I 15 and Cheyenne Blvd.	*L.V.P. 5th St. Plant*	*200*	*1000*	*460*	*7.5*	*Fail*	*1998*
I 15 and Cheyenne Blvd.	*Frehner Pit*	*80*	*325*	*861*	*8.0*	*Fail*	*1999*

I 15 and Cheyenne Blvd.	*Chey & I-15 Structure Material*	*80*	*450*	*1626*	*8.3*	*Fail*	*1999*
I 15 and Cheyenne Blvd.	*I-15 NB Structure I-1128*	*110*	*500*	*1185*	*8.4*	*Fail*	*1999*
I 15 and Lamb Blvd.	Chem Star @ Apex	50	0	3704	7.4	Pass	2004
I 515 and Charleston Blvd.	Nellis Pit- Nevada Rock & Sand	310	1600	1610	7.7	Pass	1982
I 515 and Charleston Blvd.	Henderson Pit - Nevada Rock & Sand	190	600	1920	7.2	Pass	1982
I 515 and Charleston Blvd.	Henderson Pit - Nevada Rock & Sand	100	550	3380	7.7	Pass	1982
I 515 and Charleston Blvd.	Henderson Pit - Nevada Rock & Sand	110	500	3030	7.7	Pass	1982
I 515 and Charleston Blvd.	*Nevada Rock & Sand, Henderson*	*1000*	*900*	*271*	*7.0*	*Fail*	*1982*
I 515 and Charleston Blvd.	*Nev. Rock & Sand Henderson*	*2700*	*1000*	*620*	*7.8*	*Fail*	*1982*
I 515 and Charleston Blvd.	*Nevada Rock & sand, Henderson*	*1400*	*200*	*267*	*7.1*	*Fail*	*1982*
I 515 and Las Vegas Blvd.	Tab Const. Gravel Pit near Nellis A.F.B.	70	0	-	7.1	Pass	1982
SR 160 and Jones Rd.	Blue Diamond Pit Stockpile #40	90	126	3484	8.4	Pass	2008
SR 160 and Jones Rd.	Blue Diamond Pit Stockpile #22	70	111	3215	8.3	Pass	2008
SR 160 and Jones Rd.	Blue Diamond Pit Stockpile #37	90	109	3135	8.2	Pass	2008
SR 160 and Jones Rd.	Blue Diamond Pit Stockpile #36	70	103	3521	8.2	Pass	2008
SR 160 and Jones Rd.	Blue Diamond Pit Stockpile #35	80	87	3289	8.2	Pass	2008
SR 160 and Jones Rd.	Blue Diamond Pit Stockpile #33	70	85	3077	7.8	Pass	2008
SR 160 and Jones Rd.	Blue Diamond Pit Stockpile #32	80	117	3300	8.1	Pass	2008

SR 160 and Jones Rd.	Blue Diamond Pit Stockpile #18	80	111	3274	8.5	Pass	2008
SR 160 and Jones Rd.	Blue Diamond Pit Stockpile #13	80	98	3215	8.4	Pass	2008
SR 160 and Jones Rd.	Blue Diamond Pit Stockpile #28	80	113	3344	8.3	Pass	2008
SR 160 and Jones Rd.	Blue Diamond Pit Stockpile #29	80	106	3135	8.3	Pass	2008
SR 160 and Jones Rd.	Blue Diamond Pit Stockpile #26	80	104	3521	8.3	Pass	2008
SR 160 and Jones Rd.	Blue Diamond Pit Stockpile #25	65	95	3401	8.3	Pass	2008
SR 160 and Jones Rd.	Blue Diamond Pit Stockpile #14	80	95	3289	8.5	Pass	2008
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #23*	-	-	*2801*	*8.4*	*Fail*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #24*	-	-	*2933*	*8.4*	*Fail*	*2008*
SR 160 and Jones Rd.	*Blue Diamond RD & I-15 Public Works Stockpile*	-	-	*824*	*7.8*	*Fail*	*2008*
SR 160 and Jones Rd.	*Russell Rd & Cimmaron Public Works Stockpile*	-	-	*1724*	*7.8*	*Fail*	*2008*
SR 160 and Jones Rd.	*Las Vegas Blvd. & Silverado Ranch*	-	-	*1047*	*7.5*	*Fail*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #17*	-	-	*2907*	*8.5*	*Fail*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #19*	-	-	*2950*	*8.4*	*Fail*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #17, Sample 2*	-	-	*2941*	*8.4*	*Fail*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #16, Sample 2*	-	-	*2994*	*8.3*	*Fail*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #15, Sample 2*	-	-	*2849*	*8.1*	*Fail*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #15*	-	-	*2294*	*8.1*	*Fail*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #16*	-	-	*2801*	*8.2*	*Fail*	*2008*

SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #39*	-	-	*2985*	*8.2*	*Fail*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #31*	-	-	*2717*	*8.4*	*Fail*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #30*	-	-	*2667*	*8.4*	*Fail*	*2008*

	Indicates value failing specification

Table 2-3: MSE wall backfill requirements based on NDOT 2001, used to evaluate backfill conditions

Test	Specification
Chloride	100 ppm max.
Sulfate	200 ppm max.
Resistivity	3,000 ohm-cm min.
pH	5.0 to 10.0
Organic Content[1]	1% max

[1] Used for evaluation but not part of NDOT 2001

Table 2-4: Phase I soil testing results used for evaluation, pass/fail condition is based on NDOT 2001 specifications

Site	Sample Location	Chloride [ppm]	Sulfate [ppm]	Resistivity [1] [Ω-cm]	pH	Pass/Fail [2]	Date
US 395 and Huffaker Lane	*Helms Rock & Sand*	*40*	*0*	*2884*	*8.2*	*FAIL*	*1988*
Alternate US 95 and US 50	*Gopher Pit (Fernley) Sample B*	*70*	*0*	*1957*	*9.2*	*FAIL*	*2005*
Alternate US 95 and US 50	*Gopher Pit (Fernley) Sample C*	*70*	*0*	*1918*	*9.2*	*FAIL*	*2005*
Alternate US 95 and US 50	*Gopher Pit (Fernley) Sample A*	*70*	*0*	*2191*	*9.1*	*FAIL*	*2005*
Alternate US 95 and US 50	Gopher Pit (Fernley)	0	0	3604	9.0	PASS	2005
I 15 and Cheyenne Blvd.	Chem Star Apex Pit Stockpile #3	70	0	5525	8.3	PASS	1998
I 15 and Cheyenne Blvd.	Chem Star Apex Pit Stockpile #2	70	0	5938	8.2	PASS	1998
I 15 and Cheyenne Blvd.	*Frehner Sloan Pit Stockpile #1*	*90*	*0*	*2206*	*8.5*	*FAIL*	*1998*
I 15 and Cheyenne Blvd.	Frehner Sloan Pit Stockpile #3	90	0	3839	8.5	PASS	1998
I 15 and Lamb Blvd.	*Chem Star @ Apex*	*50*	*0*	*2348*	*7.4*	*FAIL*	*2004*
I 515 and Charleston Blvd.	*Nellis Pit- Nevada Rock & Sand*	*310*	*1600*	*1052*	*7.7*	*FAIL*	*1982*
I 515 and Charleston Blvd.	*Henderson Pit - Nevada Rock & Sand*	*190*	*600*	*1247*	*7.2*	*FAIL*	*1982*
I 515 and Charleston Blvd.	*Henderson Pit - Nevada Rock & Sand*	*100*	*550*	*2150*	*7.7*	*FAIL*	*1982*
I 515 and Charleston Blvd.	*Henderson Pit - Nevada Rock & Sand*	*110*	*500*	*1935*	*7.7*	*FAIL*	*1982*
I 515 and Las Vegas Blvd.[3]	Tab Const. Gravel Pit near Nellis A.F.B.	70	0	-	7.1	Unknown	1982
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #40*	*90*	*126*	*2213*	*8.4*	*FAIL*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #22*	*70*	*111*	*2048*	*8.3*	*FAIL*	*2008*

SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #37*	*90*	*109*	*1999*	*8.2*	*FAIL*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #36*	*70*	*103*	*2236*	*8.2*	*FAIL*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #35*	*80*	*87*	*2094*	*8.2*	*FAIL*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #33*	*70*	*85*	*1964*	*7.8*	*FAIL*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #32*	*80*	*117*	*2100*	*8.1*	*FAIL*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #18*	*80*	*111*	*2085*	*8.5*	*FAIL*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #13*	*80*	*98*	*2048*	*8.4*	*FAIL*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #28*	*80*	*113*	*2127*	*8.3*	*FAIL*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #29*	*80*	*106*	*1999*	*8.3*	*FAIL*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #26*	*80*	*104*	*2236*	*8.3*	*FAIL*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #25*	*65*	*95*	*2162*	*8.3*	*FAIL*	*2008*
SR 160 and Jones Rd.	*Blue Diamond Pit Stockpile #14*	*80*	*95*	*2094*	*8.5*	*FAIL*	*2008*

[1] Converted to AASTHO T288 using the Thornley equation

[2] Based on NDOT 2001 (Same as AASHTO 2012)

[3] Resistivity testing data unavailable

Indicates value failing specification

Table 2-5: Pass/ Fail assessment of MSE backfill based on NDOT 2001 and mean testing results

Site	Chloride [ppm]			Sulfate [ppm]			Resistivity[1] [Ω-cm]			pH			Pass/Fail[2]
	Min.	Max.	Mean	Min.	Max.	Mean	Min.	Max.	Mean	Min.	Max.	Mean	
US 395 and Huffaker Lane	40	40	40	0	0	0	2884	2884	2884	8.2	8.2	8.2	FAIL
Alternate US 95 and US 50	0	70	53	0	0	0	1918	3604	2418	9	9.2	9.1	FAIL
I 15 and Cheyenne Blvd.	70	90	80	0	0	0	2206	5938	4377	8.2	8.5	8.4	PASS
I 15 and Lamb Blvd.	50	50	50	0	0	0	2348	2348	2348	7.4	7.4	7.4	FAIL
I 515 and Charleston Blvd.	100	310	178	500	1600	813	1052	2150	1596	7.2	7.7	7.6	FAIL
I 515 and Las Vegas Blvd.[3]	70	70	70	0	0	0	-	-	-	7.1	7.1	7.1	UNKNOWN
SR 160 and Jones Rd.	65	90	78	85	126	104	1964	2236	2100	7.8	8.5	8.3	FAIL

[1] Converted to AASTHO T288 using the Thornley equation

[2] Mean values, based on NDOT 2001 (Same as AASHTO 2012)

[3] Resistivity testing data unavailable

Table 2-6: ALT. US 95 and ALT. US 50, descriptive statistics of as-built soil testing results

ALT. US 95 and ALT. US 50	Chloride [ppm]	Sulfate [ppm]	Resistivity[1] [Ω-cm]	pH
Mean	53	0	2418	9.1
Median	70	0	2074	9.2
Standard Deviation	35	0	800	0.1
Coefficient of Variation	0.67	0.00	0.33	0.01
Minimum	0	0	1918	9.0
Maximum	70	0	3604	9.2
Count	4	4	4	4
Confidence Level (95.0%)	34	0	784	0.1
Confidence Range for Mean	(18 - 86)	(0)	(1633 - 3201)	(9.0 - 9.2)

[1] Converted to AASTHO T288 using the Thornley equation

Table 2-7: I-15 and Cheyenne Blvd., descriptive statistics of as-built soil testing results

I 15 and Cheyenne Blvd	Chloride [ppm]	Sulfate [ppm]	Resistivity[1] [Ω-cm]	pH
Mean	80	0	4377	8.4
Median	80	0	4682	8.4
Standard Deviation	12	0	1709	0.2
Coefficient of Variation	0.14	0.00	0.39	0.02
Minimum	70	0	2206	8.2
Maximum	90	0	5938	8.5
Count	4	4	4	4
Confidence Level (95.0%)	11	0	1674	0.1
Confidence Range for Mean	(68 - 91)	(0)	(2702 - 6051)	(8.2 - 8.5)

[1] Converted to AASTHO T288 using the Thornley equation

Table 2-8: I-515 and Charleston Blvd., descriptive statistics of as-built soil testing results

I 515 and Charleston Blvd.	Chloride [ppm]	Sulfate [ppm]	Resistivity[1] [Ω-cm]	pH
Mean	178	813	1596	7.6
Median	150	575	1591	7.7
Standard Deviation	97	527	529	0.3
Coefficient of Variation	0.55	0.65	0.33	0.03
Minimum	100	500	1052	7.2
Maximum	310	1600	2150	7.7
Count	4	4	4	4
Confidence Level (95.0%)	95	516	518	0.2
Confidence Range for Mean	(82 - 272)	(0)	(1077 - 2114)	(7.3 - 7.8)

[1] Converted to AASTHO T288 using the Thornley equation

Table 2-9: SR 160 and Jones Blvd., descriptive statistics of as-built soil testing results

SR 160 and Jones Blvd.	Chloride [ppm]	Sulfate [ppm]	Resistivity [Ω-cm]	pH
Mean	78	104	2100	8.3
Median	80	105	2094	8.3
Standard Deviation	7	11	87	0.2
Coefficient of Variation	0.09	0.11	0.04	0.02
Minimum	65	85	1964	7.8
Maximum	90	126	2236	8.5
Count	14	14	14	14
Confidence Level (95.0%)	4	6	46	0.1
Confidence Range for Mean	(74 - 82)	(98 - 110)	(2054 - 2146)	(8.2 - 8.4)

[1] Converted to AASTHO T288 using the Thornley equation

Table 2-10: Phase II MSE wall backfill electrochemical testing results

Site	Wall	Station	Sample Location	Chloride [mg/kg]	Sulfate [mg/kg]	Resistivity [Ω-cm]	pH	Organic Content [%]	Pass /Fail [1]
US 395 and Huffaker Lane	*1*	*A*	*P1*	*160*	*190*	*490*	*9.07*	*2.00*	*FAIL*
US 395 and Huffaker Lane	*1*	*A*	*P5*	*3*	*27*	*2410*	*8.96*	*3.10*	*FAIL*
US 395 and Huffaker Lane	*2*	*A*	*P3*	*3*	*17*	*3030*	*9.22*	*2.30*	*FAIL*
US 395 and Huffaker Lane	*2*	*B*	*P1*	*3*	*27*	*1390*	*9.34*	*1.50*	*FAIL*
US 395 and Huffaker Lane	*2*	*B*	*P4*	*3*	*14*	*1490*	*8.78*	*1.60*	*FAIL*
Alternate US 95 and US 50	*1*	*D*	*P2*	*23*	*230*	*570*	*9.39*	*1.20*	*FAIL*
Alternate US 95 and US 50	*1*	*A*	*P2*	*0*	*160*	*860*	*9.49*	*1.30*	*FAIL*
Alternate US 95 and US 50	*1*	*B*	*P3*	*0*	*180*	*570*	*9.44*	*1.40*	*FAIL*
Alternate US 95 and US 50	*1*	*C*	*P2*	*11*	*230*	*460*	*9.47*	*1.10*	*FAIL*
Alternate US 95 and US 50	*1*	*E*	*P2*	*45*	*380*	*630*	*9.31*	*0.86*	*FAIL*
I 15 and Cheyenne Blvd.	*1*	*A*	*B*	*68*	*220*	*1090*	*9.24*	*0.10*	*FAIL*
I 15 and Cheyenne Blvd.	*1*	*A*	*T*	*10*	*180*	*1830*	*9.32*	*0.11*	*FAIL*
I 15 and Cheyenne Blvd.	*1*	*A*	*EX*	*340*	*430*	*303*	*9.10*	*0.17*	*FAIL*
I 15 and Cheyenne Blvd.	*1*	*B*	*B*	*23*	*270*	*971*	*9.32*	*0.09*	*FAIL*
I 15 and Cheyenne Blvd.	*1*	*B*	*T*	*10*	*210*	*2460*	*9.38*	*0.10*	*FAIL*

I 15 and Cheyenne Blvd.	*1*	*C*	*B*	*11*	*200*	*971*	*9.02*	*0.18*	*FAIL*
I 15 and Cheyenne Blvd.	*1*	*C*	*T*	*10*	*190*	*1430*	*9.19*	*0.14*	*FAIL*
I 15 and Cheyenne Blvd.	1	D	B	10	64	4630	8.71	0.08	PASS
I 15 and Cheyenne Blvd.	1	D	M	10	66	3140	9.22	0.11	PASS
I 15 and Cheyenne Blvd.	1	D	T	10	21	4170	9.64	0.10	PASS
I 15 and Cheyenne Blvd.	*1*	*D*	*EX*	*10*	*190*	*2630*	*9.46*	*0.13*	*FAIL*
I 15 and Cheyenne Blvd.	2	A	S	10	5	5540	9.52	0.10	PASS
I 15 and Cheyenne Blvd.	*4*	*A*	*T*	*10*	*32*	*2860*	*9.56*	*0.11*	*FAIL*
I 15 and Cheyenne Blvd.	*4*	*A*	*B*	*10*	*30*	*2690*	*9.74*	*0.09*	*FAIL*
I 15 and Cheyenne Blvd.	*4*	*A*	*EX*	*10*	*270*	*570*	*9.55*	*0.06*	*FAIL*
I 15 and Cheyenne Blvd.	4	B	T	10	15	4860	9.50	0.09	PASS
I 15 and Cheyenne Blvd.	*4*	*B*	*B*	*10*	*29*	*2570*	*9.67*	*0.09*	*FAIL*
I 15 and Cheyenne Blvd.	4	B	EX	10	110	3890	9.76	0.07	PASS
I 15 and Cheyenne Blvd.	5	A	S	10	13	5540	9.58	0.09	PASS
I 15 and Cheyenne Blvd.	5	A	EX	10	5	6860	9.53	0.13	PASS
I 15 and Cheyenne Blvd.	6	A	T	10	46	6280	8.96	0.10	PASS

I 15 and Cheyenne Blvd.	6	A	B	10	34	7430	8.98	0.08	PASS
I 15 and Lamb Blvd.	*1*	*A*	*B*	*10*	*130*	*2300*	*9.13*	*0.12*	*FAIL*
I 15 and Lamb Blvd.	*1*	*A*	*T*	*10*	*230*	*1770*	*9.18*	*0.15*	*FAIL*
I 15 and Lamb Blvd.	*2*	*A*	*S*	*10*	*170*	*2290*	*9.15*	*0.12*	*FAIL*
I 15 and Lamb Blvd.	*2*	*A*	*EX*	*10*	*100*	*1370*	*9.23*	*0.26*	*FAIL*
I 15 and Lamb Blvd.	*3*	*A*	*S*	*10*	*260*	*1430*	*9.21*	*0.20*	*FAIL*
I 15 and Lamb Blvd.	*3*	*B*	*S*	*10*	*150*	*2290*	*9.22*	*0.11*	*FAIL*
I 15 and Lamb Blvd.	*4*	*A*	*S*	*10*	*260*	*1140*	*9.18*	*0.11*	*FAIL*
I 515 and Charleston Blvd.	*3*	*A*	*S*	*34*	*770*	*910*	*8.72*	*0.39*	*FAIL*
I 515 and Charleston Blvd.	*3*	*B*	*S*	*11*	*800*	*630*	*8.74*	*0.29*	*FAIL*
I 515 and Charleston Blvd.	*3*	*C*	*S*	*90*	*760*	*740*	*8.68*	*0.47*	*FAIL*
I 515 and Charleston Blvd.	*4*	*A*	*S*	*10*	*770*	*1600*	*8.78*	*0.49*	*FAIL*
I 515 and Charleston Blvd.	4	B	S	10	120	3370	9.02	0.20	PASS
I 515 and Charleston Blvd.	*4*	*C*	*S*	*10*	*700*	*1030*	*8.80*	*0.87*	*FAIL*
I 515 and Las Vegas Blvd.	*1*	*A*	*S*	*10*	*670*	*3940*	*8.35*	*1.00*	*FAIL*
I 515 and Las Vegas Blvd.	1	B	S	10	41	3140	8.90	0.85	PASS
I 515 and Las Vegas Blvd.	*1*	*B*	*EX*	*10*	*1700*	*3490*	*8.19*	*1.10*	*FAIL*
SR 160 and Jones Rd.	*1*	*A*	*B*	*11*	*190*	*690*	*9.30*	*0.48*	*FAIL*
SR 160 and Jones Rd.	*1*	*A*	*M*	*23*	*210*	*1140*	*9.34*	*0.59*	*FAIL*
SR 160 and Jones Rd.	*1*	*A*	*T*	*10*	*160*	*1200*	*9.33*	*0.38*	*FAIL*

SR 160 and Jones Rd.	*1*	*A*	*EX*	*34*	*180*	*2460*	*9.31*	*0.32*	*FAIL*
SR 160 and Jones Rd.	*1*	*B*	*B*	*23*	*250*	*910*	*8.76*	*0.32*	*FAIL*
SR 160 and Jones Rd.	*1*	*B*	*M*	*11*	*230*	*1030*	*9.37*	*0.34*	*FAIL*
SR 160 and Jones Rd.	*1*	*B*	*T*	*11*	*220*	*860*	*8.81*	*0.20*	*FAIL*
SR 160 and Jones Rd.	*1*	*C*	*B*	*11*	*190*	*1490*	*9.40*	*0.42*	*FAIL*
SR 160 and Jones Rd.	*1*	*C*	*M*	*34*	*290*	*630*	*9.24*	*0.45*	*FAIL*
SR 160 and Jones Rd.	*1*	*C*	*T*	*23*	*220*	*690*	*9.18*	*0.38*	*FAIL*
SR 160 and Jones Rd.	*1*	*C*	*EX*	*11*	*180*	*1090*	*9.16*	*0.49*	*FAIL*
SR 160 and Jones Rd.	*1*	*D*	*B*	*10*	*180*	*1200*	*8.97*	*0.41*	*FAIL*
SR 160 and Jones Rd.	*1*	*D*	*M*	*11*	*170*	*2000*	*9.29*	*0.42*	*FAIL*
SR 160 and Jones Rd.	*1*	*D*	*T*	*11*	*150*	*1200*	*8.96*	*0.54*	*FAIL*
SR 160 and Jones Rd.	*1*	*E*	*B*	*10*	*30*	*1090*	*9.05*	*0.65*	*FAIL*
SR 160 and Jones Rd.	*1*	*E*	*M*	*10*	*47*	*1830*	*9.06*	*0.52*	*FAIL*
SR 160 and Jones Rd.	*1*	*E*	*T*	*10*	*31*	*1490*	*9.11*	*0.32*	*FAIL*
SR 160 and Jones Rd.	*1*	*F*	*B*	*10*	*160*	*1200*	*9.26*	*0.34*	*FAIL*
SR 160 and Jones Rd.	*1*	*F*	*T*	*10*	*160*	*1310*	*8.79*	*0.38*	*FAIL*
SR 160 and Jones Rd.	*1*	*G*	*B*	*10*	*160*	*2290*	*9.27*	*0.37*	*FAIL*
SR 160 and Jones Rd.	1	G	T	10	170	4910	8.96	0.61	PASS
SR 160 and Jones Rd.	*1*	*G*	*EX*	*10*	*240*	*1140*	*8.95*	*0.31*	*FAIL*
SR 160 and Jones Rd.	*1*	*H*	*B*	*10*	*82*	*2110*	*9.22*	*0.47*	*FAIL*
SR 160 and Jones Rd.	1	H	T	11	180	3030	9.20	0.47	PASS

[1] Based on NDOT 2001 (Same as AASHTO 2012)

Indicates value failing specification

Table 2-11: Phase II MSE wall soil testing summary data

Site	Chloride [ppm or mg/kg]			Sulfate [ppm or mg/kg]			Resistivity [Ω-cm]			pH			Organic Content [%]			Pass/ Fail[1]
	Min.	Max.	Mean	Min.	Max.	Mean	Min.	Max.	Mean	Min.	Max.	Mean	Min.	Max.	Mean	
US 395 and Huffaker Lane	3	160	34	14	190	55	490	3030	1762	8.78	9.34	9.07	1.50	3.10	2.10	FAIL
Alternate US 95 and US 50	0	45	16	160	380	236	460	860	618	9.31	9.49	9.42	0.86	1.40	1.17	FAIL
I 15 and Cheyenne Blvd.	10	340	28	5	430	120	303	7430	3305	8.71	9.76	9.36	0.06	0.18	0.11	PASS
I 15 and Lamb Blvd.	10	10	10	100	260	186	1140	2300	1799	9.13	9.23	9.19	0.11	0.26	0.15	FAIL
I 515 and Charleston Blvd.	10	90	28	120	800	653	630	3370	1380	8.68	9.02	8.79	0.20	0.87	0.45	FAIL
I 515 and Las Vegas Blvd.	10	10	10	41	1700	804	3140	3940	3523	8.19	8.90	8.48	0.85	1.10	0.98	FAIL
SR 160 and Jones Rd.	10	34	14	30	290	170	630	4910	1541	8.76	9.40	9.14	0.20	0.65	0.42	FAIL

Table 2-12: Descriptive statistics for Alt. US 95 and Alt. US 95 Phase II MSE wall backfill soil testing.

Alt. US 95 and Alt. US 50	Chloride [mg/kg]	Sulfate [mg/kg]	Resistivity [Ω-cm]	pH	Organic Content [%]
Mean	16	236	618	9.4	1.17
Median	11	230	570	9.4	1.20
Standard Deviation	19	86	149	0.1	0.21
Coefficient of Variation	1.20	0.37	0.24	0.01	0.18
Minimum	0	160	460	9.3	0.86
Maximum	45	380	860	9.5	1.40
Count	5	5	5	5	5
Confidence Level (95.0%)	17	76	130	0.1	0
Confidence Range for Mean	(0 - 32)	(160 - 312)	(488 - 748)	(9.4 - 9.5)	(0.99 - 1.35)

Table 2-13: Descriptive statistics for US 395 and Huffaker Lane Phase II MSE wall backfill soil testing.

US 395 and Huffaker Lane	Chloride [mg/kg]	Sulfate [mg/kg]	Resistivity [Ω-cm]	pH	Organic Content [%]
Mean	34	55	1762	9.1	2.10
Median	3	27	1490	9.1	2.00
Standard Deviation	70	76	982	0.2	0.64
Coefficient of Variation	2.04	1.38	0.56	0.02	0.31
Minimum	3	14	490	8.8	1.50
Maximum	160	190	3030	9.3	3.10
Count	5	5	5	5	5
Confidence Level (95.0%)	62	66	861	0.2	0.56
Confidence Range for Mean	(0 - 95)	(0 - 121)	(901 - 2622)	(8.9 - 9.3)	(1.54 - 2.66)

Table 2-14: Multi-wall descriptive statistics for US 395 and Huffaker Lane, Phase II MSE wall backfill soil testing.

Wall	Chloride [ppm]			Sulfate [ppm]			Resistivity [Ω-cm]			pH			Organic Content [%]			Pass/Fail
	Min.	Max.	Mean	Min.	Max.	Mean	Min.	Max.	Mean	Min.	Max.	Mean	Min.	Max.	Mean	
1	3	160	82	27	190	109	490	2410	1450	8.69	9.07	8.88	2.00	3.10	2.55	FAIL
2	3	3	3	14	27	19	1390	3030	1970	8.78	9.34	9.11	1.50	2.30	1.80	FAIL

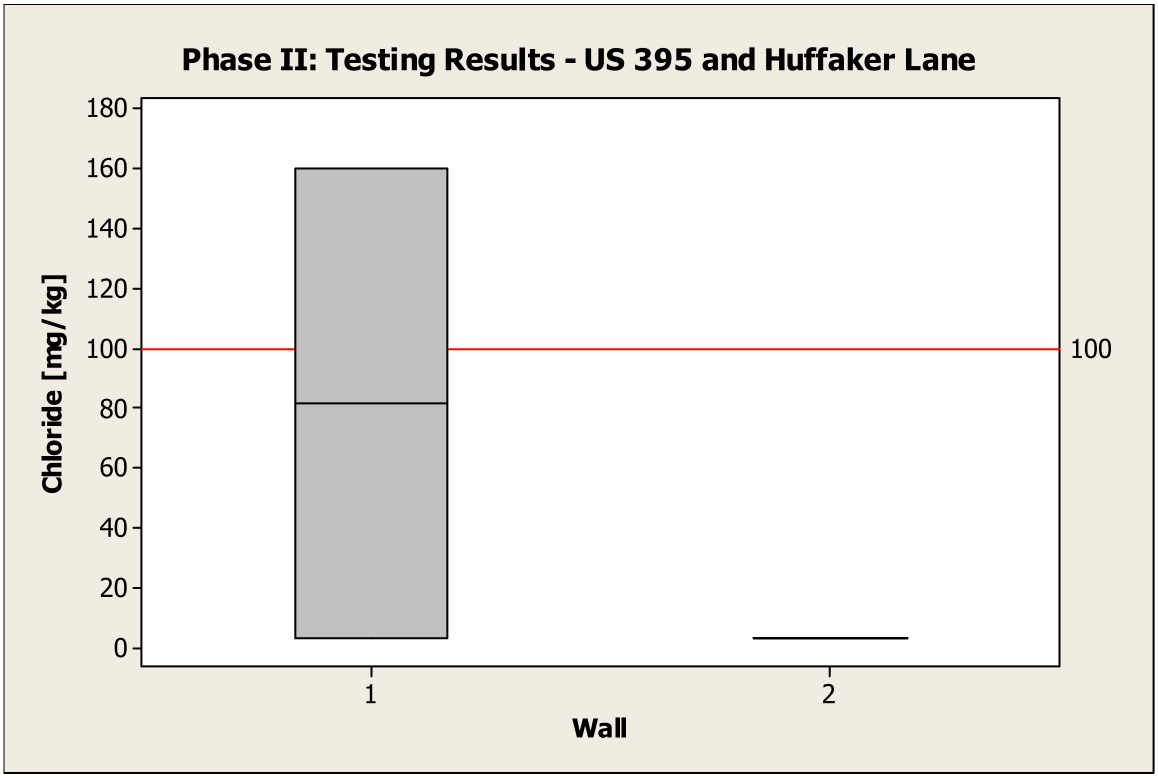

Figure 2-1: Phase II chloride results for US 395 and Huffaker Lane, separated by wall

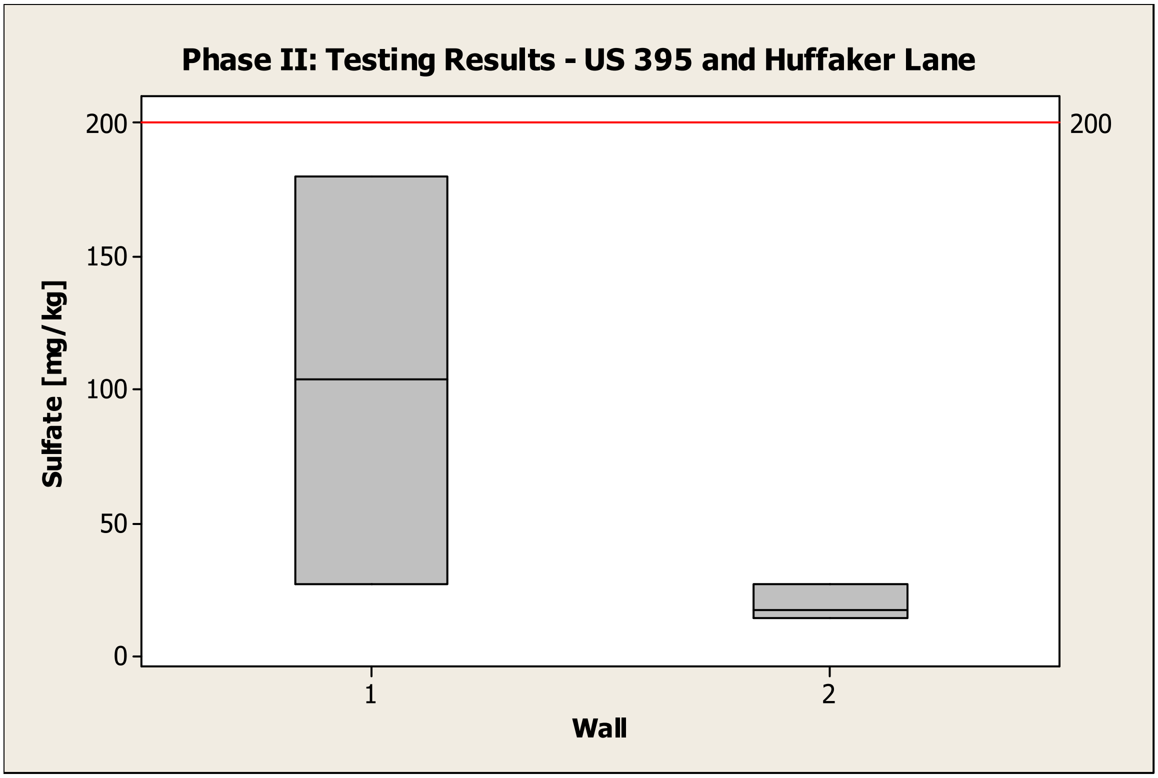

Figure 2-2: Phase II sulfate results for US 395 and Huffaker Lane, separated by wall

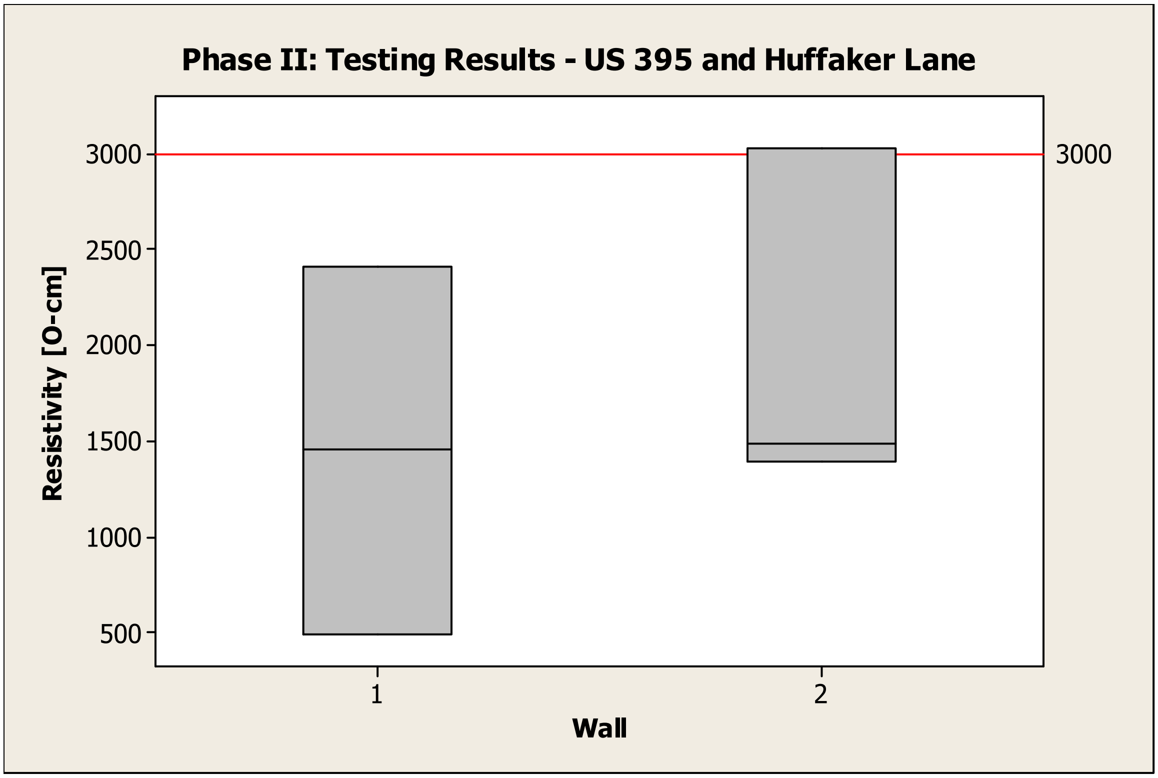

Figure 2-3: Phase II resistivity results for US 395 and Huffaker Lane, separated by wall

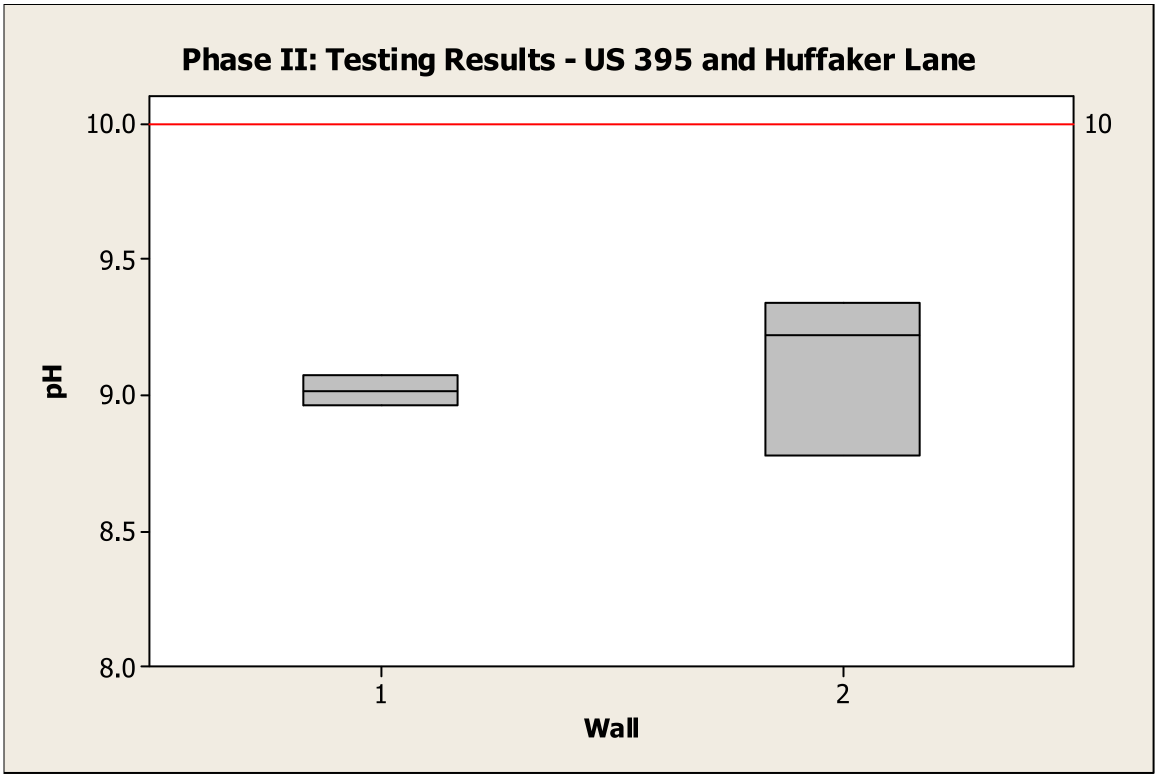

Figure 2-4: Phase II pH results for US 395 and Huffaker Lane, separated by wall

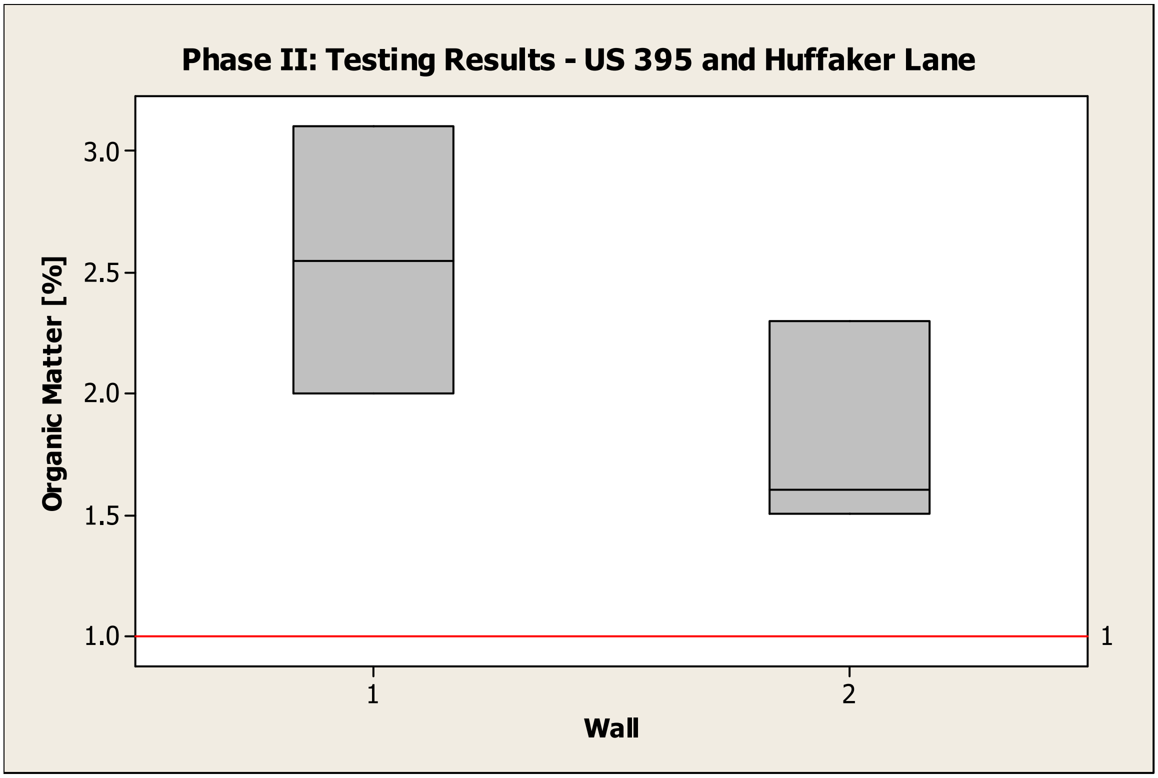

Figure 2-5: Phase II organic content results for US 395 and Huffaker Lane, separated by wall

Table 2-15: Descriptive statistics for I-15 and Cheyenne Blvd. Phase II MSE wall backfill soil testing.

I 15 and Cheyenne Blvd.	Chloride [mg/kg]	Sulfate [mg/kg]	Resistivity [Ω-cm]	pH	Organic Content [%]
Mean	28	120	3305	9.4	0.11
Median	10	65	2775	9.4	0.10
Standard Deviation	71	116	2116	0.3	0.03
Coefficient of Variation	2.50	0.97	0.64	0.03	0.28
Minimum	10	5	303	8.7	0.06
Maximum	340	430	7430	9.8	0.18
Count	22	22	22	22	22
Confidence Level (95.0%)	30	48	884	0.1	0.01
Confidence Range for Mean	(0 - 58)	(71 - 168)	(2421 - 4190)	(9.2 - 9.5)	(0.09 - 0.12)

Table 2-16: Multi-wall descriptive statistics for I-15 and Cheyenne Blvd., Phase II MSE wall backfill soil testing

Wall	Chloride [ppm]			Sulfate [ppm]			Resistivity [Ω-cm]			pH			Organic Content [%]			Pass/Fail
	Min.	Max.	Mean	Min.	Max.	Mean	Min.	Max.	Mean	Min.	Max.	Mean	Min.	Max.	Mean	
1	10	340	47	21	430	186	303	4630	2148	8.71	9.64	9.24	0.08	0.18	0.12	FAIL
2	10	10	10	5	5	5	5540	5540	5540	9.52	9.52	9.52	0.10	0.10	0.10	PASS
4	10	10	10	15	270	81	570	4860	2907	9.50	9.76	9.63	0.06	0.11	0.09	FAIL
5	10	10	10	5	13	9	5540	6860	6200	9.53	9.58	9.56	0.09	0.13	0.11	PASS
6	10	10	10	34	46	40	6280	7430	6855	8.96	8.98	8.97	0.08	0.10	0.09	PASS

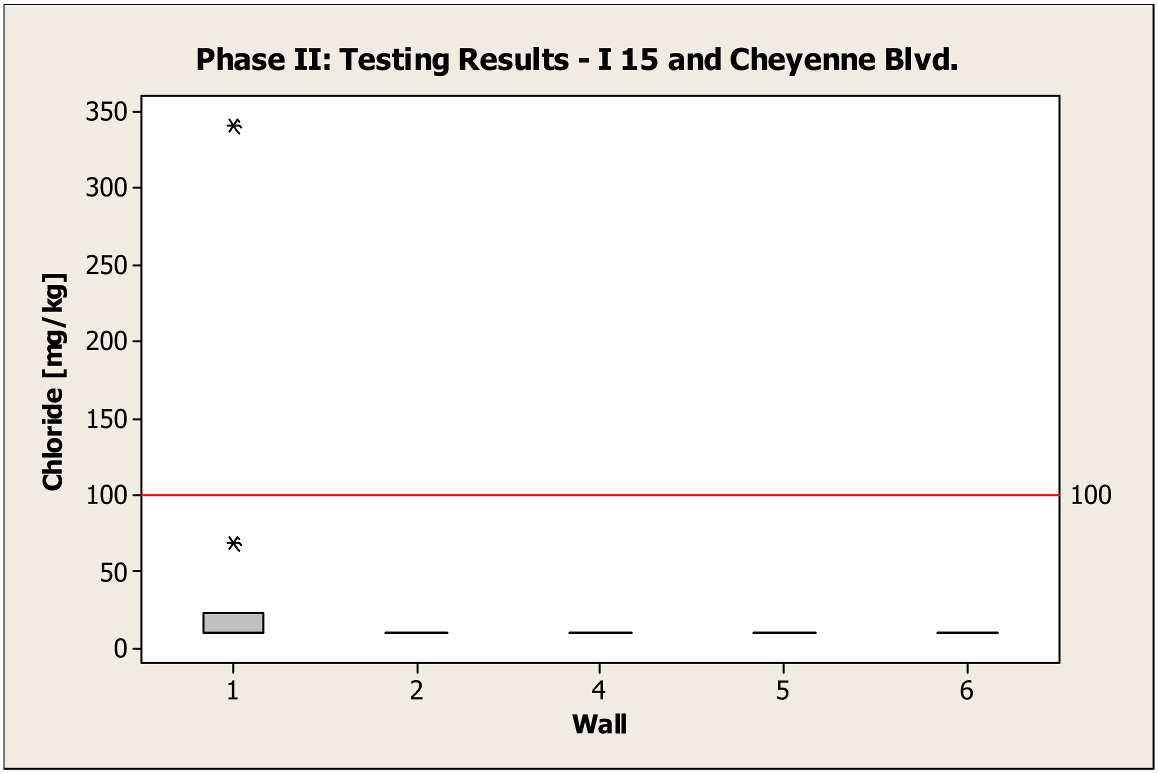

Figure 2-6: Phase II chloride results for I-15 and Cheyenne Blvd., separated by wall

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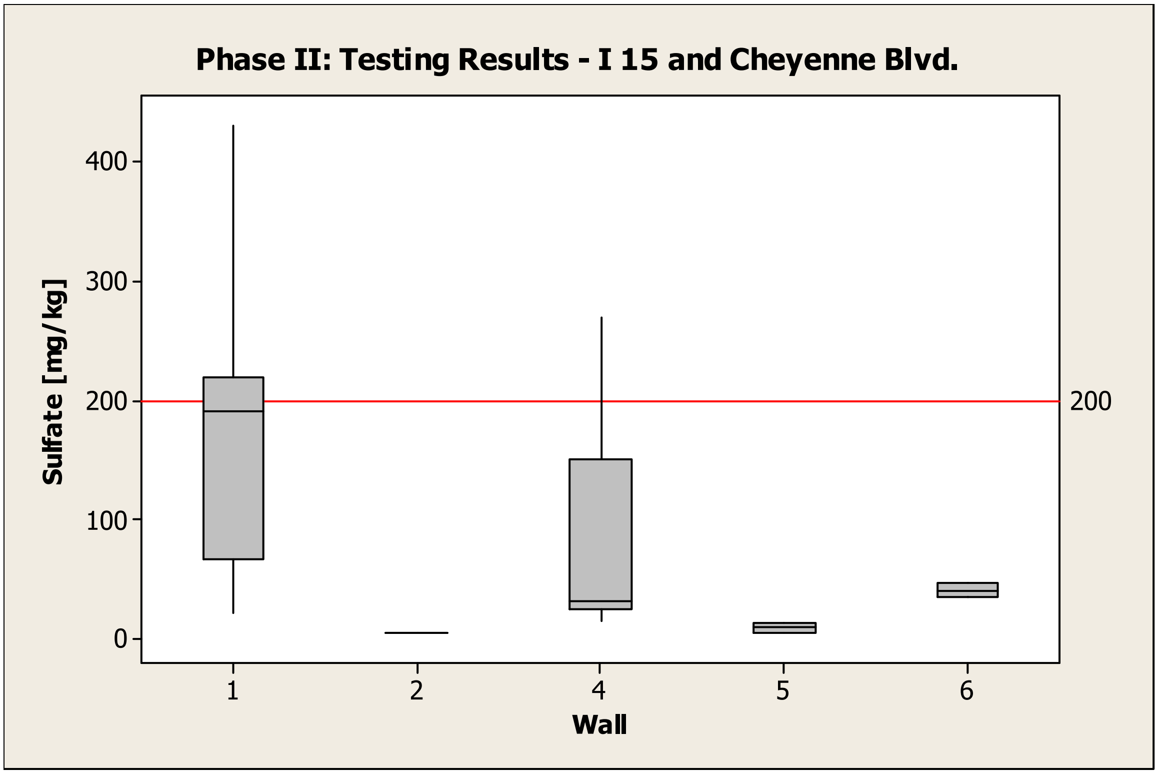

Figure 2-7: Phase II sulfate results for I-15 and Cheyenne Blvd., separated by wall

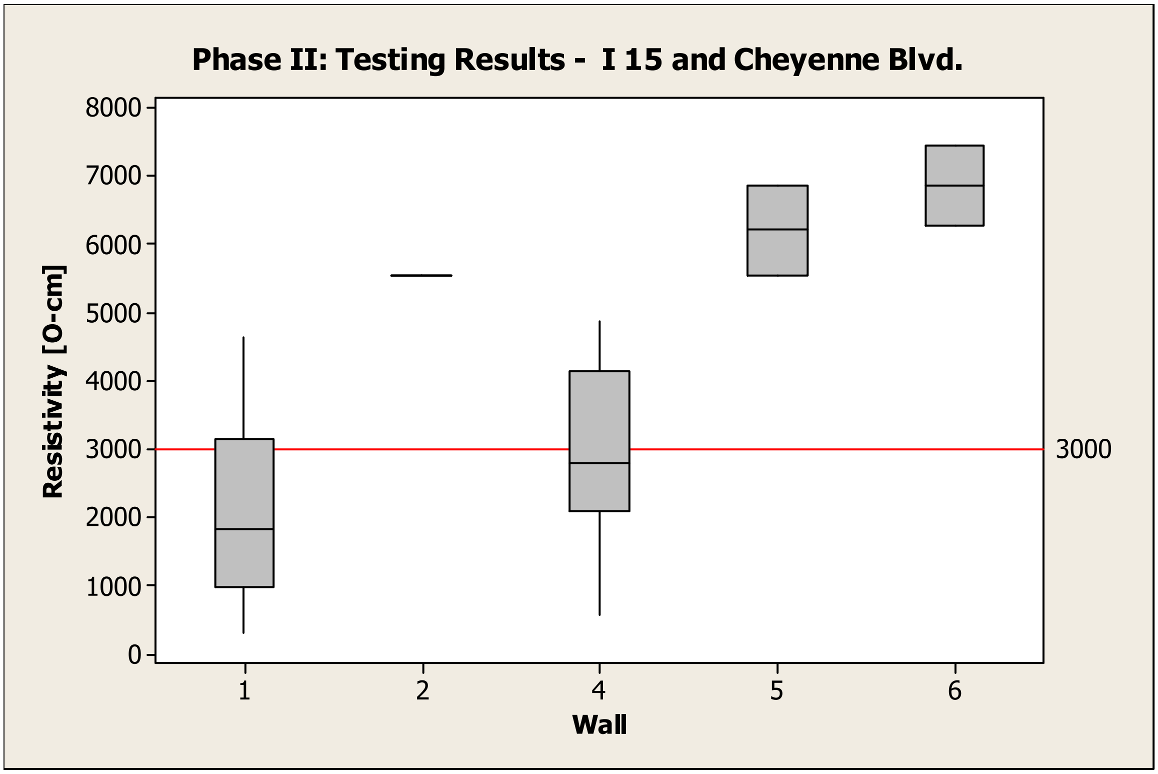

Figure 2-8: Phase II resistivity results for I-15 and Cheyenne Blvd., separated by wall

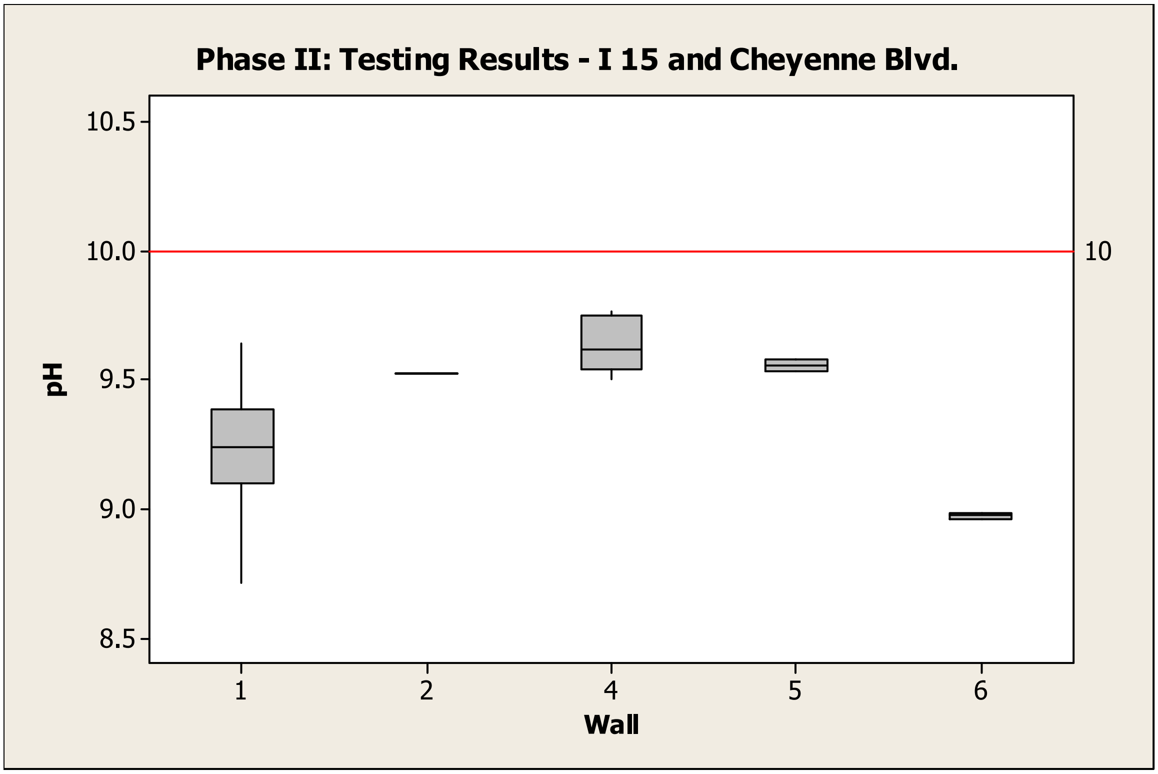

Figure 2-9: Phase II pH results for I-15 and Cheyenne Blvd., separated by wall

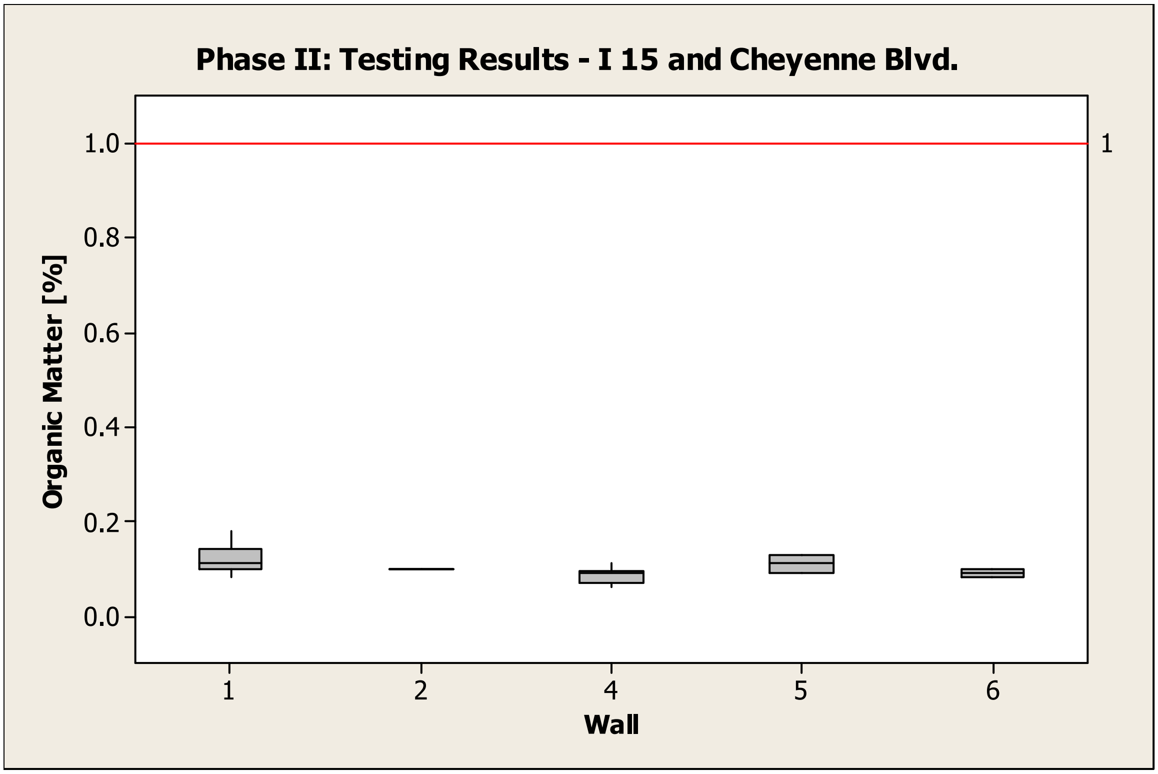

Figure 2-10: Phase II organic content results for I-15 and Cheyenne Blvd., separated by wall

Table 2-17: Descriptive statistics for I-15 and Lamb Blvd. Phase II MSE wall backfill soil testing

I 15 and Lamb Blvd.	Chloride [mg/kg]	Sulfate [mg/kg]	Resistivity [Ω-cm]	pH	Organic Content [%]
Mean	10	186	1799	9.2	0.15
Median	10	170	1770	9.2	0.12
Standard Deviation	0	65	498	0.0	0.06
Coefficient of Variation	0.00	0.35	0.28	0.00	0.37
Minimum	10	100	1140	9.1	0.11
Maximum	10	260	2300	9.2	0.26
Count	7	7	7	7.0	7
Confidence Level (95.0%)	0	48	369	0.0	0.04
Confidence Range for Mean	(10)	(138 - 234)	(1430 - 2168)	(9.2)	(0.11 - 0.2)

Table 2-18: Multi-wall descriptive statistics for I-15 and Lamb Blvd., Phase II MSE wall backfill soil testing.

Wall	Chloride [ppm]			Sulfate [ppm]			Resistivity [Ω-cm]			pH			Organic Content [%]			Pass/Fail
	Min.	Max.	Mean	Min.	Max.	Mean	Min.	Max.	Mean	Min.	Max.	Mean	Min.	Max.	Mean	
1	10	10	10	130	230	180	1770	2300	2035	9.13	9.18	9.16	0.12	0.15	0.14	FAIL
2	10	10	10	100	170	135	1370	2290	1330	9.15	9.23	9.19	0.12	0.26	0.19	FAIL
3	10	10	10	150	260	205	1430	2290	1860	9.21	9.22	9.22	0.11	0.20	0.16	FAIL
4	10	10	10	260	260	260	1140	1140	1140	9.18	9.18	9.18	0.11	0.11	0.11	FAIL

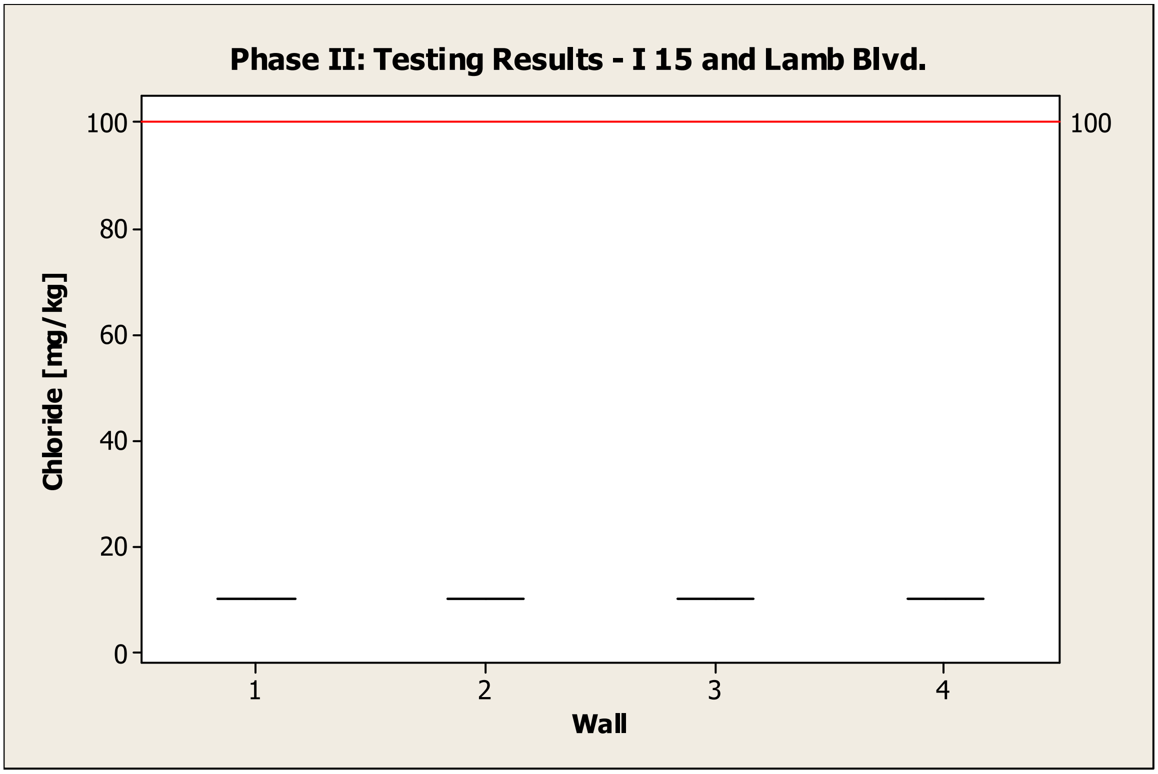

Figure 2-11: Phase II chloride results for I-15 and Lamb Blvd., separated by wall

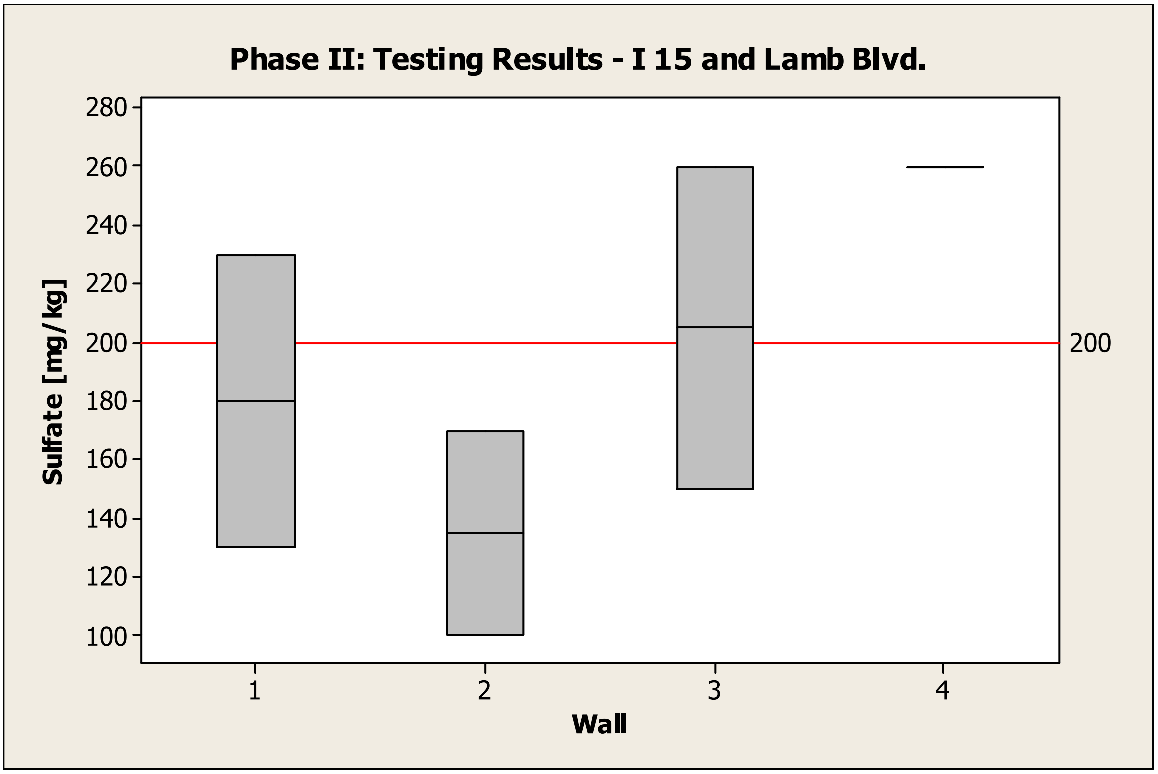

Figure 2-12: Phase II sulfate results for I-15 and Lamb Blvd., separated by wall

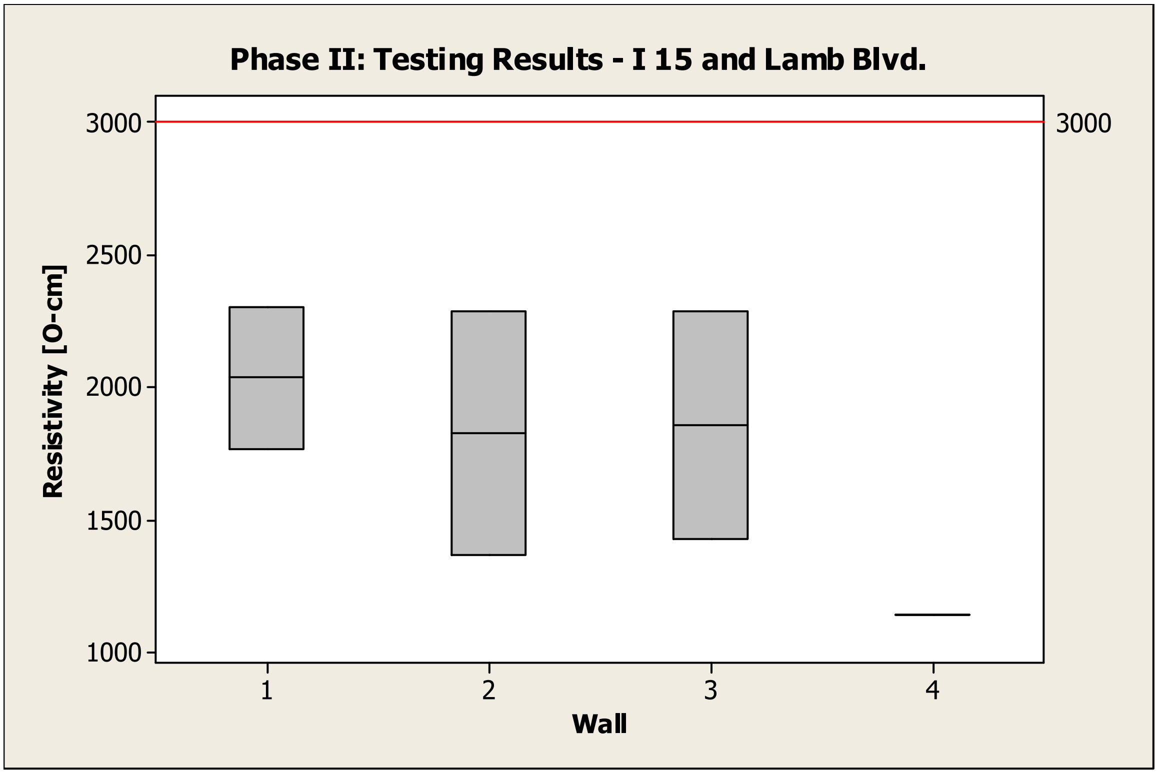

Figure 2-13: Phase II resistivity results for I-15 and Lamb Blvd., separated by wall

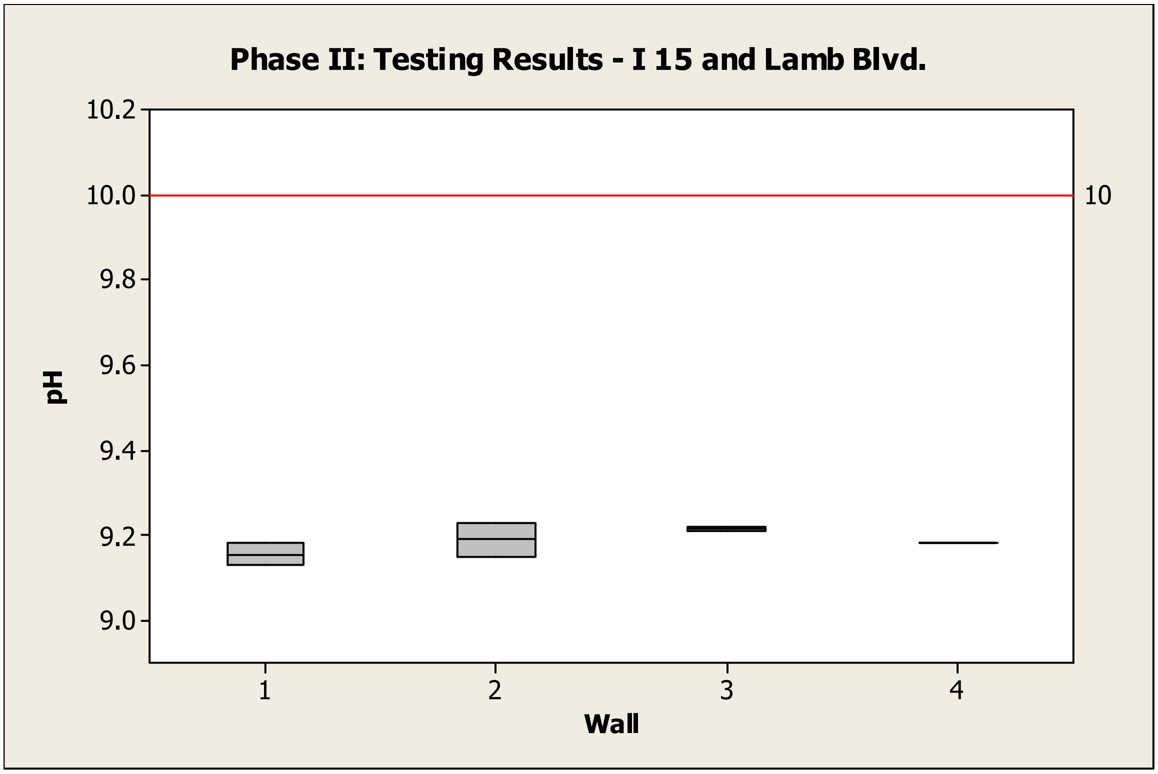

Figure 2-14: Phase II pH results for I-15 and Lamb Blvd., separated by wall

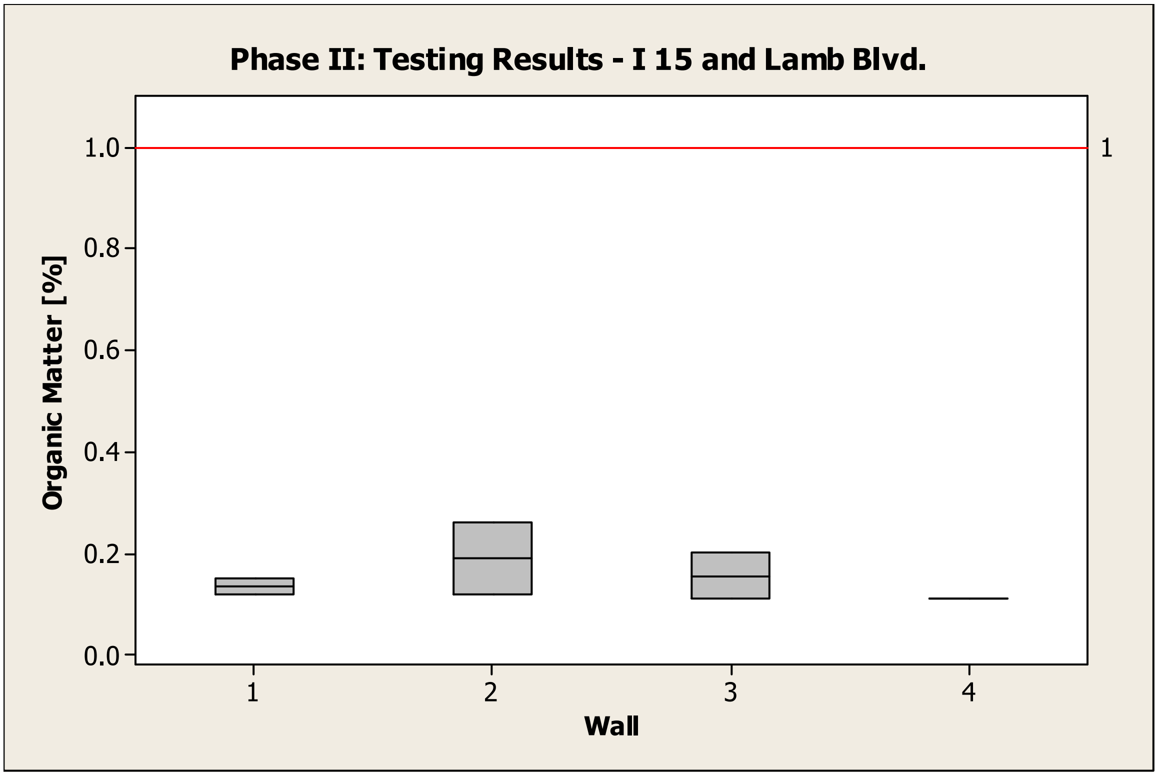

Figure 2-15: Phase II organic content results for I-15 and Lamb Blvd., separated by wall

Table 2-19: Descriptive statistics for I-515 and Charleston Blvd. Phase II MSE wall backfill soil testing

I 515 and Charleston Blvd.	Chloride [mg/kg]	Sulfate [mg/kg]	Resistivity [Ω-cm]	pH	Organic Content [%]
Mean	28	653	1380	8.8	0.45
Median	11	765	970	8.8	0.43
Standard Deviation	32	263	1032	0.1	0.23
Coefficient of Variation	1.17	0.40	0.75	0.01	0.51
Minimum	10	120	630	8.7	0.20
Maximum	90	800	3370	9.0	0.87
Count	6	6	6	6	6
Confidence Level (95.0%)	26	211	826	0.1	0.19
Confidence Range for Mean	(2 - 53)	(443 - 864)	(554 - 2206)	(8.7 - 8.9)	(0.27 - 0.64)

Table 2-20: Multi-wall descriptive statistics for I-515 and Charleston Blvd., Phase II MSE wall backfill soil testing

Wall	Chloride [ppm]			Sulfate [ppm]			Resistivity [Ω-cm]			pH			Organic Content [%]			Pass/Fail[1]
	Min.	Max.	Mean	Min.	Max.	Mean	Min.	Max.	Mean	Min.	Max.	Mean	Min.	Max.	Mean	
3	11	90	45	760	800	777	630	910	760	8.68	8.74	8.71	0.29	0.47	0.38	FAIL
4	10	10	10	120	770	530	1030	3370	2000	8.78	9.02	8.87	0.20	0.87	0.52	FAIL

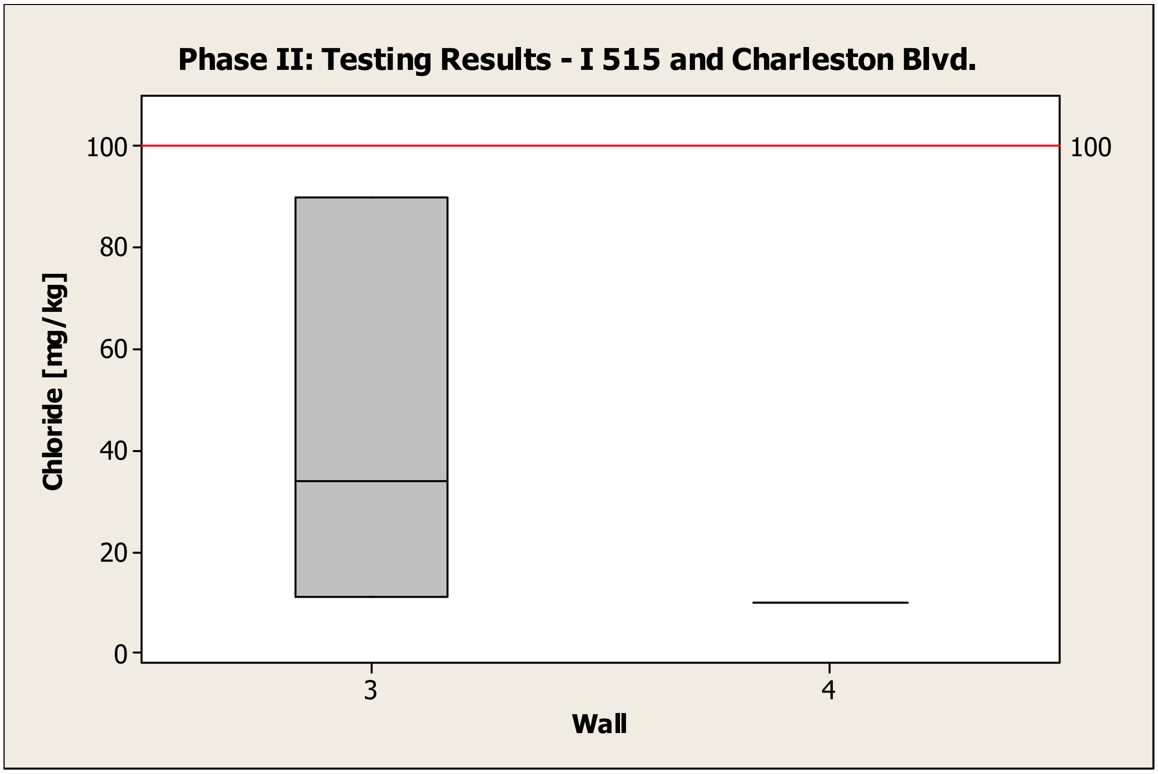

Figure 2-16: Phase II chloride results for I-515 and Charleston Blvd., separated by wall

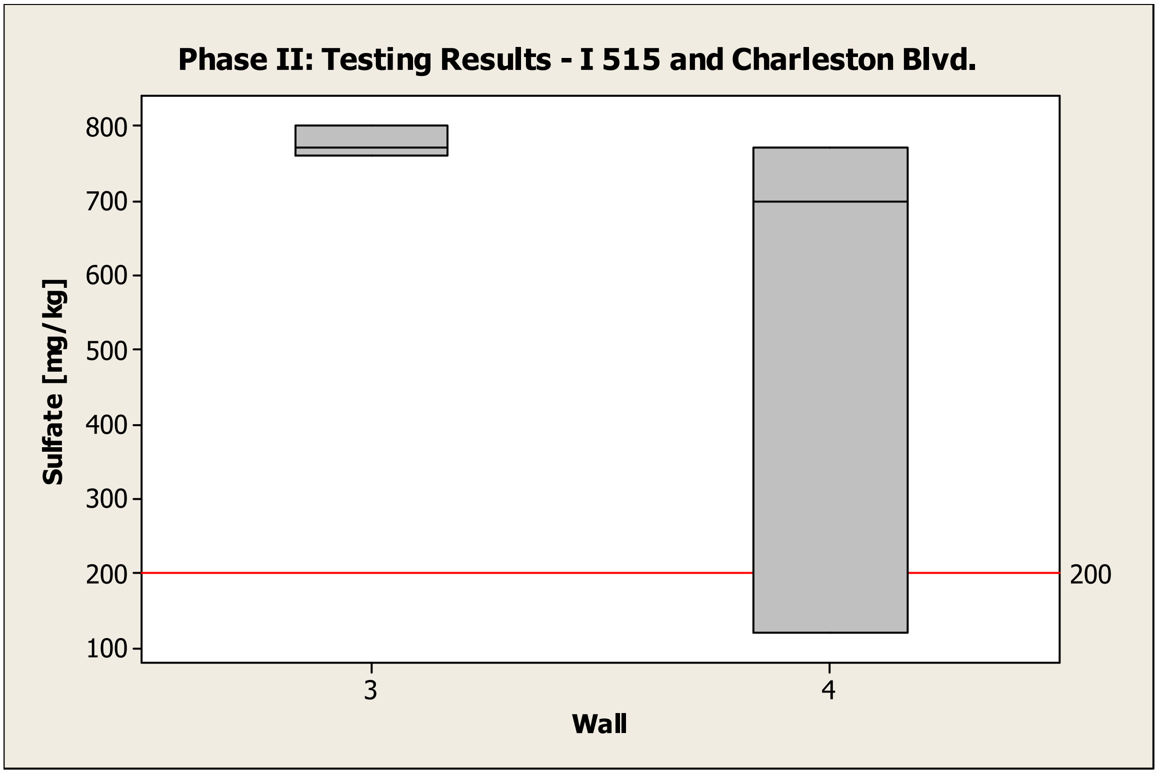

Figure 2-17: Phase II sulfate results for I-515 and Charleston Blvd., separated by wall

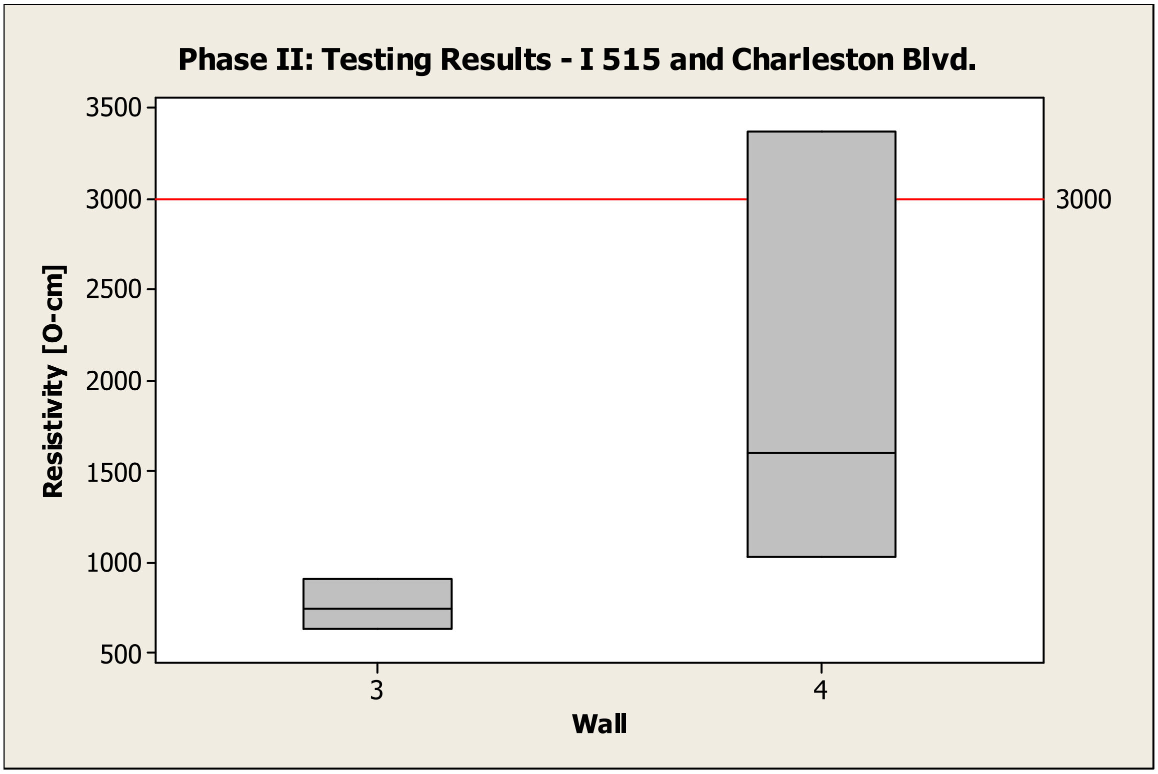

Figure 2-18: Phase II resistivity results for I-515 and Charleston Blvd., separated by wall

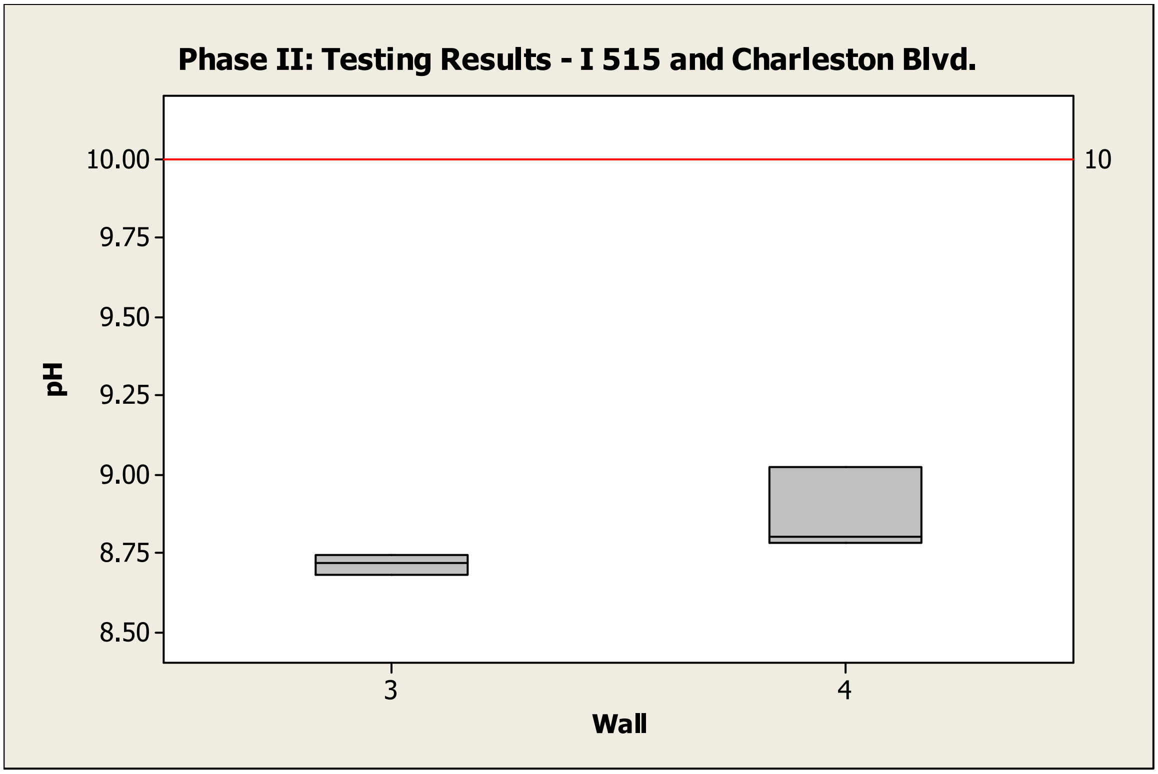

Figure 2-19: Phase II pH results for I-515 and Charleston Blvd., separated by wall

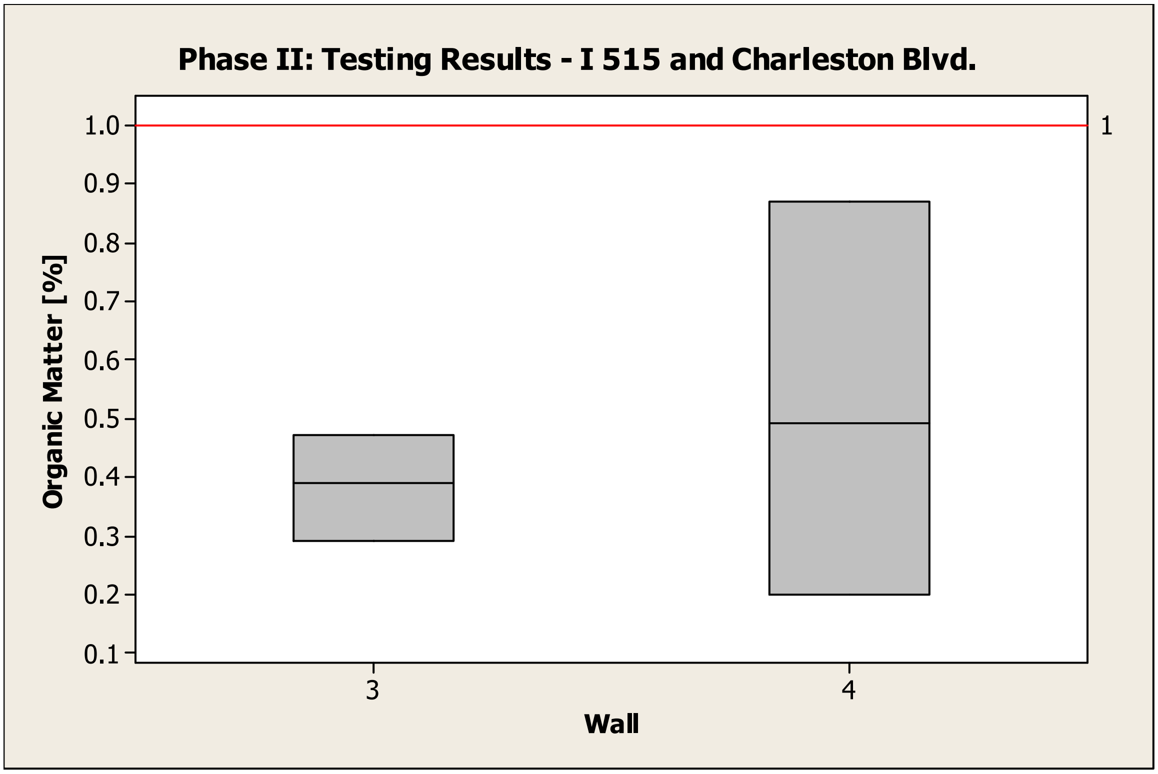

Figure 2-20: Phase II organic content results for I-515 and Charleston Blvd., separated by wall

Table 2-21: Descriptive statistics for I-515 and Las Vegas Blvd. Phase II MSE wall backfill soil testing

I 515 and Las Vegas Blvd.	Chloride [mg/kg]	Sulfate [mg/kg]	Resistivity [Ω-cm]	pH	Organic Content [%]
Mean	10	804	3523	8.5	0.98
Median	10	670	3490	8.4	1.00
Standard Deviation	0	838	401	0.4	0.13
Coefficient of Variation	0.00	1.04	0.11	0.04	0.13
Minimum	10	41	3140	8.2	0.85
Maximum	10	1700	3940	8.9	1.10
Count	3	3	3	3	3
Confidence Level (95.0%)	0	948	454	0.4	0
Confidence Range for Mean	(10)	(0 - 1751)	(3070 - 3977)	(8.1 - 8.9)	(0.84 - 1.13)

Table 2-22: Descriptive statistics for SR 160 and Jones Rd. Phase II MSE wall backfill soil testing

SR 160 and Jones Rd.	Chloride [mg/kg]	Sulfate [mg/kg]	Resistivity [Ω-cm]	pH	Organic Content [%]
Mean	14	170	1541	9.1	0.42
Median	11	180	1200	9.2	0.42
Standard Deviation	7	66	938	0.2	0.11
Coefficient of Variation	0.54	0.39	0.61	0.02	0.25
Minimum	10	30	630	8.8	0.20
Maximum	34	290	4910	9.4	0.65
Count	24	24	24	24	24
Confidence Level (95.0%)	3	26	375	0	0
Confidence Range for Mean	(11 - 17)	(144 - 196)	(1166 - 1917)	(9.1 - 9.2)	(0.38 - 0.47)

Table 2-23: Comparison of soil testing records, fundamental descriptive statistics

Indicates Change		US 395 and Huffaker Lane		Alternate US 95 and US 50		I 15 and Cheyenne Blvd.		I 15 and Lamb Blvd.		I 515 and Charleston Blvd.		I 515 and Las Vegas Blvd.		SR 160 and Jones Rd.	
		As-Built	Phase II	As-Built	Phase II	As-Built	Phase II	As-Built	Phase II	As-Built	Phase II	As-Built	Phase II	As-Built	Phase II
Chlorides [mg/kg]	Min.	40	3	0	0	70	10	50	10	100	10	70	10	65	10
	Max.	40	160	70	45	90	340	50	10	310	90	70	10	90	34
	Mean	40	34	53	16	80	28	50	10	178	28	70	10	78	14
	Std. Dev.	-	70	12	19	12	71	-	0	97	32	-	0	7	7
	COV	-	204%	22%	120%	14%	250%	-	0%	55%	117%	-	0%	9%	54%
	Count	1	5	4	5	4	22	1	7	4	6	1	3	14	24
Sulfate [mg/kg]	Min.	0	14	0	160	0	5	0	100	500	120	0	41	85	30
	Max.	0	190	0	380	0	430	0	260	1600	800	0	1700	126	290
	Mean	0	55	0	236	0	120	0	186	813	653	0	804	104	170
	Std. Dev.	-	76	-	86	-	116	-	65	97	263	-	838	11	66
	COV	-	138%	-	37%	-	97%	-	35%	12%	40%	-	104%	11%	39%
	Count	1	5	1	5	4	22	1	7	4	6	1	3	14	24
Resistivity [ohm-cm]	Min.	2884	490	1918	460	2206	303	2348	1140	1052	630	-	3140	1964	630
	Max.	2884	3030	3604	860	5938	7430	2348	2300	2150	3370	-	3940	2236	4910
	Mean	2884	1762	2418	618	4377	3305	2348	1799	1596	1380	-	3523	2100	1541
	Std. Dev.	-	982	800	149	1709	2116	-	498	529	1032	-	401	87	938
	COV	-	56%	33%	24%	39%	64%	-	28%	33%	75%	-	11%	4%	61%
	Count	1	5	4	5	4	22	1	7	4	6	-	3	14	24
pH	Min.	8.2	8.8	9.0	9.3	8.2	8.7	7.4	9.1	7.2	8.7	7.1	8.2	7.8	8.8
	Max.	8.2	9.3	9.2	9.5	8.5	9.8	7.4	9.2	7.7	9.0	7.1	8.9	8.5	9.4
	Mean	8.2	9.1	9.1	9.4	8.4	9.4	7.4	9.2	7.6	8.8	7.1	8.5	8.3	9.1
	Std. Dev.	-	0.2	0.1	0.1	0.2	0.3	-	0.0	0.3	0.1	-	0.4	0.2	0.2
	COV	-	2%	1%	1%	2%	3%	-	0%	3%	1%	-	4%	2%	2%
	Count	1	5	4	5	4	22	1	7	4	6	1	3	14	24

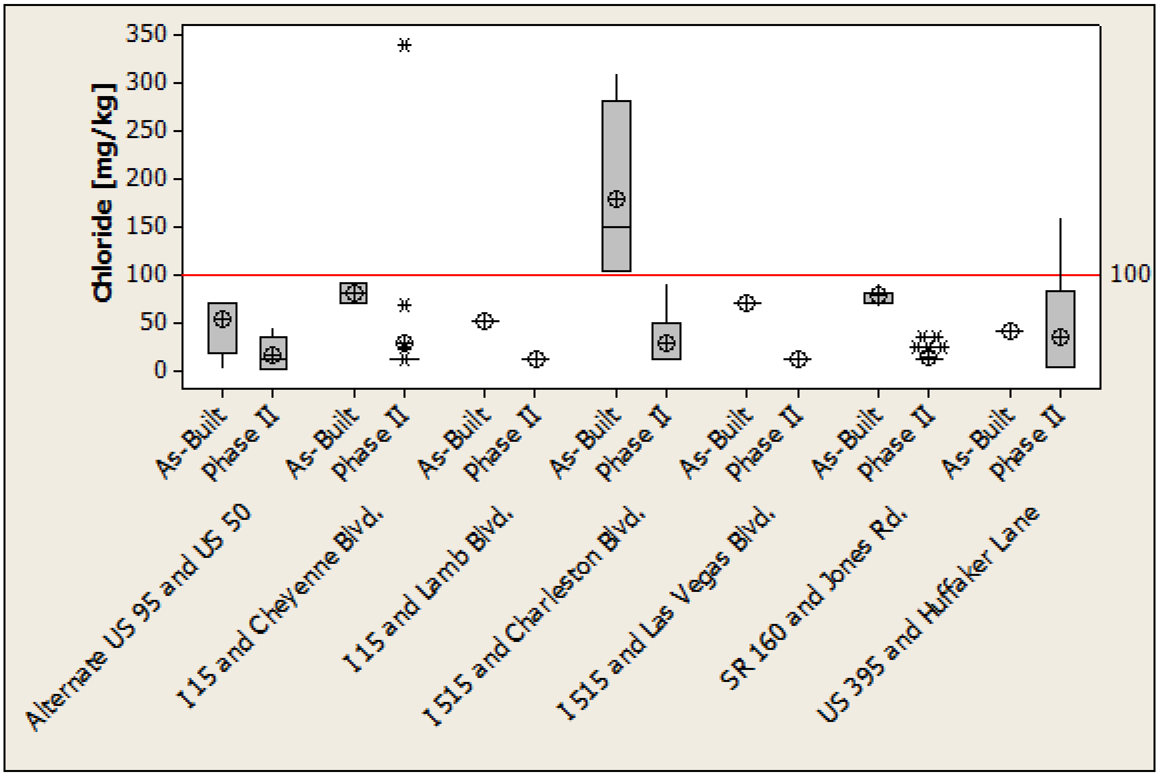

Figure 2-21: Comparison of As-built and Phase II soil testing results for chloride content, the majority of all data is below specification.

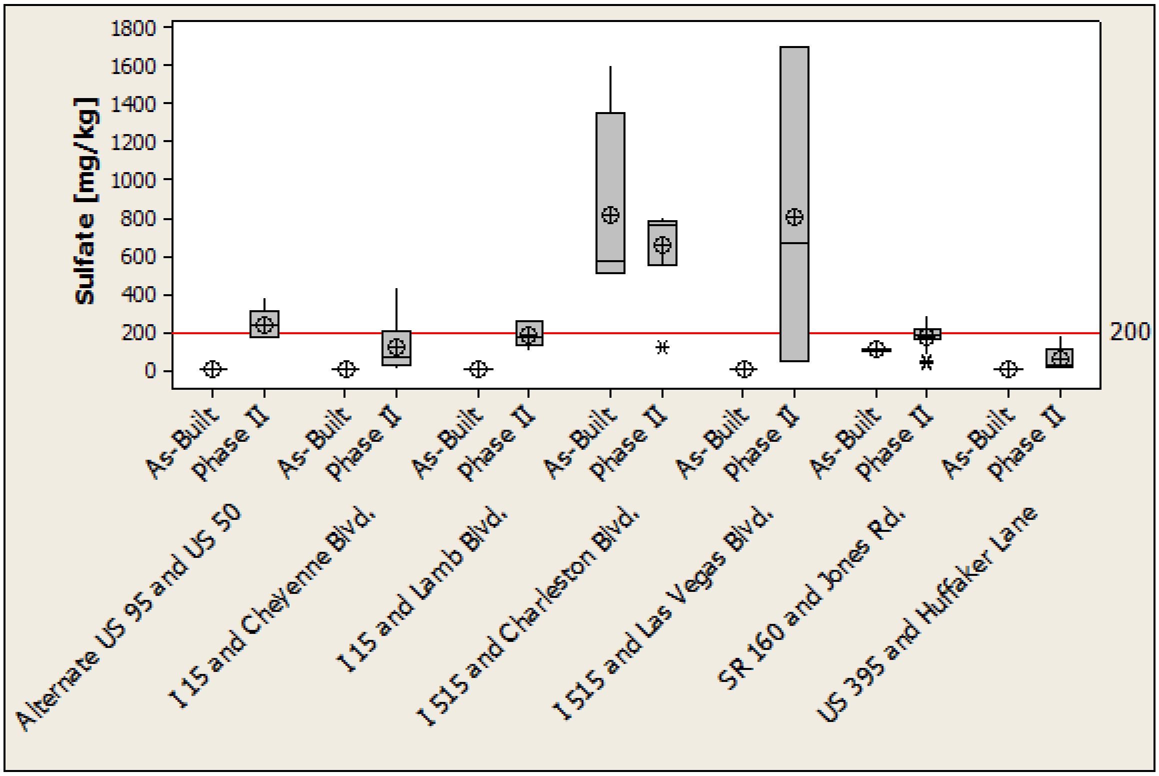

Figure 2-22: Comparison of As-built and Phase II soil testing results for sulfate content, the majority of all data is below specification, Charleston Blvd. fails to meet requirements for during either test period.

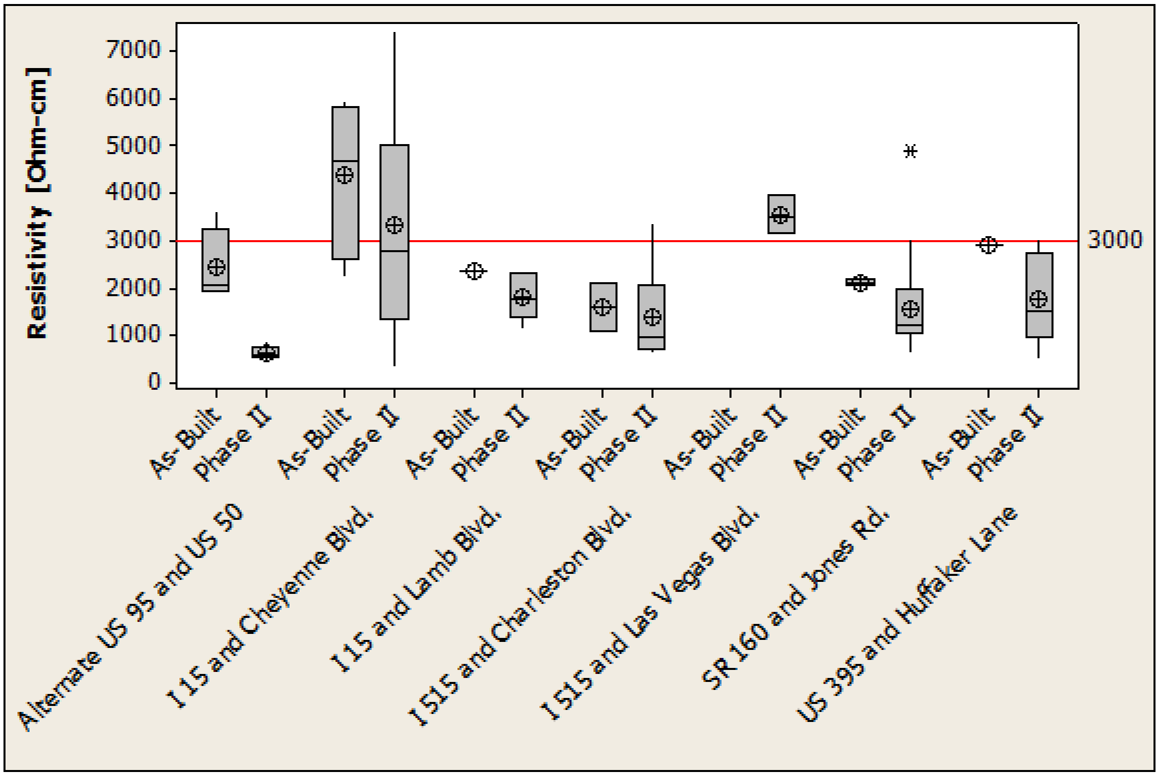

Figure 2-23: Comparison of As-built and Phase II soil testing results for minimum soil resistivity. The mean value is indicated with a circle and cross.

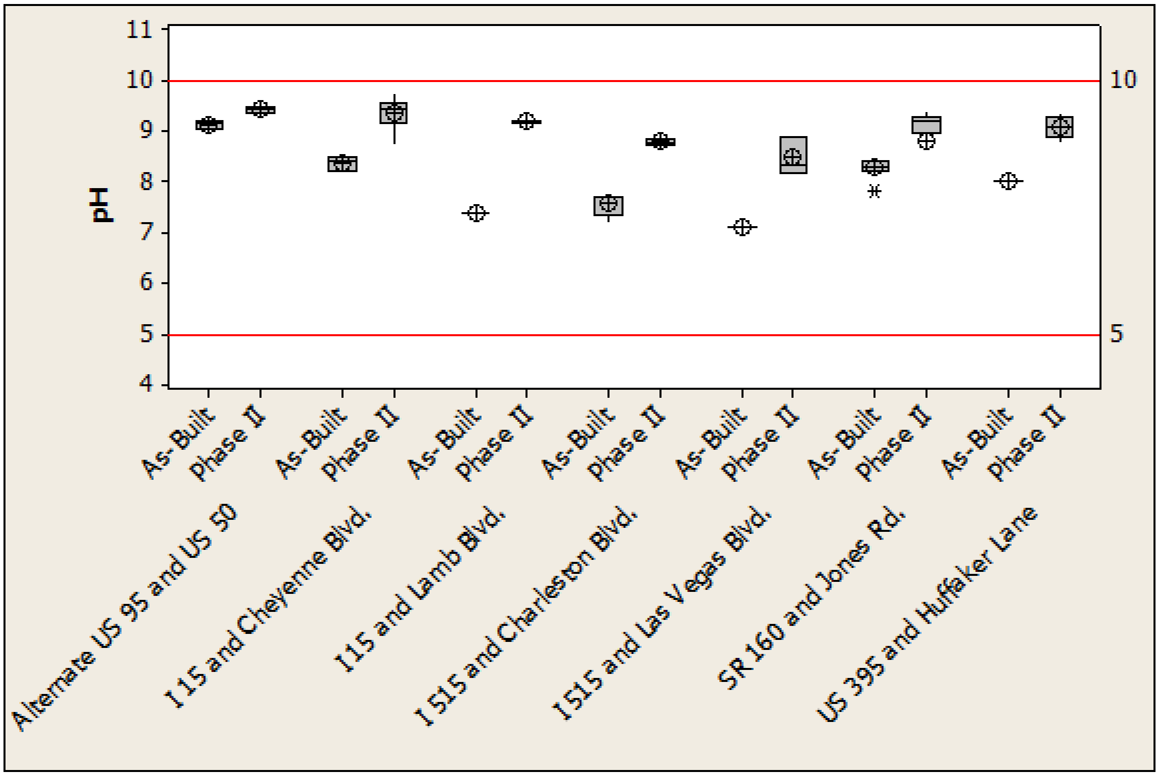

Figure 2-24: Comparison of As-built and Phase II soil testing results for pH, all data is within specification, and is narrowly banded as mildly alkaline.

Results for: Alternate US 95 and US 50

Mann-Whitney Test and CI: As-Built Resistivity, Phase II Resistivity

```
                       N  Median
As-Built Resistivity   4  2074.0
Phase II Resistivity   5   570.0


Point estimate for ETA1-ETA2 is 1477.5
96.3 Percent CI for ETA1-ETA2 is (1096.9,3034.0)
W = 30.0
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.0200
The test is significant at 0.0195 (adjusted for ties)
```

Figure 2-25: Mann-Whitney hypothiese testing reuslts, P-value = 0.0200 which is below the 0.05 significance level, Alt. US 95 and US 50

Results for: I 515 and Charleston Blvd.

Mann-Whitney Test and CI: As-Built Resistivity, Phase II Resistivity

```
                       N  Median
As-Built Resistivity   4  1591.0
Phase II Resistivity   6   970.0


Point estimate for ETA1-ETA2 is 379.5
95.7 Percent CI for ETA1-ETA2 is (-1434.9,1305.1)
W = 28.0
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.2410
```

Figure 2-26: Mann-Whitney hypothiese testing reuslts, P-value = 0.2410 which is above the 0.05 significance level, I-515/Charleston Blvd.

Results for: SR 160 and Jones Rd.

Mann-Whitney Test and CI: As-Built Resistivity, Phase II Resistivity

```
                        N  Median
As-Built Resistivity  14  2094.0
Phase II Resistivity  24  1200.0


Point estimate for ETA1-ETA2 is 885.0
95.3 Percent CI for ETA1-ETA2 is (509.0,1036.1)
W = 373.0
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.0026
The test is significant at 0.0026 (adjusted for ties)
```

Figure 2-27: Mann-Whitney hypothiese testing reuslts, P-value = 0.0026 which is below the 0.05 significance level, SR 160/ Jones Rd.

Table 2-24: Summary of Mann-Whitney hypotheses testing for statistical difference in populations.

Site	Data Sets	Significance level	P-Value	Null Hypothesis
Alternate US 95 and US 50	2004-2013	0.05	0.0200	*Reject*
I 515 and Charleston Blvd.	1982-2013	0.05	0.2410	Accept
SR 160 and Jones Rd.	2007-2013	0.05	0.0026	*Reject*

Table 2-25:Complete MSE wall backfill testing results for Cheyenne Blvd. including As-built, Phase I and Phase II results.

Date	Sample Location	Chloride [ppm]	Sulfate [ppm]	Resistivity[1] [Ω-cm]	pH	Organic Content [%]	Pass/Fail[2]
1998*	Chem Star Apex Pit Stockpile #3	70	0	*5525*	8.3	-	PASS
	Chem Star Apex Pit Stockpile #2	70	0	*5938*	8.2	-	PASS
	Frehner Sloan Pit Stockpile #1	90	0	*2206*	8.5	-	FAIL
	Frehner Sloan Pit Stockpile #3	90	0	*3839*	8.5	-	PASS
2008	12' From Face	90	70	1477	8.1	-	FAIL
	Near Top Face	30	48	3319	8.2	-	PASS
	Near Rusty Strip	210	126	604	8.0	-	FAIL
	Stockpile Sample	70	81	1805	8.2	-	FAIL
2013	Wall 1-StationA-B	68	220	1090	9.24	0.10	FAIL
	Wall 1-StationA-T	10	180	1830	9.32	0.11	FAIL
	Wall 1-StationA-EX	340	430	303	9.10	0.17	FAIL
	Wall 1-StationB-B	23	270	971	9.32	0.09	FAIL
	Wall 1-StationB-T	10	210	2460	9.38	0.10	FAIL
	Wall 1-StationC-B	11	200	971	9.02	0.18	FAIL
	Wall 1-StationC-T	10	190	1430	9.19	0.14	FAIL
	Wall 1-StationD-B	10	64	4630	8.71	0.08	PASS
	Wall 1-StationD-M	10	66	3140	9.22	0.11	PASS
	Wall 1-StationD-T	10	21	4170	9.64	0.10	PASS
	Wall 1-StationD-EX	10	190	2630	9.46	0.13	FAIL
	Wall 2-StationA-S	10	5	5540	9.52	0.10	PASS
	Wall 4-StationA-T	10	32	2860	9.56	0.11	FAIL
	Wall 4-StationA-B	10	30	2690	9.74	0.09	FAIL
	Wall 4-StationA-EX	10	270	570	9.55	0.06	FAIL
	Wall 4-StationB-T	10	15	4860	9.50	0.09	PASS
	Wall 4-StationB-B	10	29	2570	9.67	0.09	FAIL
	Wall 4-StationB-EX	10	110	3890	9.76	0.07	PASS
	Wall 5-StationA-S	10	13	5540	9.58	0.09	PASS
	Wall 5-StationA-EX	10	5	6860	9.53	0.13	PASS
	Wall 6-StationA-T	10	46	6280	8.96	0.10	PASS
	Wall 6-StationA-B	10	34	7430	8.98	0.08	PASS

[1] AASHTO T288

[2] Based on NDOT 2001 (Same as AASHTO 2012) | Indicates value failing specification

* Values converted using Thornley equation

Table 2-26: Summary of Mann-Whitney hypotheses testing for statistical difference in populations, for the three test periods at I-15 and Cheyenne Blvd.

Data Sets	Significance level	P-Value	Null Hypothesis
1998-2008	0.05	0.0606	Accept
1998-2013	0.05	0.4138	Accept
2008-2013	0.05	0.2410	Accept

Mann-Whitney Test and CI: As-Built Resistivity, Phase I Resistivity

```
                      N  Median
As-Built Resistivity  4  4682.0
Phase I Resistivity   4  1641.0


Point estimate for ETA1-ETA2 is 2490.5
97.0 Percent CI for ETA1-ETA2 is (-1112.9,5334.0)
W = 25.0
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.0606
```

Figure 2-28: Mann-Whitney hypothiese testing reuslts, P-value = 0.0606 which is above the 0.05 significance level, I-15/Cheyenne Blvd. As-built vs. Phase I

Mann-Whitney Test and CI: As-Built Resistivity, Phase II Resistivity

```
                        N  Median
As-Built Resistivity    4  4682.0
Phase II Resistivity   22  2775.0


Point estimate for ETA1-ETA2 is 1222.0
95.7 Percent CI for ETA1-ETA2 is (-1491.9,3477.9)
W = 66.0
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.4138
The test is significant at 0.4136 (adjusted for ties)
```

Figure 2-29: Mann-Whitney hypothiese testing reuslts, P-value = 0.4138 which is above the 0.05 significance level, I-15/Cheyenne Blvd. As-built vs. Phase II

Mann-Whitney Test and CI: Phase I Resistivity, Phase II Resistivity

```
                        N  Median
Phase I Resistivity     4  1641.0
Phase II Resistivity   22  2775.0


Point estimate for ETA1-ETA2 is -1219.5
95.7 Percent CI for ETA1-ETA2 is (-4026.1,715.0)
W = 37.0
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.2410
The test is significant at 0.2408 (adjusted for ties)
```

Figure 2-30: Mann-Whitney hypothiese testing reuslts, P-value = 0.2410 which is above the 0.05 significance level, I-15/Cheyenne Blvd. Phase I vs. Phase II

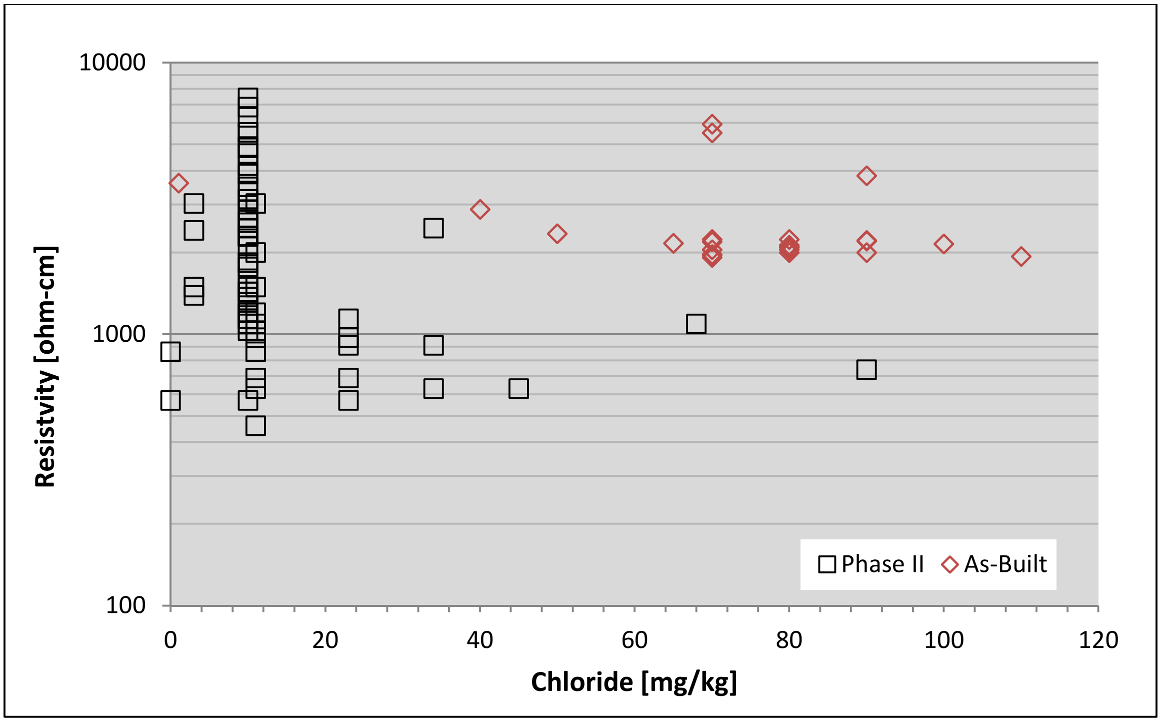

Figure 2-31: Relationship between chloride and resistiviy using the combined As-builta and Phase II data, no clear realationship exist.

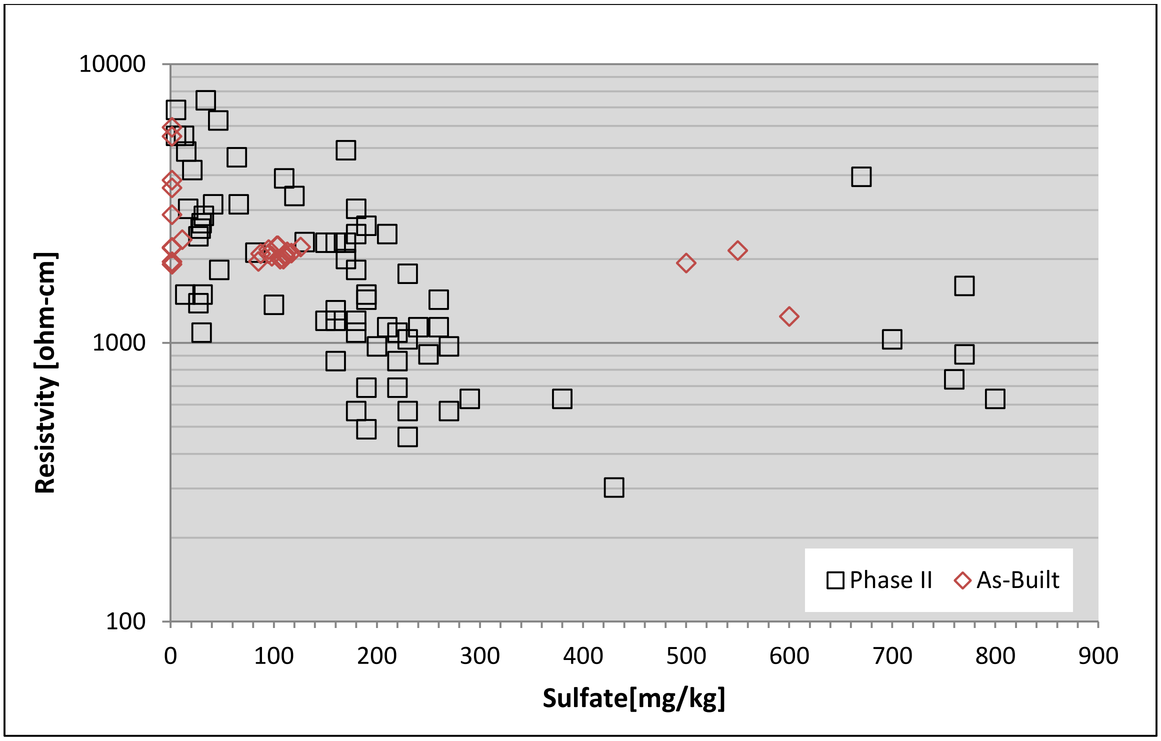

Figure 2-32: Relationship between sulfate and resistiviy using the combined As-builta and Phase II data, a weak realationship exist, that mimics the expected inversly porpotional realationsip.

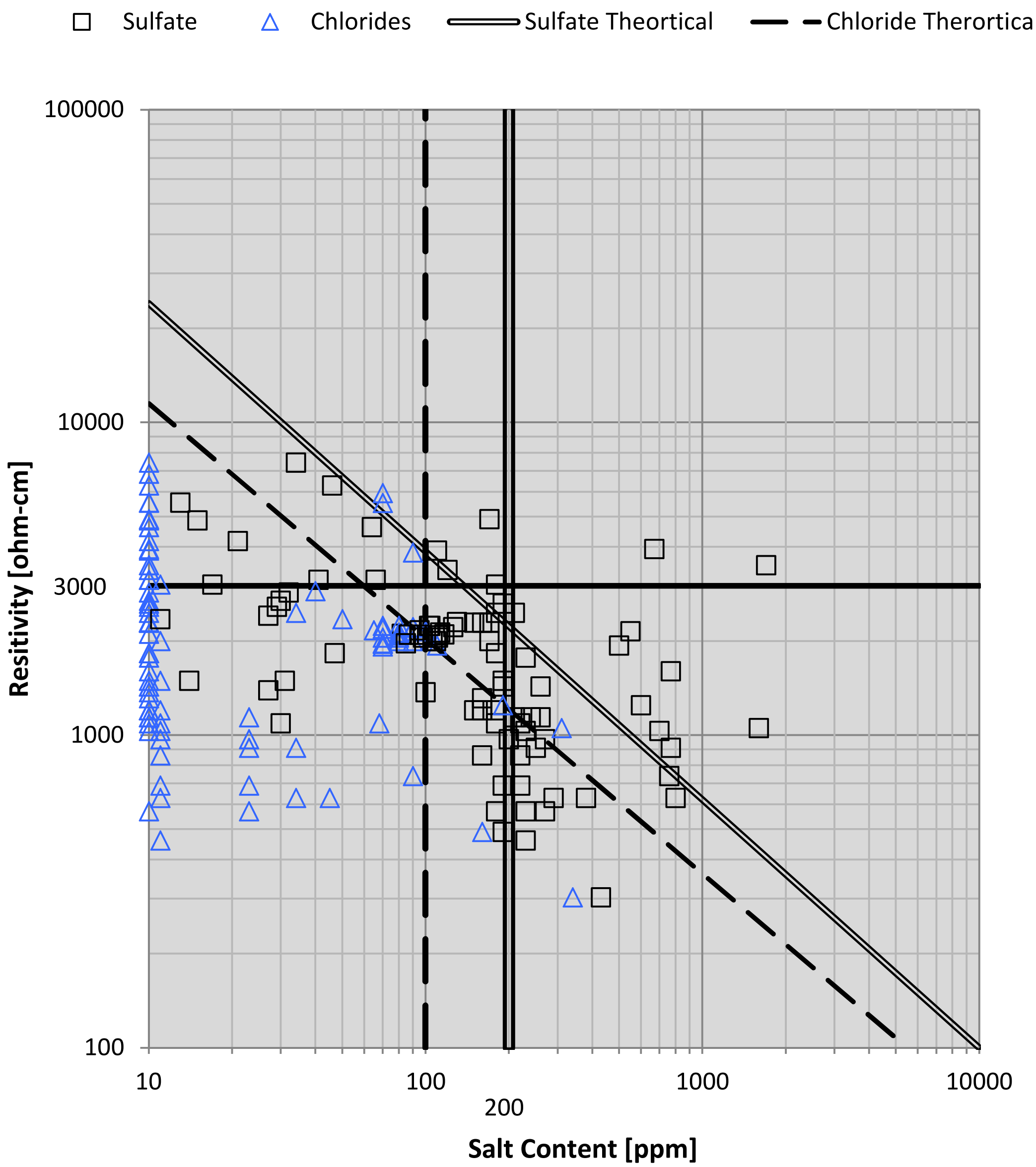

Figure 2-33: Phase II salt and resistivity data overlaid on the theoretical resistivity values based on salt content, it is seen that the general trend is for resistivity values measured in Nevada backfills that the resistivity value is below what is predicted using the NACE relationships (solid lines represent specification limits)

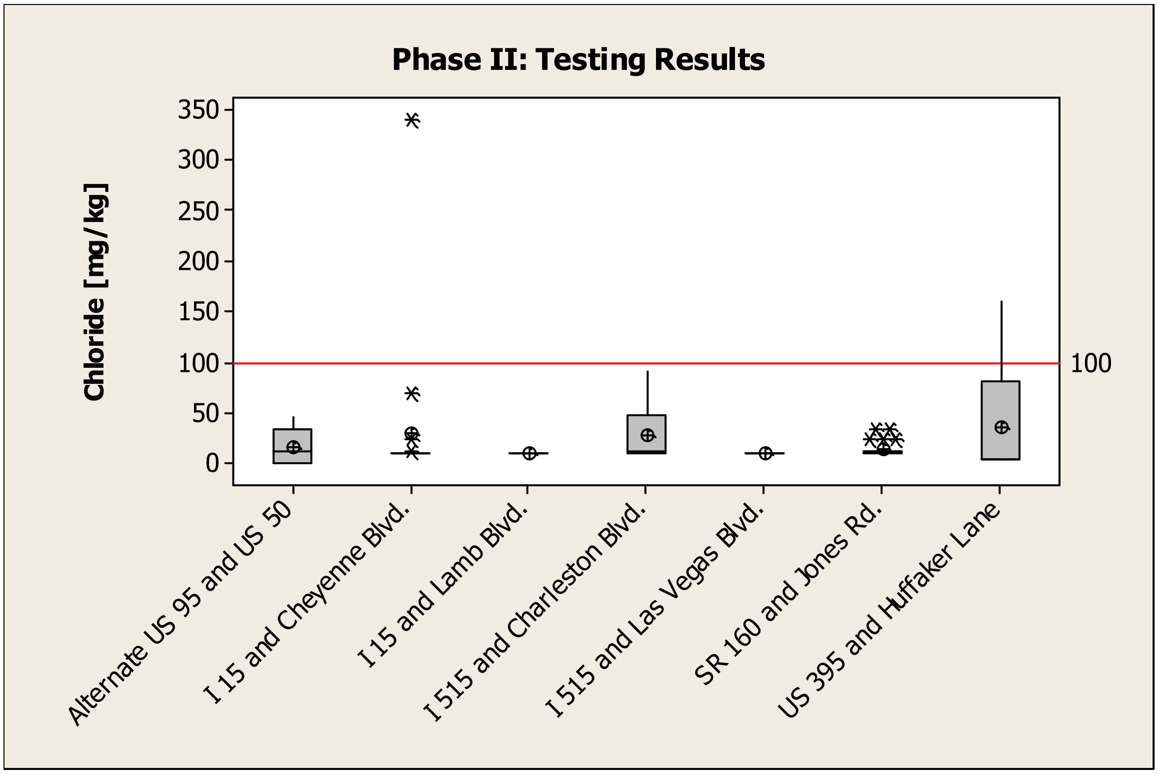

Figure 2-34: Site comparison of Phase II data, and severity rankings. Rankings as follow (most severe to least severe): Cheyenne Blvd, Huffaker Lane, Charleston Blvd., Alt. US 50, Las Vegas Blvd., Lamb Blvd. and Jones Rd

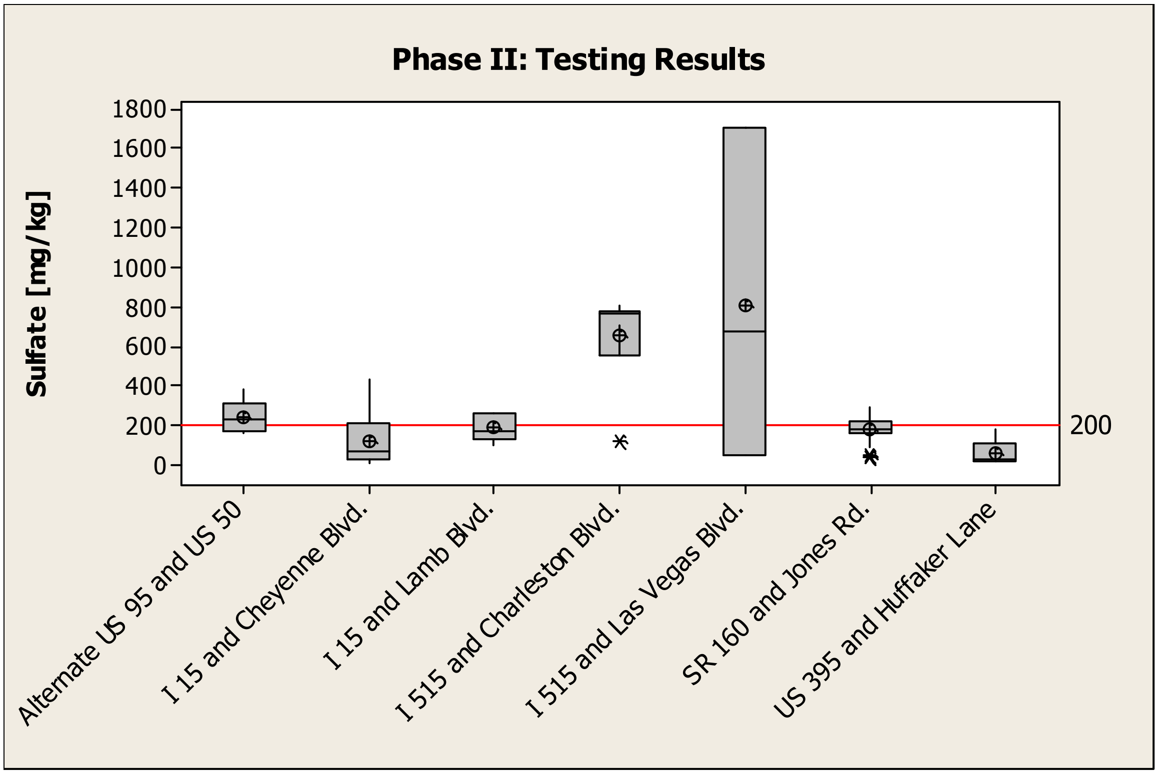

Figure 2-35: Site comparison of Phase II data, and severity rankings. Rankings as follow (most severe to least severe): Las Vegas Blvd., Charleston Blvd., Cheyenne Blvd., Alt. US 50, Lamb Blvd., Jones Rd., and Huffaker Lane

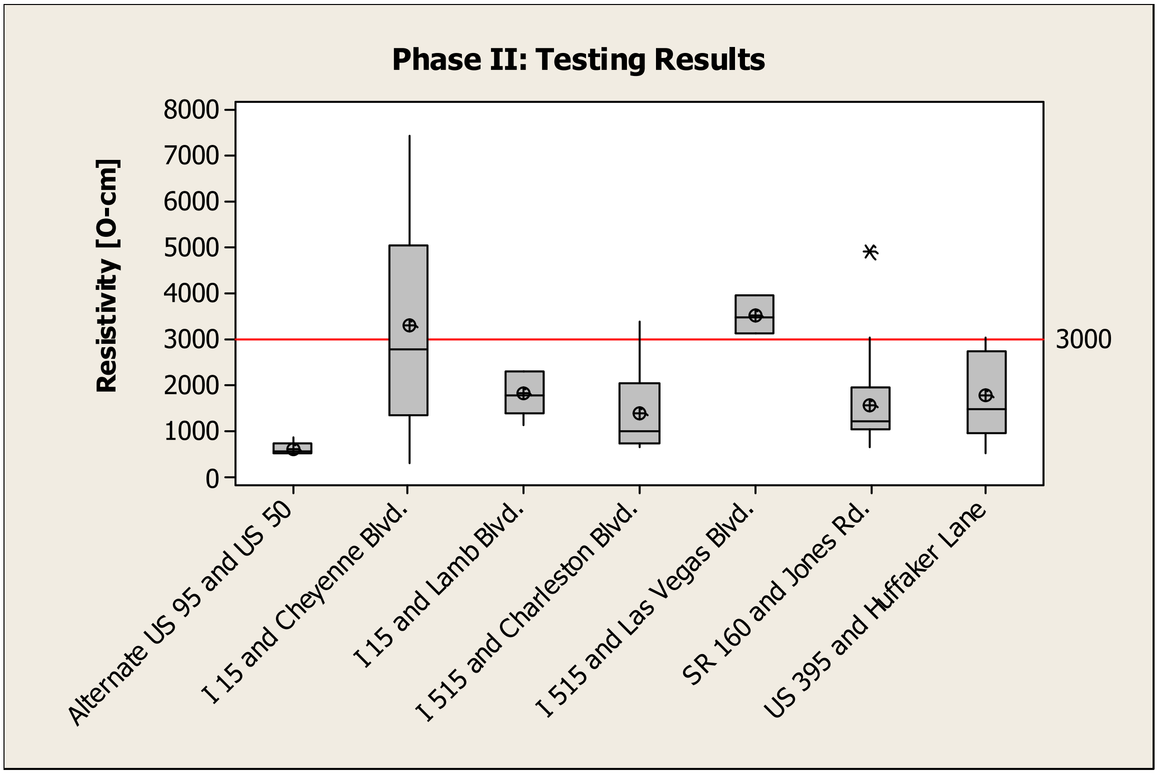

Figure 2-36: Site comparison of Phase II data, and severity rankings. Rankings as follow (most severe to least severe): US 50, Huffaker Lane, Cheyenne Blvd., Jones Rd., Lamb Blvd., Charleston Blvd., and Las Vegas Blvd.

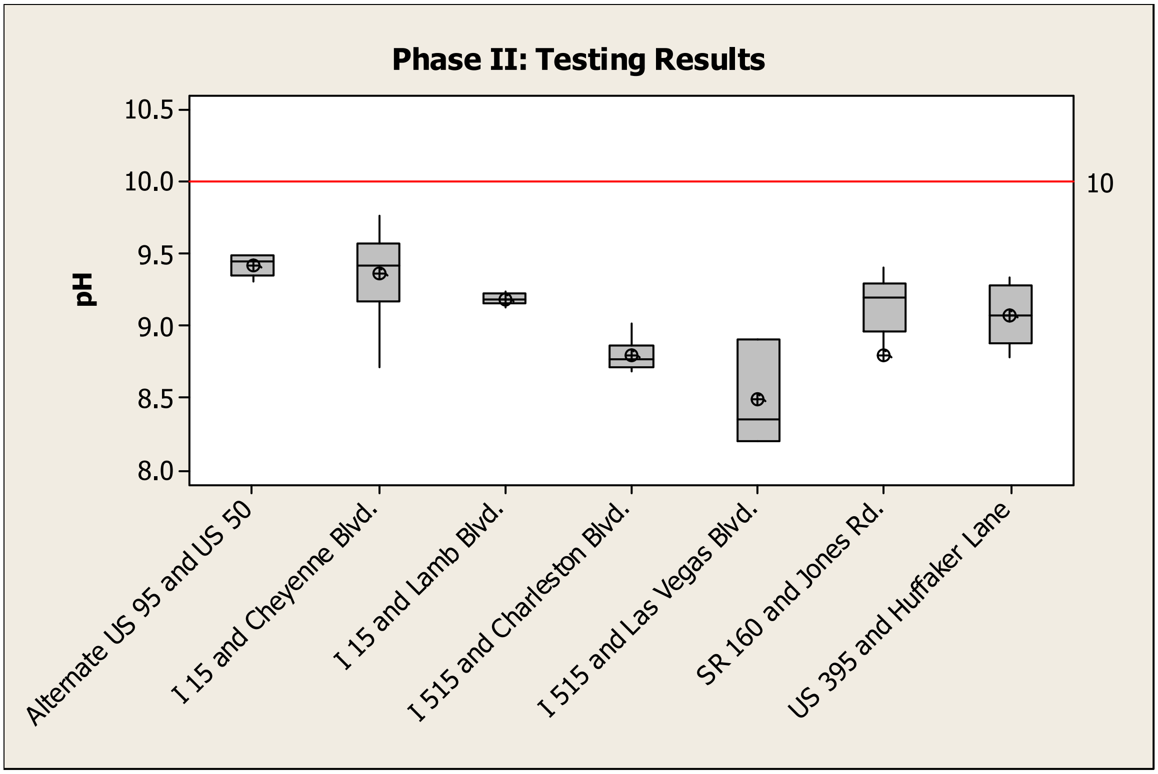

Figure 2-37: Site comparison of Phase II data, and severity rankings. Rankings as follow (most severe to least severe): Cheyenne Blvd., US 50, Huffaker Lane, Jones Rd., Las Vegas Blvd., Charleston Blvd., and Lamb Blvd

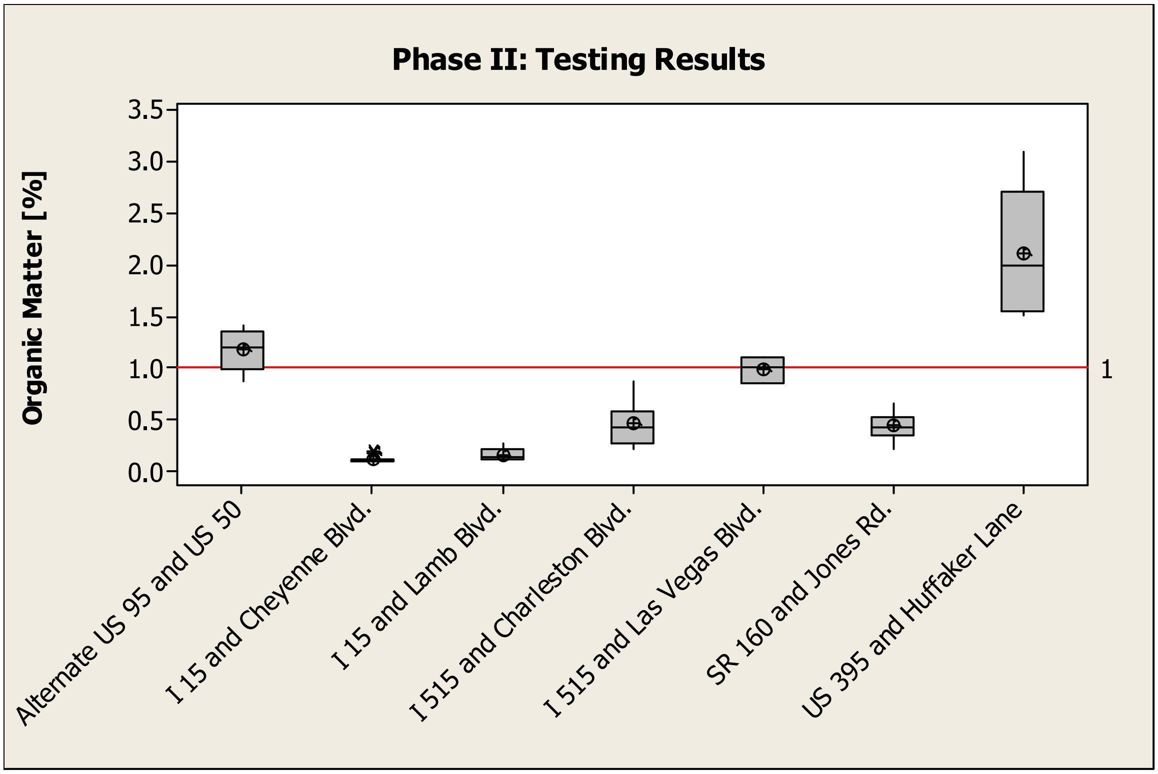

Figure 2-38: Site comparison of Phase II data, and severity rankings. Rankings as follow (most severe to least severe): Huffaker Lane, US 50, Las Vegas Blvd., Charleston Blvd., Jones Rd., Lamb Blvd., and Cheyenne Blvd.

Table 2-27: Summary of severity ratings for MSE wall backfill conditions, severity is indicated with increasing values.

	Sulfate	Chloride	Resistivity	pH	Organic Content	Severity Rating
US 395 and Huffaker Lane	1	6	6	5	7	25
Alternate US 95 and US 50	4	4	7	6	6	27
I 15 and Cheyenne Blvd.	5	7	5	7	1	25
I 15 and Lamb Blvd.	3	2	3	1	2	11
I 515 and Charleston Blvd.	6	5	2	2	4	19
I 515 and Las Vegas Blvd.	7	3	1	3	5	19
SR 160 and Jones Rd.	2	1	4	4	3	14

Table 0-1: Summarization of the metal loss model details that are used within Nevada's MSE wall inventory

AASHTO Galvanized Metal Loss Model*			
Corrosion Stage	Description	Time Period [years]	Corrosion Rate [µm/yr]
1	Zinc Consumption	0-2	15
2	Galvanized Passivation	2-16*	4
3	Base Steel Consumption	16+	12
Plain Steel Metal Loss Model			
1	Aggressive equalization	0-2	45
2	Base Steel Consumption	2+	12

* Assumes initial zinc thickness of 86µm, ASTM standards

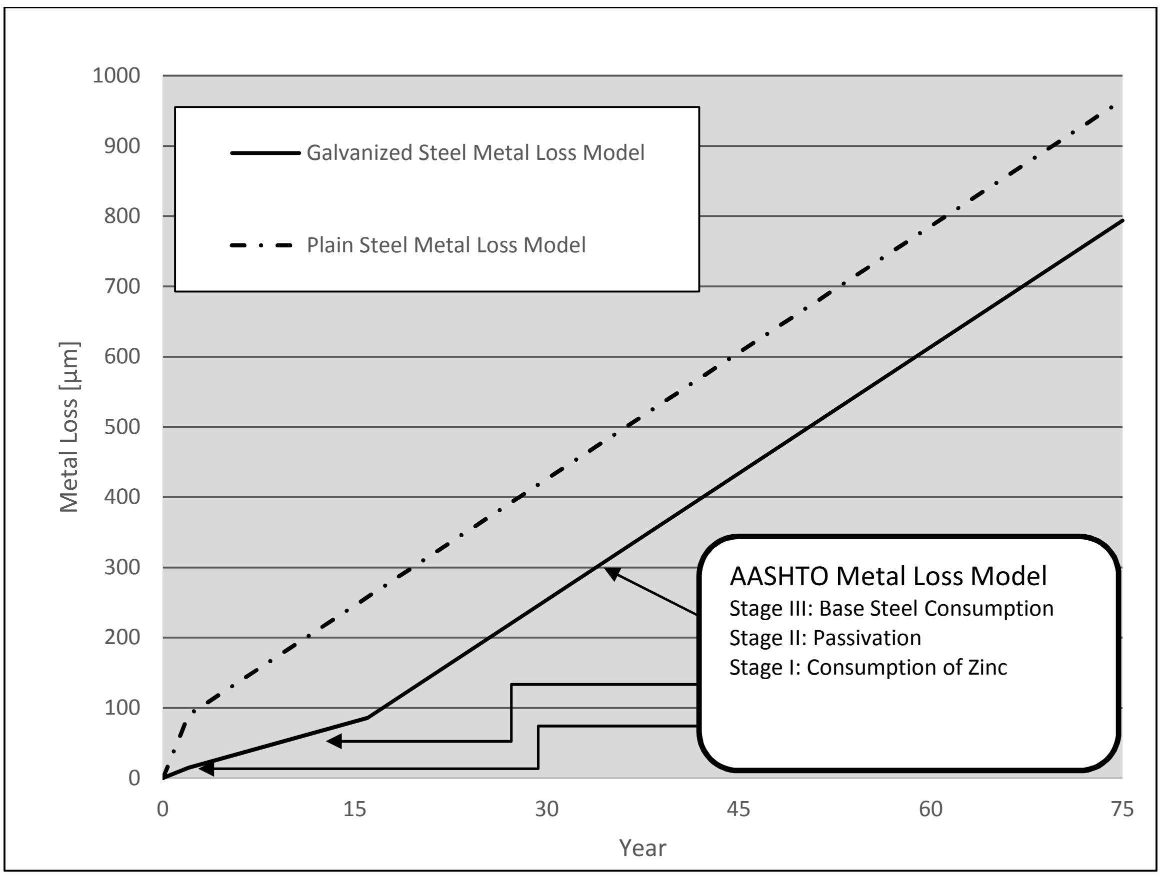

Figure 0-1: Nation metal loss models used for the design of sacrificial reinforcement thickness to compensate for metal loss due to corrosion

Table 0-2: MMCE LPR corrosion rates and soil characteristics (2004) (I-515/Flamingo Rd.)

Station	Site	Layer	E_{corr} [mV] March/ August	PR [Ω] March/ August	A_s [in^2]	CR [μm/yr.] March /August	pH[1]	Resisitvity[1] [Ω-cm]	Chloride [1] [ppm]	Sulfate[1] [ppm]	Organic Content[1] [%]
Wall #1											
1	12	II	-330	9.6	3354	1.94	9.37	450	15	910	2.30
1	12	III	-449/-370	4.0/7.7	3354	4.63/2.42	9.37	450	15	910	2.30
1	11	VIII	-465/-420	3.2/2.6	3354	5.84/7.18	9.50	450	19	3700	2.30
1	11	IX	-440	2.6	3354	7.18	9.50	450	19	3700	2.30
1	1	XIII	-428/-390	2.3/2.3	3907	7.09/6.97	8.86	450	15	1400	2.30
1	1	XIV	-370	2.1	4391	6.79	8.86	450	15	1400	2.30
2	TP2	II	-580/-440	4.5/3.8	3633	3.83/4.54	8.98	5200	15	15	5.20
2	TP2	III	-410	7.7	3633	2.24	8.98	5200	15	15	5.20
2	13	VI	-569/-510	2.0/1.4	3633	8.75/12.31	9.01	5200	15	7500	5.20
2	13	VII	-510	3.2	3633	5.39	9.04	5200	15	7500	5.20
2	2	XIII	-565/-500	2.7/1.8	4233	5.48/8.22	8.06	5200	15	430	5.20
2	2	XIV	-520	1.0	4757	13.17	8.06	5200	15	430	5.20
3	TP3	II	-453/-310	2.1/2.0	3074	9.56/10.19	8.14	420	15	380	1.30
3	TP3	III	-310	5.1	3074	3.99	8.14	420	15	380	1.30
3	14	V	-441/-340	2.4/4.7	3074	8.59/4.33	9.08	420	100	2900	1.30
3	14	VI	-340	1.4	3074	14.55	9.08	420	25	2900	1.30
3	3	X	-456/-360	2.0/1.6	3582	8.61/10.93	8.62	410	20	300	1.30
3	3	XI	-390	3.3	3582	5.3	8.62	410	20	300	1.30
4	TP4	II	-436/-420	2.6/6.5	2515	9.47/3.83	8.28	1247	15	390	
4	TP4	III	-410	16.4	2515	1.52	8.28	1247	15	390	
4	4	VIII	-424/-440	1.5/3.2	2515	16.60/7.78	8.23	1018	15	390	
4	4	IX	-430	33.0	2515	0.75	8.23	1018	15	390	
5	TP5	II	-130	5.9	1956	5.43	8.12	420	78	4600	
5	TP5	III	-300/-130	0.4/4.1	1956	76.22/7.81	8.12	420	78	4600	
5	5	V	-315/-143	8.1/8.1	1956	3.94/3.95	8.48	420	15	470	
5	5	VI	-153	5.6	1956	5.72	8.48	420	15	470	
6	TP6	II	-463/-640	2.8/11.6	1677	13.20/3.22	8.40	1307	15	160	
6	TP6	III	-650	4.7/33.8	1677	7.95/1.11	8.40	1307	15	160	
6	S6	III	-400/-610	4.5/14.5	1677	8.25/2.58	8.40	1234	15	160	
6	S6	IV	-620	6.7	1677	5.57	8.40	1234	15	160	
Wall #2											
8	15	II	-447/-280	1.63/3.7	2795	13.75/6.06	9.38	7800	15	240	
8	15	III	-430/-266	2.8/3.8	2795	8.00/5.90	9.38	7800	15	240	
8	8	X	-413/-290	3.6/2.3	3256	5.39/8.36	9.38	7800	15	240	
9	16	II	-449/-390	1.4/2.9	2515	17.79/8.59	9.14	7800	15	3000	6.40
9	16	III	-448/-390	2.1/1.6	2515	12.03/15.56	9.14	7800	15	3000	6.40
9	9	VIII	-433/-400	1.6/0.48	2930	13.61/44.53	9.50	7800	70	93	6.40
10	17	II	-407/-360	2.9/4.5	1956	11.04/7.11	8.46	7800	230	6900	
10	17	III	-490/-380	3.9/4.1	1956	8.21/7.81	8.46	7800	230	6900	
10	10	VI	-480/-420	3.9/1.5	1956	8.21/21.24	8.46	7800	230	6900	
Wall #3											
11	18	I	-317	2.9	1677	12.88					
11	18	II	-309	40.3	1677	0.93					
11	18	III	-324	5.7	1677	6.55					
12	19	II	-154	5.2	1677	7.18					
12	19	III	-142	~	1677	~					
12	19	IV	-163	5.5	1677	6.79					

[1] Mix of different test methods

\# Measurement in excess of metal loss model or failing electrochemical specifications

Table 0-3:2004 MMCE LPR data set used for evaluation with Phase II LPR testing results, combined data from the March and August LPR measurements (I-515/Flamingo Rd.)

Station	Site	Layer	E_{corr} [mV]	PR [Ω]	A_s [in^2]	r^1 [μm/yr.]	r^2 [μm/yr.]
Wall #1							
1	12	II	-330	9.6	3354	1.94	1.44
1	12	III	-449	4.0	3354	4.63	3.44
1	12	III	-370	7.7	3354	2.42	1.80
1	11	VIII	-465	3.2	3354	5.84	4.34
1	11	VIII	-420	2.6	3354	7.18	5.33
1	11	IX	-440	2.6	3354	7.18	5.33
1	11	IX	-428	2.3	3354	7.09	5.27
1	1	XIII	-390	2.3	3907	6.97	5.18
1	1	XIV	-370	2.1	4391	6.79	5.04
2	TP2	II	-580	4.5	3633	3.83	2.85
2	TP2	II	-440	3.8	3633	4.54	3.37
2	TP2	III	-410	7.7	3633	2.24	1.66
2	13	VI	-5.69	2.0	3633	8.75	6.50
2	13	VI	-510	1.4	3633	12.31	9.14
2	13	VII	-510	3.2	3633	5.39	4.00
2	2	XIII	-565	2.7	4233	5.48	4.07
2	2	XIII	-500	1.8	4233	8.22	6.11
2	2	XIV	-520	1.0	4757	13.17	9.78
3	TP3	II	-453	2.1	3074	9.56	7.10
e	TP3	II	-310	2.0	3074	10.19	7.57
3	TP3	III	-310	5.1	3074	3.99	2.96
3	14	V	-441	2.4	3074	8.59	6.38
3	14	V	-340	4.7	3074	4.33	3.22
3	14	VI	-340	1.4	3074	14.55	10.81
3	3	X	-456	2.0	3582	8.61	6.40
3	3	X	-360	1.6	3582	10.93	8.12
3	3	XI	-390	3.3	3582	5.30	3.94
4	TP4	II	-436	2.6	2515	9.47	7.03
4	TP4	II	-420	6.5	2515	3.83	2.85
4	TP4	III	-410	16.4	2515	1.52	1.13
4	4	VIII	-424	1.5	2515	16.60	12.33
4	4	VIII	-440	3.2	2515	7.78	5.78
4	4	IX	-430	33.0	2515	0.75	0.56
5	TP5	II	-130	5.9	1956	5.43	4.03
5	TP5	III	-300	0.4	1956	76.22	56.62
5	TP5	III	-130	4.1	1956	7.81	5.80
5	5	V	-315	8.1	1956	3.94	2.93
5	5	V	-143	8.1	1956	3.95	2.93
5	5	VI	-153	5.6	1956	5.72	4.25
6	TP6	II	-463	2.8	1677	13.20	9.81

6	TP6	II	-640	11.6	1677	3.22	2.39
6	TP6	III	-650	4.7	1677	7.95	5.91
6	TP6	III	-650	33.8	1677	1.11	0.82
6	S6	III	-400	4.5	1677	8.25	6.13
6	S6	III	-610	14.5	1677	2.58	1.92
6	S6	IV	-620	6.7	1677	5.57	4.14
Wall #2							
8	15	II	-447	1.63	2795	13.75	10.21
8	15	II	-280	3.7	2795	6.06	4.50
8	15	III	-430	2.8	2795	8	5.94
8	15	III	-266	3.8	2795	5.9	4.38
8	8	X	-413	3.6	3256	5.39	4.00
8	8	X	-290	2.3	3256	8.36	6.21
9	16	II	-449	1.4	2515	17.79	13.22
9	16	II	-390	2.9	2515	8.59	6.38
9	16	III	-448	2.1	2515	12.03	8.94
9	16	III	-390	1.6	2515	15.56	11.56
9	9	VIII	-433	1.6	2930	13.61	10.11
9	9	VIII	-400	0.48	2930	44.53	33.08
10	17	II	-407	2.9	1956	11.04	8.20
10	17	II	-360	4.5	1956	7.11	5.28
10	17	III	-490	3.9	1956	8.21	6.10
10	17	III	-380	4.1	1956	7.81	5.80
10	10	VI	-480	3.9	1956	8.21	6.10
10	10	VI	-420	1.5	1956	21.24	15.78
Wall #3							
11	18	I	-317	2.9	1677	12.88	9.57
11	18	II	-309	40.3	1677	0.93	0.69
11	18	III	-324	5.7	1677	6.55	4.87
12	19	II	-154	5.2	1677	7.18	5.33
12	19	III	-142	~	1677	~	~
12	19	IV	-163	5.5	1677	6.79	5.04

[1] Original Reported Values
[2] Corrected values for different environmental constant

In excess of design model

Table 0-4: Descriptive statistics of the expanded and modified MMCE LPR corrosion rates (I-515/Flamingo Rd.)

	E_{corr} [mV]	PR [Ω]	CR [µm/yr]
Mean	-400	5.4	6.81
Median	-417	3.2	5.33
Standard Deviation	123	7.2	7.58
Coefficient of Variation	29%	224%	142%
Minimum	-650	0.4	0.56
Maximum	-130	40.3	56.62
Confidence Level (95%)	29	2	1.79
Confidence Range for Mean	(-429 to -371)	(3.7 to 7.1)	(5.02 to 8.60)
Count	70	69	69

Table 0-5: Phase II LPR Data for I-515/Flamingo Rd. includes all element types (I-515/Flamingo Rd.)

Station	Site	WE	E_{corr} [mV]	PR [Ω]	A_s [in^2]	CR [μm/yr]
Wall #1						
1	12	GC	-634	1486.56	13	3.26
1	12	IV	-338	5.87	3354	2.38
1	12	SC	-436	533.64	13	6.75
1	12	III	-322	3.61	3654	3.54
1	11	GC	-676	1455.86	13	3.33
1	11	IX	-397	4	3354	3.49
1	11	SC	-384	679.25	13	5.3
1	11	VIII	-391	3.79	3354	3.69
1	1	SC1	-359	191.66	13	18.79
1	1	SC2	-386	260.57	13	13.82
1	1	XIII	-354	2.76	3907	4.33
2	2	GC	-660	1602.49	13	3.03
2	2	II	-341	6.91	3633	1.87
2	2	III	-388	4.37	3633	2.95
2	2	SC	-303	1878.23	13	1.92
2	2	XIII	-401	2.05	4233	5.41
2	2	XIV	-413	1.92	4233	5.75
3	14	GC	-736	1832.95	13	2.65
3	14	SC	-256	1716.96	13	2.1
3	14	V	-307	2.86	3074	5.32
3	14	V	-312	2.92	3074	5.22
3	3	III	-291	5.48	3074	2.78
3	3	SC1	-350	1040.93	13	3.46
3	3	SC2	-343	703.35	13	5.12
3	3	X	-432	574883.06	3582	0
3	3	XI	-342	3.1	3582	4.22
4	4	II	-333	3.38	2515	5.5
4	4	III	-327	2.94	2515	6.33
5	5	SC1	-338	435.52	13	8.27
5	5	SC2	-348	588.95	13	6.12
5	5	VI	-371	12397.5	1956	0
6	6	II	-298	3.47	1677	8.05
6	6	III	-317	6.18	1677	4.51
6	6	III	-296	5.49	1677	5.09
6	6	IV	-306	5.72	1677	4.88
6	6	SC1	-309	263.18	13	13.69

6	6	SC2	-301	226.56	13	15.9
Wall #2						
8	15	GC	-763	6522.83	13	0.74
8	15	II	-190	2.55	2795	6.58
8	15	III	-194	2.41	2795	6.94
8	15	SC	-262	2390.62	13	1.51
8	8	S1	-199	8777.77	13	0.41
8	8	S1	-216	9603.74	13	0.38
8	8	X	-248	470	3256	0.03
9	16	GC	-558	10689.94	13	0.45
9	16	II	-228	2.78	2515	6.7
9	16	S1	-221	9204.6	13	0.39
9	16	S2	-105	16911.55	13	0.21
9	16	SC	-239	2160.49	13	1.67
9	9	VIII	-267	0.27	2930	60.17
10	17	GC	-408	120806.8	13	0.04
10	17	II	-174	88	1956	0.27
10	17	II	-168	109.05	1956	0.22
10	17	SC	-122	58466.47	13	0.06
10	17	VI	-431	376442.81	1956	0
10	10	S2	-175	12381.53	13	0.29
10	10	SC1	-214	393321.81	13	0.01
10	10	SC2	-177	40187.89	13	0.09
Wall #3						
11	18	A	-225	2.64	1677	10.56
11	18	B	-228	4.58	1677	6.1
11	18	GC	-748	1889.96	13	2.57
11	18	IV	-232	5.19	1677	5.38
11	18	SC	-310	500.1	13	7.2
12	19	GC	-875	2248.38	13	2.16
12	19	I	-353	2.31	1677	12.09
12	19	II	-333	5.54	1677	5.04
12	19	III	-344	5.08	1677	5.5
12	19	SC	-390	859.91	13	4.19

Table 0-6: 2004 MMCE LPR data set used for evaluation with Phase II LPR testing results, combined data from the March and August LPR measurements (I-515/Flamingo Rd.)

Station	Site	Layer	E_{corr} [mV]	PR [Ω]	A_s [in^2]	CR [μm/yr]	pH[1]	Resisitvity[1] [Ω-cm]	Chloride[1] [ppm]	Sulfate[1] [ppm]	Organic Content[1] [%]
Wall #1											
1	12	II	-338	5.87	3354	2.38	9.37	450	15	910	2.3
1	12	III	-322	3.61	3654	3.54	9.37	450	15	910	2.3
1	11	IX	-397	4.00	3354	3.49	9.5	450	19	370	2.3
1	11	VIII	-391	3.79	3354	3.69	9.5	450	19	370	2.3
1	1	XIII	-354	2.76	3907	4.33	8.86	450	15	1400	2.3
2	*TP2*	II	-341	6.91	3633	1.87	8.98	5200	15	15	5.2
2	*TP2*	III	-388	4.37	3633	2.95	8.98	5200	15	15	5.2
2	2	XIII	-401	2.05	4233	5.41	8.06	5200	15	430	5.2
2	2	XIV	-413	1.92	4757	5.12	8.06	5200	15	430	5.2
3	14	V	-307	2.86	3074	5.32	9.08	420	100	2900	1.3
3	14	*VI*	-312	2.92	3074	5.22	9.08	420	25	2900	1.3
3	*TP3*	III	-291	5.48	3074	2.78	8.14	420	15	380	1.3
3	3	X	-432	574883	3582	0.00	8.62	410	20	300	1.3
3	3	XI	-342	3.10	3582	4.22	8.68	410	20	300	1.3
4	*TP4*	II	-333	3.38	2515	5.50	8.28	1247	15	390	
4	*TP4*	III	-327	2.94	2515	6.33	8.28	1247	15	390	
5	*5*	VI	-371	12397	1956	0.00	8.48	420	15	470	
6	*TP6*	II	-298	3.47	1677	8.05	8.4	1307	15	160	
6	*TP6*	III	-317	6.18	1677	4.51	8.4	1307	15	160	
6	6	III	-296	5.49	1677	5.09	8.4	1234	15	160	
6	6	IV	-306	5.72	1677	4.88	8.4	1234	15	160	
Wall #2											

8	15	II	-190	2.55	2795	6.58	9.38	7800	15	240	
8	15	III	-194	2.41	2795	6.94	9.38	7800	15	240	
8	8	X	-248	470.00	3256	0.03	9.38	7800	15	240	
9	16	II	-228	2.78	2515	6.70	9.14	7800	15	3000	
9	9	VIII	-267	0.27	2930	60.17	9.5	7800	70	93	
10	17	II	-174	88.00	1956	0.27	846	7800	230	6900	
10	17	*III*	-168	109.05	1956	0.22	846	7800	230	6900	
10	*10*	VI	-431	376443	1956	0.00	846	7800	230	6900	
Wall #3											
12	*19*	A	-225	2.64	1677	10.56					
12	*19*	B	-228	4.58	1677	6.10					
12	*19*	IV	-232	5.19	1677	5.38					
11	*18*	I	-353	2.31	1677	12.09					
11	*18*	II	-333	5.54	1677	5.04					
11	*18*	III	-344	5.08	1677	5.50					

\# Measurement in excess of metal loss model or failing electrochemical specifications

[1] Mix of different testing methods

Assumed based on data available

Table 0-7: Phase II LPR corrosion rates for in-service reinforcements (I-515/Flamingo Rd.)

Station	Site	WE	E_{corr} [mV]	PR [Ω]	A_s [in^2]	CR [μm/yr]
Wall #1						
1	12	II	-338	5.87	3354	2.38
1	12	III	-322	3.61	3654	3.54
1	11	IX	-397	4	3354	3.49
1	11	VIII	-391	3.79	3354	3.69
1	1	XIII	-354	2.76	3907	4.33
2	*TP2*	II	-341	6.91	3633	1.87
2	*TP2*	III	-388	4.37	3633	2.95
2	2	XIII	-401	2.05	4233	5.41
2	2	XIV	-413	1.92	4757	5.12
3	14	V	-307	2.86	3074	5.32
3	14	*VI*	-312	2.92	3074	5.22
3	*TP3*	III	-291	5.48	3074	2.78
3	3	X	-432	574883	3582	0
3	3	XI	-342	3.1	3582	4.22
4	*TP4*	II	-333	3.38	2515	5.5
4	*TP4*	III	-327	2.94	2515	6.33
5	*5*	VI	-371	12398	1956	0
6	*TP6*	II	-298	3.47	1677	8.05
6	*TP6*	III	-317	6.18	1677	4.51
6	6	III	-296	5.49	1677	5.09
6	6	IV	-306	5.72	1677	4.88
Wall #2						
8	15	II	-190	2.55	2795	6.58
8	15	III	-194	2.41	2795	6.94
8	8	X	-248	470	3256	0.03
9	16	II	-228	2.78	2515	6.7
9	9	VIII	-267	0.27	2930	60.17
10	17	II	-174	88	1956	0.27
10	17	*III*	-168	109.05	1956	0.22
10	*10*	VI	-431	376443	1956	0
Wall #3						
12	*19*	A	-225	2.64	1677	10.56
12	*19*	B	-228	4.58	1677	6.1
12	*19*	IV	-232	5.19	1677	5.38
11	*18*	I	-353	2.31	1677	12.09
11	*18*	II	-333	5.54	1677	5.04
11	*18*	III	-344	5.08	1677	5.5

Table 0-8: Descriptive statistics for Phase II LPR corrosion rates obtained from in-service reinforcements (I-515/Flamingo Rd.)

Phase II: LPR Corrosion Rates: In-Service	E_{corr} [mV]	CR µm/yr]
Mean	-311	6.03
Median	-322	5.04
Standard Deviation	73	9.84
Coefficient of Variation	23%	195%
Minimum	-432	0
Maximum	-168	60.17
Confidence Level (95%)	24	3.26
Confidence Range for Mean	(-335 to -287)	(2.77 to 9.28)
Count	35	35

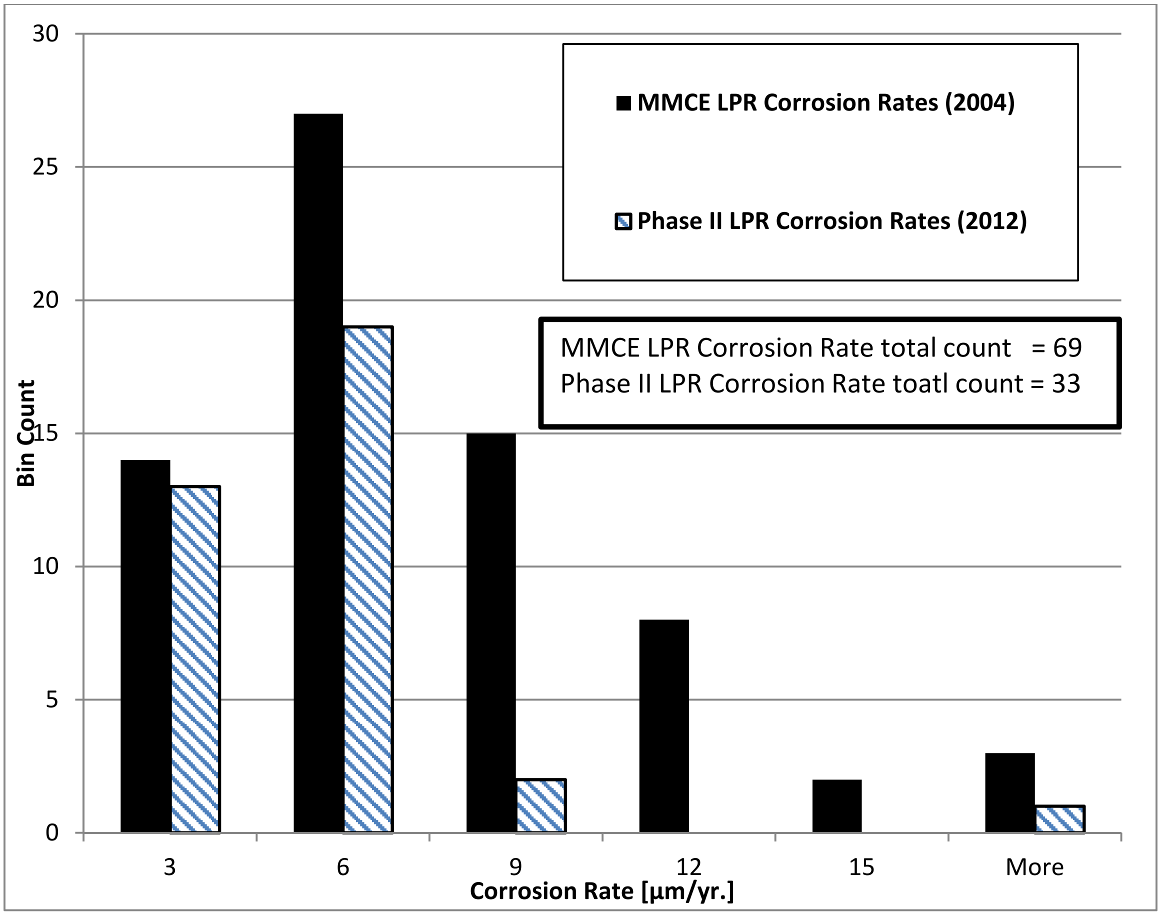

Figure 0-2: Histogram of LPR corrosion rates for both the MMCE and Phase II in-service reinforcements (I-515/Flamingo Rd.)

Table 0-9: Phase II LPR corrosion rates for sacrificial coupons (I-515/Flamingo Rd.)

Station	Site	WE	E_{corr} [mV]	PR [Ω]	A_s [in^2]	CR [μm/yr]
Wall #1						
1	12	GC	-634	1486.56	13	3.26
1	12	SC	-436	533.64	13	6.75
1	11	GC	-676	1455.86	13	3.33
2	2	GC	-660	1602.49	13	3.03
1	11	SC	-384	679.25	13	5.3
1	1	SC1	-359	191.66	13	18.79
1	1	SC2	-386	260.57	13	13.82
2	2	SC	-303	1878.23	13	1.92
3	14	GC	-736	1832.95	13	2.65
3	14	SC	-256	1716.96	13	2.1
3	3	SC1	-350	1040.93	13	3.46
3	3	SC2	-343	703.35	13	5.12
5	5	SC1	-338	435.52	13	8.27
5	5	SC2	-348	588.95	13	6.12
6	6	SC1	-309	263.18	13	13.69
6	6	SC2	-301	226.56	13	15.9
Wall #2						
8	15	GC	-763	6522.83	13	0.74
8	15	SC	-262	2390.62	13	1.51
8	8	S1	-199	8777.77	13	0.41
8	8	S1	-216	9603.74	13	0.38
9	16	GC	-558	10689.94	13	0.45
9	16	S1	-221	9204.6	13	0.39
9	16	S2	-105	16911.55	13	0.21
9	16	SC	-239	2160.49	13	1.67
10	17	GC	-408	120806.8	13	0.04
10	17	SC	-122	58466.47	13	0.06
10	10	S2	-175	12381.53	13	0.29
10	10	SC1	-214	393321.81	13	0.01
10	10	SC2	-177	40187.89	13	0.09
Wall #3						
11	18	GC	-748	1889.96	13	2.57
11	18	SC	-310	500.1	13	7.2
12	19	GC	-875	2248.38	13	2.16
12	19	SC	-390	859.91	13	4.19

Table 0-10: Phase II Descriptive statistics for sacrificial coupons (I-515/Flamingo Rd.)

Phase II LPR Corrosion Rates: Sacrificial Coupons	Galvanized Coupons		Steel Coupons	
	E_{corr} [mV]	CR [µm/yr]	E_{corr} [mV]	CR [µm/yr]
Mean	-673	2.02	-281	4.9
Median	-676	2.57	-302	2.78
Standard Deviation	134	1.27	89	5.55
Coefficient of Variation	20%	63%	32%	113%
Minimum	-875	0	-436	0
Maximum	-408	3.33	-105	18.79
Confidence Level (95%)	88	0.83	36	2.22
Confidence Range for Mean	(-761 to -586)	(1.19 to 2.86)	(-317 to -245)	(2.68 to 7.12)
Count	9	9	24	24

Table 0-11: CSR statistics based on the MMCE (2004) LPR corrosion rates for in-service reinforcements (I-515/Flamingo Rd.)

MMCE Data	In-Service Reinforcements CSR
Mean	0.57
Median	0.44
Standard Deviation	0.63
Coefficient of Variation	142%
Minimum	0.05
Maximum	4.72
Confidence Level (95%)	0.15
Confidence Range for Mean	(0.42 to 0.72)
Total Count	69
CSR < 0.80	59
0.80 ≤ CSR ≤ 1.0	6
1.0 ≤ CSR ≤ 2.0	3
CSR > 2.0	1
Concern Decision Criteria	
CSR ≥ 0.8	10
	14%
CSR ≥ 1.0	4
	6%

Table 0-12: CSR statistics based on the Phase II (2012) LPR corrosion rates for in-service reinforcements (I-515/Flamingo Rd.)

Phase II Data	In-Service Reinforcements CSR
Mean	0.53
Median	0.42
Standard Deviation	0.83
Coefficient of Variation	157%
Minimum	0
Maximum	5.01
Confidence Level (95%)	0.28
Confidence Range for Mean	(0.25 to 0.82)
Total Count	33
CSR < 0.80	30
0.80 ≤ CSR ≤ 1.0	1
1.0 ≤ CSR ≤ 2.0	1
CSR > 2.0	1
Concern Decision Criteria	
CSR ≥ 0.8	3
	9%
CSR ≥ 1.0	2
	6%

Table 0-13: CSR statistics based on the Phase II (2012) LPR corrosion rates for sacrificial coupons (I-515/Flamingo Rd.)

Phase II Data	Galvanized Coupons CSR	Plain Steel CSR
Mean	0.51	0.41
Median	0.64	0.23
Standard Deviation	0.32	0.46
Coefficient of Variation	63%	113%
Minimum	0.01	0
Maximum	0.83	1.57
Confidence Level (95%)	0.21	0.19
Confidence Range for Mean	(0.30 to 0.71)	(0.22 to 0.59)
Total Count	9	24
CSR < 0.80	7	7
0.80 ≤ CSR ≤ 1.0	2	0
1.0 ≤ CSR ≤ 2.0	0	4
CSR > 2.0	0	0
Concern Decision Criteria		
CSR ≥ 0.8	2	4
	22%	*17%*
CSR ≥ 1.0	0	4
	0%	*17%*

Table 0-14: CSR statistics based on the Phase II (2012) LPR corrosion rates for all elements (I-515/Flamingo Rd.)

Phase II	All Elements CSR
Mean	0.48
Median	0.42
Standard Deviation	0.66
Coefficient of Variation	136%
Minimum	0
Maximum	5.01
Confidence Level (95%)	0.16
Confidence Range for Mean	(0.32 to 0.64)
Total Count	66
CSR < 0.80	57
0.80 ≤ CSR ≤ 1.0	3
1.0 ≤ CSR ≤ 2.0	5
CSR > 2.0	1
Concern Decision Criteria	
CSR ≥ 0.8	9
	14%
CSR ≥ 1.0	6
	9%

Table 0-15: CSR statistics based on all LPR corrosion rates for all elements at I-515/Flamingo Rd.

Combined MMCE and Phase II Data	Lifetime CSR
Mean	0.53
Median	0.44
Standard Deviation	0.64
Coefficient of Variation	122%
Minimum	0
Maximum	5.01
Confidence Level (95%)	0.11
Confidence Range for Mean	(0.42 to 0.64)
Total Count	135
CSR < 0.80	115
0.80 ≤ CSR ≤ 1.0	9
1.0 ≤ CSR ≤ 2.0	8
CSR > 2.0	3
Concern Decision Criteria	
CSR ≥ 0.8	20
	15%
CSR ≥ 1.0	11
	8%
CSR ≥ 2.0	3
	2%

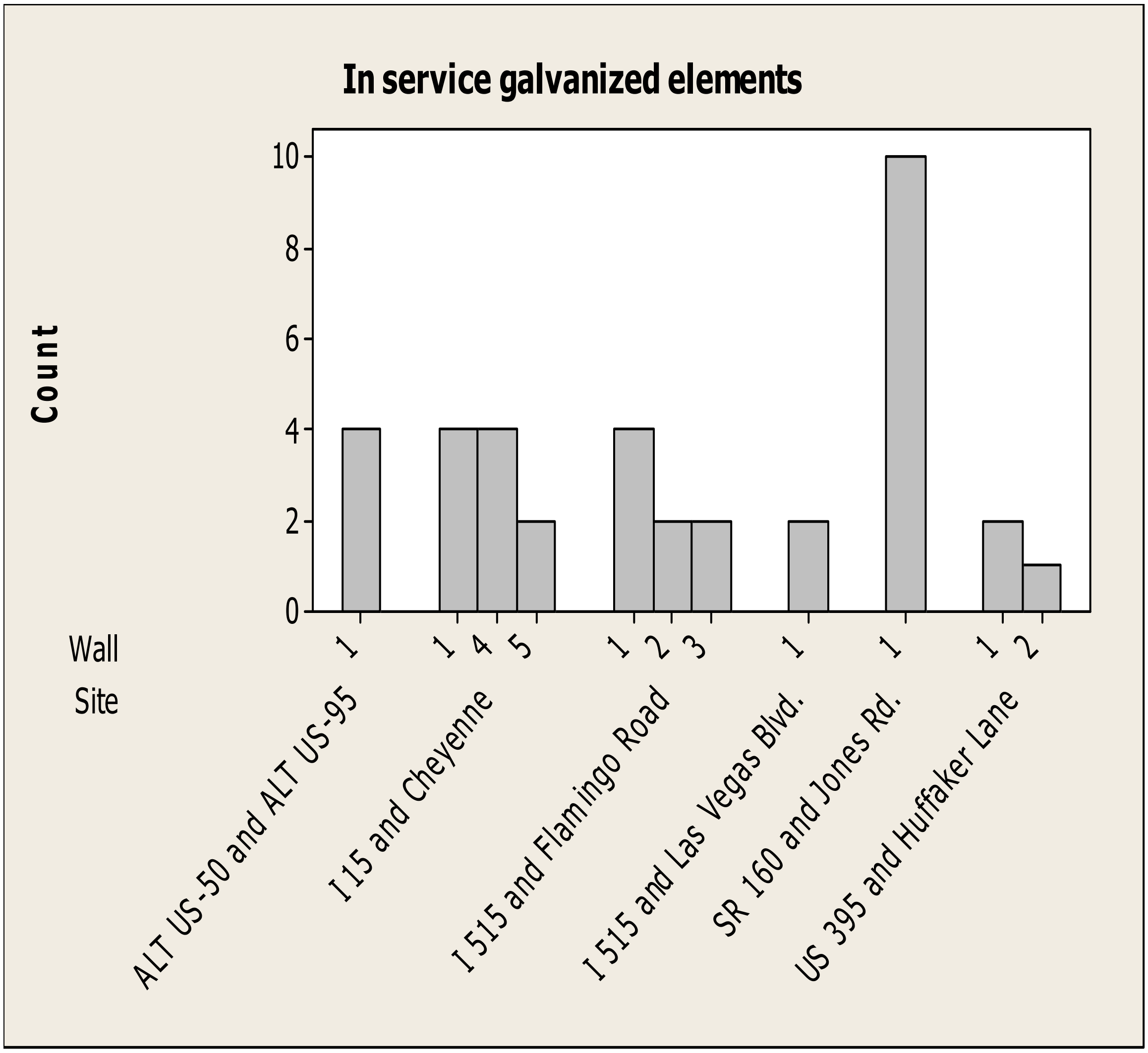

Figure 0-3: Number of In-service Galvanized Elements per wall

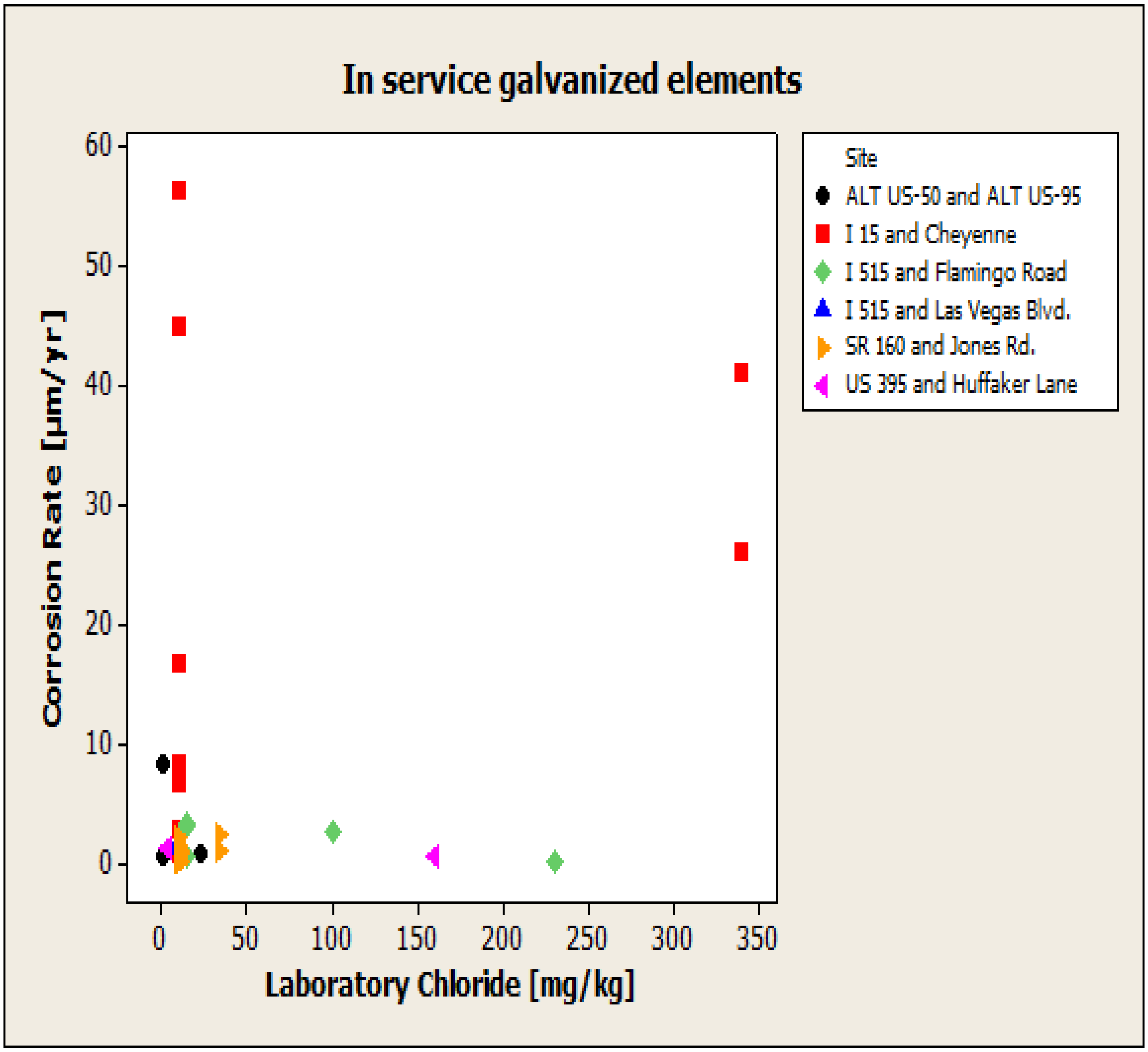

Figure 0-4: Phase II In-service galvanized LPR corrosion rates compared to the chloride content

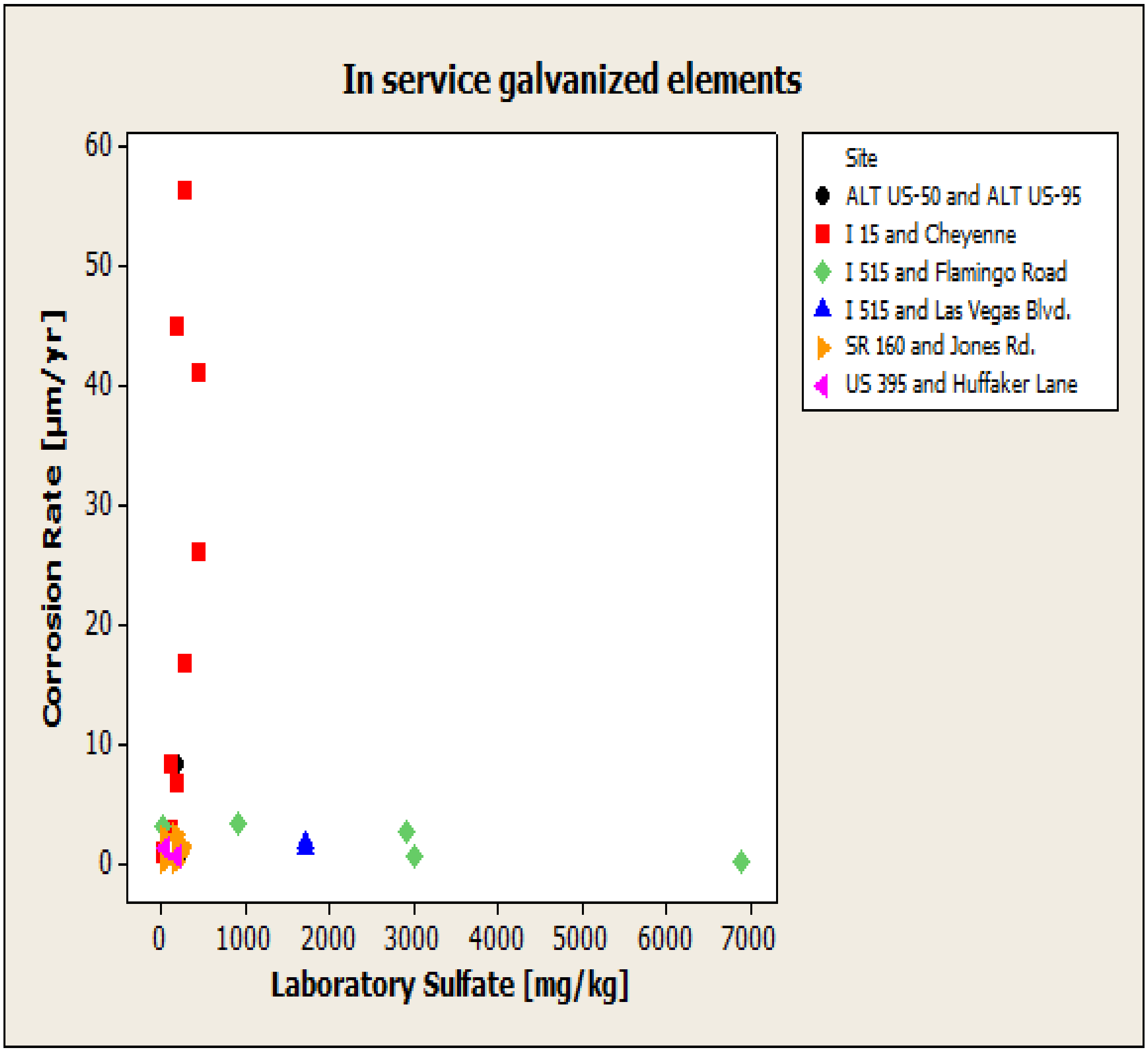

Figure 0-5: LPR corrosion rates compared to the sulfate content, full spectrum of Phase II data

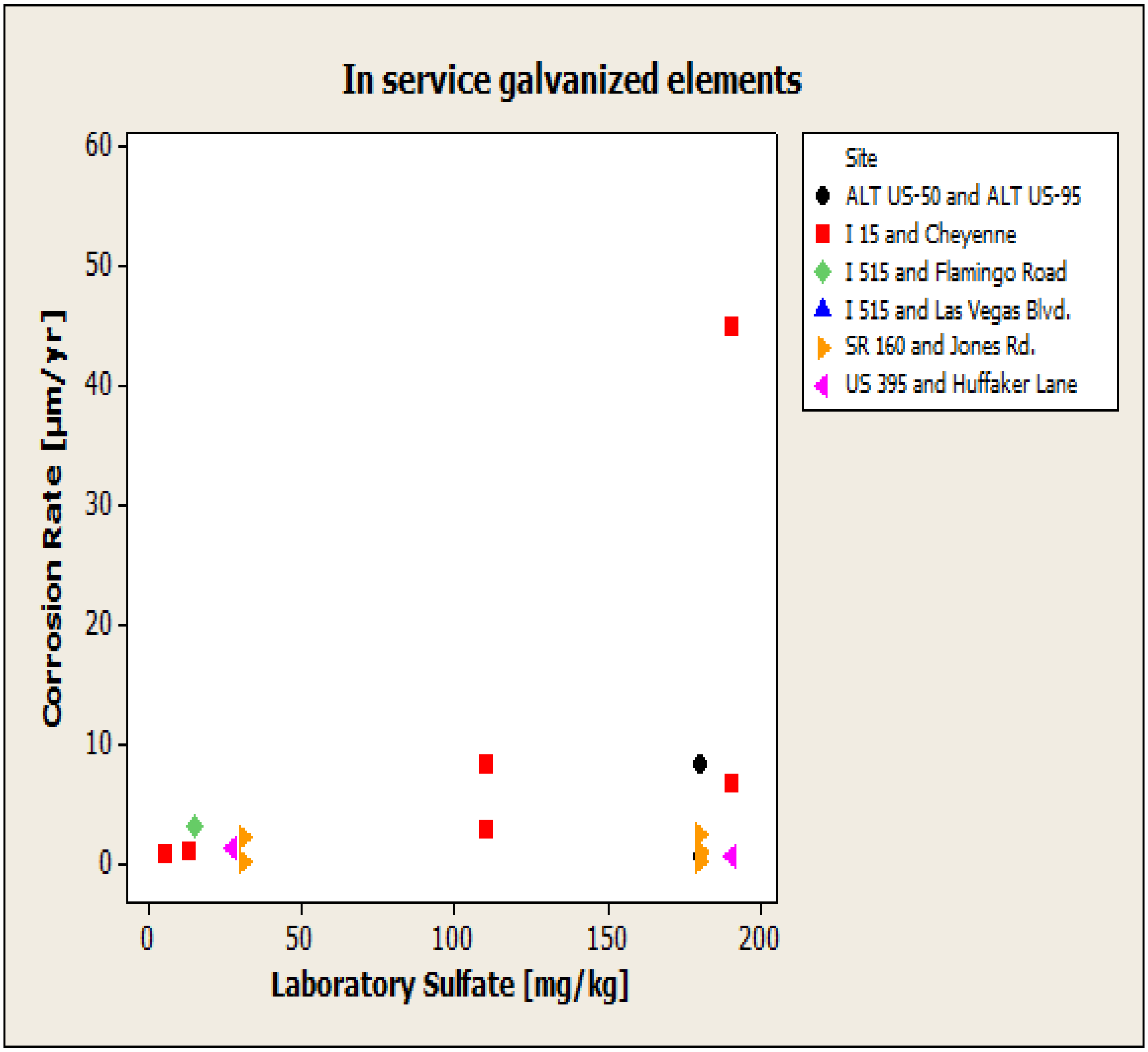

Figure 0-6: Phase II LPR corrosion rates and sulfate content data for galvanized in-service elements which PASS the maximum limit of 200 ppm sulfate

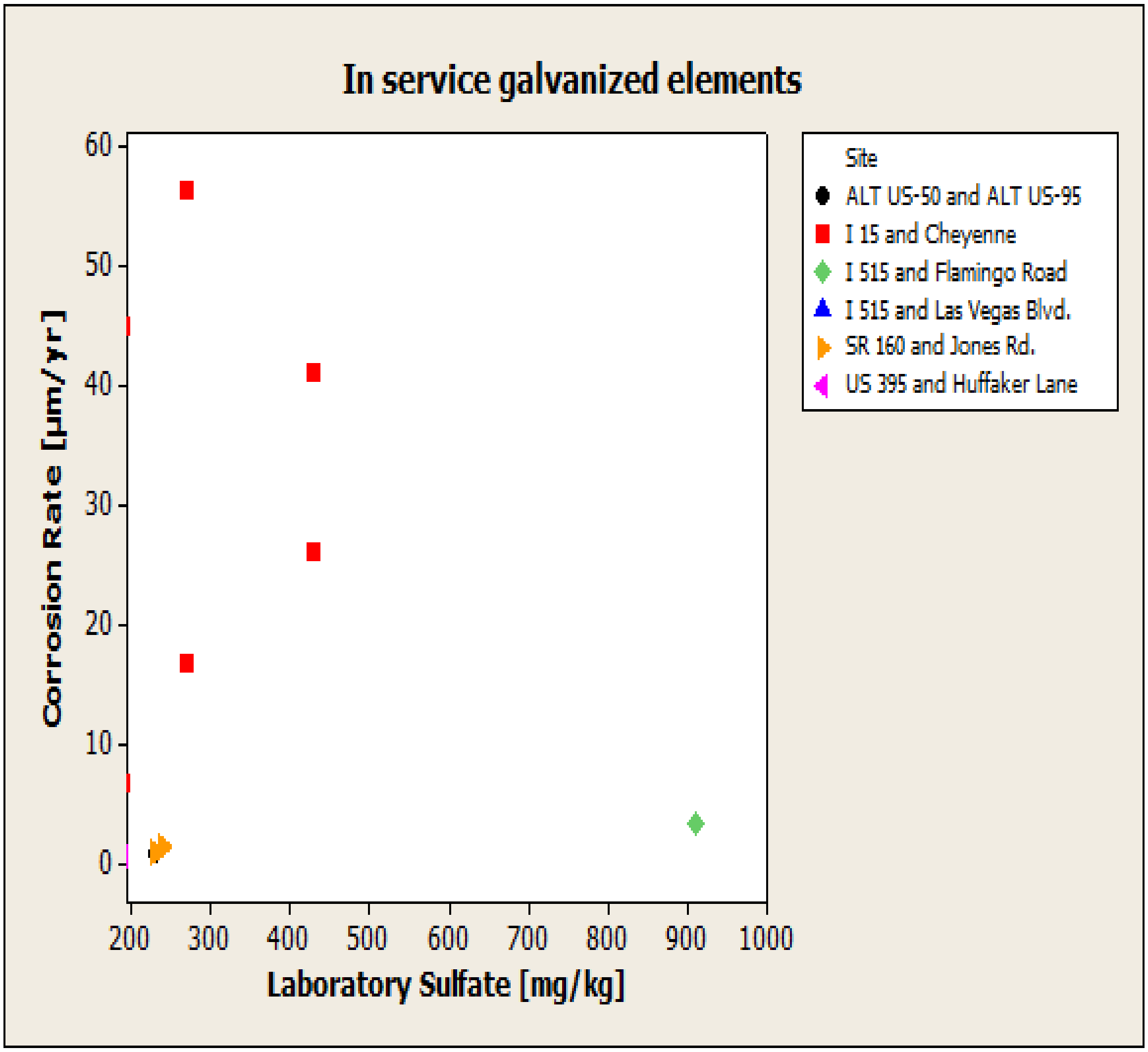

Figure 0-7: Phase II LPR corrosion rates and sulfate content data for galvanized in-service elements which FAIL the maximum limit of 200 ppm sulfate

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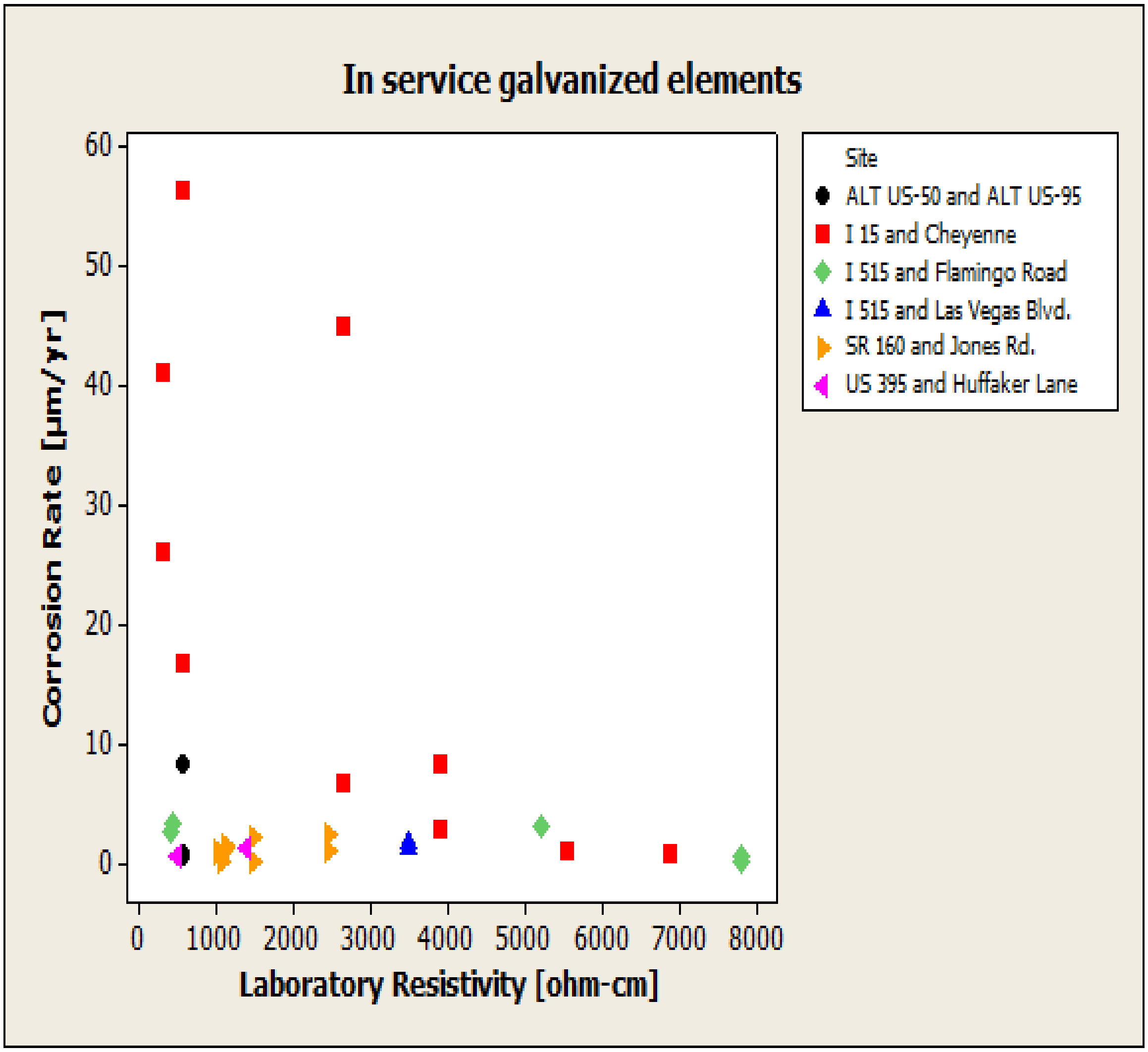

Figure 0-8: LPR corrosion rates compared to the minimum soil resistivity, full spectrum of Phase II

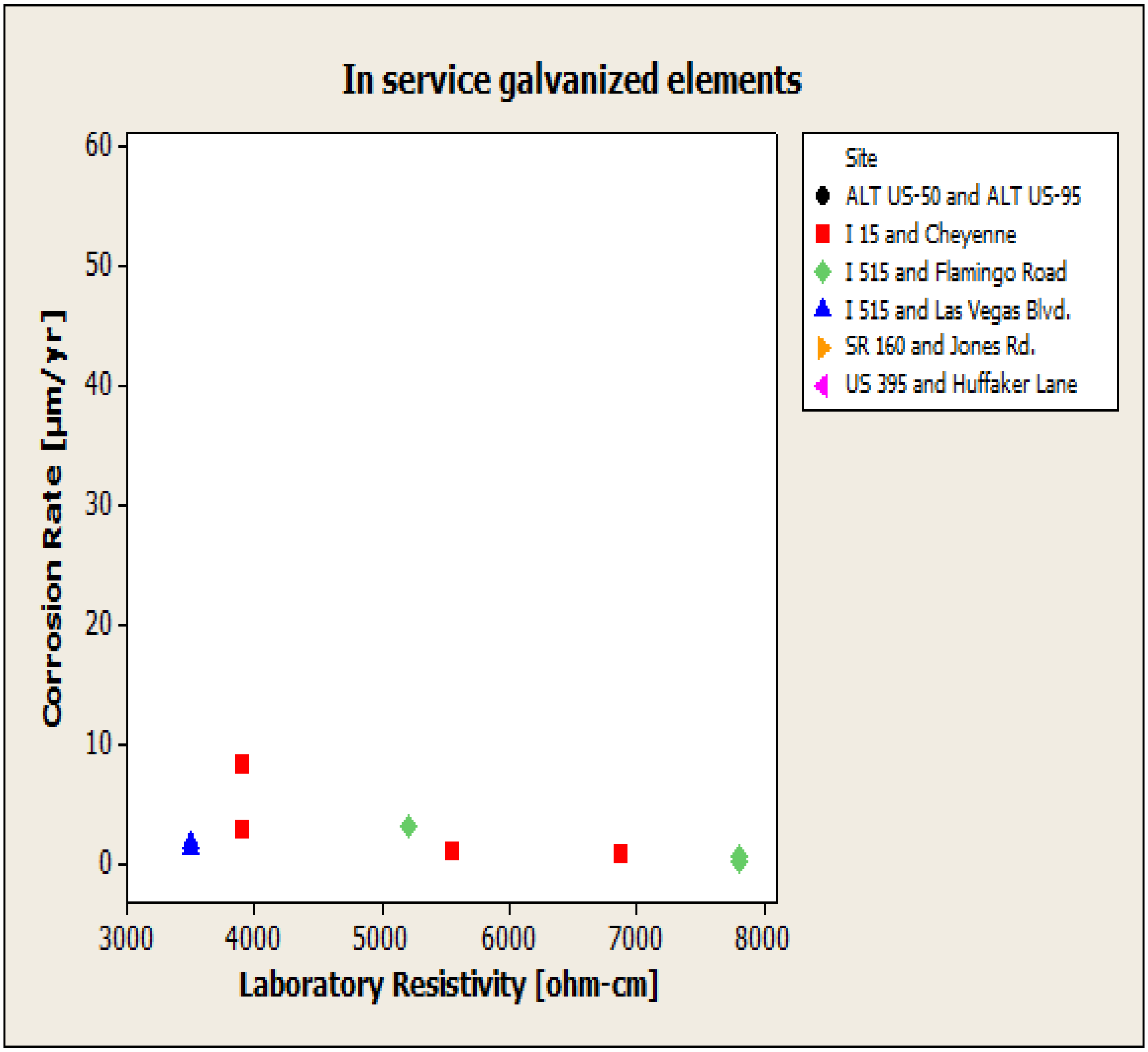

Figure 0-9: Phase II LPR corrosion rate data in soils that PASS the 3,000 ohm-cm specification for minimum soil resistivity

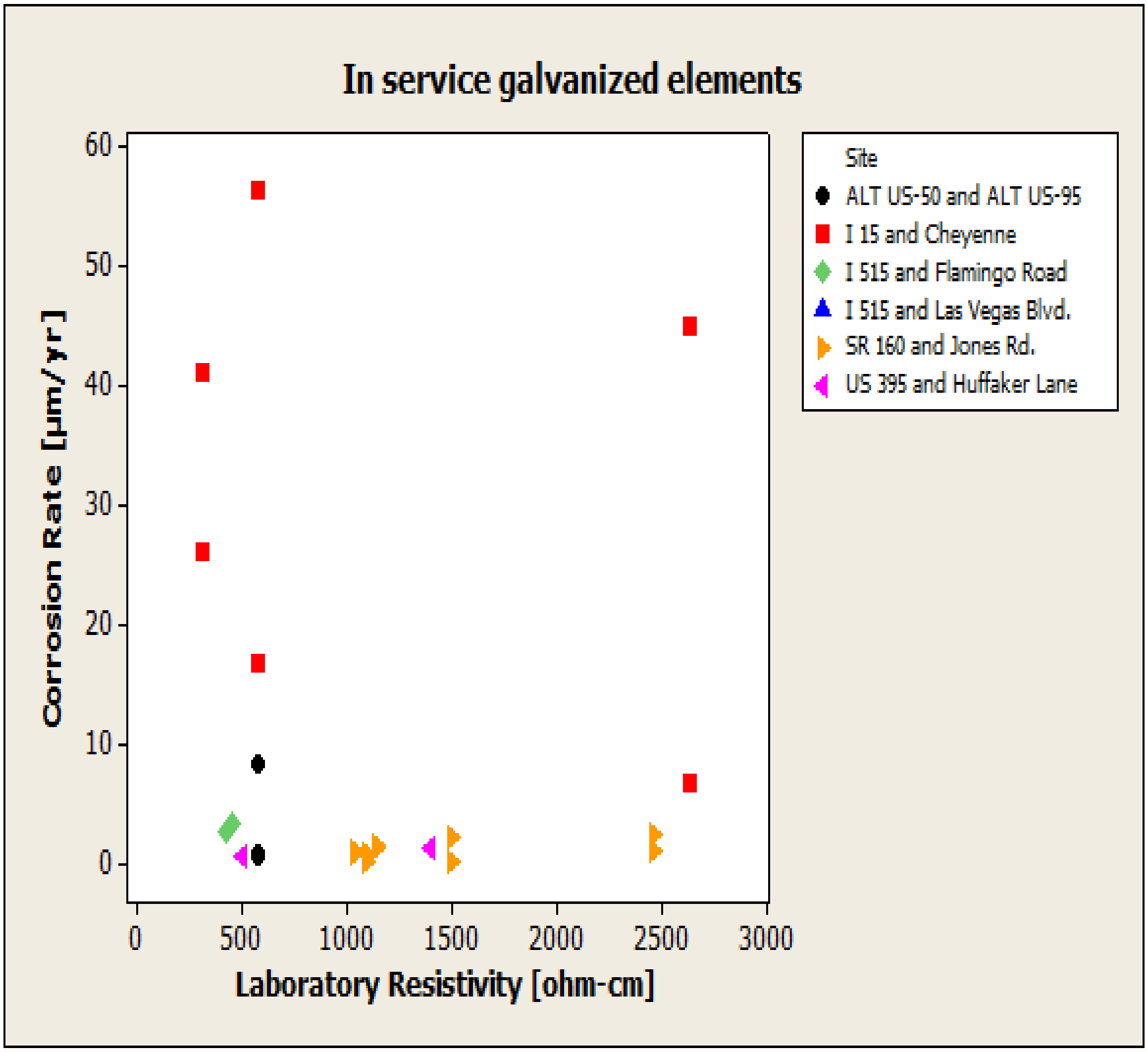

Figure 0-10: Phase II LPR corrosion rate data in soils that FAILS the 3,000 ohm-cm specification for minimum soil resistivity

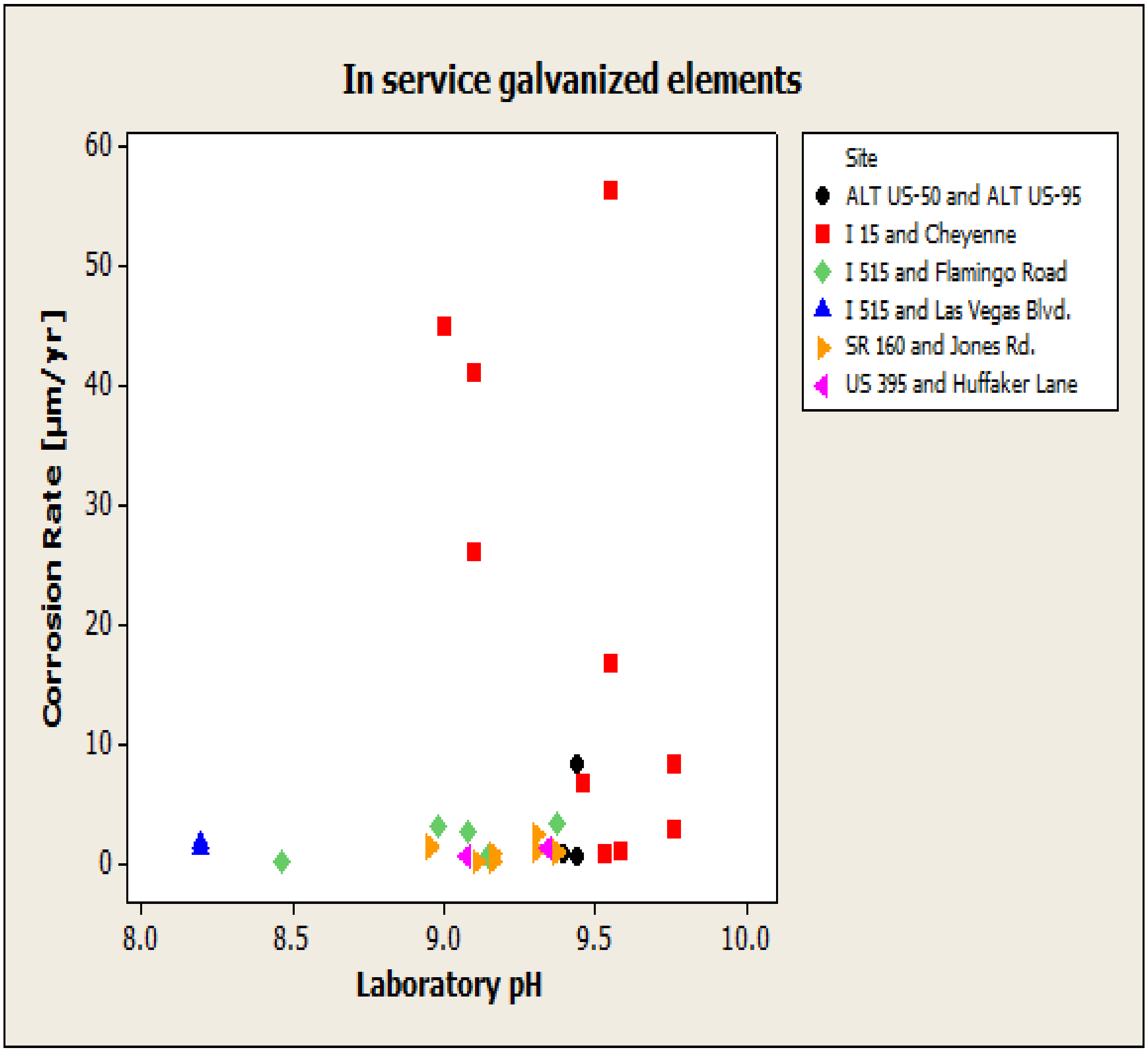

Figure 0-11: Phase II LPR corrosion rate data compared with backfill pH test results

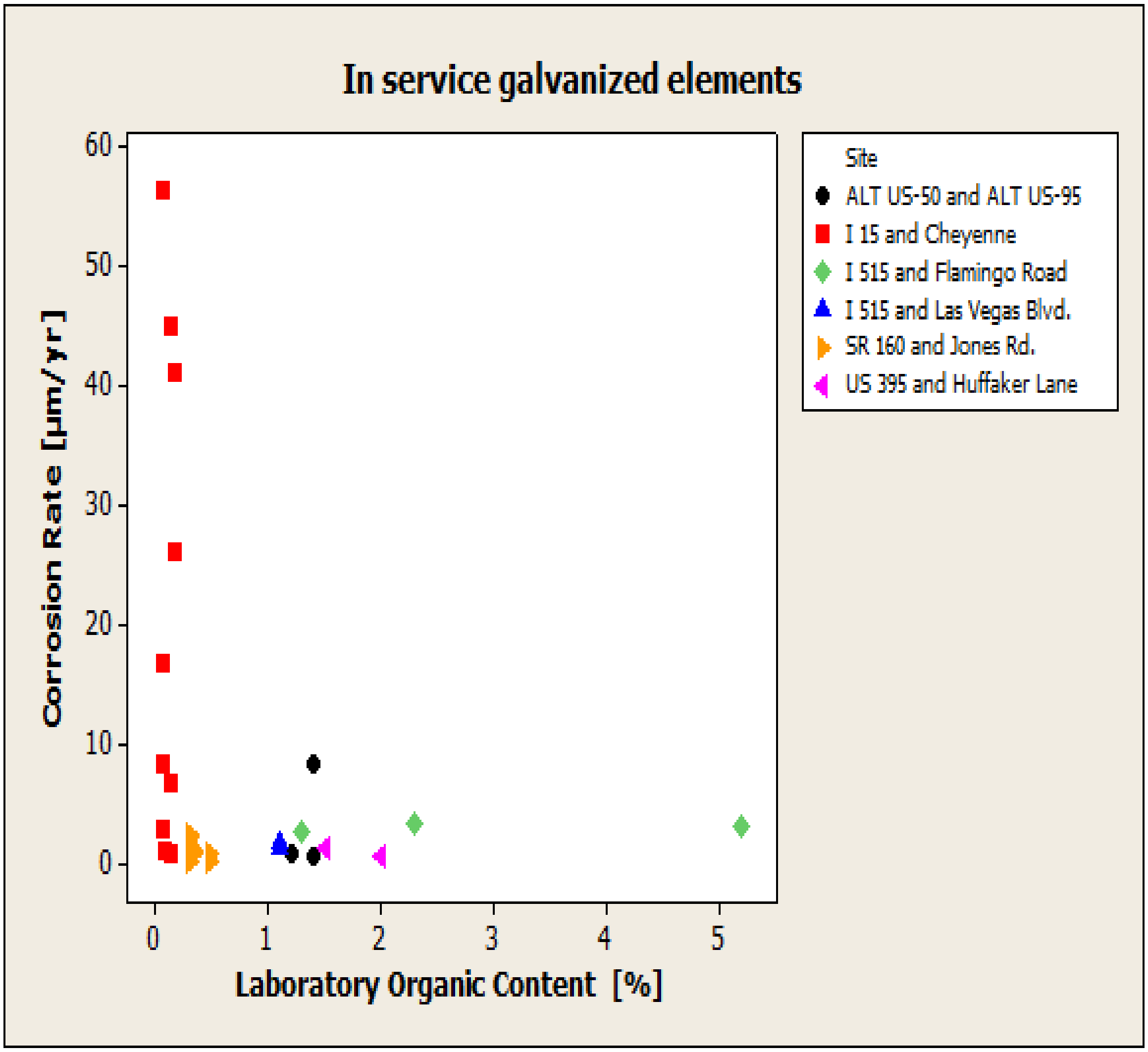

Figure 0-12: II LPR corrosion rates compared with backfill organic content test results

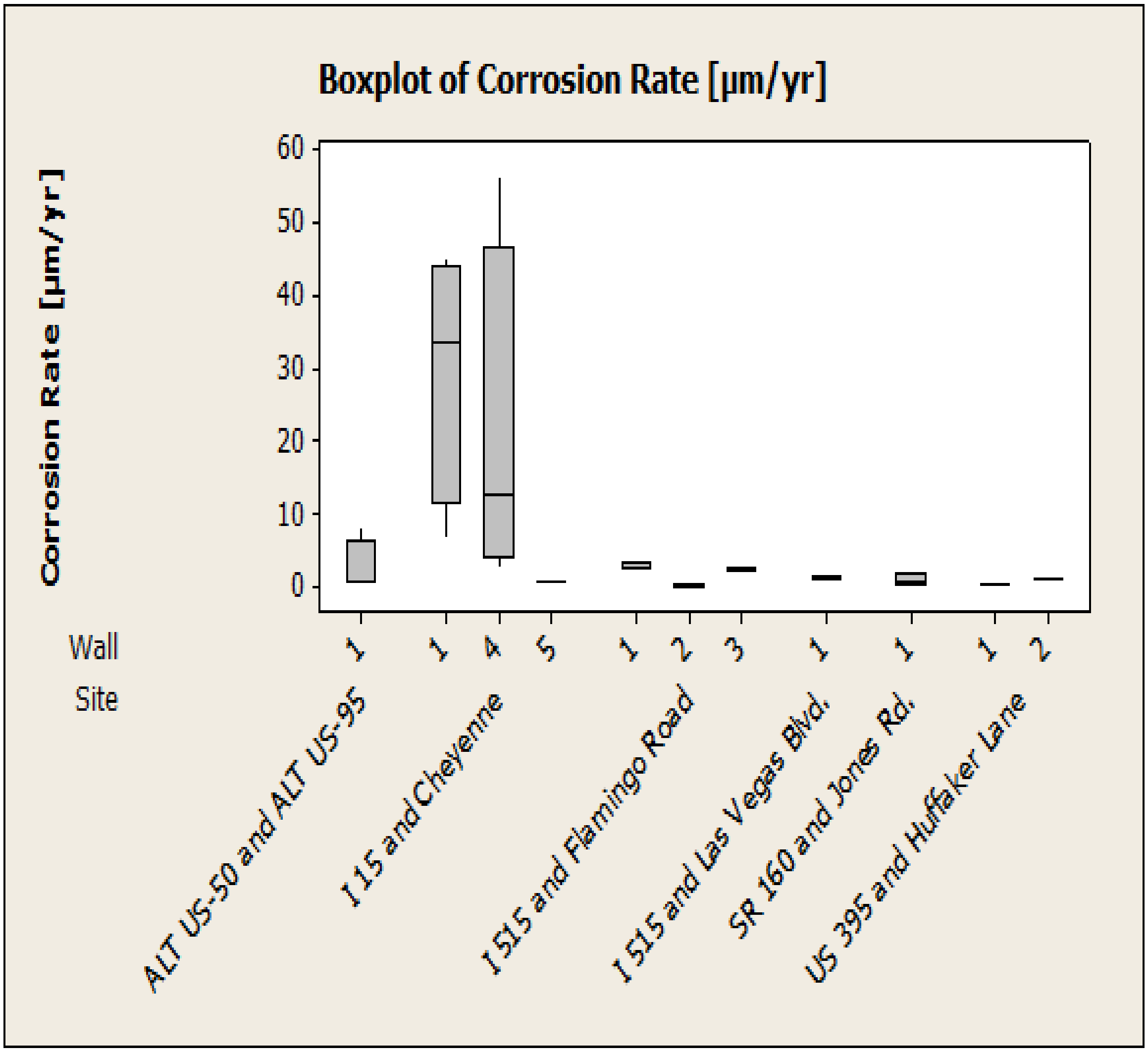

Figure 0-13: Phase II in-service galvanized corrosion rates distribution information based on MSE wall location

Table 0-16: Descriptive statistics for in-service galvanized elements based on wall location

Site	Wall	Count	Mean [μm/yr]	Standard Deviation [μm/yr]	COV [%]	Confidence Level (95%) [μm/yr]	Confidence Range for Mean [μm/yr]
Alt. US 50 and Alt. US 95	1	4	2.6	3.8	149%	3.7	(0.00 - 6.3)
I 15 and Cheyenne	1	4	29.7	17.5	59%	17.1	(12.6 - 46.8)
	4	4	21	24.3	115%	23.8	(0.00 - 44.8)
	5	2	0.7	0.1	20%	0.2	(0.5 - 0.9)
I 515 and Flamingo Road	1	4	3.1	0.3	10%	0.3	(2.8 - 3.4)
	2	2	0.2	0.3	119%	0.4	(0.00 - 0.7)
	3	2	2.4	0.3	12%	0.4	(2.0 - 2.8)
I 515 and Las Vegas Blvd.	1	2	1.3	0.3	23%	0.4	(0.9 - 1.8)
SR 160 and Jones Rd.	1	10	1	0.8	74%	0.5	(0.6 - 1.5)
US 395 and Huffaker Lane	1	2	0.5	0.1	16%	0.1	(0.4 - 0.5)
	2	1	1.3	-	-	-	-

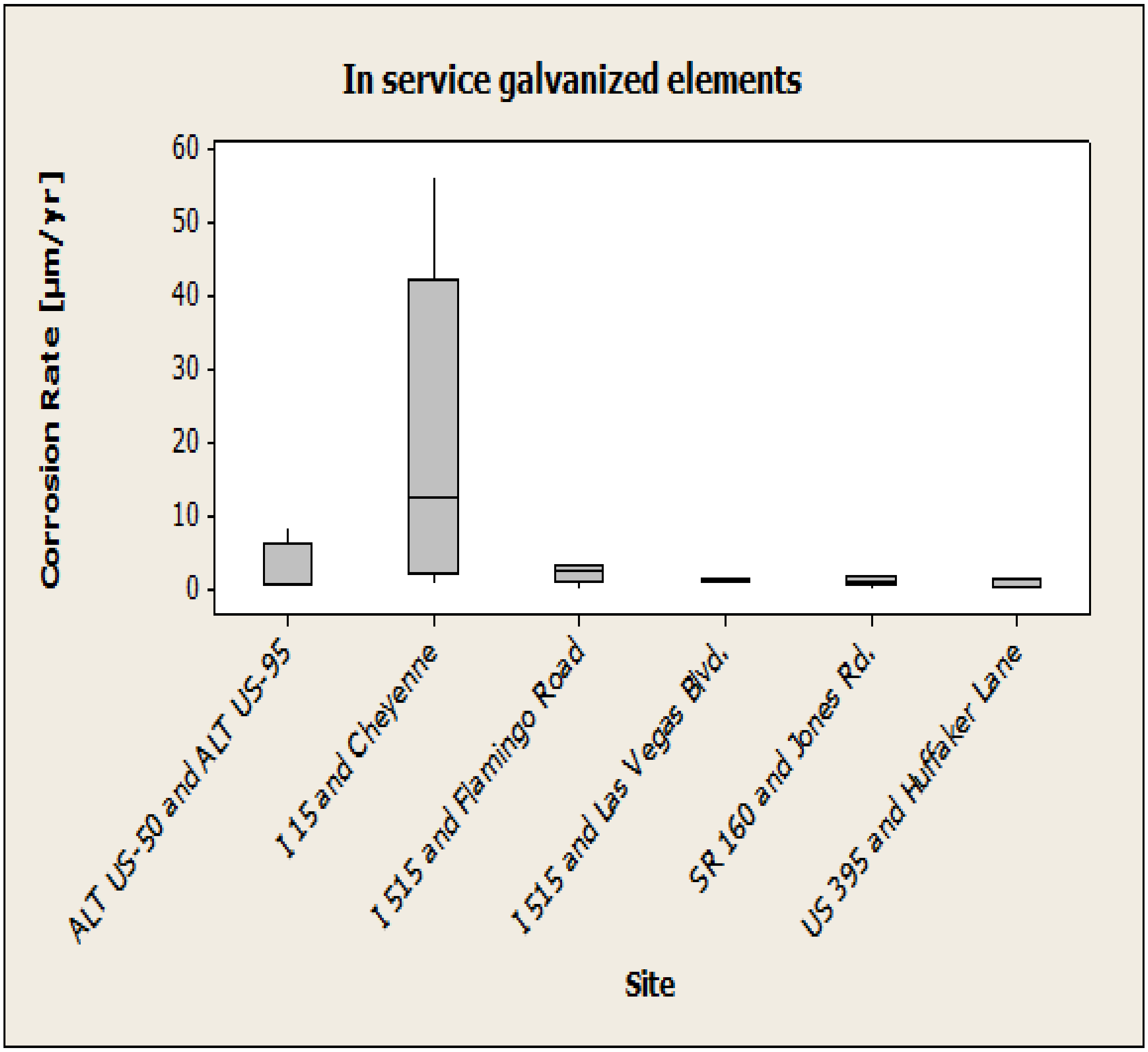

Figure 0-14: II in-service galvanized corrosion rates distribution information based on site wide information

Table 0-17: Descriptive statistics for in-service galvanized elements based on site wide information

Site	Mean [μm/yr]	Median [μm/yr]	Standard Deviation [μm/yr]	COV [%]	Min. [μm/yr]	Max. [μm/yr]	Count	Confidence Level (95%) [μm/yr]	Confidence Range for Mean [μm/yr]
Alt. US 50 and Alt. US 95	2.6	0.7	3.8	149%	0.5	8.3	4	3.7	(0.00 - 6.32)
I-15 and Cheyenne Blvd.	20.4	12.5	20.5	101%	0.6	56.4	10	12.7	(7.70 - 33.17)
I-515 and Flamingo Rd.	2.2	2.6	1.3	58%	0.0	3.3	8	0.9	(1.31 - 3.06)
I-515 and Las Vegas Blvd.	1.3	1.3	0.3	23%	1.1	1.6	2	0.4	(0.91 - 1.77)
SR 160 and Jones Rd.	1.0	0.9	0.8	74%	0.0	2.3	10	0.5	(0.57 - 1.53)
US 395 and Huffaker Lane	0.7	0.5	0.5	66%	0.4	1.3	3	0.5	(0.18 - 1.26)

Table 0-18: In-service galvanized elements CSR values based on site wide LPR test results

Site	Mean	Median	Standard Deviation	COV	Min.	Max	Count	Confidence Level (95%)	Confidence Range for Mean
Alt. US 50 and Alt. US 95	0.6	0.2	1.0	149%	0.1	2.1	4	0.9	(0.00 - 1.58)
I-15 and Cheyenne Blvd.	5.1	3.1	5.1	101%	0.2	14.1	10	3.2	(1.93 - 8.29)
I-515 and Flamingo Rd.	0.5	0.7	0.3	58%	0	0.8	8	0.2	(0.33 - 0.76)
I-515 and Las Vegas Blvd.	0.1	0.1	0.0	23%	0.1	0.1	2	0.0	(0.08 - 0.15)
SR 160 and Jones Rd.	0.3	0.2	0.2	74%	0	0.6	10	0.1	(0.14 - 0.38)
US 395 and Huffaker Lane	0.1	0	0	66%	0	0.1	3	0	(0.02 - 0.11)

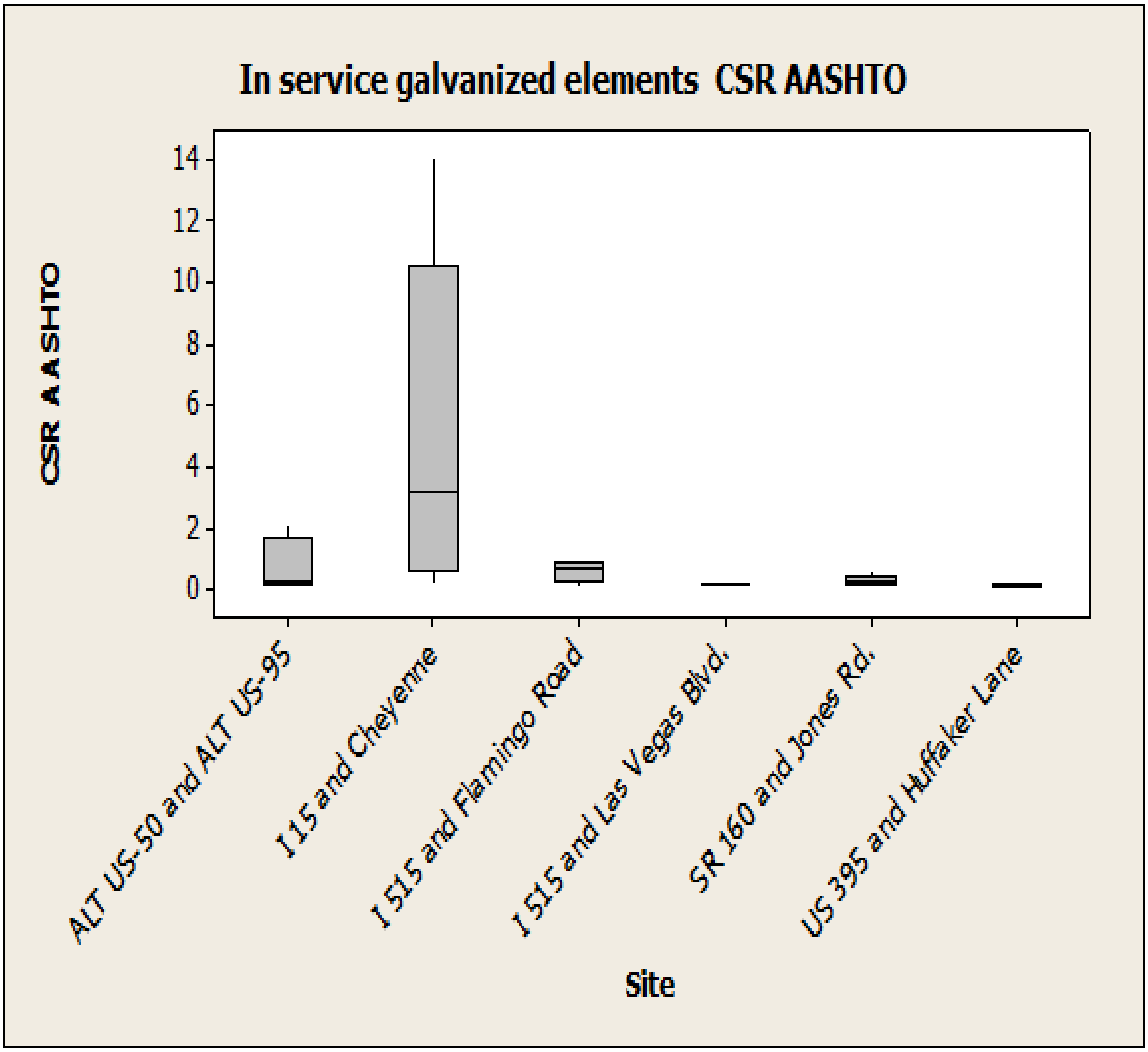

Figure 0-15: In-service galvanized elements CSR values based on site wide information

Table 0-19: Phase II in-service reinforcement LPR corrosion rates descriptive statistics, I-15/Cheyenne Blvd. removed from data set so that its influence on the data population demonstrated

	Phase II In-Service Galvanized Elements	Phase II In-Service Galvanized Elements *
Mean [µm/yr]	6.7	1.6
Median [µm/yr]	1.5	1.1
Standard Deviation [µm/yr]	13.4	1.7
COV [%]	200%	105%
Min. [µm/yr]	0.0	0.0
Max. [µm/yr]	56.4	8.3
Count	37	27
Confidence Level (95%) [µm/yr]	4.3	0.6
Confidence Range for Mean [µm/yr]	(2.37 - 11.01)	(0.96 - 2.23)

* I-15/Cheyenne Blvd. omitted from data set

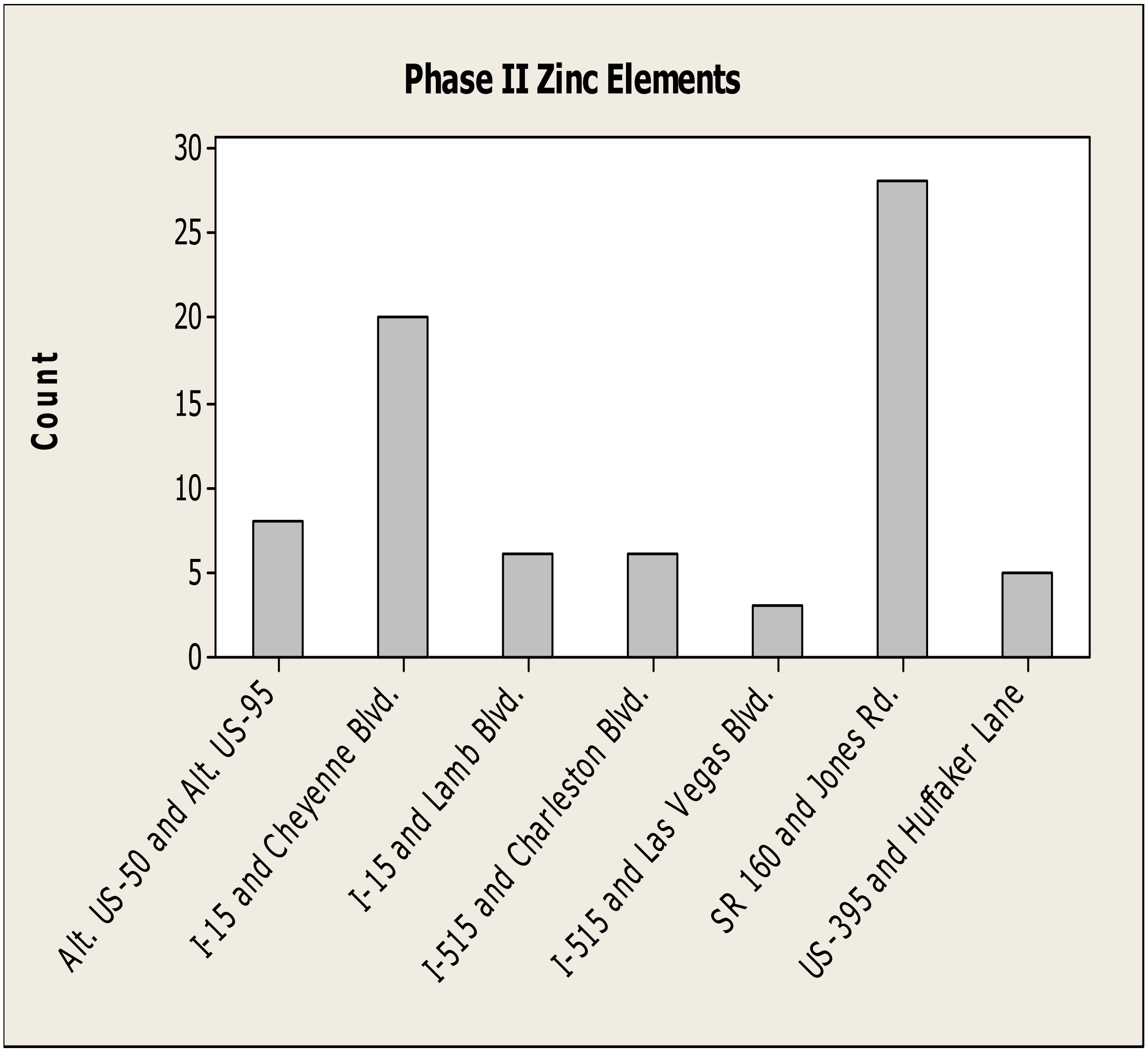

Figure 0-16: Number of zinc elements per site for LPR analysis

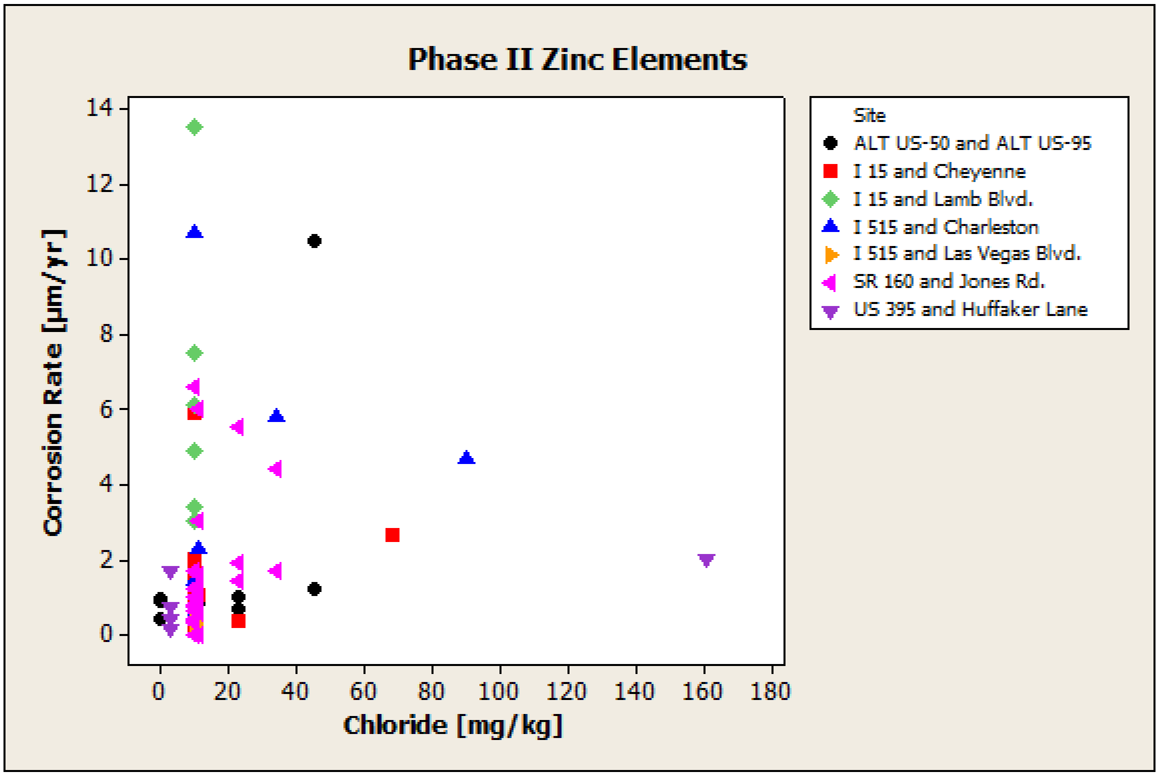

Figure 0-17: Phase II Zinc element LPR corrosion rates compared to the chloride content

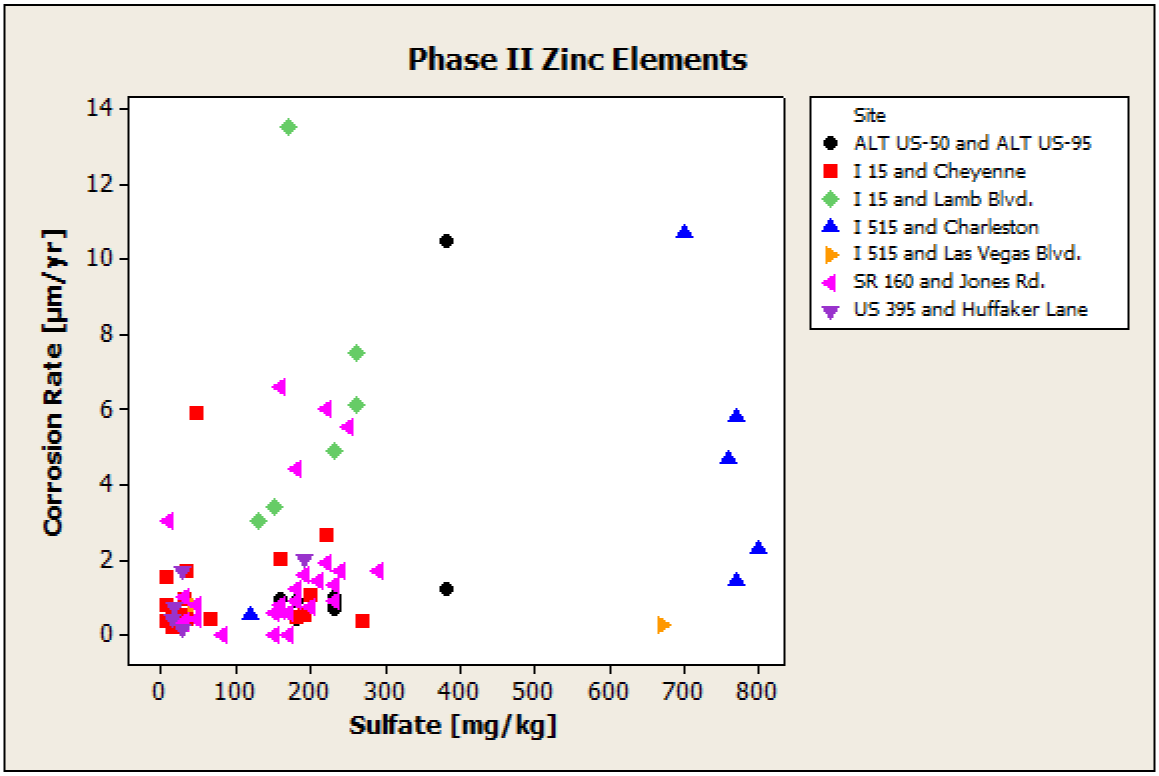

Figure 0-18: Phase II Zinc element LPR corrosion rates compared to the sulfate content

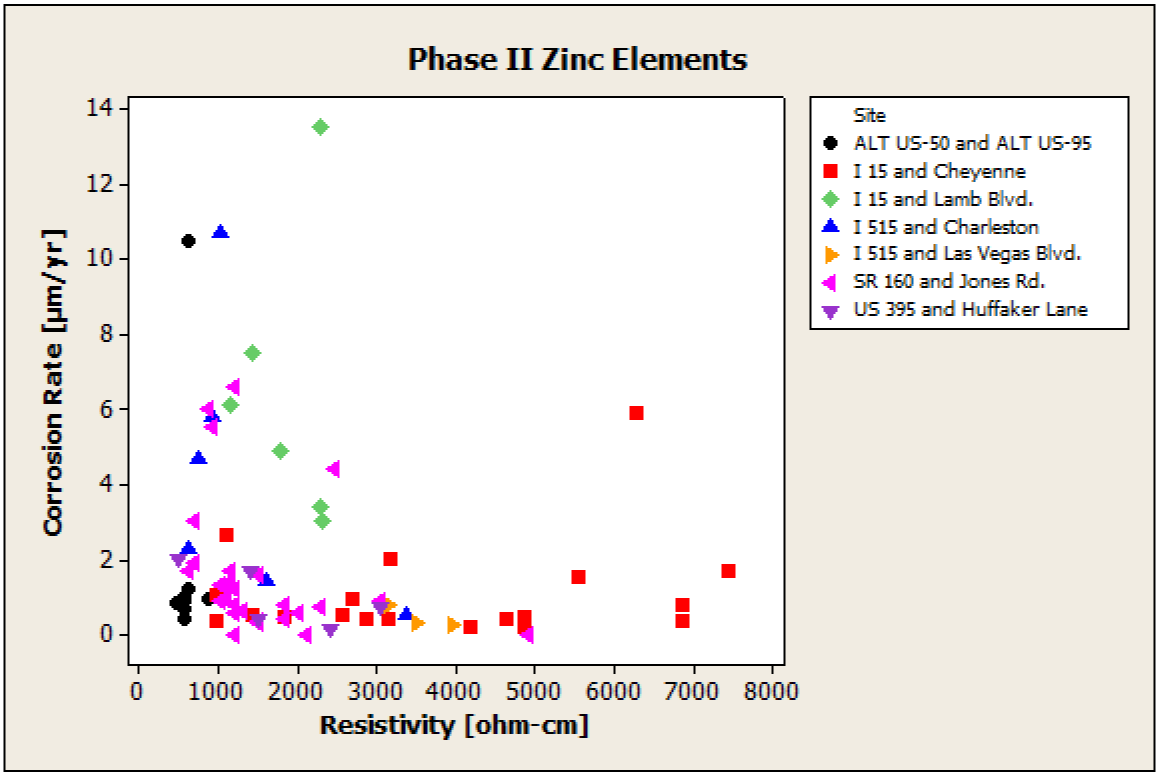

Figure 0-19: Phase II Zinc element LPR corrosion rates compared to the complete minimum soil resistivity testing results

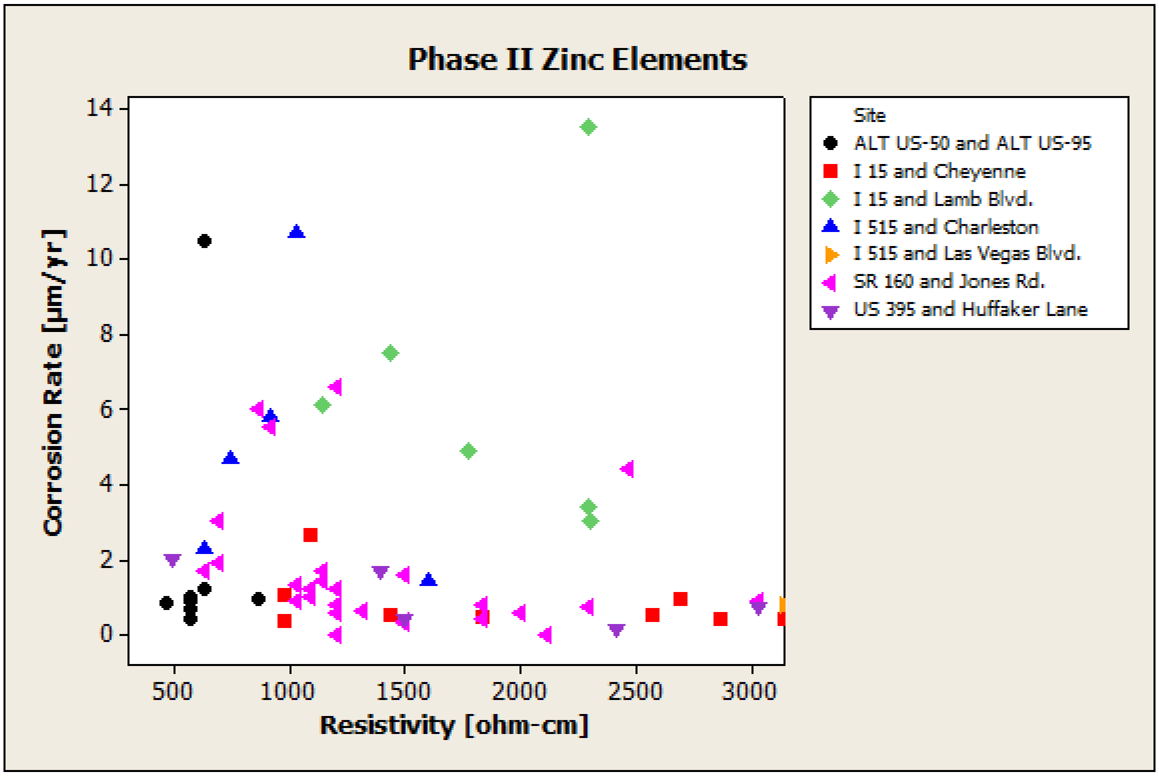

Figure 0-20: Phase II Zinc element LPR corrosion rates for elements in backfills that fail specifications for minimum soil resistivity

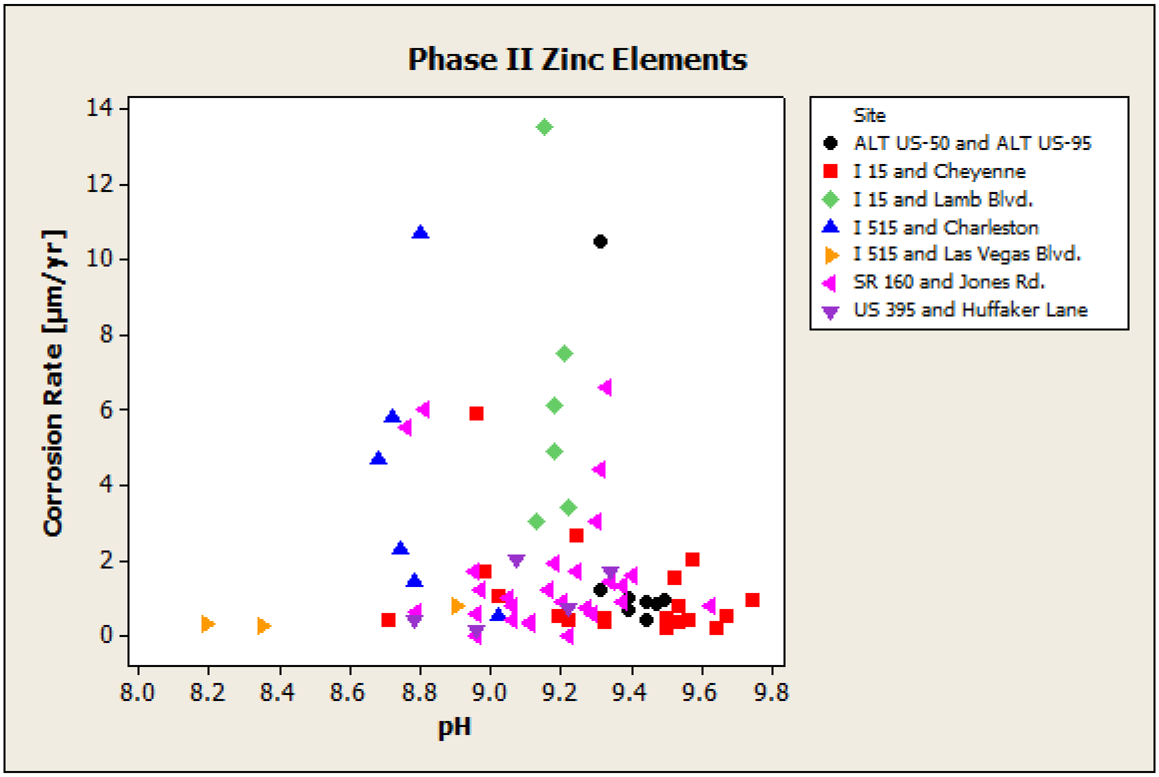

Figure 0-21: Phase II Zinc element LPR corrosion rates as they relate to backfill pH

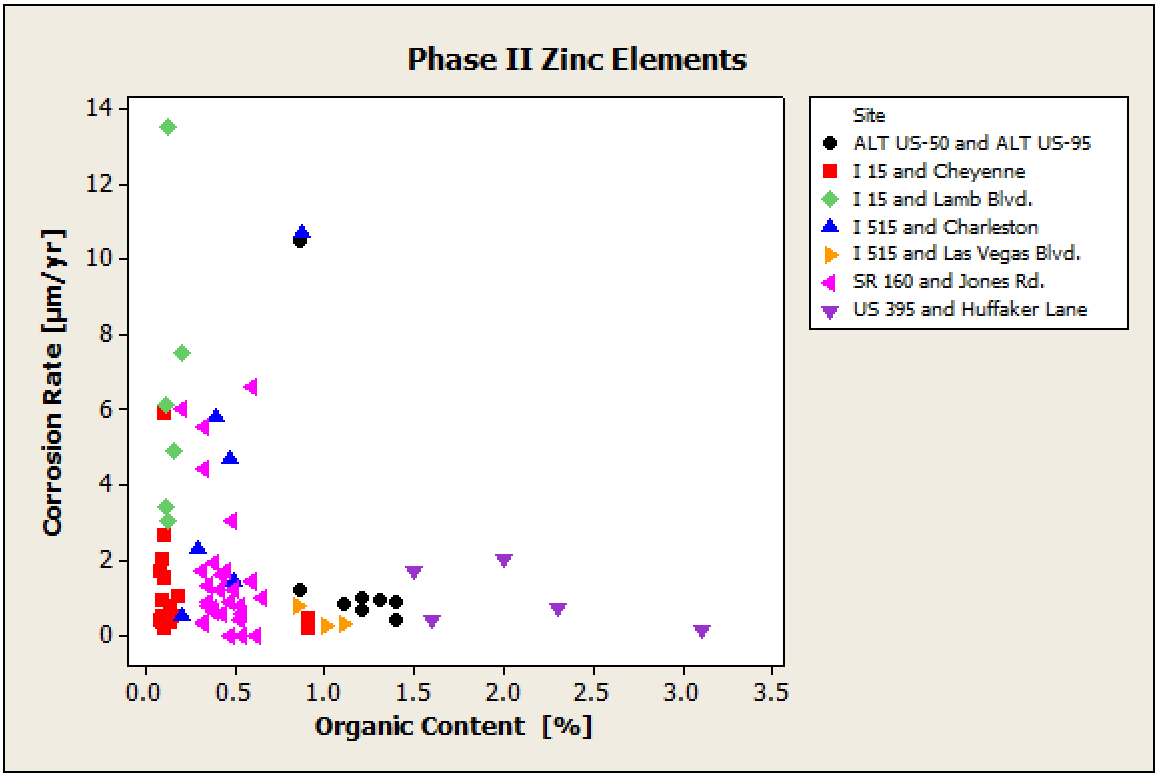

Figure 0-22: Phase II Zinc element LPR corrosion rates as they relate the organic content of the backfill

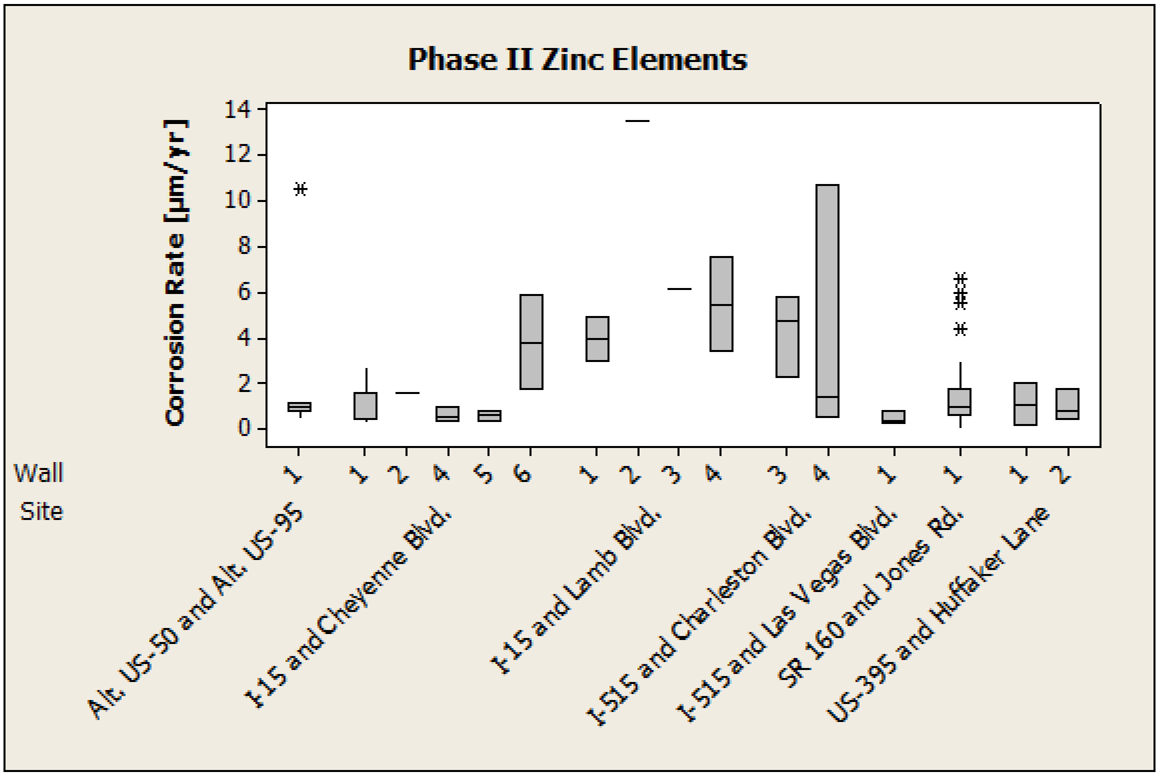

Figure 0-23: Phase II zinc elements corrosion box-plots, per wall distribution of LPR corrosion data

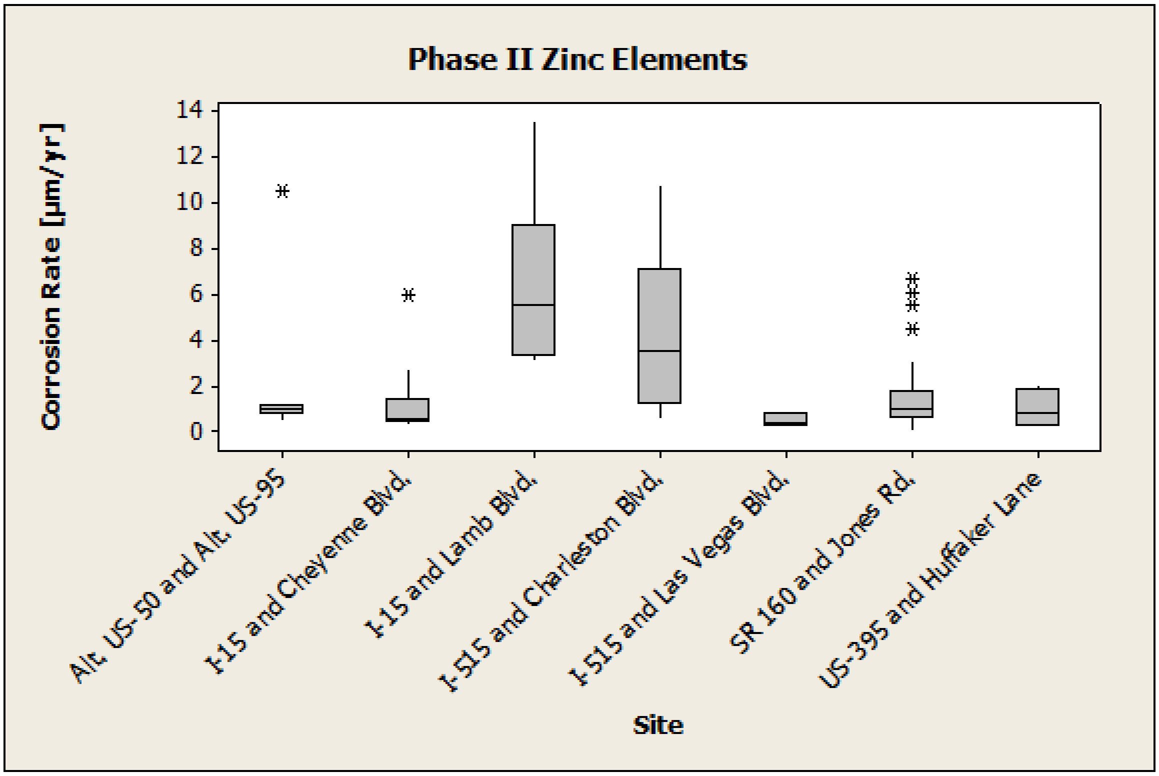

Figure 0-24: Phase II zinc elements corrosion box-plots based on site wide LPR testing results

Table 0-20: Phase II zinc element, descriptive statistics for LPR corrosion rates

Site	Mean [μm/yr]	Median [μm/yr]	Standard Deviation [μm/yr]	COV	Min. [μm/yr]	Max. [μm/yr]	Confidence Level (95%) [μm/yr]	Confidence Range for Mean [μm/yr]	Count
ALT US-50 and ALT US-95	0.8	0.9	0.3	30%	0.4	1.2	0.2	(0.7 - 1.0)	7
I 15 and Cheyenne	1.1	0.5	1.3	120%	0.2	5.9	0.6	(0.5 - 1.7)	20
I 15 and Lamb Blvd.	6.4	5.5	3.9	60%	3.0	13.5	3.1	(3.3 - 9.5)	6
I 515 and Charleston	4.2	3.5	3.7	88%	0.5	10.7	3.0	(1.2 - 7.2)	6
I 515 and Las Vegas Blvd.	0.5	0.3	0.3	62%	0.3	0.8	0.3	(0.1 - 0.8)	3
SR 160 and Jones Rd.	1.6	0.9	1.8	112%	0.0	6.6	0.7	(0.9 - 2.3)	28
US 395 and Huffaker Lane	1.0	0.6	0.8	85%	0.2	2.0	0.7	(0.2 - 1.7)	5

Table 0-21: Phase II zinc element, descriptive statistics, for AASHTO CSR

Site	Mean	Median	Standard Deviation	COV	Min.	Max.	Confidence Level (95%)	Confidence Range for Mean	Count
ALT US-50 and ALT US-95	0.1	0.1	0.0	30%	0.0	0.1	0.0	(0.0 - 0.1)	7
I 15 and Cheyenne	0.1	0.0	0.1	120%	0.0	0.4	0.0	(0.0 - 0.1)	20
I 15 and Lamb Blvd.	0.4	0.4	0.3	60%	0.2	0.9	0.2	(0.2 - 0.6)	6
I 515 and Charleston	0.3	0.2	0.2	88%	0.0	0.7	0.2	(0.1 - 0.5)	6
I 515 and Las Vegas Blvd.	0.0	0.0	0.0	62%	0.0	0.1	0.0	(0.0 - 0.1)	3
SR 160 and Jones Rd.	0.1	0.1	0.1	112%	0.0	0.4	0.0	(0.1 - 0.2)	28
US 395 and Huffaker Lane	0.1	0.0	0.1	85%	0.0	0.1	0.0	(0.0 - 0.1)	5

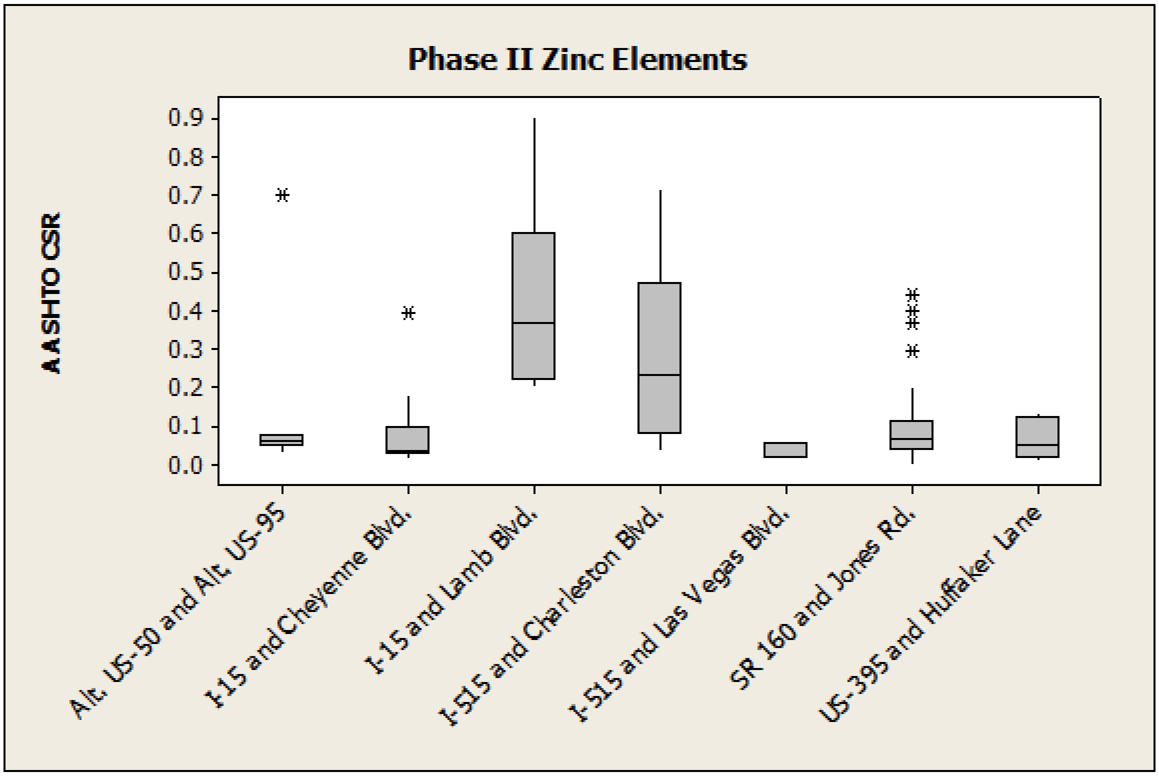

Figure 0-25: Phase II zinc element AASHTO CSR box-plots based entire site sub-datasets

Table 0-22: Descriptive statistics for Phase II zinc elements

Zinc Samples	Corrosion Rate [µm/yr]	CSR
Mean	1.9	0.1
Median	0.9	0.1
Standard Deviation	2.5	0.2
Coefficient of Variation	130%	130%
Minimum	0.0	00.
Maximum	13.5	0.9
Confidence Level (95%)	0.6	0
Confidence Range for Mean	(0.0 - 4.4)	(0.0 - 0.3)
Count	75	75

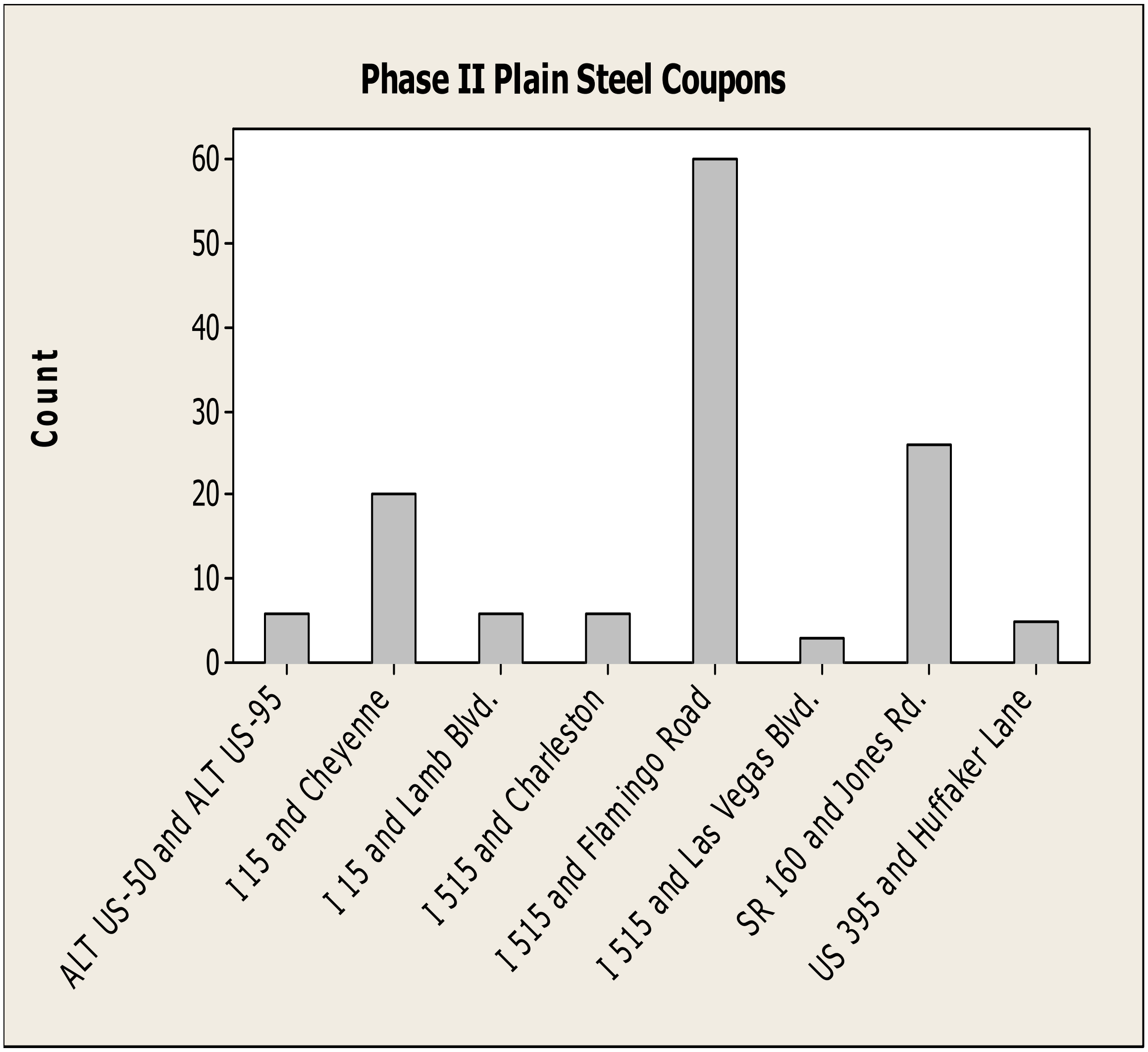

Figure 0-26: Number of plain steel coupons per site for LPR analysis

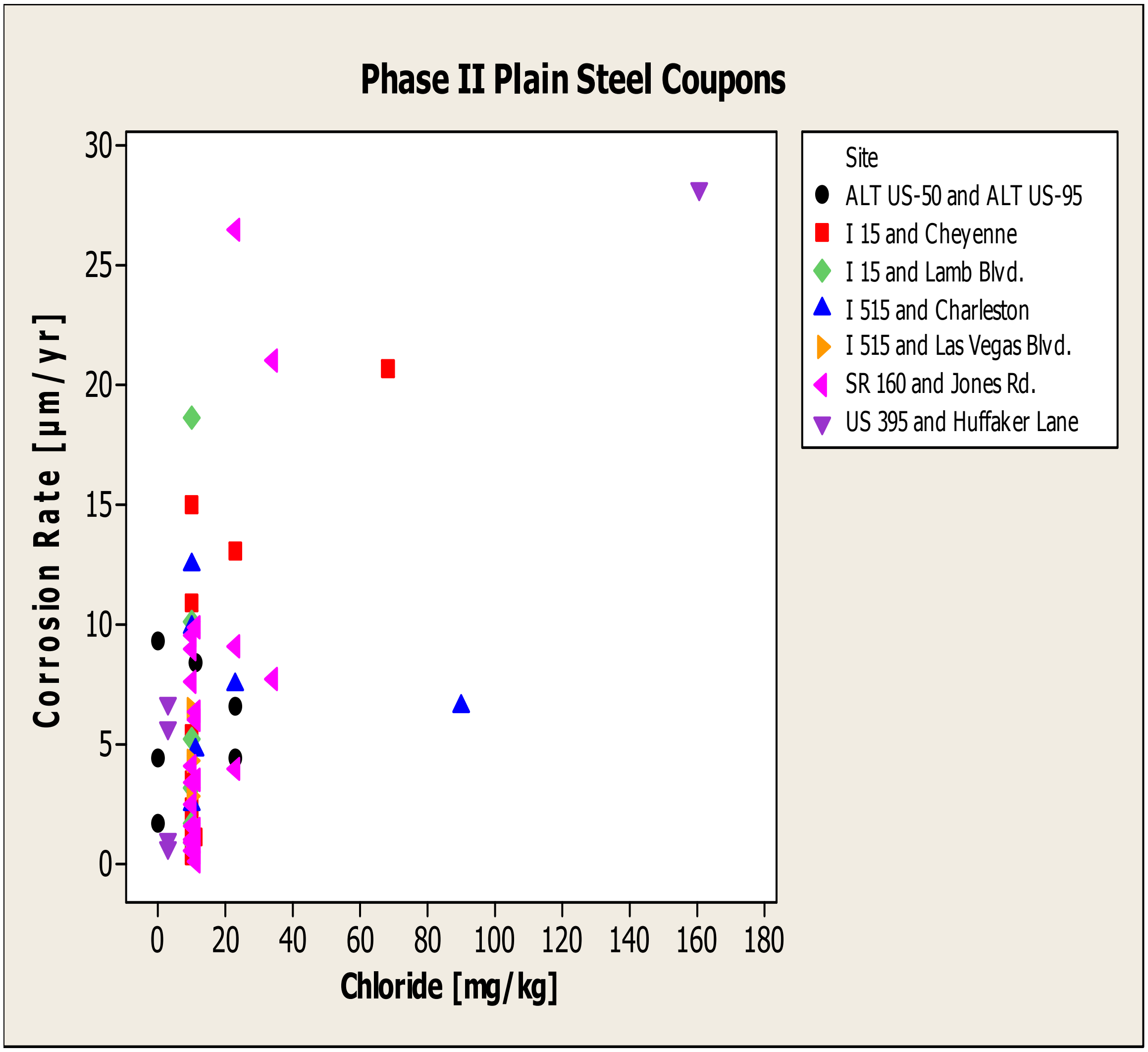

Figure 0-27: Phase II carbon steel coupon LPR corrosion rates with respect to backfill chloride content.

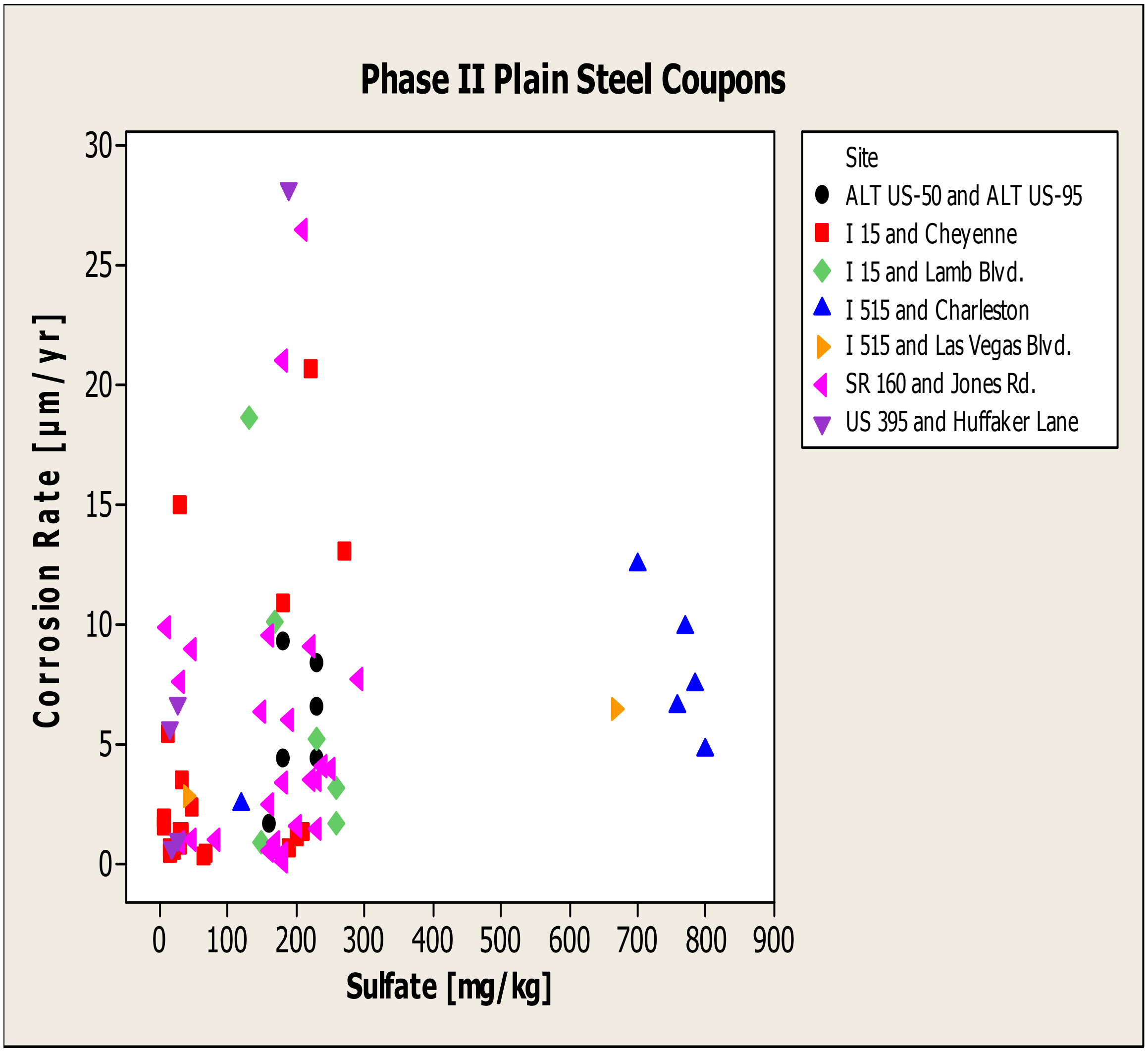

Figure 0-28: Phase II carbon steel coupon LPR corrosion rates with respect to backfill sulfate content.

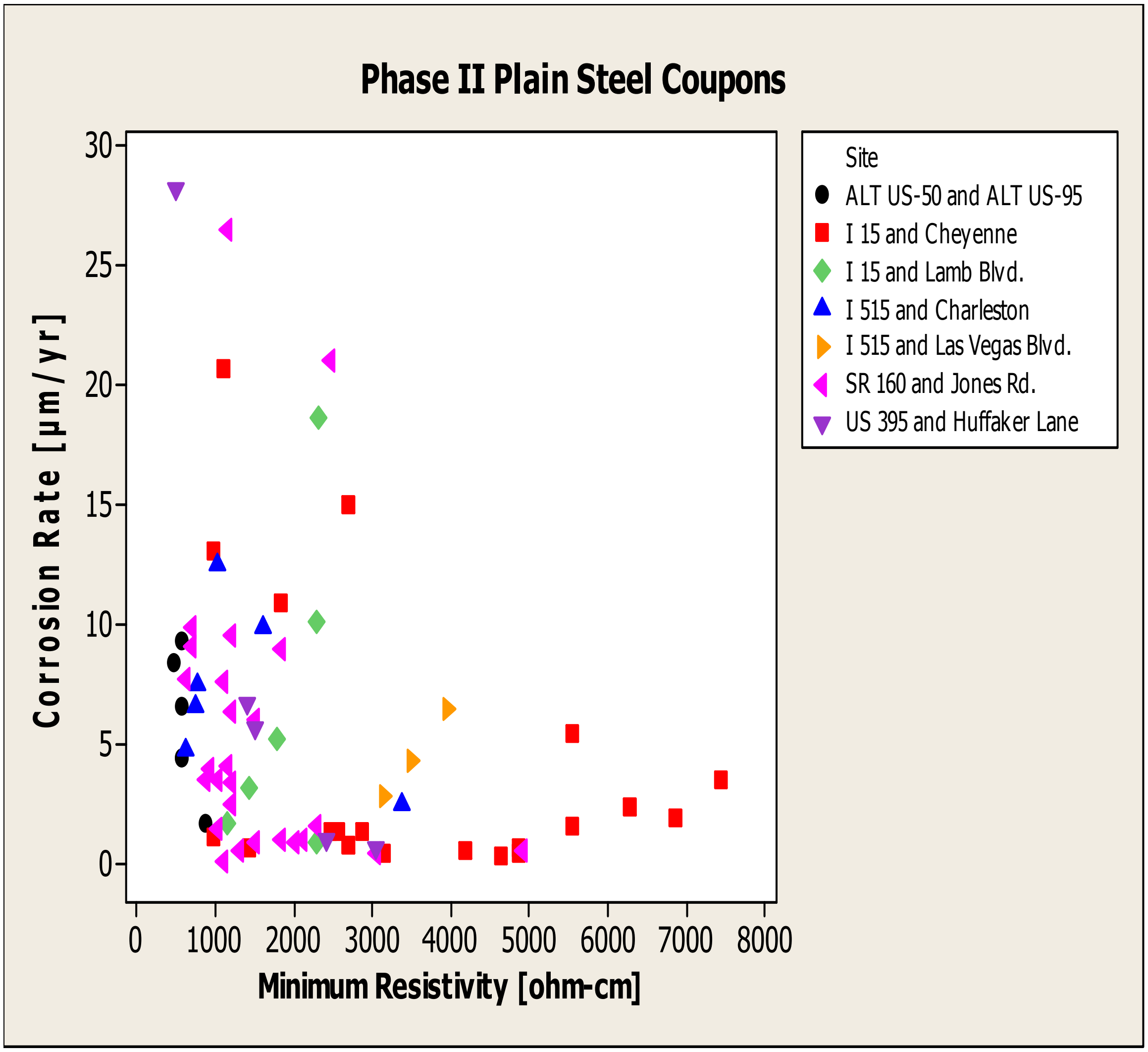

Figure 0-29: Phase II plain steel coupons LPR corrosion rates with respect to the minimum soil resistivity, note the pseudo-random trend

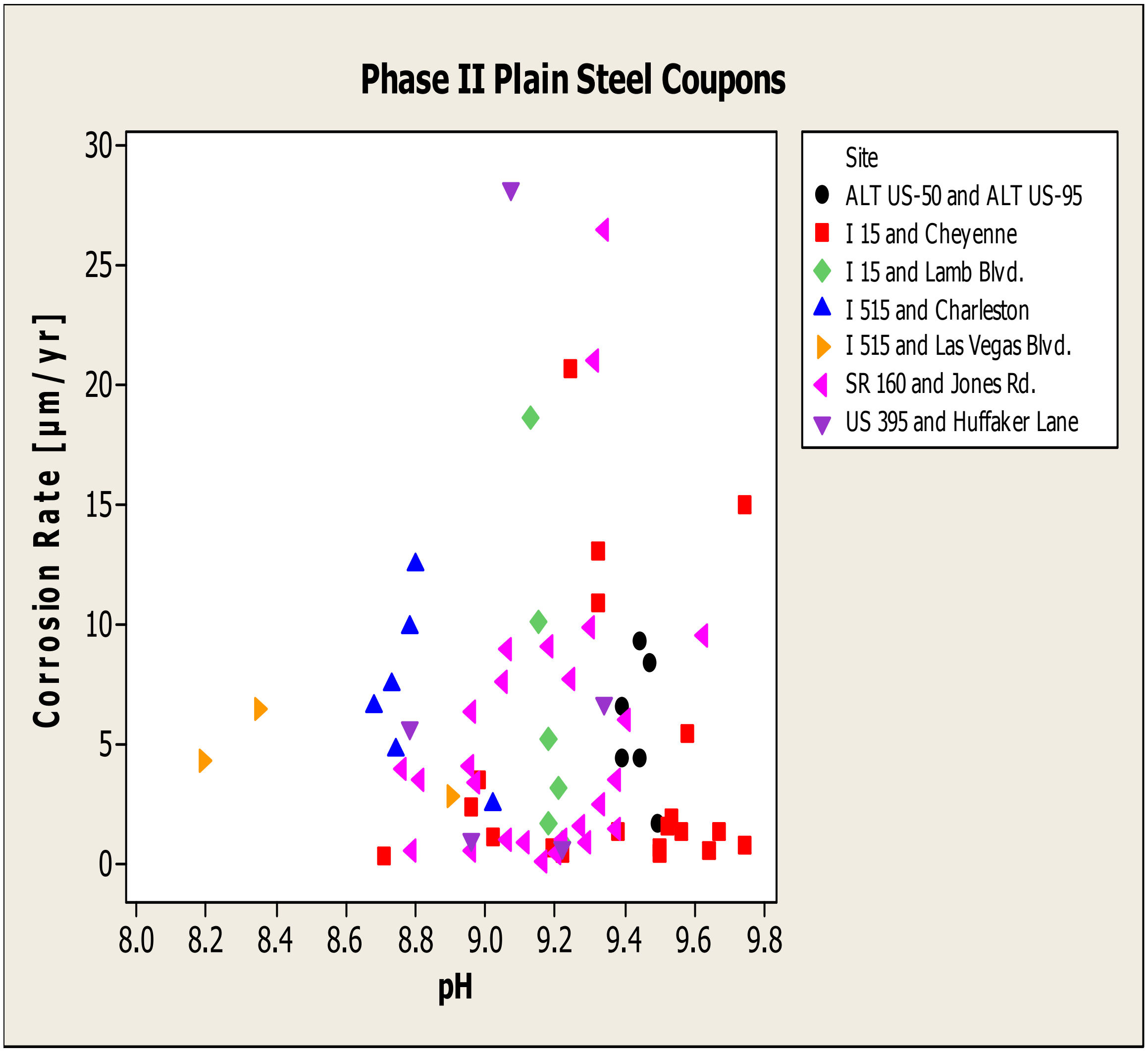

Figure 0-30: Phase II plain steel coupons LPR corrosion rates with respect to backfill pH

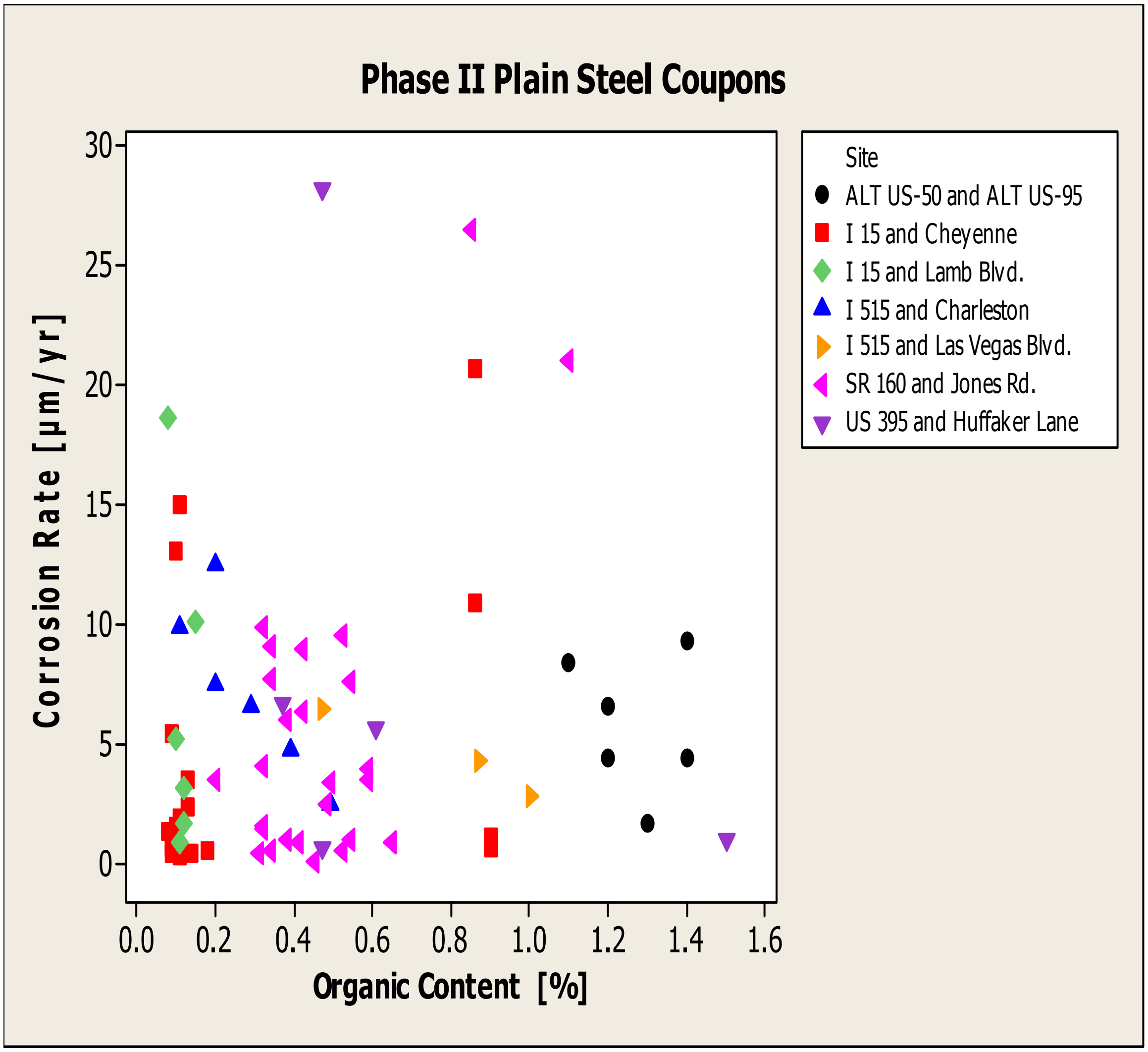

Figure 0-31: Phase II plain steel coupon LPR corrosion rates with respect to organic content of MSE wall backfill

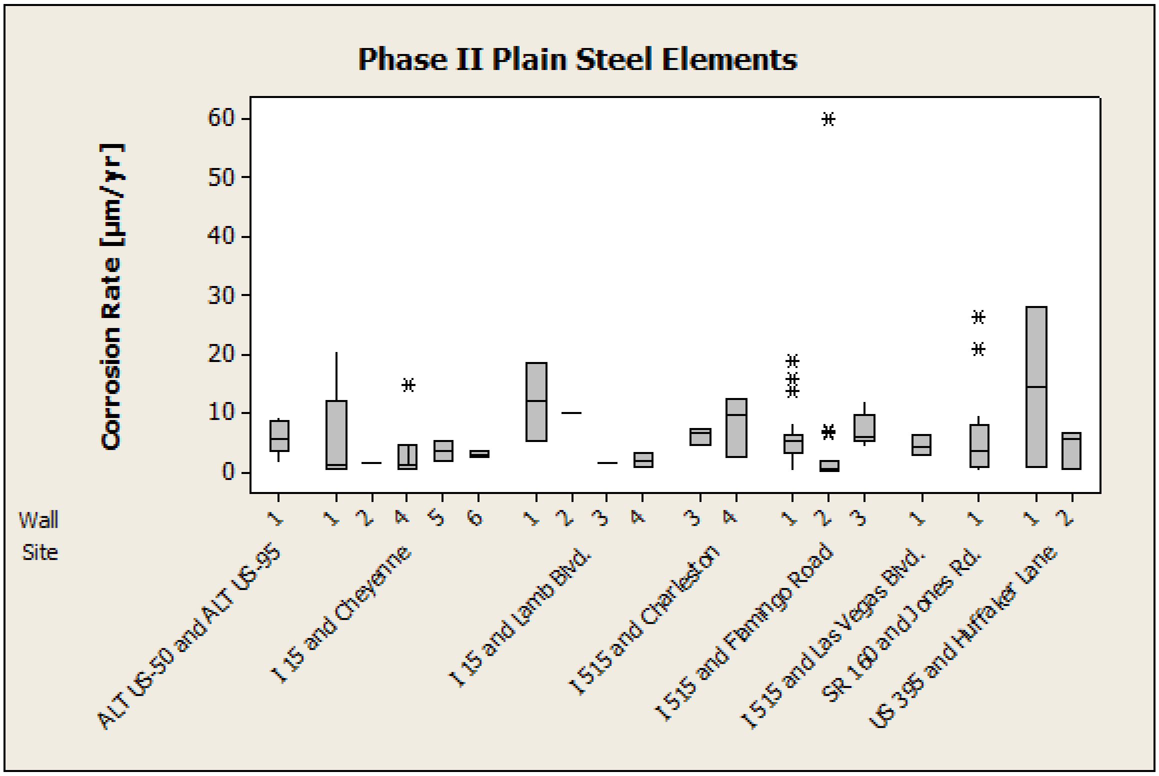

Figure 0-32: Wall specific LPR corrosion rates of Phase II plain steel coupons data, I-515/Flamingo Rd. included as reference

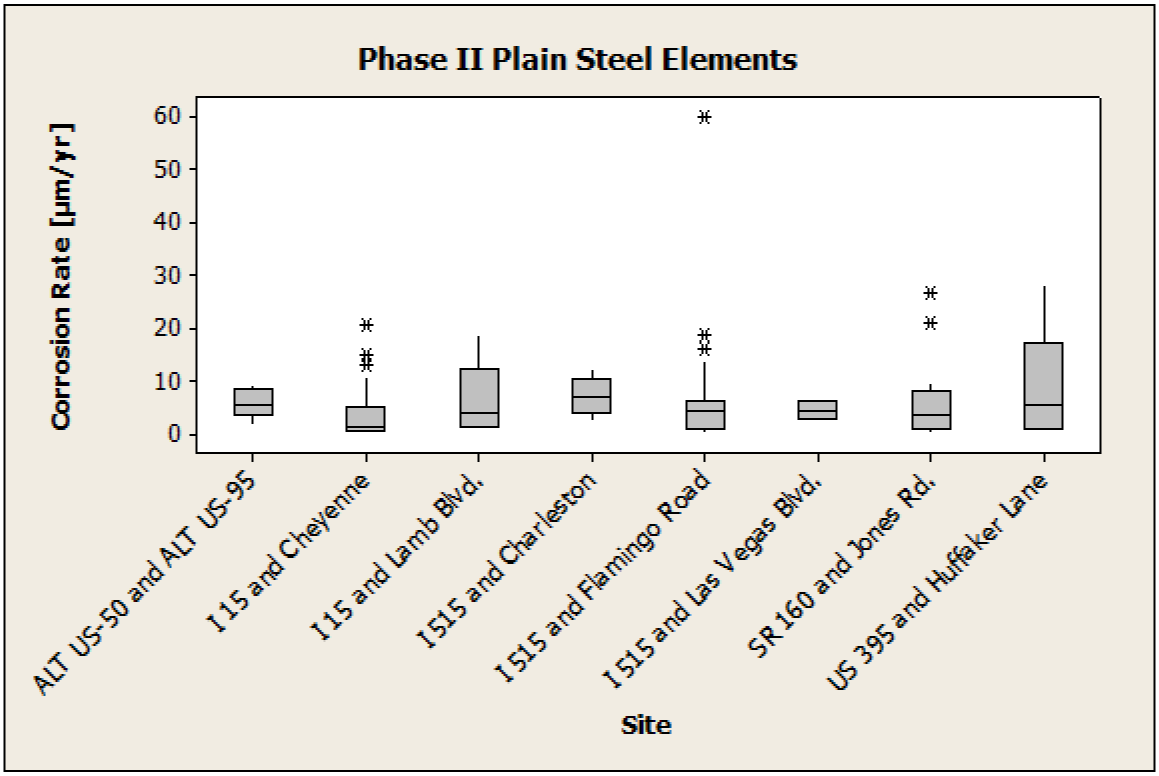

Figure 0-33: Phase II plain steel LPR corrosion rates grouped by site, I-515/Flamingo Rd. included as reference

Table 0-23: Descriptive statistic for Phase II plain steel coupons, LPR corrosion rates, I-515/Flamingo Rd. included as reference

Site	Mean [µm/yr]	Median [µm/yr]	Standard Deviation [µm/yr]	COV	Min. [µm/yr]	Max. [µm/yr]	Confidence Level (95%) [µm/yr]	Confidence Range for Mean [µm/yr]	Count
Alt. US-50 and Alt. US-95	5.7	5.5	2.9	50%	1.6	9.3	2.3	(3.5 - 8.0)	6
I-15 and Cheyenne	4.1	1.3	5.9	143%	0.3	20.6	2.6	(1.5 - 6.7)	20
I-15 and Lamb Blvd.	6.5	4.2	6.8	103%	0.8	18.6	5.4	(1.1 - 12.0)	6
I-515 and Charleston	7.2	7.0	3.6	49%	2.5	12.4	2.8	(4.4 - 10.1)	6
I-515 and Las Vegas Blvd.	4.5	4.3	1.8	40%	2.8	6.4	2.0	(2.5 - 6.5)	3
SR 160 and Jones Rd.	5.4	3.4	6.3	117%	0.0	26.5	2.4	(3.0 - 7.8)	26
US-395 and Huffaker Lane	8.4	5.6	11.4	136%	0.6	28.2	10.0	(0.0 - 18.4)	5
I-515 and Flamingo Rd.	5.5	4.4	8.3	151%	0.0	60.2	2.1	(3.4 - 7.6)	60

Table 0-24: Descriptive statistics for Phase II plain steel coupons corosion severity ratio, I-515/Flamingo Rd. included as reference

Site	Mean	Median	Standard Deviation	COV	Min.	Max.	Confidence Level (95%)	Confidence Range for Mean	Count
Alt. US-50 and Alt. US-95	0.1	0.1	0.1	50%	0.0	0.2	0.1	(0.1 - 0.2)	6
I-15 and Cheyenne	0.1	0.0	0.1	143%	0.0	0.5	0.1	(0.0 - 0.1)	20
I-15 and Lamb Blvd.	0.1	0.1	0.2	103%	0.0	0.4	0.1	(0.0 - 0.3)	6
I-515 and Charleston	0.2	0.2	0.1	49%	0.1	0.3	0.1	(0.1 - 0.2)	6
I-515 and Las Vegas Blvd.	0.1	0.1	0.0	40%	0.1	0.1	0.0	(0.1 - 0.1)	3
SR 160 and Jones Rd.	0.1	0.1	0.1	117%	0.0	0.6	0.1	(0.1 - 0.2)	26
US-395 and Huffaker Lane	0.2	0.1	0.3	136%	0.0	0.6	0.2	(0.0 - 0.4)	5
I-515 and Flamingo Rd.	0.5	0.4	0.7	151%	0.0	5.0	0.2	(0.3 - 0.6)	60

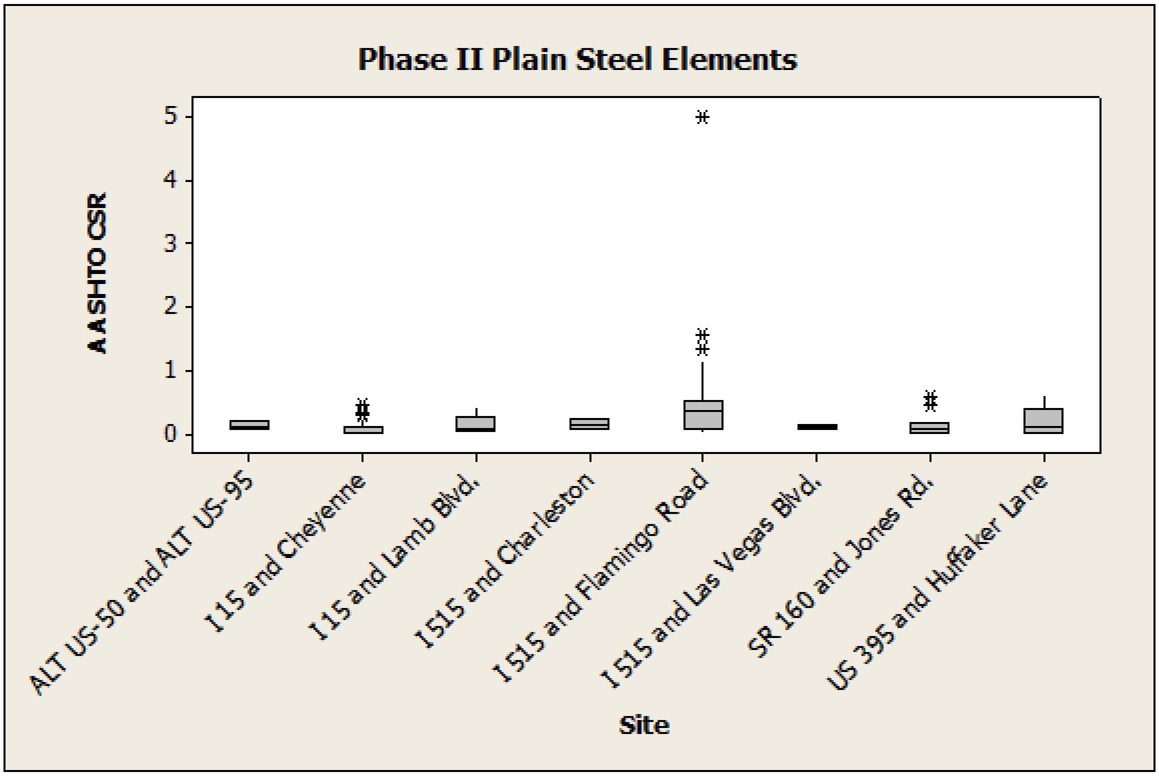

Figure 0-34: Phase II plain steel coupon AASHTO corrosion severity ratios based on site wide information, I-515/Flamingo Rd. included as reference

Table 3-1: Descriptive statistics for site wide 75-year AASHTO equivalent metal loss model (AEML) CSR values

Site	Mean	Median	Standard Deviation	Coefficient of Variation	Minimum	Maximum	Confidence Level (95%)	Confidence Range for Mean	Count
Alt. US-50 and Alt. US-95	0.32	0.10	0.34	105%	0.04	0.99	0.16	(0.17 - 0.48)	18
I-15 and Cheyenne Blvd.	0.58	0.11	1.14	195%	0.02	5.32	0.32	(0.27 - 0.90)	50
I-15 and Lamb Blvd.	0.61	0.48	0.50	81%	0.07	1.75	0.28	(0.33 - 0.89)	12
I-515 and Charleston Blvd.	0.54	0.50	0.36	67%	0.05	1.17	0.20	(0.34 - 0.74)	12
I-515 and Las Vegas Blvd.	0.21	0.13	0.20	99%	0.03	0.60	0.14	(0.06 - 0.35)	8
SR 160 and Jones Rd.	0.29	0.12	0.43	150%	0.00	2.50	0.11	(0.18 - 0.40)	64
US-395 and Huffaker Lane	0.36	0.08	0.72	202%	0.01	2.66	0.39	(0.00 - 0.75)	13

Table 3-2: Equivalent 75-year uniform corrosion rates based on AEML CSR values of all element types per site, >10.6 µm/yr indicates reduction of service life

Site	AEML-CSR Corrosion Rates [µm/yr]					
	Mean	Median	Minimum	Maximum	Lower Mean Bound	Upper Mean Bound
Alt. US-50 and Alt. US-95	3.4	1.1	0.4	10.5	1.7	5.0
I-15 and Cheyenne Blvd.	6.2	1.2	0.2	56.4	2.8	9.5
I-15 and Lamb Blvd.	6.5	5.1	0.8	18.6	3.5	9.4
I-515 and Charleston Blvd.	5.7	5.3	0.5	12.4	3.6	7.9
I-515 and Las Vegas Blvd.	2.2	1.3	0.3	6.4	0.7	3.7
SR 160 and Jones Rd.	3.1	1.3	0.0	26.5	1.9	4.2
US-395 and Huffaker Lane	3.8	0.9	0.2	28.2	0.0	7.9

Table 3-3: Service life estimates using mean equivalent 75-year uniform corrosion rates

Site	Service Life [yr]			
	Mean	Median	Upper Bound	Lower Bound
Alt. US-50 and Alt. US-95	233.6	721.8	453.9	157.3
I-15 and Cheyenne Blvd.	128.7	678.1	280.8	83.5
I-15 and Lamb Blvd.	122.6	157.2	226.6	84.1
I-515 and Charleston Blvd.	138.7	151.2	222.5	100.8
I-515 and Las Vegas Blvd.	362.5	591.9	1152.8	215.1
SR 160 and Jones Rd.	258.9	635.2	410.0	189.2
US-395 and Huffaker Lane	210.4	882.2	75.0	100.3

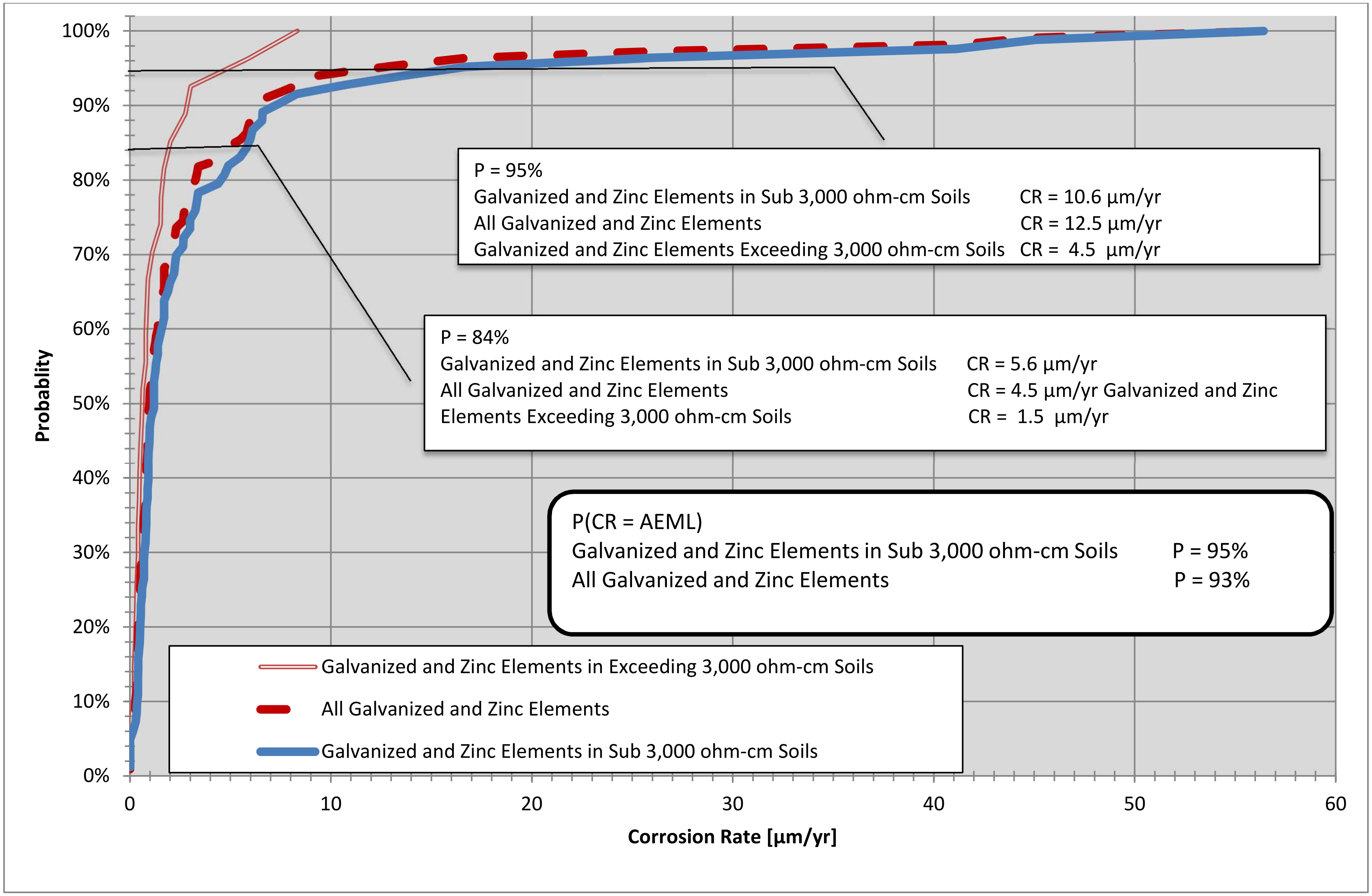

Figure 3-1: Cumulative distribution function (CDF) for the probability that a corrosion rates will exceed a value based on relevant metal types for galvanized in-service soil reinforcements, using all Phase II LPR data

Table 3-4: Probability table for corrosion rates based on the metal type relevant CDF (Fig. 5-1), corrosion rates are based on uniform metal loss and are a function of minimum soil resistivity

	P		
	0.5	0.84	0.95
Galvanized and Zinc Elements in Sub 3,000 ohm-cm Soils [μm/yr]	1.2	6.9	16.5
All Galvanized and Zinc Elements [μm/yr]	1.0	4.8	10.7
Galvanized and Zinc Elements in Exceeding 3,000 ohm-cm Soils [μm/yr]	0.6	1.9	4.9

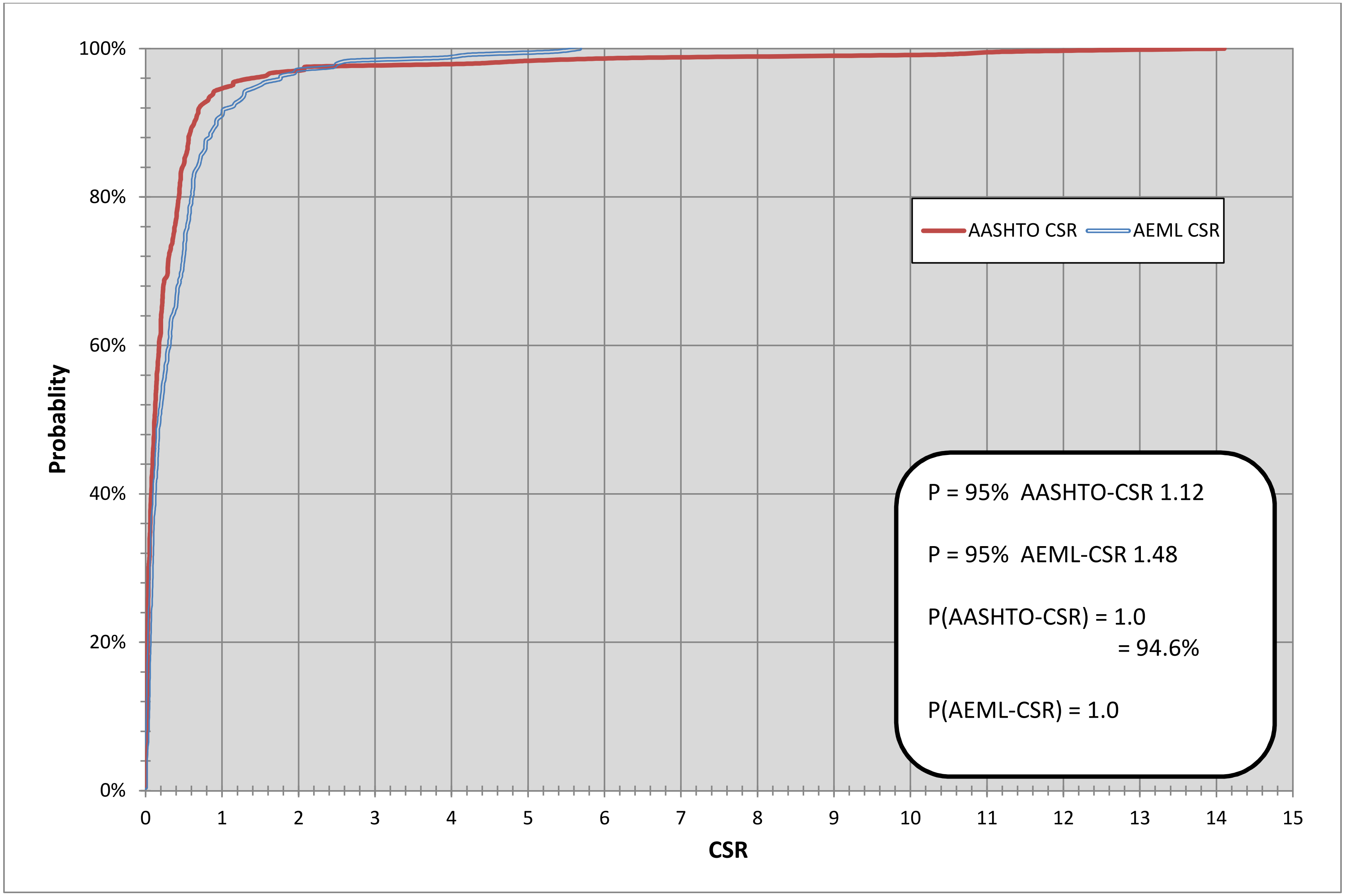

Figure 3-2: Corrosion severity ratio (CSR) CDF curves based on entire Phase II dataset, inflection point CSR ≈ 2.2

Table 3-5: Severity rating and rankings for Phase II site based on electrochemical backfill testing

Site	Severity Rating Based on Mean Values				*Median Severity Rating*	Severity Rating of Resistivity COV	Severity Rating of Minimum Resistivity	*Composite Rating*	**Soil Severity Rank[1]**
	Chloride	Sulfate	Resistivity	Organic Content					
US 395 and Huffaker Lane	1	7	4	1	*2.5*	3	3	*8.5*	**3**
Alternate US 95 and US 50	4	3	1	2	*2.5*	2	2	*6.5*	**1**
I 15 and Cheyenne Blvd.	2	6	6	7	*6.0*	1	1	*8.0*	**2**
I 15 and Lamb Blvd.	6	4	5	6	*5.5*	6	6	*17.5*	**6**
I 515 and Charleston Blvd.	3	2	2	4	*2.5*	4	4	*10.5*	**4**
I 515 and Las Vegas Blvd.	7	1	7	3	*5.0*	7	7	*19.0*	**7**
SR 160 and Jones Rd.	5	5	3	5	*5.0*	5	5	*15.0*	**5**

[1] 1 = Most severe conditions

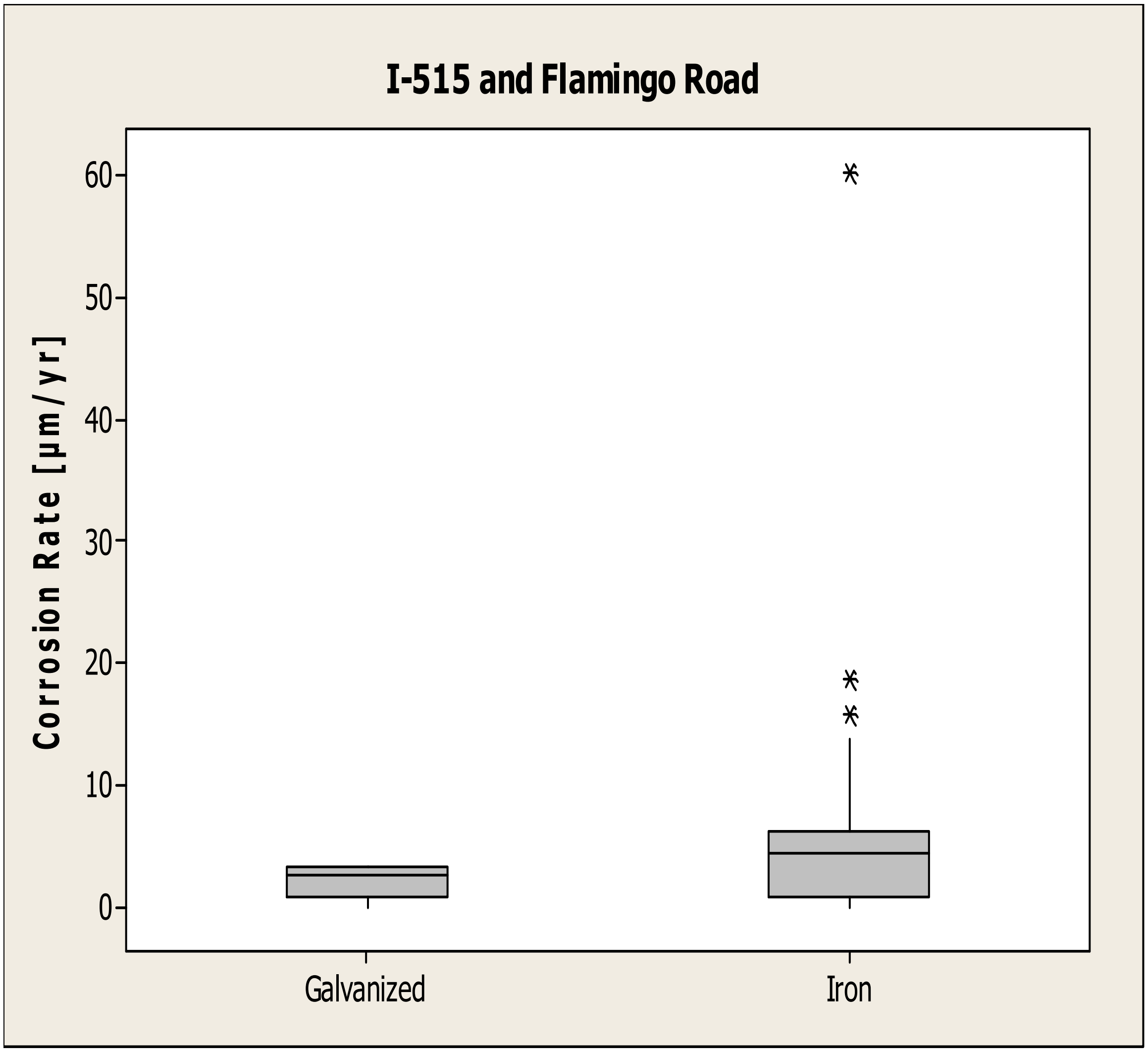

Figure 3-3: Phase II LPR corrosion rate box-plots by element t for I-515/ Flamingo Rd.

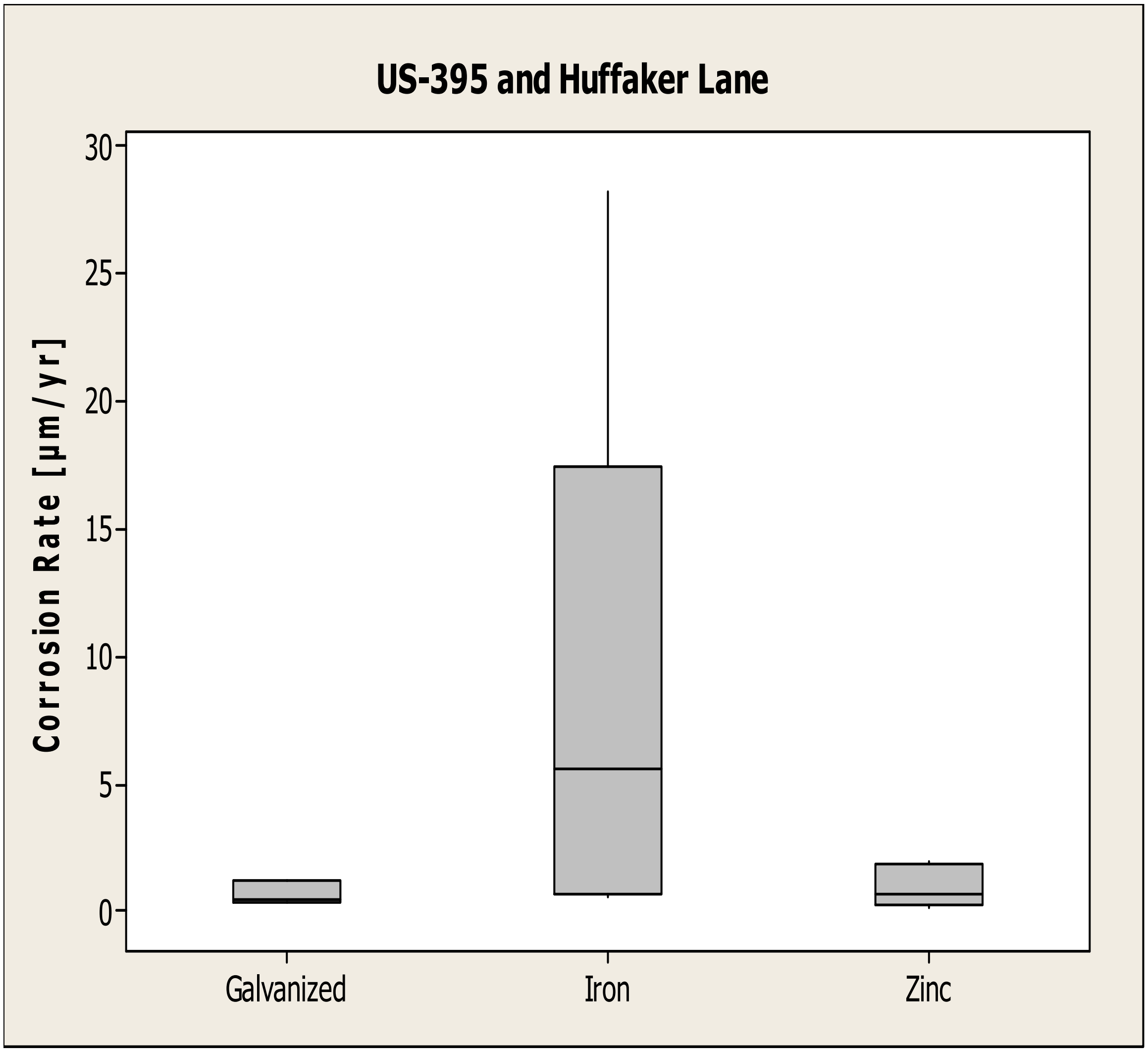

Figure 3-4: Phase II LPR corrosion rate box-plots by element type for US-395 and Huffaker Lane

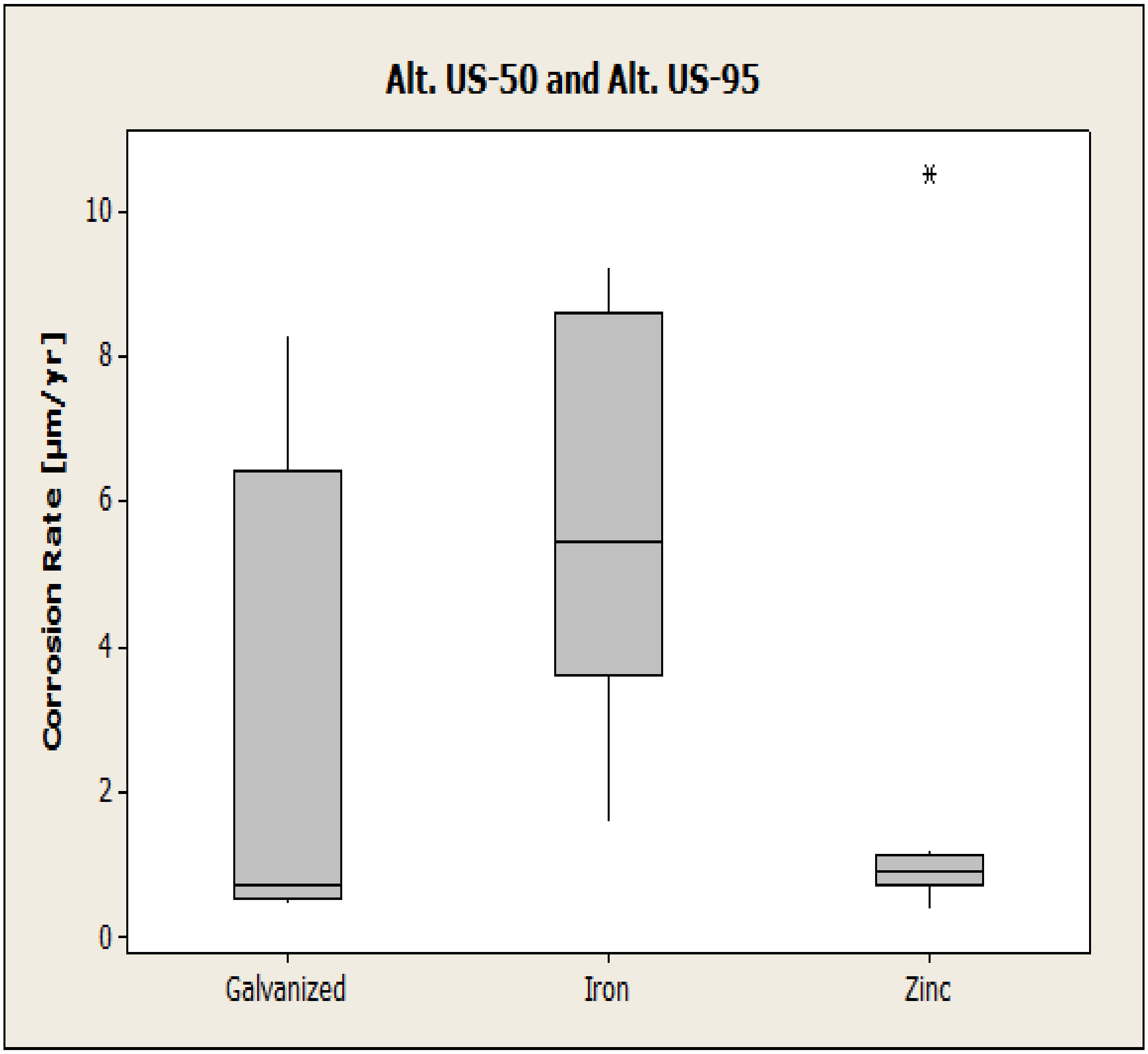

Figure 3-5: Phase II LPR corrosion rate box-plots by element type for Alt. US 50 and Alt. US 95

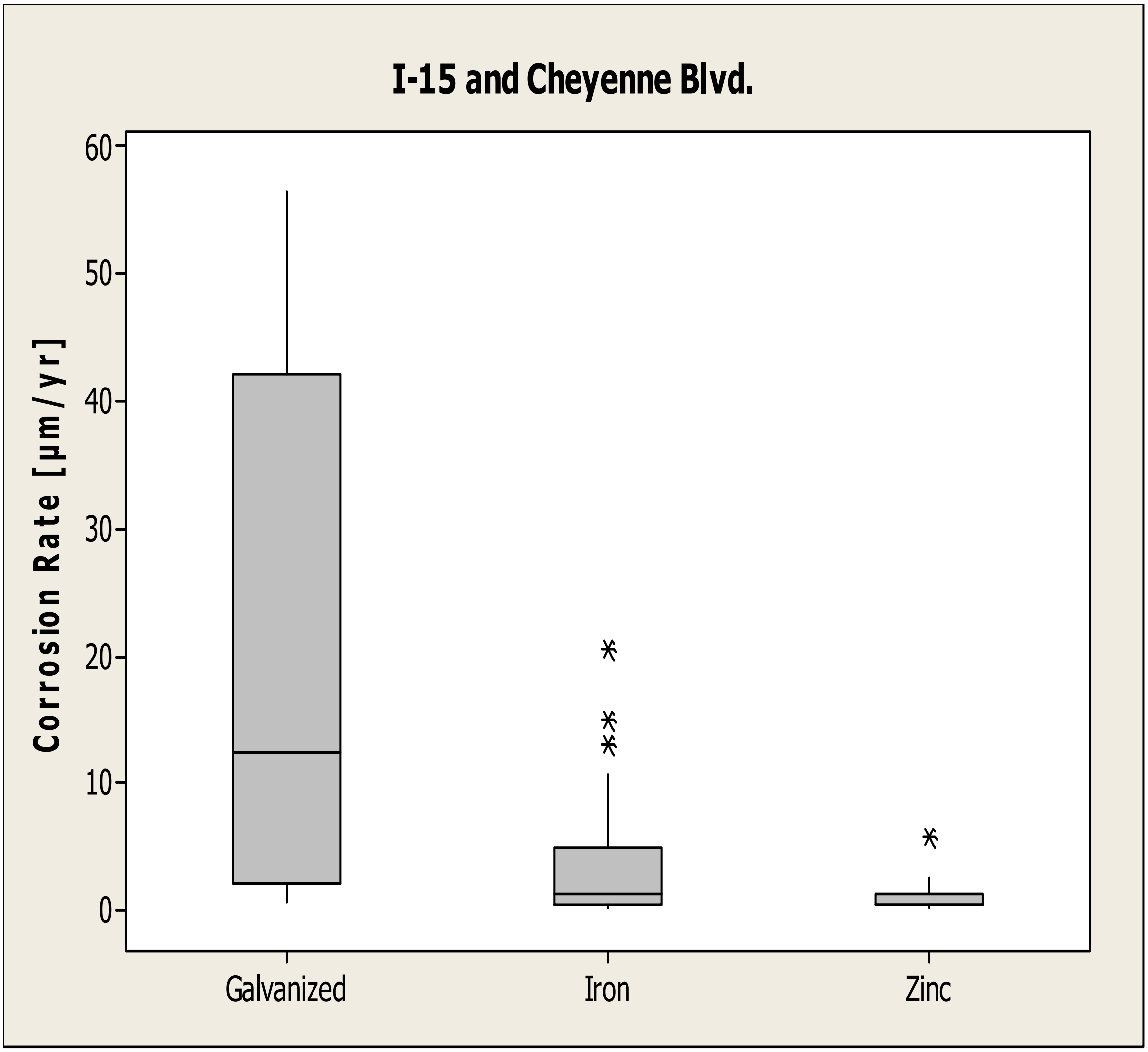

Figure 3-6: Phase II LPR corrosion rate box-plots by element type for I-15 and Cheyenne Blvd.

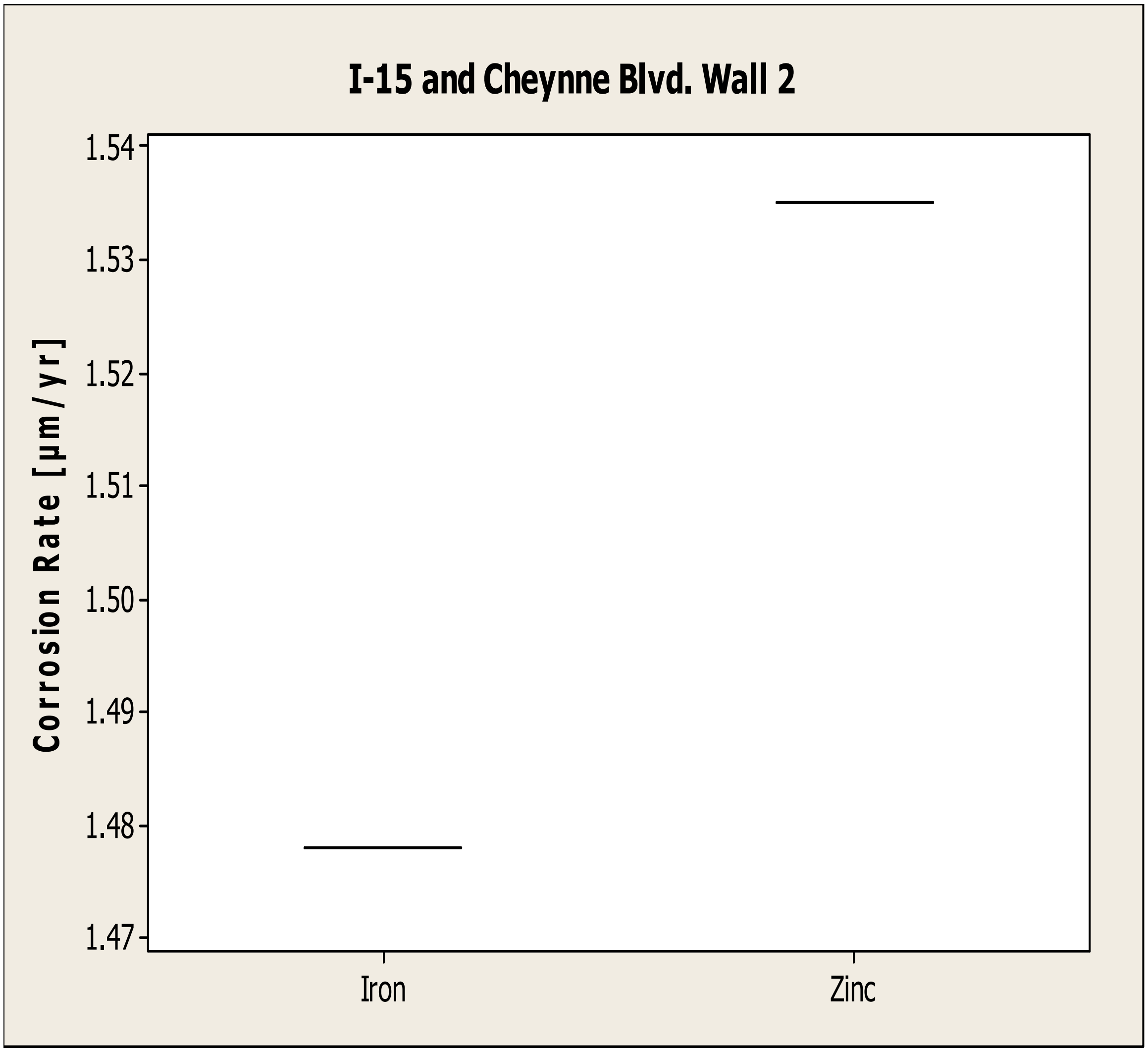

Figure 3-7: Phase II LPR corrosion rate box-plots by element type for I-15/ Cheyenne Blvd. Wall 2, figure represent only one data point per type

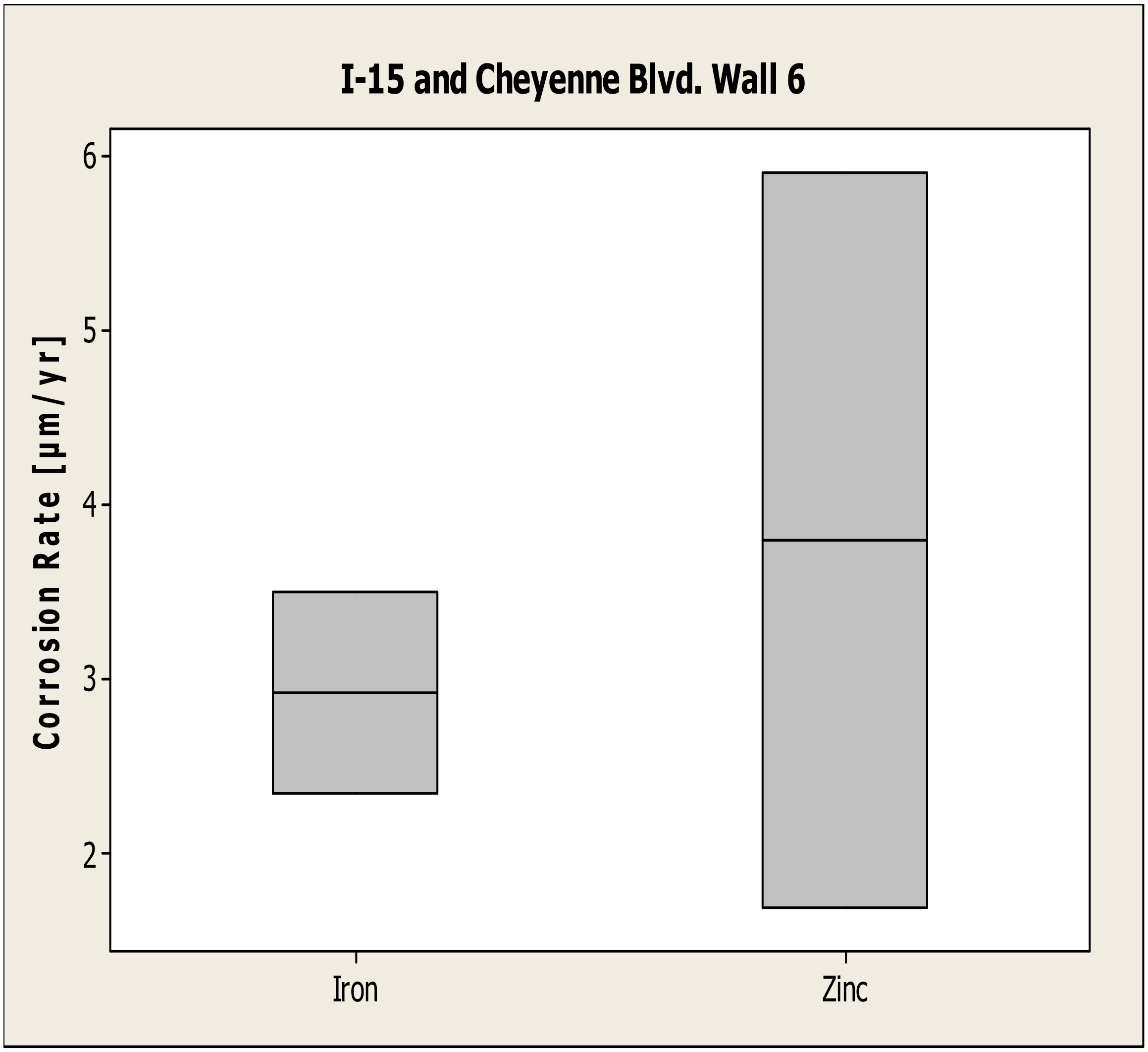

Figure 3-8: Phase II LPR corrosion rate box-plots by element type for I-15/ Cheyenne Blvd. Wall 6

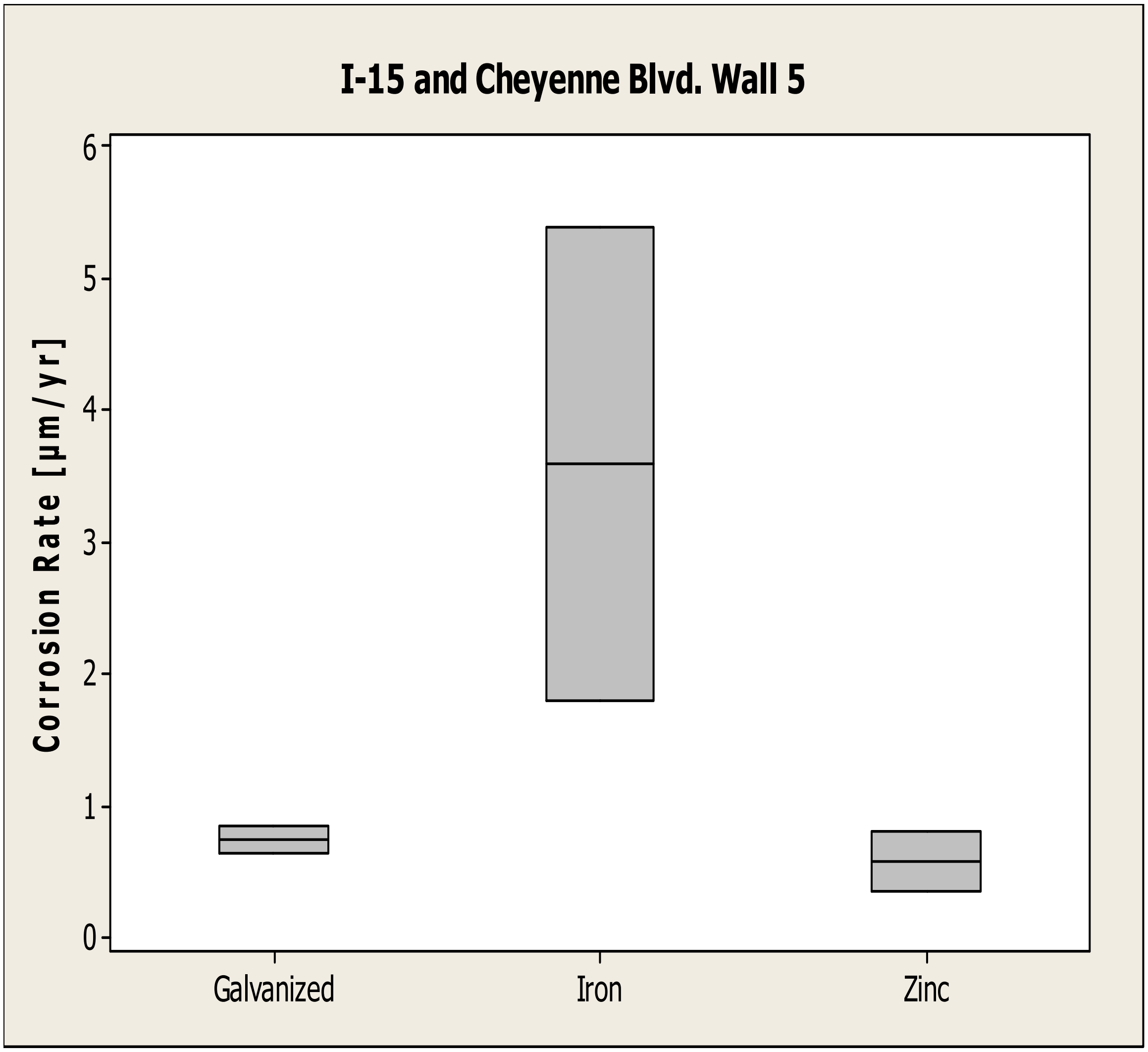

Figure 3-9: Phase II LPR corrosion rate box-plots by element type for I-15/ Cheyenne Blvd. Wall 5

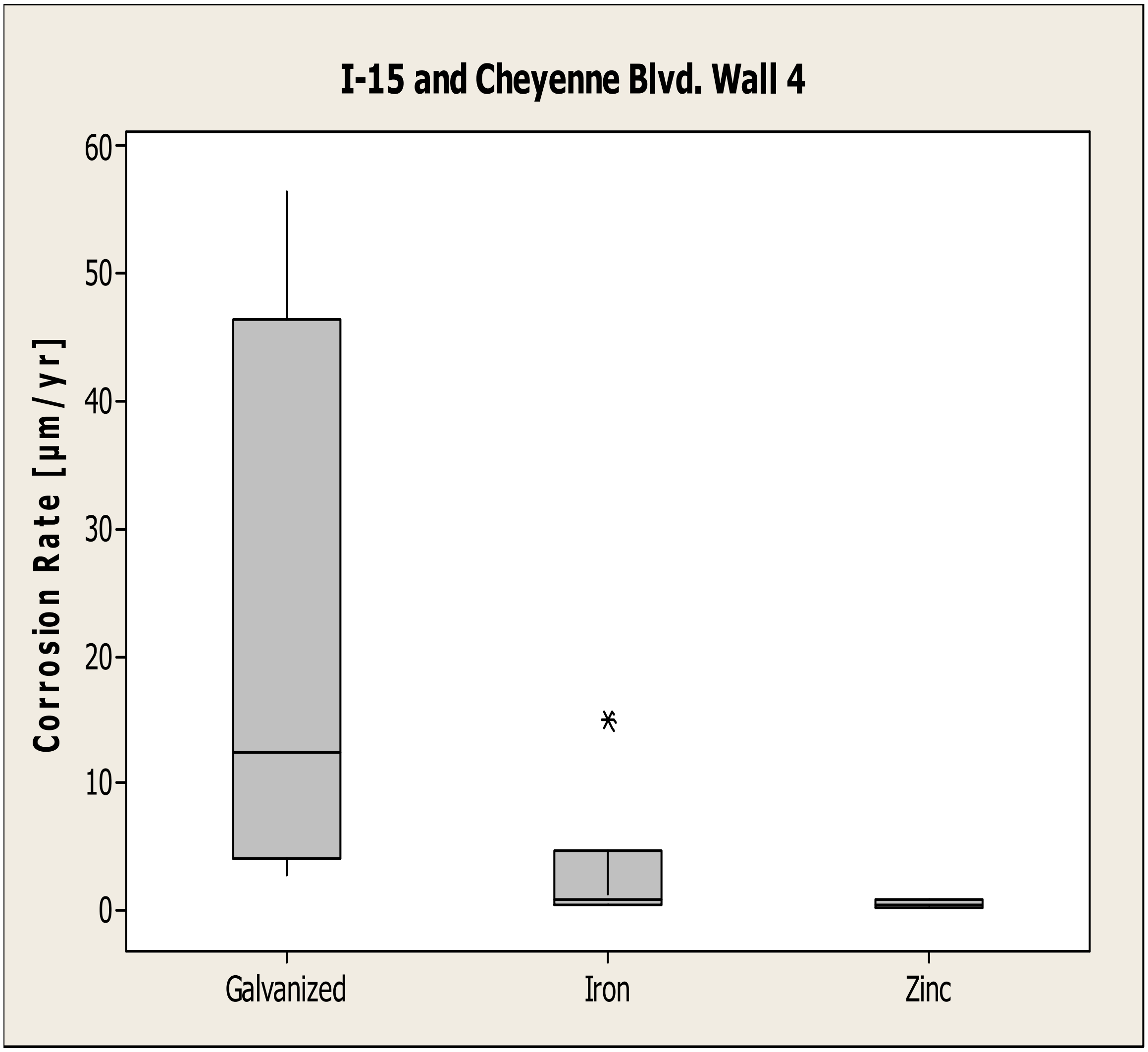

Figure 3-10: Phase II LPR corrosion rate box-plots by element type for I-15/ Cheyenne Blvd. Wall 4

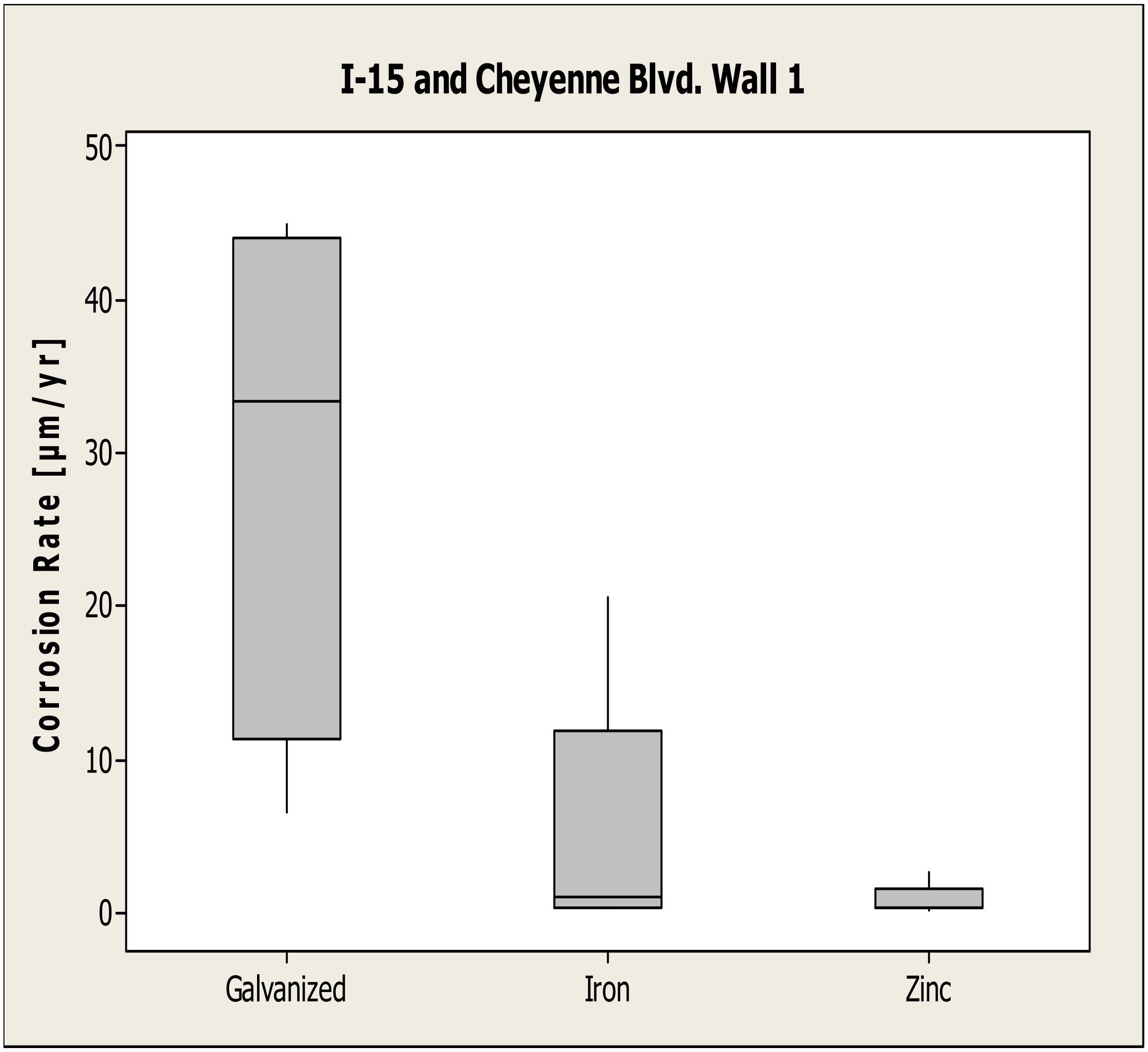

Figure 3-11: Phase II LPR corrosion rate box-plots by element type for I-15/ Cheyenne Blvd. Wall 1

Figure 3-12: In-service galvanized soil reinforcement for I-15/ Cheyenne Blvd. Wall 1, 15 years old. (W1-SA-E1)

Figure 3-13: In-service galvanized soil reinforcement for I-15/ Cheyenne Blvd. Wall 1, 15 years old. (W1-SA-E2)

Figure 3-14: In-service galvanized soil reinforcement for I-15/ Cheyenne Blvd. Wall 1, 15 years old. (W1-SD-E1)

Figure 3-15: In-service galvanized soil reinforcement for I-15/ Cheyenne Blvd. Wall 1, 15 years old. (W1-SD-E2)

Figure 3-16: In-service galvanized soil reinforcement for I-15/ Cheyenne Blvd. Wall 1, 15 years old. (W2-SA-E1)

Figure 3-17: In-service galvanized soil reinforcement for I-15/ Cheyenne Blvd. Wall 2, 15 years old. (W2-SA-E2)

Figure 3-18: I-15/ Lamb Blvd. in-service reinforcement WA-SA-E1

Figure 3-19: I-15/ Lamb Blvd. in-service reinforcement WA-SA-E2

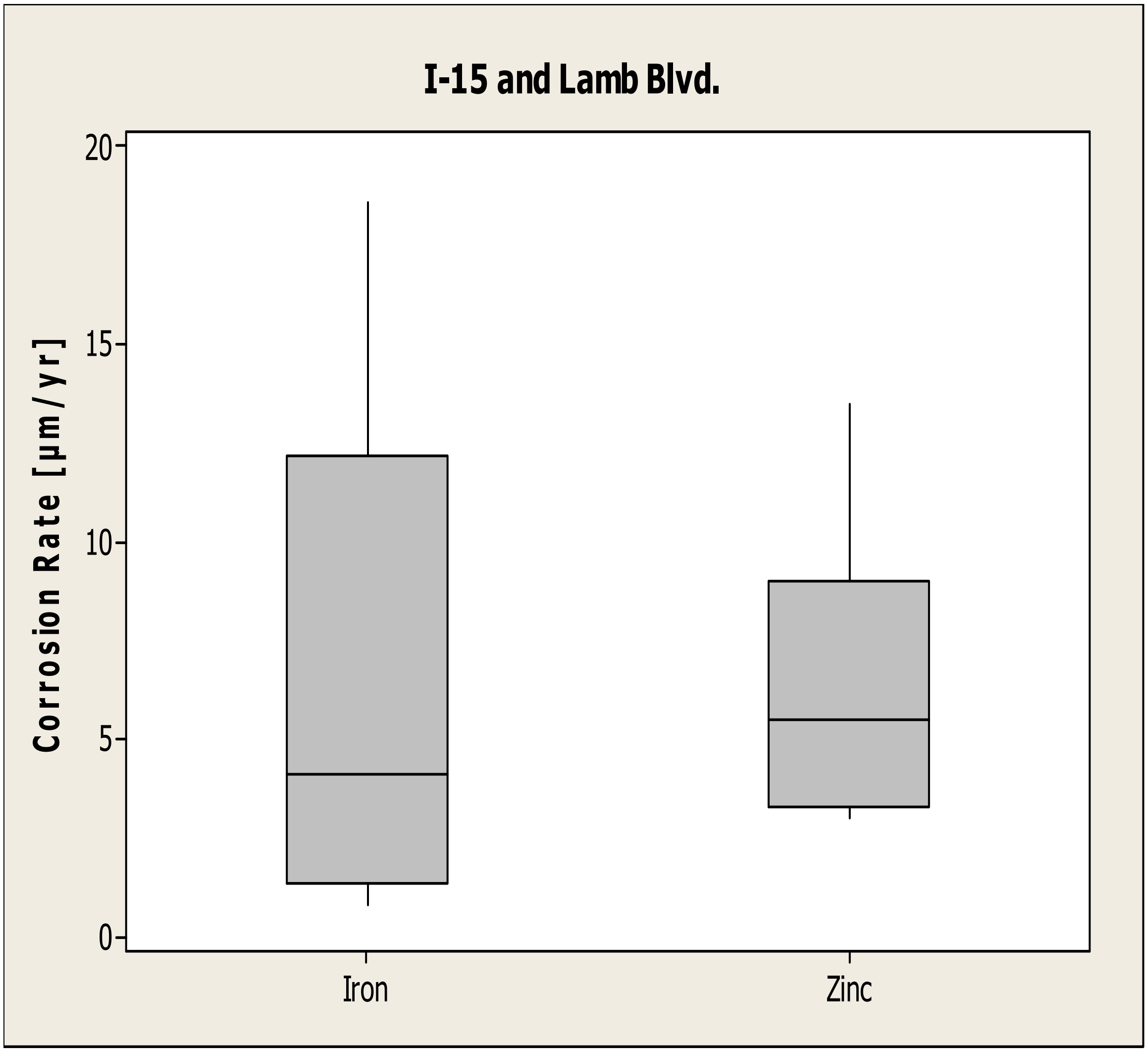

Figure 3-20: Phase II LPR corrosion rate box-plots by element type for I-15/ Lamb Blvd.

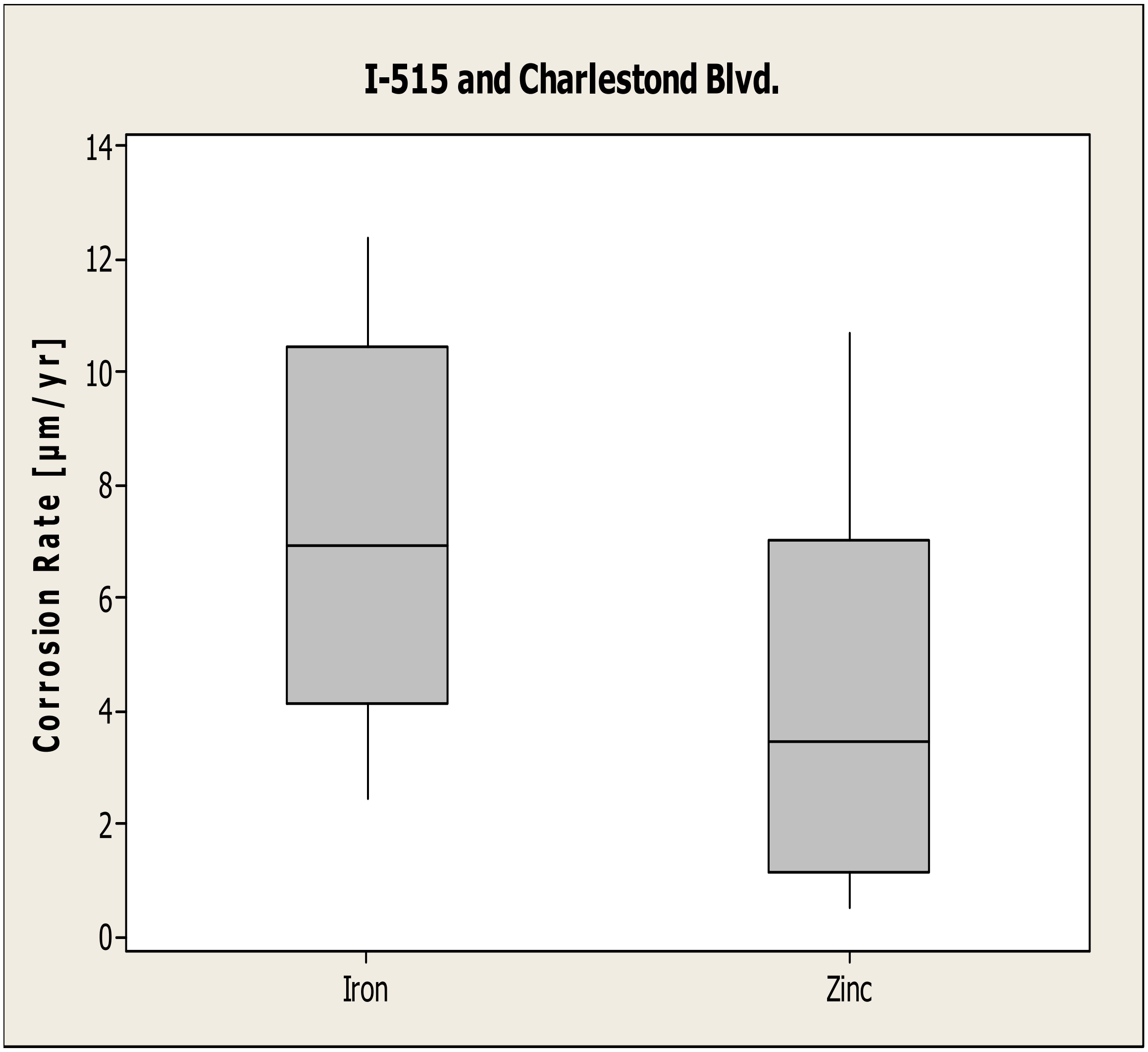

Figure 3-21: Phase II LPR corrosion rate box-plots by element type for I-515/ Charleston Blvd.

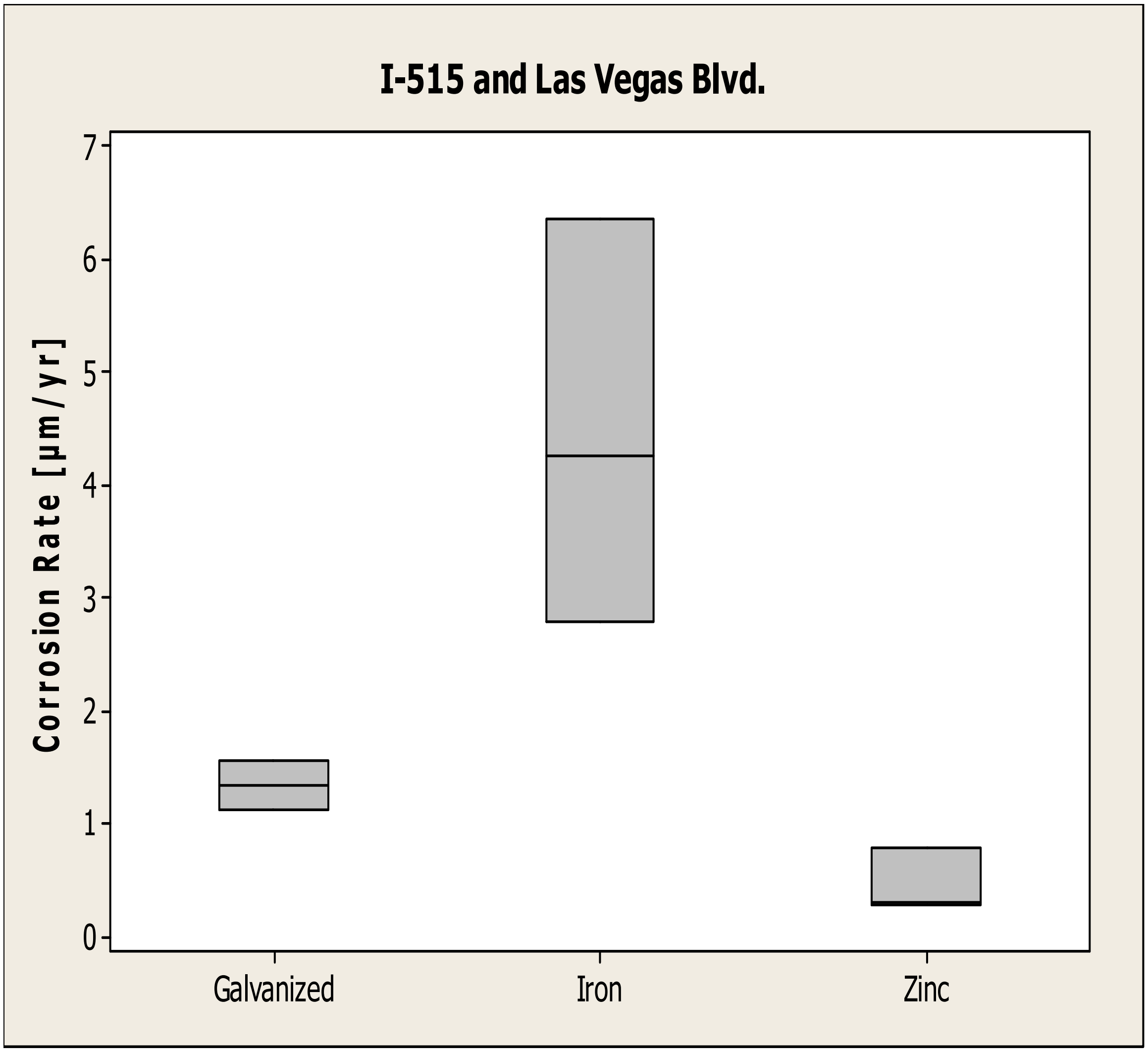

Figure 3-22: Phase II LPR corrosion rate box-plots by element type for I-515/ Las Vegas Blvd.

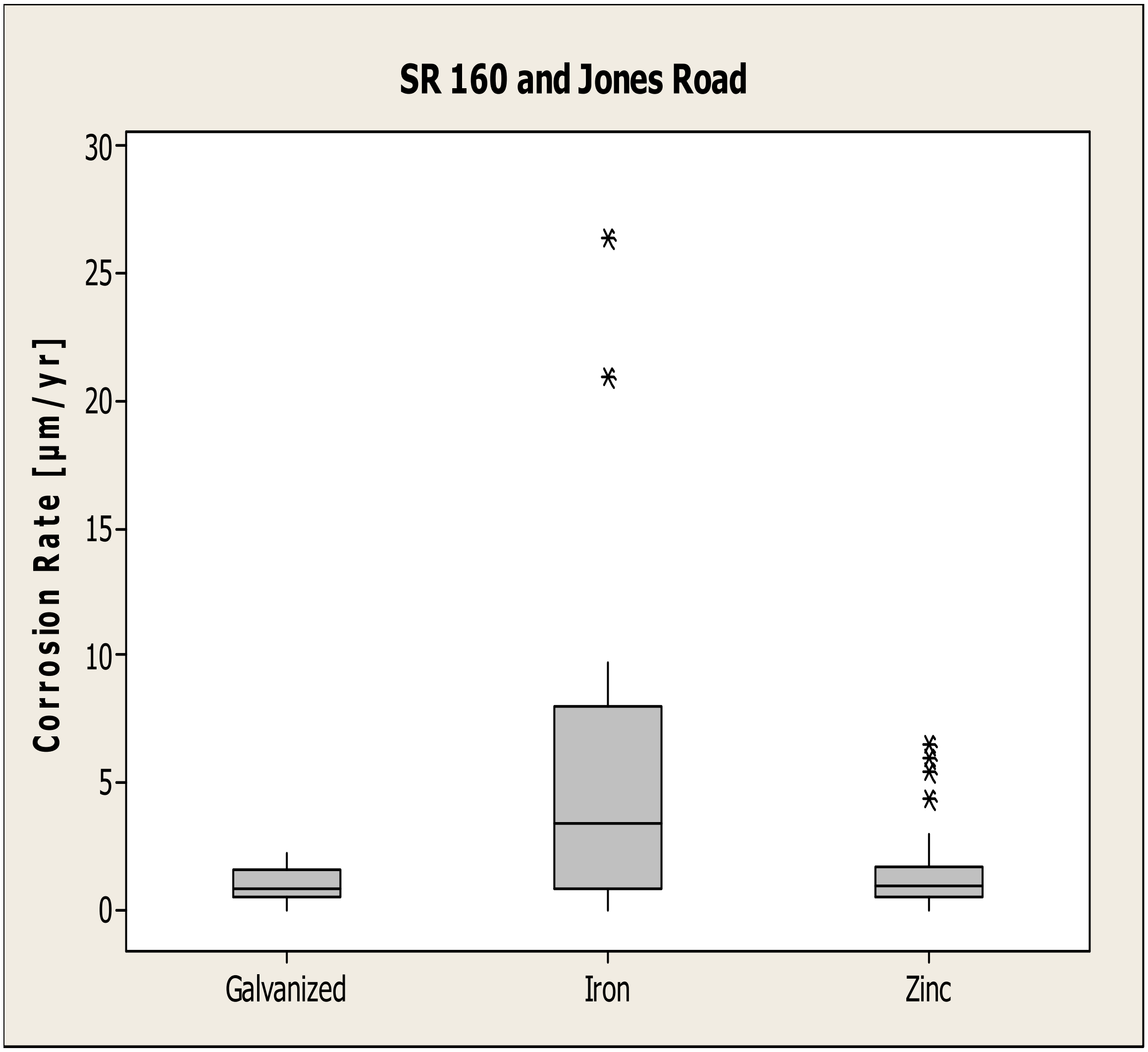

Figure 3-23: Phase II LPR corrosion rate box-plots by element type for SR 160 and Jones

Table 3-6: Corrosion Severity Ranking Based on Corrosion Rates of All Metal Types

Site	In-service Elements		Steel Coupons		Galvanized Coupons		*Composite Median*	**Corrosion Severity Rank**[1]
	Mean Corrosion Rate Rating	Corrosion Rate COV Rating	Mean Corrosion Rate Rating	Corrosion Rate COV Rating	Mean Corrosion Rate Rating	Corrosion Rate COV Rating		
US 395 and Huffaker Lane	6	5	1	2	5	4	*4.5*	**5**
ALT US-50 and ALT US-95	2	1	4	6	6	7	*5.0*	**6**
I 15 and Cheyenne	1	2	7	1	4	1	*1.5*	**1**
I 15 and Lamb Blvd.	3	3	3	4	1	5	*3.0*	**2**
I 515 and Charleston	3	3	2	5	2	3	*3.0*	**3**
I 515 and Las Vegas Blvd.	4	6	6	7	7	6	*6.0*	**7**
SR 160 and Jones Rd.	5	4	5	3	3	2	*3.5*	**4**

[1] 1 = Most severe conditions

Table 4-1: Phase II soil severity rankings based on mean values and resistvity discriptors

Site	Severity Rating Based on Mean Values				*Median Severity Rating*	Severity Rating of Resistivity COV	Severity Rating of Minimum Resistivity	*Composite Rating*	**Soil Severity Rank**[1]
	Chloride	Sulfate	Resistivity	Organic Content					
US 395 and Huffaker Lane	1	7	4	1	*2.5*	3	3	*8.5*	**3**
Alternate US 95 and US 50	4	3	1	2	*2.5*	2	2	*6.5*	**1**
I 15 and Cheyenne Blvd.	2	6	6	7	*6.0*	1	1	*8.0*	**2**
I 15 and Lamb Blvd.	6	4	5	6	*5.5*	6	6	*17.5*	**6**
I 515 and Charleston Blvd.	3	2	2	4	*2.5*	4	4	*10.5*	**4**
I 515 and Las Vegas Blvd.	7	1	7	3	*5.0*	7	7	*19.0*	**7**
SR 160 and Jones Rd.	5	5	3	5	*5.0*	5	5	*15.0*	**5**

[1] 1 = Most severe conditions

Table 4-2: Phase II Corrosion rate severity rankings

Site	In-service Elements		Steel Coupons		Galvanized Coupons		*Composite Median*	**Corrosion Severity Rank[1]**
	Mean Corrosion Rate Rating	Corrosion Rate COV Rating	Mean Corrosion Rate Rating	Corrosion Rate COV Rating	Mean Corrosion Rate Rating	Corrosion Rate COV Rating		
US 395 and Huffaker Lane	6	5	1	2	5	4	*4.5*	**5**
ALT US-50 and ALT US-95	2	1	4	6	6	7	*5.0*	**6**
I 15 and Cheyenne	1	2	7	1	4	1	*1.5*	**1**
I 15 and Lamb Blvd.	3	3	3	4	1	5	*3.0*	**2**
I 515 and Charleston	3	3	2	5	2	3	*3.0*	**3**
I 515 and Las Vegas Blvd.	4	6	6	7	7	6	*6.0*	**7**
SR 160 and Jones Rd.	5	4	5	3	3	2	*3.5*	**4**

[1] 1 = Most severe conditions

Table 4-3: Severity ranking of the Phase II sites based on composite data of backfill conditions and LPR corrosion rates

Site	Soil Severity Rank	Corrosion Severity Rank	Composite Rank	**Phase II MSE Wall Corrosion Condition Severity Rank[1]**
I 15 and Cheyenne Blvd.	2	1	3	**1**
ALT US-50 and ALT US-95	1	6	7	**2**
I 515 and Charleston Blvd.	4	3	7	**3**
I 15 and Lamb Blvd.	6	2	8	**5**
US 395 and Huffaker Lane	3	5	8	**4**
SR 160 and Jones Rd.	5	4	9	**6**
I 515 and Las Vegas Blvd.	7	7	14	**7**

[1] 1 = Most severe conditions