

CAUSES OF CORROSION IN THE BOTTOM REINFORCEMENT OF PIER
CAPS SUPPORTING A PEDESTRIAN BRIDGE AT YSU AND POSSIBLE
SOLUTIONS

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Causes of Corrosion in the Bottom Reinforcement of Pier Caps Supporting a
Pedestrian Bridge at YSU and Possible Solutions

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ABSTRACT

A pedestrian bridge linking Moser Hall and Cushwa Hall at Youngstown State University (YSU), Youngstown, Ohio, was built almost 40 years ago. Three hammerhead pier caps supporting the bridge deck have experienced severe corrosion at the bottom. This corrosion has led to spalling of concrete at multiple locations in all three pier caps and thereby exposed the rebars to open air facilitating more corrosion. At some locations, corroded steel bars are clearly visible at spalled locations and through larger cracks. Some patches of repair are seen on the site, but the corrosion problem still continues to degrade the substructure.

An investigative study was undertaken to identify the causes of corrosion in these substructures and to propose a viable remedy to this problem. The replacement of the pier caps was not a preferable option, unless no remedial option is available. The researchers studied the construction drawings available at the time of this study, and some important documents containing information on material strength and clear cover were lost in time. This situation created a significant challenge in performing structural analysis of the existing substructure. Therefore, Schmidt Hammer and Profoscope were used to determine the approximate concrete strength, rebar location and clear cover.

Reviews of previous research suggest that substructure corrosion can likely be chloride-induced or carbonation-induced. Chloride-induced corrosion is more common in structures subject to deicing salts or saline water. Being a pedestrian bridge between two educational buildings with no access to deicing salts or saline water, chloride-induced phenomenon was ruled out as an active cause of corrosion in the substructure. Ingression of aggressive ions through cracks due to physical deterioration was ruled out as well. No signs of crack due to sulfate attack, alkali-silica reaction, freezing and

thawing action, shrinkage or support settlement were seen on the structure. Although carbonation-induced corrosion is a slower process in a dry environment and mostly occurs in older structures, it was predicted that the corrosion in those pier caps was due to carbonation of concrete. Free carbon dioxide in the atmosphere reacts with alkaline hydroxides in the concrete to form carbonic acid. Carbonic acid, unlike other acids, does not attack the concrete paste but it lowers the pH value. Carbonation reaction takes place on the outer surface of concrete and gradually penetrates into the concrete cover resulting in a low pH concrete cover.

A full-scale structural analysis revealed no major reduction in strength of the pier caps. Visual inspections and analytical study were conducted to ensure no major deficiency exists in the corroded concrete and steel reinforcement. After considering various repair methods, patching with low-slump dense concrete coupled with hydrophobic coating is suggested as a remedial measure to protect the substructure from further deterioration.

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TABLE OF CONTENTS

ABSTRACT.....	iii
ACKNOWLEDGEMENTS.....	v
LIST OF FIGURES	viii
LIST OF TABLES.....	ix
Chapter 1 Introduction.....	1
1.1 General overview	1
1.2 Problem statement.....	2
1.3 Research objectives.....	5
Chapter 2 Corrosion.....	6
2.1 Background.....	6
2.2 Types and causes of corrosion	7
2.2.1 Carbonation-induced corrosion.....	7
2.2.2 Chloride-induced corrosion	9
2.2.3 Concrete deterioration.....	10
Chapter 3 Repair Approach	14
3.1 Reactive repair techniques	17
3.1.1 Conventional repair.....	17
3.1.2 Electrochemical techniques	21
3.1.3 Corrosion inhibitor.....	24
3.2 Proactive repair techniques	25

3.2.1	Durability design.....	25
3.2.2	Corrosion resistant reinforcement.....	26
3.2.3	Surface protection system.....	29
3.2.4	Cathodic prevention.....	31
3.3	Summary.....	31
Chapter 4	Structural Condition and Repair Assessment	33
4.1	Current condition of pier cap.....	33
4.1.1	Prediction of carbonation depth.....	37
4.1.2	Current condition of corrosion in reinforcement.....	40
4.1.3	Strength of concrete in existing pier cap.....	41
4.2	Structural analysis of pier cap.....	52
4.3	Recommended repair	54
Chapter 5	Summary, Conclusions and Recommendations.....	57
5.1	Summary.....	57
5.2	Conclusions.....	58
5.3	Limitations of the study.....	59
5.4	Recommendations for future work	60
REFERENCES	61
APPENDICES	69

LIST OF FIGURES

Figure 1-1: Pier caps supporting bridge between Moser and Cushwa Hall at YSU.....	1
Figure 1-2: Bottom face of a pier cap.	2
Figure 1-3: Close-up picture showing corroded reinforcement.	4
Figure 4-1: On-site concrete cover measurement using a Vernier Caliper.....	34
Figure 4-2: On-site concrete cover measurement using Profoscope.	35
Figure 4-3: Longitudinal section of a Schmidt hammer.	43
Figure 4-4: Conversion curves for different positions of Schmidt Hammer.	44
Figure 4-5: Standard N-type Schmidt Hammer.	50
Figure 4-6: Plot of rebound number and compressive strength of test cylinders.	51
Figure 4-7: On-site Rebound hammer test.....	52
Figure 4-8: 2D elevation of pier cap.	53
Figure 4-9: Section at cap-column interface (A-A) showing study locations.	53

LIST OF TABLES

Table 4-1: Information extracted from structural drawings.....	33
Table 4-2: Weights of primary aggregates used in the construction	36
Table 4-3: Strength transformation factors, Aydin et al. (2010).....	46
Table 4-4: Strength reduction coefficients (AIJ 1983)	48
Table 4-5: Comparison of moment capacity and stresses at various locations in different conditions	54

Chapter 1 Introduction

1.1 General overview

Concrete has very low tensile strength compared to its compressive strength. In order to overcome this shortcoming, reinforcement is provided in the tension zone of a structure. Thus, reinforced concrete (RC), being capable of bearing compression and tension force as well as bending moments is very suitable for the construction of structures. Due to its large range of applicability, reinforced concrete is the most widely used material in the world.

Reinforced concrete consists of concrete and steel reinforcement. Concrete is a composite material made with coarse aggregates, fine aggregates, cement, and admixtures. Steel reinforcing bars can be made of mild steel or high strength steel, and these bars can be galvanized, or epoxy-coated for additional protection from corrosion. The traditional practice of reinforced concrete construction might not include coated steel. As a result, corrosion has been a predominant issue with old reinforced concrete structures.



Figure 1-1: Pier caps supporting bridge between Moser and Cushwa Hall at YSU.

A pedestrian bridge linking Moser Hall and Cushwa Hall at Youngstown State University (YSU), as shown in Fig. 1-1, was constructed almost 40 years ago. Three hammerhead pier caps supporting the bridge deck are experiencing corrosion at the bottom reinforcement. Signs of corrosion at the bottom face of a pier cap are shown in Fig. 1-2. This corrosion has led to spalling of concrete at several locations, where noticeable loss of cross-sectional area of reinforcing bars is visible. In addition, at some other sites, corroded steel bars are visible through cracks generated due to increase in volume of rusty bars.

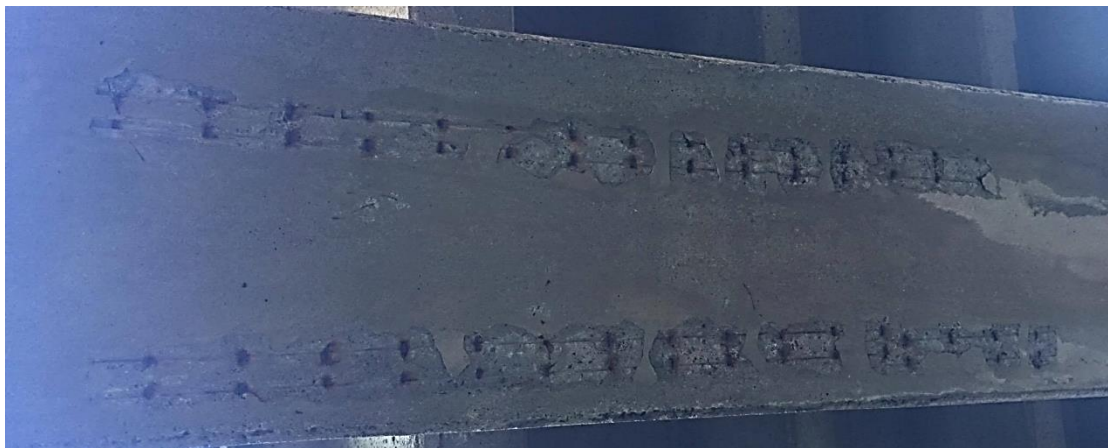


Figure 1-2: Bottom face of a pier cap.

1.2 Problem statement

Corrosion in reinforcing steel is one of the most common and severe problems in old RC structures. It decreases the tensile strength of reinforcing bars through the loss of cross-sectional area, and consequently, reduces the service life of a structure. In addition to generating cracks in the outer shell of tensile concrete, it weakens the bond between concrete and steel. This may lead to a structural, functional or aesthetic problem, if left untreated.

The excellent durability, rigidity and resilience of concrete make it the most widely used construction material in the world. During and after World War II, a large number

of reinforced concrete structures, mainly bridges, were built and they are still functioning in the transportation system of the United States. Reviewing the construction practice during that period, it can be concluded that conventional reinforced concrete was the prime material used in the construction. Most of those bridges were constructed for 50 years design life with an average current life of 43 years (NACE 2013; ASCE 2013). As a result, most of those bridges need repair or replacement. According to the American Road and Transport Builders Association (ARTBA), almost 28% of the U.S. bridges are over 50-year old, and they had never undergone any major reconstruction work (ARTBA 2017). National Bridge Inventory (NBI) rates a bridge as structurally deficient (SD) if it has considerable section loss, deterioration, spalling, or scour in their deck, substructure, or superstructure. The average age of structurally deficient bridges is 67 years, while it is 39 years for non-deficient bridges (ARTBA 2017). In the latest ASCE Infrastructure Report Card, 56,007 bridges (9.1%) in the U.S. are structurally deficient (ASCE 2017). If proper maintenance or reconstruction is not done in these deficient bridges, there might be a huge socio-economic loss in the near future.

The Silver Bridge between Point Pleasant, West Virginia, and Kanawha, Ohio, collapsed on December 15, 1967, leaving 46 dead and nine injured. In 1983, Mianus River Bridge over I-95 failed due to hinge connection failure resulting three fatalities and five injuries. Both bridges were considered to fail due to corrosion and fatigue (Lichtenstein 1993). Though the corrosion of reinforcing steel is not the only cause of the structural deficiency, it is one of the significant contributors, and therefore, is a matter of big concern in this study. In a study of 92 sample bridge failures, Cook (2014) categorized the causes of bridge failure as hydraulic total, collision total, overload, deterioration total, fire, construction, fatigue-steel, bearing, soil, and miscellaneous.

Among these reasons for the failure of bridges in the U.S., deterioration of steel and concrete properties encompasses 6.5% of the failures. Cook (2014) documented that the age of bridges collapsed due to this deterioration ranged from 44 to 100 years.

Almost 40-year old pedestrian bridge between Moser Hall and Cushwa Hall has the deck supported by three single-column hammerhead pier caps (Fig 1-1). The pier caps have several spots where lump of concrete has been chipped out. On a closer inspection, section loss of reinforcing bars can be seen as well. Examining the affected areas, it can be easily concluded that the corrosion process is continuing. Some patches of repair are seen on the site, but the corrosion problem continues to degrade the structure. If this issue is left unaddressed, corrosion in steel bars will be worsen with time creating a significant risk of potential damage and degradation of the substructure. Pictures of the pier cap with spalled concrete and corroded steel bars can be seen in Figs. 1-2 and 1-3, respectively.



Figure 1-3: Close-up picture showing corroded reinforcement.

1.3 Research objectives

Corrosion can be chloride-induced or carbonation-induced. Carbonation induced corrosion is a slow process that usually occurs in old structures. Due to climate change, it has gained more attention in recent days (Yoon et al. 2007). Corrosion caused by chloride induction is more common in concrete structures that experience deicing chemicals. Bridges in marine environment and those subjected to deicing salts during the winter suffer from chloride-induced corrosion (Zhou et al. 2014).

The primary goal of this research is to study the causes of corrosion in the bottom reinforcement of the pier cap, to analyze its structural condition, and to provide a suitable solution of the problem. Furthermore, construction approach to prevent corrosion in reinforced concrete structure in future will also be discussed.

Chapter 2 Corrosion

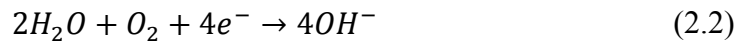
2.1 Background

The microscopic pores of concrete contain a high concentration of soluble calcium, sodium and potassium oxides, which has a high alkalinity level with a pH of around 13. This alkaline solution, in the presence of oxygen, creates a passivating layer in the proximity of steel bars in RC structures. Higher pH values and higher availability of oxygen create a thicker protective layer around steel bars. With carbonation and chloride attack, the alkalinity of the solution reduces and the passive layer breaks down. Following the breakdown of the protective layer, a complex electro-chemical process, which is called corrosion, takes place leading to the formation of rust in steel bars (Bertolini 2013).

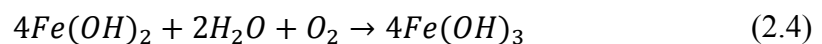
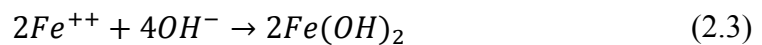
An anodic reaction, as shown in Eq. 2.1, occurs in steel bars causing the dissolution of iron.

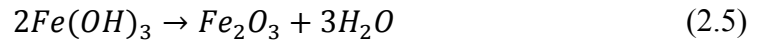


Steel bars with more positive charged area acts as a cathode in this process, where the reduction reaction takes place, as shown in Eq. 2.2:



These free electrons tend to migrate again to maintain electrical neutrality and thus, rust (Fe_2O_3) is formed per Eq. 2.3 - 2.5 in presence of moisture and oxygen.





As the reinforcing steel corrodes, the rust and the remaining steel together occupy more volume than original steel bars, and as a result, the increased volume exerts outward pressure on surrounding concrete (Luckovic et al. 2014; Chen and Leung 2015; Khan et al. 2015). When this outward pressure exceeds the concrete tensile strength, cracks are formed often resulting in spalling of concrete. These cracks or openings provide pathways for further corrosion by exposing steel bars to open moist air. Thus, steel corrosion leads to cracking, delamination and spalling of concrete leading to the loss of service life of the structure.

2.2 Types and causes of corrosion

There are mainly two types of corrosion that occur in reinforced concrete structures. They are carbonation-induced corrosion and chloride-induced corrosion.

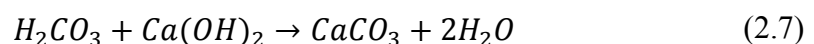
The main causes of corrosion are discussed below:

2.2.1 Carbonation-induced corrosion

This type of corrosion is caused by carbonation, which is the reaction of carbon dioxide present in the air with hydrated cement solution to neutralize the alkalinity of concrete, as shown in Eq. 2.6.



Equation 2.7 shows that this carbonic acid does not attack the cement paste directly, instead, reacts with pore solution and forms calcium carbonate.



The formation of calcium carbonate reduces the alkalinity of concrete to the level that facilitates corrosion in steel bars.

The carbonation rate depends on concrete properties as well as on environmental factors. The permeability of concrete greatly influences the diffusion of carbon dioxide, which needs to penetrate the pores before it can disrupt the alkaline layer. Pores can be closed, accessible, or continuous. Most of the ions, moisture and gas ingress is through the continuous pores. Higher porosity or greater permeability increases the carbonation rate as well. Concrete composition with higher water-cement ratio produces a more porous concrete. Type and composition of cement used also influences the porosity. Porous cement matrix allows carbon dioxide to diffuse into the hydrated cement paste through capillary action. It results in pH value drop, which progresses with time.

Carbonation front indicates the depth of concrete to which carbonation has occurred. Carbonation depth increases nonlinearly with time (McPolin et al. 2009). Therefore, the damage is very much dependent on the concrete cover provided to the reinforcement as well. For low concrete cover, the risk of corrosion is higher since the carbonation process destroys the passivating layer in a very short period of time. In order to resist the carbonation induced corrosion, it is essential to have a sufficient cover in well-compacted and non-porous concrete.

Environmental factors also influence the carbonation process. It occurs only in the presence of water, so the process is very slow in a dry environment. In addition, pores entirely filled with water will also slow down the process. Carbonation rate is high in the high humid environment. Humidity range of 50% - 70% is found to be critical in promoting carbonation (Cigna et al. 2003). Higher relative humidity significantly increases the concrete conductivity, which results in increased risk of corrosion for carbonated concrete (Glass et al. 1991). Carbon dioxide concentration and temperature

are other environmental factors that affect the process. Higher concentration of CO₂ increases the carbonation rate, and so does the rise in temperature.

2.2.2 Chloride-induced corrosion

Chlorides directly attack the passive layer around steel bars to break it down and start the corrosion process. They are present either in the concrete itself or diffused in concrete through pores from the atmosphere. Chlorides from sea water in marine structures and de-icing salts in colder region ingress through the concrete cover in reinforced concrete structures. When the exposed structure undergoes dry and wet cycle, there is an increased possibility of suction of chlorides through capillary action.

Unlike carbonation, they are not consumed in the process, and the pH of concrete does not drop. Chlorides break the passive layer on the steel, which allows the corrosion process to proceed rapidly. Firstly, chlorides react with ferric ions to form complex ions, which reacts with hydroxyl ions to form corrosion product. Then the chloride ions are again released to repeat the process. Due to this effective recycling process, chlorides act as catalyst in corrosion after the initiation of corrosion.



This effective recycling reaction (Eq. 2.8 - 2.10) of chloride ions makes the corrosion process complex to eliminate once it has started. Thus, chlorides lead to a localized corrosion in a region forming a pit of rust, called pitting corrosion. Steel reinforcement with no longer protected by passivating layer acts as an active zone (anode) and the surrounding portion of steel, which is still passive, serves as a cathode. Anodic and cathodic reactions in these zones initiate the corrosion process. However, for corrosion

process to start, a certain level of chloride content is required, which is called the chloride threshold. The chloride threshold value, as well as the time to reach that value, depends on the type of cement used in concrete. Like carbonation, the process and the threshold value rely on both concrete properties and environmental factors.

The chloride threshold value depends on pores in concrete near the steel surface (Page 2009). Reduction of voids by decreasing the entrapped air from 1.5% to 0.2% (by volume) in steel-concrete interface improved the threshold value from 0.2% to 2% (by mass of cement). Other factors affecting the limiting value are temperature, composition, and surface roughness of steel reinforcement (Bertolini and Redaelli 2009).

2.2.3 Concrete deterioration

Concrete deteriorates due to a variety of factors that result in cracking. Though these cracks are not direct cause of corrosion, they provide pathways to harmful substances, which can initiate or boost the corrosion process.

2.2.3.1 Physical mechanisms

The physical mechanism of deterioration is mainly associated with the volume change of concrete, which leads to cracking, if effective ways to reduce the effect of volume change are not applied. Contraction of concrete may take place by the reduction of moisture in concrete, fall of temperature, or by reaction of concrete with free carbon dioxide in the atmosphere. Due to the change in moisture content in concrete, there is a different type of shrinkage. When the rate of evaporation exceeds the rate of bleeding in fresh concrete, cement particles at the concrete surface will lose the water molecules. Sufficient loss of water will result in contraction of concrete proceeding to cracking. This process is called plastic shrinkage. Drying shrinkage takes place in dry concrete

when its volume tends to reduce the evaporation of pore water. Even if the moisture is lost from a hydrated surface other than pores, shrinkage takes place, which is called autogenous shrinkage. Use of admixtures, as well as fibers, has been found helpful in reducing this type of shrinkage cracks (Sivakumar and Santhanam 2007).

The volume of water molecules increases while freezing occurs. Expansion of water particles present in concrete pores may lead to cracking with repeated cycles of freezing and thawing. Concrete with the high water-cement ratio, generally used in the old structure, is more susceptible to this type of damage or cracks. Aggregates may also pop-out if the water absorbed by them expands after freezing. The Portland Cement Association (PCA 2002) encourages the use of air-entraining admixture to avoid this type of cracks.

The hydration process of Portland cement is exothermic. Heat is involved during setting and hardening of cement paste resulting an increase in volume. Concrete shrinks as it cools down slowly, which leads to cracking if volume change is restrained. While concrete is in the setting process, external temperature coupled with temperature rise due to internal reactions also causes cracks. (Bamforth 2007).

Due to the action of traffic or outer material, concrete surface may undergo abrasion that leads to loss of material. Moreover, if cavitation is created in the concrete surface by means of water or air, lose material may erode as well. ACI 210R-93 (Graham et al. 1987) gives approximate threshold values of cavitation ranging from 0.19 to 0.30. When a moving particle collides with a concrete surface and creates an impact, there may be a loss of a small portion of concrete due to crushing. All these actions leading to material loss reduce the effectiveness of concrete cover.

Support settlement or structural movement also generates stresses in a structure (PCA 2002). If the current load is more than the design capacity, it increases stress beyond the design value in structures. These stresses may cause cracking in concrete if they are higher than the tensile strength of concrete.

2.2.3.2 Chemical mechanisms

Concrete surface may encounter different environmental conditions throughout its service period. If the cement matrix comes in contact with any harmful chemicals, it reacts to disintegrate the cement molecules. Sulfate attack, alkali-aggregate reaction and acid attack are three primary mechanisms of chemical degradation in concrete.

When dissolved sulfate ions ingress into the hardened concrete to combine with Calcium-Silicate-Hydrate (C-S-H) gel, it reacts to alter the concrete properties either by expansion and cracking or loss of strength and integrity. The most conventional type of sulfate attack includes the cations with sulfate ions of sodium, potassium, or calcium (Taylor 1997). Sulfate attack including magnesium ions has been found to have greater damage to concrete integrity (Binici and Orhan 2006). Brucite is produced as a result of magnesium sulfate attack, which reduces the pH of concrete pore solution. Thaumasite may be formed on the buried structures exposed to sulfate ions. In all these processes, there is a formation of new crystals, which occupy the voids in the concrete. Formations of these crystals in a continuous manner cause the concrete to crack, generally characterized by whitish powder on the spalled section.

Hardened concrete contains water with dissolved cations of alkaline potassium and sodium from cement. In the presence of these alkali ions, aggregate minerals react with hydroxide ions. This reaction absorbs water, which leads to expansion and cracking of concrete (Dyer 2014). When alkali reacts with silica, it forms an expansive gel called

alkali-silica reaction (ASR) gel. In dolomitic rock aggregates, alkali-carbonate reaction (ACR) has been observed. It involves the decomposition of dolomite to form brucite and calcite resulting in the reduction of net volume. But clay particles within carbonate minerals expand as they absorb water. However, ACR is rarely encountered than ASR, as dolomite aggregates are not often used in concrete.

Some ingredients of concrete may dissolve when acid encounters the concrete surface. This dissolution of materials may lead to increase in porosity, decreasing the effectiveness of concrete cover. Acids such as nitric acid, sulphuric acid, acetic acid and oxalic acid, damage the concrete layer, generally characterized by layers of colored zones. (Gutberlet et al. 2015).

Chapter 3 Repair Approach

Reinforced concrete structures undergoing corrosion in steel reinforcement need maintenance or repair to restore the safety and serviceability of the structure. Repairs are also necessary for providing a better remaining service life to the structure. A structure's integrity can be maintained by reactive approach or proactive approach, depending on whether a structure is already built or is still in the design phase (Code 2010).

The reactive approach uses unplanned remedial measures or interventions to stop alterations that have changed the serviceability of a concrete structure. Many old structures without a precise durability design at the time of construction are now facing concrete degradation as well as corrosion of reinforcement. In order to control the propagation of corrosion to prolong a structure's life and to guarantee its expected performance, maintenance is required. Realizing this importance of maintenance, there have been many studies to recommend repair techniques. Standard codes have been developed by RILEM Technical Committee (RILEM 1994) and ACI (ACI 2013), which define structural repair techniques, rehabilitation processes, and strengthening procedures of those structures.

A proactive approach is based on the preventive measures that are set forth in the design phase and implemented during the construction. Through the approach, damages can be prevented before they become critical to compromise the intended serviceability of the structure. On the other hand, the reactive approach aims to check the ongoing degradation that might have already damaged the structure. For an existing structure, which is undergoing corrosion related degradation, a reactive approach is the only option.

In order to repair concrete structures with corrosion problems, the very first step is to define the repair option, that is, to decide whether to take control actions against corrosion or to adopt another approach. The second step would be selecting the repair principle to avoid further corrosion. Choosing a repair method and design details of its application would be the final step (RILEM 1994).

When defining repair option, a preliminary evaluation of the structure's condition should be completed. Visual inspection, as well as electrochemical inspections, should be conducted to diagnose the extent of damage in a structure. The visual inspection includes a recording of visible cracks and other physical defects, and measurement of concrete cover, whereas detailed survey including electrochemical investigation uses techniques, such as half-cell potential mapping and resistivity measurement, to identify the affected sites and corrosion rate in those locations.

As per RILEM 124-SRC, after a preliminary assessment, the structure can be classified as one of the following:

- a) Reinforcement is still passive, and corrosion has not started yet, i.e., carbonation or chloride penetration has not reached the steel surface.
- b) Reinforcement has begun corroding, but the propagation is in an early stage, negligible loss of steel cross-section and no cracks in concrete cover.
- c) Profound corrosion in reinforcement and significant loss of rebar cross-section, loss in serviceability of structure due to cracking, spalling or delamination of concrete cover.

Depending upon the extent of damage environment around the structure, availability of repair products of competent workforce to implement the repair procedures, ACI 222R (ACI 2010) has listed some of the options. These options include (i) doing nothing, (ii)

removing spalled concrete and replacing with a patch of overlay, (iii) removing spalled and delaminated concrete and replacing with a patch of overlay, (iv) removing chloride contaminated concrete or carbonated concrete and patching with an overlay, (v) installing cathodic protection to protect the steel reinforcement from further corrosion, (vi) using electrochemical chloride extraction (ECE) to remove chloride from the surface of the reinforcing bars and to improve the alkalinity at the bar surface, (vii) using re-alkalization technique to restore the concrete pH, or (viii) using corrosion inhibitors on the surface of concrete to reduce the corrosion rate of the embedded reinforcing bars.

In order to reduce the corrosion in reinforcement, either anodic process or electrolytic process needs to be applied. Basic principles to stop anodic process include repassivation of steel reinforcement by coating reinforcement or cathodic protection. Reducing the moisture content in concrete can stop the electrolytic process (RILEM 1994). Based on the cause of corrosion and desired life of a structure, different techniques can be used to address the problem. Rehabilitation can be done by conventional repair or by electrochemical methods.

If a structure is diagnosed to be structurally deficient for the original design load, strengthening process should also be considered along with the rehabilitation process. Several options are available for structural strengthening, such as concrete jacketing or the use of carbon fiber reinforced polymer (CFRP). These options are not in the scope of this study.

3.1 Reactive repair techniques

3.1.1 Conventional repair

Conventional repair methods aim at restoring protection to reinforcement by replacing non-protective concrete with a suitable cementitious material. This approach is also called patch repair as it is done by removing the affected concrete and replacing it with alkaline mortar or concrete. This process is suitable when the corrosion attack region is small as the removal of concrete will be small in volume. Even though coating of steel is not required in this method of repair, loose rust from the affected reinforcement should be removed before patching fresh alkaline concrete or mortar. In this approach, affected concrete, as well as concrete which is expected to be affected within the remaining design life should be removed. In places where concrete undergoes wetting-drying cycles, it is not required to remove sound concrete if sufficiently thick (generally more than 20 mm) cement mortar is applied over the affected concrete (Bier et al. 1987). This method is based on the diffusion of hydroxyl ions from the new external alkaline layer towards the carbonated concrete and may lead to repassivation of steel bars. However, this method is not suitable if the carbonation depth is 20 mm beyond the reinforcement.

The conventional method is divided into four phases:

- i) conditional assessment of the structure
- ii) removal of concrete in affected areas up to a specific depth
- iii) cleaning of exposed/affected reinforcement, and
- iv) application of suitable repair material to provide an adequate cover to the reinforcement.

Each of these steps needs to be executed properly to ensure the effectiveness of the repair work (ACI 2004).

The condition assessment of a structure is one of the important stages of the repair process. In this phase, the cause of deterioration and extent of damage in the structure are determined. For carbonated structures, effective concrete cover depth (x), and carbonation depth (d_c) at the moment defines the present condition of the structure. Depending on these two values, the thickness of concrete that needs to be removed is determined. Removal of concrete cover is necessary not only in the affected zone where cracks or damage can be seen but also in the areas where existing carbonation is expected to damage the structure within the remaining life. Even though the cover is structurally sound, it needs to be removed if it is affected.

Concrete removal can be done in many ways. Depending on the thickness of concrete to be removed, it can be milling method, abrading method, or water-jetting method. It is easy to do so when affected concrete is weak or cracked. If the thickness of concrete to be removed is lesser than the cover depth, scarifier can be used as milling method or grit blasting can be used as abrading method. If concrete should be removed beyond reinforcement, the pneumatic breaker is used as high-pressure water jetting. While removing concrete, appropriate care should be taken so that reinforcement is not damaged and the surface of concrete is left rough and clean. Also, cracking of concrete substrate or disturbance in the bond between concrete and the reinforcement in areas where concrete is not being removed should be avoided (Babaei and Weyers 1996).

Concrete surface should be carefully prepared to provide sufficient bond between original concrete substrate and new repair material. The surface should be rough, but dust particles should be removed. If the repair material is to be directly applied to the

concrete substrate, first the surface should be saturated with water to avoid further absorption of water, which may lead to plastic shrinkage in fresh concrete. Otherwise, some bonding materials, such as cement or mortar paste, polymer latex, or epoxy systems, should be used to enhance the adhesion of the repair mortar. If concrete is removed beyond steel reinforcement, exposed bars should also be cleaned by grit blasting or any other suitable methods. Corrosion byproducts, i.e., rust and any loose concrete mortar on the reinforcing bars, should be removed without incurring any damage. For chloride-induced corrosion, special care should be taken. Loose rust particles from corrosion pits and other hidden parts should also be removed. This removal can be done using high-pressure water blasting.

After the surface preparation, suitable repair material of designed thickness should be applied. The cement-based repair material should be resistant to carbonation and chloride penetration and other harmful environments, such as sulfate attack or freezing and thawing actions. Ordinary Portland cement mortar is used along with admixtures to achieve these properties. Water cement ratio should be low (less than 0.4) to ensure resistance to carbonation and chloride penetration. Because of high cement content, there may be drying shrinkage that leads to cracking or loss of bond. In order to avoid this situation, expansive cement can be used, or expansive agents such as calcium oxide or anhydrous calcium sulfoaluminates, can be added to conventional cement (Li et al. 2009). Expansion should be constrained, and wet curing should be performed if these materials are used.

Some part of the water in repair mortar can be replaced by synthetic latex to make a polymer modified mortar. The use of latex may improve the workability, carbonation and chloride resistance, and tensile and flexural strength of repair mortar (Martins et al. 2012). It also increases the bond strength to the substrate and reduces the rate of

shrinkage. In order to improve the durability of the overlying concrete, fly ash or silica fume can be added. Superplasticizers can be used to maintain workability in concrete with low water-cement ratio. Fibers can be used as well to reduce slump and to increase tensile strength of concrete that helps reduce cracks.

Specifications of repair material should be based on the tests that can be carried out before repair. Standard cover tests should be performed to determine workability and materials properties, such as compressive, tensile and flexural strength, elastic modulus and adhesion. Further, accelerated durability test should also be conducted to find carbonation and chloride penetration depth. These tests should be able to find the coefficient of carbonation, K , or apparent diffusion coefficient, D_{app} . Cover thickness should be designed based on the carbonation rate or chloride ingress rate and the design life of the structure. Based on the aggressiveness of environment and resistance properties of repair material to deterioration, the cover thickness can vary significantly.

The primary objective of repair work to passivate the reinforcement can be increased by using additional protections. Corrosion inhibitors can improve the protection of steel reinforcement. Inhibitors, such as calcium nitrite, can be mixed with repair mortar to increase the chloride threshold. Applying migrating inhibitors on the surface of concrete may reduce or stop the corrosion process, which makes the removal of concrete for repair unnecessary in future. Surface treatments can be applied over conventional patches for the effectiveness of protection. Organic coatings, hydrophobic treatment, and other cementitious coating over the repair layer can help to prolong the degradation of concrete and steel (Söylev et al. 2008).

Besides coating the concrete surface with moisture repellent substances, steel reinforcement can also be coated. The coating on rebars can improve resistance to

further corrosion and enhance the adhesion to the repair mortar as well (Bertolini et al. 2004). This can also be used when it is impossible to provide sufficient repair mortar thickness, but in other cases, it is recommended to use it along with design thickness of patch.

3.1.2 Electrochemical techniques

Rehabilitation of corroded steel reinforcement can be done using electrochemical techniques as well. Cathodic protection, electrochemical chloride extraction and electrochemical realkalization are the electrochemical methods that control corrosion of steel in concrete (Redaelli et al. 2011).

3.1.2.1 Cathodic Protection

Cathodic protection (CP) is one of the most effective and popular methods in corrosion control, where electric potential of steel reinforcement is decreased by providing a current of electrons from external artificial anodes. Originally, cathodic protection was invented to prevent future corrosion after a repair in damaged structures. Nowadays, it has been widely used in new construction to check the corrosion process from the beginning. Adopting this repair technique in repairing spalls and detached concrete significantly reduces the repair work and cost, as contaminated concrete should not be removed if it is structurally strong (Polder 1998).

There are two types of cathodic protection systems: galvanic cathodic protection and impressed current cathodic protection. Both systems use a simple process supplying free electrons from an external source to the steel reinforcement so that anodic reaction (Eq. 2.1) decreases and consequently the corrosion process is reduced.

Galvanic cathodic protection system is also popularly known as sacrificial anode system. In this type of protection system, the current for cathodic protection is supplied

from galvanic anodes, such as zinc, aluminum, or magnesium, which are more active metals than steel in the galvanic series. Current flows from sacrificial metals to steel reinforcement due to the development of a potential difference between more active external metal and less active steel reinforcement. Thus, galvanic anode is the sources of free electrons to the process, and it sacrifices itself by galvanically corroding first to protect more expensive steel bars (Virmani and Clemena 1998). If the current for cathodic protection is supplied from an external power source, those systems are termed as impressed current cathodic protection system. Both systems have been used successfully in the U.S. (Kepler et al. 2000). A suitable type is employed in repair depending on the structure and its surrounding. While the impressed current system is often used to repair bridge decks, sacrificial anode system is used for substructures.

A cathodic protection system has different components, such as an anode, reinforcement in protection (cathode), a power source, concrete covering the steel reinforcement, a monitoring system, and wire connections to carry currents. In order to apply CP in existing structures, deteriorated concrete surface should be repaired first, and the anode is placed on top of the repaired surface. Anode distributes the current over the surface while oxygen and moisture present in concrete act as electrolyte. External current shifts the potential of reinforcing steel in negative direction, and if the current is designed properly to make all steel reinforcement cathodic, corrosion can be stopped (Kepler et al. 2000).

Corrosion of steel reinforcement depends on many factors, so does the selection of appropriate material for cathodic protection. For the transfer of current throughout the structure, all steel reinforcement should be linked or electrically continuous. Appropriate anode material should be selected to resist deterioration that may appear in a structure in future. Furthermore, it should be durable enough to reach the design

life of the protection system. It is essential to determine the appropriate amount of potential difference or current to make sure that the anode is capable of protecting the structure through a continuous supply of current.

Different technical specifications have recommended different current densities for varying steel conditions (NACE 2007; ISO 2012). In order to be considered effective, 24-hour depolarization value should be greater than 100 mV. At the course of a repair life, the number of aggressive ions decreases gradually, and the smaller current density can also satisfy the depolarization requirement. Regular monitoring is required to avoid any overprotection of structures, which makes the repair system uneconomic as well as may cause deleterious effects like steel-concrete bond loss, hydrogen evolution, alkali-silica reaction and anode deterioration (Kepler et al. 2000, Bertolini 2013)

3.1.2.2 Electrochemical chloride extraction and realkalization

Corrosion of reinforcement in RC structures is an electrochemical process, where electrical charge from a positively charged area of steel or anode moves to a negatively charged area of steel or cathode. Electrochemical chloride extraction (ECE) and electrochemical realkalization (ER) use the application of temporary current directly between steel reinforcement (cathode) and an anode that is placed on the outer surface of concrete. ECE is used for chloride-contaminated structure, whereas ER is used for carbonated structures.

Both of these methods share a similar technique to rejuvenate the passivating layer with different electrolytes. A temporary anode can be an activated titanium wire mesh or a steel mesh. For ECE, the anode is surrounded by tap water or saturated calcium hydroxide solution, whereas for ER, it is surrounded by sodium carbonate solution of 1

mole per liter. The paste can be sprayed onto the surface, or the structure can be enclosed making a pond or a tank.

Typically, 1-2 A/m² current density is applied from the direct source. In order to realkalize the carbonated concrete using ER, current needs to be applied for one to two weeks. Hydroxyl ions (OH⁻) are generated in reinforcement due to the electrical field, which imbibes alkali into the concrete. ECR is more time-consuming and a susceptible process. It normally takes 4 to 8 weeks to extract the chloride ions depending on several factors, such as chloride content and environmental conditions. Application of direct current in this process causes the negatively charged ions in steel reinforcement to migrate to the temporary anode. It reduces the potential in reinforcement, increasing the concentration of hydroxyl ions and decreasing the chloride content in the concrete surrounding the steel bar.

3.1.3 Corrosion inhibitor

A corrosion inhibitor is a chemical substance that improves the durability of concrete by reducing corrosion of metals without decreasing the concentration of a corrosive agent. It reduces the rate of reaction in anode and cathode to subdue the overall corrosion rate. These organic-based substances can be mixed in fresh concrete as an admixture or can be applied to the surface of hardened concrete. They can also be mixed with repair mortar while performing patch repairs. Most commonly used inhibitors are amino-alcohols (AMAs), calcium nitrites (CN) and sodium monofluorophosphates (MFPs) (Söylev 2008).

Corrosion inhibitors can be applied to prevent or delay corrosion initiation or to reduce the corrosion rate once it is initiated. These substances migrate through the concrete by

vapor diffusion. The absorbed ingredient hinders anodic reaction at iron, and it also interrupts the oxygen level at the cathode to inhibit corrosion process.

For chloride-induced corrosion, its performance is largely dependent on chloride level at the reinforcement. At different chloride levels, Page et al. (2000) applied MFP and Alkanolamine-based inhibitors to reinforced concrete specimens with a water-cement ratio of 0.65 to learn about the corrosion behavior of rebars. Application of these inhibitors reduced the corrosion rate of rebars for low chloride content ($<1.2\%$ by mass of cement), while they had no effect on concrete with chloride content 2.4% . Confirming to this study, several works (Elsener et al. 2000; Trépanier et al. 2001) have revealed that these inhibitors perform well to delay the initiation of corrosion but have minimal effect once the corrosion has started. Its effect is found to be enhanced when used with hydrophobic coatings. Laboratory studies and field monitoring of repairs have demonstrated this repair method to be effective for carbonation-induced corrosion damage (Mackechnie and Alexander 2001).

3.2 Proactive repair techniques

3.2.1 Durability design

Proper design considerations can prevent any possible corrosion of reinforcement in RC structures. A simple error in design phase may significantly increase the cost of a structure. De Sitter's "Law of Five" (1984) approximates the effect as: if the cost of properly designing and building a structure is a dollar, repairs would cost \$5, and if repairs are not made in time, the cost would be five times higher, i.e., \$25. The cost goes on increasing if the maintenance is deferred and the level of severity increases in a structure. Therefore, in order to save the possible high maintenance expenditures, care should be taken at design phase. If the structure shows some symptoms of degradation,

timely maintenance may save a substantial amount of maintenance cost in the long run. Therefore, regular monitoring of structures is vital to the service life of a structure as well as in reducing future maintenance cost.

More important factors affecting durability of a structure are environmental aggressiveness, quality of concrete used, and cover thickness. Depending upon the location of the structure and the aggressiveness of the surrounding, appropriate concrete cover should be considered. Adequate cover and low water-cement ratio will increase the time for chloride and carbonate ions to reach reinforcing steel. ACI-318 (ACI 2008) and AASHTO LRFD Bridge Design Specifications (AASHTO 2012) have set their respective minimum cover to be provided in RC structures under different exposure conditions. ACI 318 limits the maximum water-cement ratio to 0.4 for effective corrosion protection. Sherman et al. (1996) found that concrete mix with a water-cement ratio of 0.30 to 0.32 can produce almost impermeable concrete. Furthermore, the porosity of concrete can also be reduced by using admixtures, such as fly ash, slag, silica fume and other material. However, in order to avoid deleterious effects of different elements present in these materials, proper curing should be ensured.

3.2.2 Corrosion resistant reinforcement

Conventional reinforcing bars can be replaced by corrosion resistant bars, which may be epoxy- or metal-coated reinforcement or fiber reinforced plastic reinforcement. Federal Highway Administration (FHWA) estimates 16% of existing highway bridges in the U.S. use corrosion-resistant reinforcing steel (Stroia 2017). This type of reinforcement has barrier system that shields the flow of oxygen and prevent the corrosion process.

Fusion-bonded epoxy-coated reinforcement (FBE CR) has been widely used in the U.S. and Canada for aggressive chloride environment. Departments of Transportation (DOT) of many states and FHWA recommend the use of FBE CR as one of the main protection of reinforcement in highway bridges from chloride attack. As waterproofing membranes are difficult to install and monitor in bridge decks, use of FBE CR can serve as a tough defense line against corrosion. While it was first installed in a bridge deck in Philadelphia in 1973, by the end of 1989, FHWA estimated 100 million square feet use of FBE CR (Virmani and Clemena 1998).

Epoxy coatings are applied in reinforcing bars in a factory. They are first cleaned by grit blasting, heated in a furnace to about 230 °C and are passed through a coating unit the epoxy powder is sprayed. The epoxy powder melts and is cured on the bar, which is then quenched to get a corrosion-resistant epoxy-coated bar. These processes are operated in a sequence following standard guidelines of ASTM A775, A934, and D3963, and AASHTO M284. Epoxy can be coated in individual reinforcing bars before fabrication or can be coated in a cage of reinforcement after fabrication and welded together. Firstly, epoxy coating act as a barrier shielding oxygen and chloride ions from the steel surface. Secondly, it increases the electrical resistance between adjacent steel reducing the magnitude of microcell current.

Beside epoxy, reinforcing steel can also be coated by metals to prevent corrosion. Galvanizing or zinc-coating in steel bars is one of the proven methods to improve performance against corrosion. Traditionally, steel bars can be galvanized through the hot-dipped galvanizing process (ASTM A767), where iron bars go through multiple stages of surface preparation and are dipped in a molten zinc bath at temperature of 450-500 °C. In recent years, galvanizing is also performed as a continuous process

following the guidelines of ASTM A1094, which are similar to the technology used in sheet metal industries.

In addition to providing barrier protection to the underlying rebar against any aggressive environment, zinc coating also serves as a sacrificial anode once the chloride or carbon threshold is exceeded. Moreover, it forms a stable passivating layer within few hours of concrete pouring. Compared to unprotected black steel, the passivating layer in galvanized bar remains steady even at low pH, which increases the amount of chloride ions required to initiate corrosion by four times. In a recent study done by National Research Council of Canada (Stroia 2017), galvanized steel was found to have 5 to 10 times lower corrosion rate than carbon steel even in highly aggressive areas. On a cost analysis, Goodwin (2014) found that continuous galvanized reinforcement has the lowest cost ratio (1.1) with respect to the cost of unprotected black steel. The ratio of cost for epoxy-coated, batch-galvanized, stainless cladding, solid stainless, and the combination of continuously galvanized and epoxy-coated rebar were studied to be 1.25, 1.5, 3.2, 7, and 1.2 respectively.

Stainless steel reinforcement is one of the very effective corrosion resistant reinforcement, which contains minimum 12% chromium. Depending upon the chromium content, it is divided into four types: ferritic (<17% chromium), ferritic-austenitic (22-28% chromium and 4-8% nickel), austenitic (18% chromium and 8% nickel) and martensitic (12-18% chromium and carbon content up to 1.2%). Chromium makes the stainless steel corrosion resistant by creating an invisible surface film that resists oxidation. Because of its high corrosion resistance, it is not required to couple stainless steel with other protection systems and presence of cracks does not seem to compromise its performance. However, its high cost, i.e., almost six times to that of black steel, has limited its use in locations where adequate cover cannot be provided or

at construction joints. Virmani and Clemena (1998) compared the cost of three bridge constructions and found stainless steel would increase the cost by 6 to 16%.

Lately, fiber reinforced plastic (FRP) bars are also being used and studied as corrosion resistant reinforcing bars. In the U.S., the first bridge built using FRP bars was in 1996 in West Virginia, where glass fiber reinforced plastic (GFRP) bars were used in the concrete deck. Regular monitoring and studies have been conducted on that bridge to learn about its behavior and performance. At the same time, FRP bars are also being studied for prestressed concrete in a new system of patented construction (Grace 2016). However, carbon FRP strands have a low tensile strain at failure, and allow smaller deflection. Glass composites contain silica, which may react with calcium hydroxide in concrete resulting in the loss of reinforcement.

3.2.3 Surface protection system

Coatings, sealers, inhibitors and waterproofing membranes can be applied in concrete to protect it from the aggressive environment. They increase the time for chlorides or carbonates to reach the reinforcement. Waterproofing membranes are of two types, one is a liquid that solidifies after application, and the other is a sheet that is directly stuck on the concrete. Care must be taken while installing these membranes, and no defects, such as blow holes, penetrations or mechanical damage, should be present in the finished surface.

Penetrating sealers have been found effective in stopping chloride intrusion into concrete. When applied, it penetrates into the concrete and reacts with the moisture present in the pores to form a water repelling layer. This hydrophobic layer stops the water from entering but allows the water vapor movement, which facilitates drying. These hydrophobic coatings using silanes and siloxanes are most effective on the

concrete surface, which is uncontaminated and free of cracks and surface defects (Mackechnie and Alexander 2001).

FHWA accepts the use of corrosion inhibitor as an alternative to epoxy-coated steel in protection against chloride induced corrosion. Studies have shown that if sufficient nitrite concentration can be ensured such that nitrite to chloride ratio is 1 at rebar depth, then corrosion will be prevented (Broomfield 2006). A typical corrosion inhibitor used during the casting of concrete is calcium nitrite, which has no serious effects on construction. But it should be mixed in such a proportion that there will be sufficient nitrite throughout the service life to stop corrosion, as it gets consumed with the exposure to chlorides.

Unlike bridges, buildings are less prone to chloride attack unless they are maritime structures. For such less severe environmental conditions, cladding can also be used to increase durability of concrete by controlling chloride and carbon dioxide ingress. Cladding and other type of coatings will be a better choice for buildings, if appropriate drainage system is maintained.

In a recent study, self-assembling siloxane coating was studied to produce super hydrophobic concrete (Flores-Vivian et al. 2013). In order to make this advanced coating, three emulsions were used for the study. The super-hydrophobic emulsion was produced by mixing polymethyl hydrogen siloxane oil hydrophobic agent, metakaolin (MK) or silica fume (SF), and polyvinyl alcohol (PVA). This emulsion makes the concrete surface rough in micro-level and creates hierarchical surface that increases the contact angle as much as 156°. Authors concluded use of this type of coating can increase the durability of concrete as water absorption is controlled.

3.2.4 Cathodic prevention

If rebars in RC structures are prevented from corrosion using external current from the beginning, it is classified as cathodic prevention. It can be either impressed current or a sacrificial anode system. This technique can be used along with surface coatings on concrete but may need complex installation, if epoxy-coated rebars are used. Regular monitoring and maintenance of this technique can prolong the life of this protection system to 50 years or more, but this process is technically complex and require highly skilled personnel.

3.3 Summary

As discussed above, different methods can be applied in new and old structures to protect them from corrosion. Depending on the severity of the environment, a designer can select any single technique or a combination of techniques to ensure good health of the structure. The combination of two or more techniques has proven to be a better performer in most of the successful rehabilitation programs, such as Franklin Avenue Bridge, Minneapolis, Minnesota; Carrollton Bridge (Carroll County Bridge No. 132), Carroll County, Indiana; and Robert A. Booth (Winchester) Bridge, Douglas County, Oregon.

Some techniques are very effective and have long service life (50+ years), but they have high cost of installation and maintenance. With proper design and installation, cathodic prevention and use of stainless steel can have a service life of more than 50 years. But they are technically complex at production and installation stages. At the same time, some cheaper techniques can be adopted, which may require frequent maintenance.

Lack of understanding of ongoing processes, inadequate investigations, improper selection of a repair method, and poor implementation and supervision on site often

lead to an unsuccessful repair. Past case studies have shown that there are some critical factors for any successful rehabilitation. They can be summarized in order as below:

1. A thorough and comprehensive investigation including field testing and lab work is the backbone to any repair approach. Collecting samples from site, performing tests and analyzing results to find the cause of deterioration is the first step. Further steps in rehabilitation process rely on accurate prediction of cause and state of a structure's condition.
2. Exploration of repair techniques for an identified cause is the second step. A detailed economic analysis should be carried out on available methods. Coupling of two or more methods is usually found to have a synergic effect, which could be more effective than applying a single method. Based on the desired life of a structure, cost effectiveness can vary. Repair method should not be selected considering the cost and desired life only in some cases. In historical and cultural structures, post-repair appearance also plays a vital role in the selection process.
3. Implementation of designed repair method in a structure should be carried out carefully. Every procedure must be followed strictly for a particular method.
4. After the application of a repair technique, regular monitoring and maintenance should be carried out. This step helps to ensure that the maintenance performed on the structure lasts until the desired service life.

Chapter 4 Structural Condition and Repair Assessment

4.1 Current condition of pier cap

From the structural drawings (attached in Appendix A) provided by the Facilities and Maintenance Services (FMS) at Youngstown State University, the following parameters, as shown in Table 4-1, were collected for various load calculations:

Table 4-1: Information extracted from structural drawings

Number of spans	3
Span length	32 ft
Length of pier cap	25 ft 9 in.
Width of pier cap	2.5 ft
Depth of pier cap	Tapered section with 3 ft at center and 2 ft at end
Size of pier	2.5 ft x 2.5 ft
Width of Deck	21 ft 9 in.
Number of beams	6
Design Live Load	100 psf.
Reinforcement used in pier cap	3 layers of 8 #11bars (two layers at top and one layer at bottom)

Concrete cover dimension was not provided in the available construction drawings. Besides, information regarding the type of concrete and reinforcement used could not be confirmed with the FMS due to the lack of necessary construction documents. In order to learn more about the condition of the pier cap and to investigate the unknown properties of concrete, field investigations were performed.

At the bottom face of the pier cap, several previous patch repairs were visible. The patches were also delaminating in most of the repairs. Investigating the delaminated area, following conclusions were made:

1. Clear cover was measured using a Vernier Caliper (Fig. 4-1) and was found to be around 0.30 in.

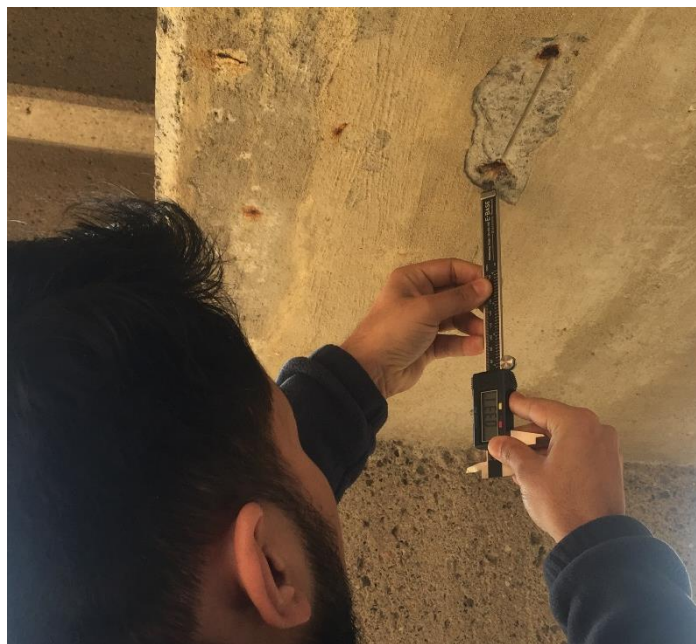


Figure 4-1: On-site concrete cover measurement using a Vernier Caliper.

2. Steel bars, which were undergoing corrosion and were exposed to open air after spalling, were smaller in size than shown in the drawings. From the measurement, it was found to have an average diameter of 0.25 in.

3. Besides the steel bars, some parts of stirrups were also exposed at the location of spalling. Stirrups were visibly in contact with the exposed steel bars.

The corroded steel bars found on site were different and smaller in size, which researcher assumes contractors used those smaller bars as temperature and shrinkage reinforcements. In order to know more about the structure, a more advanced investigation was considered. The purpose of additional investigations was to confirm the use of #11 bars in the pier cap and the clear cover provided to that reinforcement. In order to learn more about the reinforcement inside the hardened concrete, a Profoscope was used (Fig. 4-2). Profoscope is a simple, handy scanning tool that uses magnetic lines to locate rebars inside concrete. It is capable of predicting rebar diameter as well as the clear cover provided to that reinforcement. Using a Profoscope at the bottom of hammer-head cap, it was confirmed that #11 rebars were used with a clear cover of 1.50 in. on average.



Figure 4-2: On-site concrete cover measurement using Profoscope.

Besides clear cover, the properties of concrete and steel used were also unknown. In order to conduct this study, general construction practices during 1970-1980 were explored. According to Ohio Department of Transportation (ODOT) Specification, the trend concrete used between 1970s and 1980s was a prescriptive mix with 600 lb of cement and a water-cement ratio of 0.50. Specific gravities of construction materials were standard, such as 2.62, 2.68, 2.65, and 2.62 for natural sand, manufactured sand, limestone, and gravel respectively. Weights of aggregates used in this construction are given in Table 4-2.

Table 4-2: Weights of primary aggregates used in the construction

	Fine Aggregate (lb)	Coarse Aggregate (lb)	Total (lb)
Gravel	1160	1735	2895
Limestone	1285	1630	2915
Slag	1350	1360	2710

Specification records indicate that reinforcement was required to meet the ASTM A615 standard. Since it did not specify the grade of steel, an ODOT Cement and Concrete Engineer recommended Grade 60 reinforcing steel for this study as it is the current requirement for the construction of similar structures

Analyzing the conclusions made from different literature reviews and concrete clear cover measured on site, the cause of corrosion has been predicted to be the carbonation of concrete. Chloride attack may not be considered as a cause of corrosion herein since the sources of chloride ions, i.e. de-icing salt and saline water, is not present in the proximity of the site. If it was a highway bridge, there might have been the use of de-

icing salts during the winter. Ingression of aggressive ions through cracks due to physical deterioration was ruled out as well. Cracking due to sulfate attack was ruled out as there is no accumulation of white powder at the delaminated locations. Alkali-aggregate reaction is also eliminated from probable reasons of cracking since no sign of distributed hair cracks was visible on the concrete surface.

Furthermore, historical weather records were also analyzed to learn about the relative humidity trend in Youngstown, Ohio. Based on the analysis of weather records from 1961 till to-date found in the web, the average relative humidity of Youngstown-Warren area was found to be 73%. As discussed earlier, study performed by Cigna et al. (2003) suggest that probability of carbonation process is high in this range of humidity. This suggestion reinforces the assumption that the cause of corrosion in this case might be carbonation. However, the carbonation depth and the effective concrete depth should be compared to confirm this assumption.

4.1.1 Prediction of carbonation depth

Free CO₂ in the atmosphere reacts with alkaline hydroxides in the concrete to form carbonic acid. Carbonic acid, unlike other acids, does not attack the concrete paste, but it lowers its pH value. Carbonation reaction takes place on the outer surface of concrete and gradually penetrates into the concrete cover resulting in a low pH front. The outer part of concrete, which has lower pH value, is called the carbonation front. The depth of carbonation greatly depends on concrete composition, humidity, CO₂ concentration and temperature (Bertolini et al. 2013).

Carbon dioxide diffusion through concrete follows Fick's law of diffusion. Several studies (Pade et al. 2007; Broomfield 2006) have shown carbonation depth as a function of time, as given in Eq. (4.1).

$$d = k * \sqrt{t} \quad (4.1)$$

where, d is the depth of carbonation in mm, k is the carbonation coefficient in $\text{mm}/\text{year}^{1/2}$, and t is the time in years.

The carbonation coefficient depends on concrete composition and exposure conditions, such as the water-cement ratio of concrete, the CO_2 concentration at the surface, diffusion coefficient, some alkaline components in concrete, humidity, and temperature.

Broomfield (2001) performed a study with a range of old structures to determine the carbonation coefficient, k . For 11 buildings aged 8 to 24 years, k was determined to range from $1.2 \text{ mm}/\text{year}^{1/2}$ to $6.7 \text{ mm}/\text{year}^{1/2}$, whereas for seven car parking aged 14 to 41 years, k was determined to range from $2.2 \text{ mm}/\text{year}^{1/2}$ to $7.6 \text{ mm}/\text{year}^{1/2}$. In a similar study, Broomfield (2006) determined the range of k for the different porosity of concrete. For less porous concrete, with low water-cement ratio and well compacted, the value of k ranged from 2 to $6 \text{ mm}/\text{year}^{1/2}$; for medium porosity, k value ranged from 6 to $9 \text{ mm}/\text{year}^{1/2}$; and the value of k was more than $9 \text{ mm}/\text{year}^{1/2}$ for highly porous concrete i.e. poorly compacted with the high water-cement ratio.

The pier cap in this study was constructed almost 40 years ago. As a general practice of construction during that time, a water-cement ratio of 0.5 was used, which would create concrete with medium porosity. But the use of slag makes concrete overcome that low porosity. Analyzing both studies discussed earlier, carbonation coefficient is estimated as $4 \text{ mm}/\text{year}^{1/2}$ for this study. Using the square-root formula and the above-mentioned time and carbonation coefficient, carbonation depth is estimated as 25.30 mm (1 in.).

In the restoration program of Franklin Avenue Bridge located in Minneapolis, Minnesota, piers with the shallow concrete cover were affected by carbonation induced corrosion (Johnson et al 2017). On petrographic analysis, concrete used in the bridge was found to have a water-cement ratio of 0.50. This bridge was constructed in 1923 and was repaired in 1970. On an experimental investigation in 2013, carbonation depth was found to be 3/8 to 5/8 in. The average annual relative humidity from meteorological data for the area is found to be 70%. Pier caps of the bridge at YSU were also built in 1970s and the concrete had 0.50 water-cement ratio. The average annual relative humidity of Minneapolis, Minnesota, i.e. 70% is comparable to that of Youngstown, Ohio, i.e. 73%. Considering these similarities, carbonation depth of concrete in the pier cap structure should be comparable with that of the Franklin Avenue Bridge.

As an experimental method, pH indicator can be used to find the carbonation depth in existing structures. A pH indicator solution, 1% phenolphthalein solution mixed in ethanol or a mixture of 70% ethyl alcohol and 30% water, can be sprayed onto the surface of concrete fragments taken from the site (Villain et al. 2007). The indicator can be sprayed into concrete cores, fragments and drilled holes to find the extent of carbonation in the concrete. This indicator is pink at high pH value and remains clear at low pH. The change in color of solution from pink (un-carbonated zone) to colorless (carbonated zone) will determine the carbonation front.

Since concrete cores were not extracted from the structure, carbonation depth could not be calculated experimentally. Although conventional square-root law overestimates the carbonation depth, the result is comparable to the case study in Northern part of the U.S. Therefore, a carbonation depth of 1 in. will be used in this study.

Based on the clear cover (0.30 in. for smaller bars) and the estimated carbonation depth (1 in.), it can be concluded that the 1 in. carbonation front has already extended passed the outer layer of reinforcement in the structure. Therefore, the cause of corrosion in the reinforcement is concluded as the carbonation of the outer layer of concrete in the pier cap. But an important thing to notice is this corrosion has only affected the smaller bars, not the main reinforcement yet.

4.1.2 Current condition of corrosion in reinforcement

The current condition of corrosion in reinforcement inside the concrete pier cap can be examined by different electrochemical techniques and the corrosion rate can be determined as well. The half-cell potential mapping (ASTM C876) is one of the widely used and standard techniques to investigate the state of corrosion in rebars inside concrete. Measurement of concrete resistivity can also be used to estimate corrosion rate. Recent technique uses piezo-impedance transducers to investigate carbonation progression and corrosion rate in carbonated concrete.

Upon unavailability of these techniques, an analytical study was performed to predict the corrosion rate in the pier cap. Many studies relating to prediction of corrosion rate were found (Bertolini et al. 2013; Broomfield 2006), however, no study was found to predict the percentage area loss with time in a carbonated concrete structure. Therefore, an analytical study was considered for the prediction of the state of corrosion in rebars in the concrete pier cap.

A set of data from a study performed by Apostolopoulos and Papadakis (2007) was used for this study, which documented the percentage loss of steel mass in a 30-year old industrial building located in Aegion, Greece. This study was selected for a few reasons. Firstly, the structure was in an industrial city similar to Youngstown, where

the relative humidity is between 50 to 70% throughout the year. Secondly, the corrosion in that study was predicted to be due to the carbonation of concrete, similar to this case. Finally, the age of structure studied matches the age of the structure in this study. Using the percentage loss of mass in reinforcement, the percentage loss in the area of reinforcement in this study was calculated to be 5.52% (Appendix D).

Using the calculated percentage loss in area, the current area of reinforcement in the pier cap was calculated to be 1.47 in² for a #11 bar, which has an original area of 1.56 in².

4.1.3 Strength of concrete in existing pier cap

Hardened concrete can be tested in two ways, i.e., destructive testing and non-destructive testing. The destructive testing is performed by testing a concrete sample where load is applied until failure to find its properties. Non-destructive testing (NDT) is a method of testing materials where the material continues to remain in service for which it is intended. It is performed on the structural elements without taking out any sample from the structure. Compressive strength test, split tensile test of concrete cylinders, and flexure test are standard destructive tests, whereas rebound hammer test, ultrasonic pulse velocity test, and pull out test are often used as non-destructive tests for concrete.

Quite often, it becomes necessary to test concrete properties of existing structures to learn the reliability and appropriateness of the structure for the intended purpose. In such cases, it is ideal to use non-destructive testing techniques without damaging the concrete. Fundamental parameters, such as strength, surface hardness, modulus of elasticity, density, surface absorption as well as reinforcement location, size, and cover distance can be examined from this type of testing. Non-destructive testing can be

completely non-destructive in nature, such as rebound hammer test and Profoscope test, where there is no damage in concrete at all. In case of partially destructive testing, such as the pull-out test, the concrete surface needs some repair after the test.

4.1.3.1 Schmidt Hammer Rebound Test

Developed in 1948 by Schmidt, Schmidt hammer rebound test is one of the fastest as well as cheapest non-destructive tests. It is a surface hardness test that measures the rebound number. In a simple understanding, it works on the principle that the harder the material, the higher the rebound. Although there is no direct theoretical relationship between surface hardness and compressive strength of concrete, empirical correlation can be established between these two parameters (Bungey et al. 2006). Standard correlation curves generated from extensive data collection and tests can be used to measure in-situ concrete strength, or calibration charts can be generated in the lab with the known strength of concrete specimens (Szilagyi et al. 2011).

A standard rebound hammer consists of a casing, spring loaded weight that slides on a guide bar, locking and release mechanism, and a pointer to measure the rebound number. An N-type Schmidt hammer weighs about 2 kg and transfer the impact energy of 2.207 N.m onto the concrete surface through the plunger (Proceq 2016). A typical section of a Schmidt hammer is shown in Fig. 4-3.

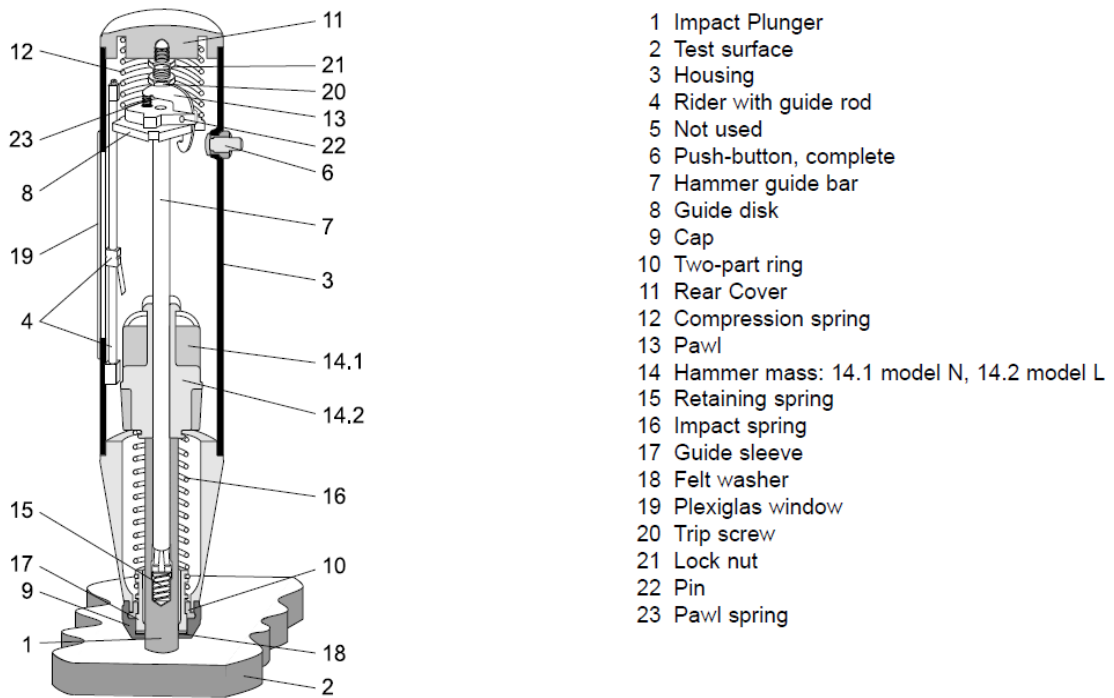


Figure 4-3: Longitudinal section of a Schmidt hammer.

During the test, the plunger hits the concrete surface and bounces back through the hammer guide. The rebound number is due to the elastic reaction of the impacting mass and the hardness of the surface it strikes. Pressing the lock button, rebound number can be recorded. The hammer can be used in horizontal, vertically downward or vertically upward position as well as at some angles, provided that the hammer mass impacts the concrete surface perpendicularly. However, the position of the relative mass affects the rebound number due to the action of gravity. Therefore, a graph representing different positions should be used according to the testing position. A typical graph provided by the manufacturer for vertical, horizontal upward and downward position of equipment is shown in Fig. 4-4.

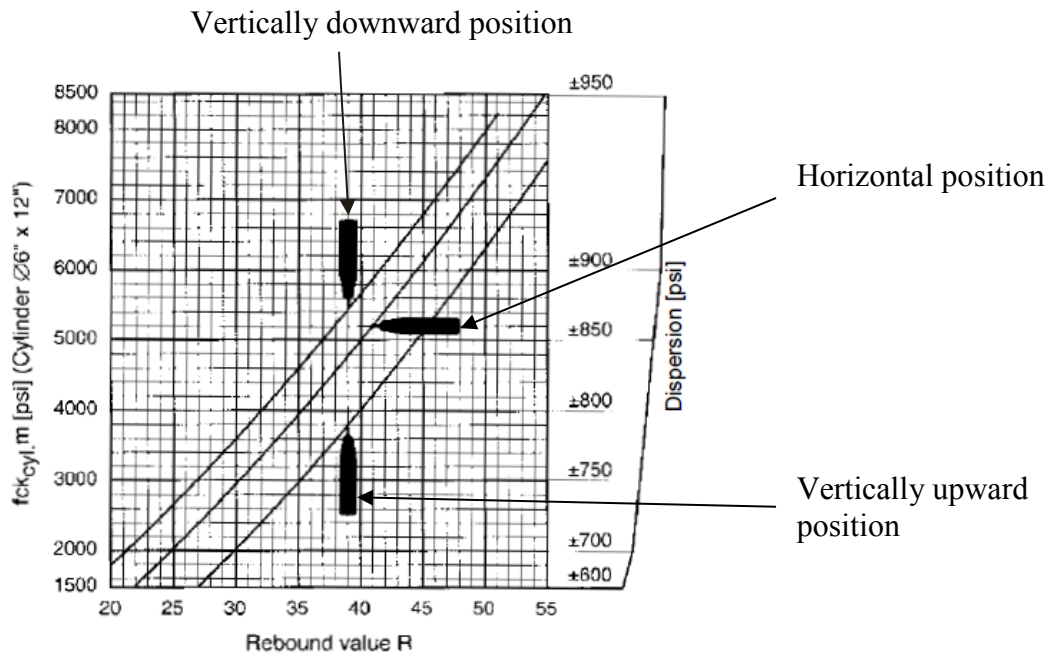


Figure 4-4: Conversion curves for different positions of Schmidt Hammer.

The graphs provided by the manufacturer are based on a very large number of test results for a wide range of material properties. It may overestimate or underestimate the concrete strength according to some literature (Brencich et al. 2013). So, it has been recommended to develop case-specific graphs from laboratory tests for more reliable prediction of the compressive strength of concrete. Case-specific charts can be obtained by performing rebound hammer tests on standard samples of known strength.

The rebound hammer depends on various factors. As discussed earlier, a rebound number varies according to the position of the hammer. In addition, it depends on the thickness of the cement paste, surface finish, edge distance, aggregate type, carbonation, and the type of cement and sand used in the concrete. The rebound number also varies due to local variations within the concrete itself. If the hammer is struck near the steel reinforcement or large aggregates, it will show a higher rebound value than if it is struck on the cement paste. Similarly, a test carried out away from the edge would

provide higher value than taken near the edge. Carbonation tends to make the concrete cover harder as it is exposed to the external environment. Thus, a higher value obtained from the carbonated surface would overestimate the compressive strength of concrete. In order to account for all these factors affecting the rebound number, ASTM encourages taking 12 readings in close intervals, disregarding any dubious value, and applying the correction factors for hammer position and carbonation.

4.1.3.1.1 Corrections

Gravity is one of the factors that directly affect the rebound number. Considering identical conditions, vertically downward position of the hammer gives the lowest value, whereas vertically upward position gives the highest value (Barton and Choubey 1977). These authors performed a number of tests on rocks to learn the shear strength of rock joints. They defined general correction values for different positions. All rebound numbers were normalized to the vertically upward position. For the vertically downward position, correction values for 20, 30, 40, 50 and 60 were defined as -8.8, -7.8, -6.6 and -4.0, respectively. Correction values for intermediate angles between vertically upward and vertically downward positions were also given in the study.

Rebound numbers obtained from old concrete structures are suggested to be corrected according to the age and the strength of concrete. Aydin et al. (2010) performed rebound hammer tests on several cube specimens and on the surface of an existing reinforced concrete structure. Later, in-situ sample cores (75 mm diameter) were extracted from the structures, and destructive tests were performed to find the compressive strength of the concrete in those specimens. Results were correlated with the strength of concrete specimens. Although the curves were almost parallel, strength obtained from the rebound value for the existing building was higher than the strength

obtained from the core samples. So, various strength transformation factors, as shown in Table 4-3, were suggested to account for the age of concrete.

Table 4-3: Strength transformation factors, Aydin et al. (2010)

Strength interval (ksi)	Strength transformation factors
< 1.45	0.51
1.45 – 2.18	0.62
2.18 – 2.90	0.67
2.90 – 3.63	0.72
3.63 – 4.35	0.75
4.35 – 5.08	0.78
5.08 – 5.80	0.80
5.80 – 6.53	0.81
6.53 – 7.25	0.82

Moreover, for existing structures, it was also suggested to use correction factors for other influencing factors, such as surface carbonation, moisture condition, characteristics of the mixture, rate of hardening, and curing type.

Carbonation results in a reduction of porosity of concrete that increases the hardness of the surface. It greatly influences the rebound number and strength obtained from NDT (Kim et al. 2009). As a result of carbonation, calcium carbonate (CaCO_3) is formed, which has a larger volume than the calcium hydroxide [$\text{Ca}(\text{OH})_2$]. This phenomenon increases the strength and hardness of the concrete exposed to the external environment. This effect can be seen more in older structures. Hannachi et al. (2014) found rebound numbers obtained from carbonated concrete that can be 50% more than those without carbonation.

A rebound number obtained from the rebound hammer test is mainly based on the properties of concrete near the surface layer. For older concrete, the outer layer is usually harder than the core concrete due to carbonation. As a result, a rebound hammer test overestimates the strength of concrete if the correlation between rebound number and the compressive strength established in 28 days is used (Kim et al. 2009).

In order to account for the overestimation of rebound number for older concrete, various studies have been performed to suggest correction factors for different ages of concrete (Tanigawa et al. 1984; Kim et al. 2009; Kim et al. 2002, Breccolotti et al. 2013).

Tanigawa et al. (1984) recommended an equation to estimate the compressive strength of concrete in existing structures with strength reduction factors as 1.0, 0.9 - 1.0 and 0.9 for concrete aged less than 4 weeks, 4 to 26 weeks, and over 26 weeks, respectively. Architectural Institute of Japan (AIJ 1983) also proposed strength reduction coefficients for the adjustment of carbonation effect in old concrete structures, as demonstrated in Table 4-4.

Though both of these sets of coefficients are for the same purpose, studies have shown that recommendation by Tanigawa et al. (1984) is suitable for medium strength concrete with 28-day compressive strength of 5.80 ksi. However, recommendations made by AIJ is found to be more appropriate for low strength concrete with 28-day compressive strength of 3.63 ksi (Kim et al. 2009). This infers that carbonation has a significant effect on concrete strength according to their study, and thus influences the rebound number. Therefore, Kim et al. (2009) performed a series of experiments using concrete with different strengths and developed a new equation (Eq. 4.2)

$$\alpha(R, t) = 1 - \frac{0.3064}{(R/34.7)^{4.2}} \left(\frac{t - 28}{t + 94} \right) \quad (4.2)$$

Where, R is rebound number obtained from the concrete structure and t is time in days.

Table 4-4: Strength reduction coefficients (AIJ 1983)

Age of concrete (days)	Strength reduction coefficient [α]
28	1.0
50	0.87
70	0.84
100	0.78
200	0.72
500	0.67
1000	0.65
3000	0.63

Breccolotti et al. (2013) performed finite element analysis to study the impact of the plunger in rebound hammer test on the concrete surface at different carbonation stages. Based on the analysis, two formulas for correction factor (k_c) were developed, one of them is given by Eq. (4.3).

$$k_c = 0.7 + e^{-1.2 * e^{\frac{x_c}{7.5}}} \quad (4.3)$$

Where, x_c is the thickness of the carbonated concrete layer. This formula does not consider the strength of concrete but only the carbonation depth. The second formula that incorporates the strength of concrete as well as its carbonation depth is shown in Eq. (4.4).

$$k_c = 0.7 + e^{-1.2 * e^{\frac{x_c}{7.5}}} \times e^{\left[(x_c)^{\frac{1}{3}} \times 0.08 \times \left(1 - \frac{24}{f_c} \right) \right]} \quad (4.4)$$

Where, k_c is the correction factor, x_c is the thickness of carbonated concrete layer, and f_c is the compressive strength of concrete.

Szilágyi et al. (2011) introduced a phenomenological constitutive model (SBZ-model) and formulated that surface hardness of concrete is a time-dependent material property. Rebound index and concrete strength were formulated as a function of carbonation depth and time, as shown by Eqs. (4.5) and (4.6).

$$R(t)/R_{28} = \frac{1}{1 + a_{11} \cdot x_c^{a_{12}}} \quad (4.5)$$

$$f_c(t)/f_{c,28} = \exp(a_4 \cdot (1 - (28/t)^{a_5})) \quad (4.6)$$

Where, a_{11} , a_{12} , a_4 , and a_5 are coefficients as defined in the study, t is the age of concrete, $R(t)$ and R_{28} are rebound numbers at time t and 28 days, respectively. The coefficients were dependent on the water-cement ratio of the concrete and were found empirically in the study.

4.1.3.1.2 Calibration

In a research conducted by Aydin et al. (2010), it was found that the application of rebound hammer test on existing structures is not suitable for estimating the compressive strength of in-situ concrete since the direct use of the 12 rebound hammer numbers resulted in a high variation. However, they made a recommendation that the rebound hammer can be used for calculating the compressive strength of concrete after proper calibrations. Although manufacturer provides calibration charts for the equipment, it cannot be used to obtain reliable estimation as rebound number depends on various factors discussed earlier. Since strength and rebound values do not have any particular relationship to different circumstances, users are encouraged to develop their own charts according to their situations (Carino 1994).

For this study, a standard N-type Schmidt hammer, as shown in Fig. 4-5, was used to estimate the compressive strength of concrete.



Figure 4-5: Standard N-type Schmidt Hammer.

Paudel (2016) recently used the same Schmidt hammer for similar tests for his Master's thesis, where he calibrated it in a lab at YSU. So, the calibration data obtained by him in the lab has been used for this study. Nine standard cylinders (4 in. diameter, 8 in. height) were cast in the Strength of material lab in Moser Hall at YSU. For each cylinder, 10 rebound numbers, excluding the highest and the lowest, were recorded for the vertically downward position of the hammer. These samples were later tested for destructive compressive strength tests following ASTM C 39. They were loaded until failure to find the actual compressive strength of the samples. Using the average rebound number for each specimen and the results from the compressive tests, a calibration graph was developed as shown in Fig. 4-6.

From the analysis of the data, a regression equation (Eq. 4.7) was established to correlate the rebound number and the compressive strength of concrete, with a regression coefficient (R^2) of 0.89.

$$y = 105.04x + 818.08 \quad (4.7)$$

Where, y is the compressive strength of concrete in psi and x is the rebound number.

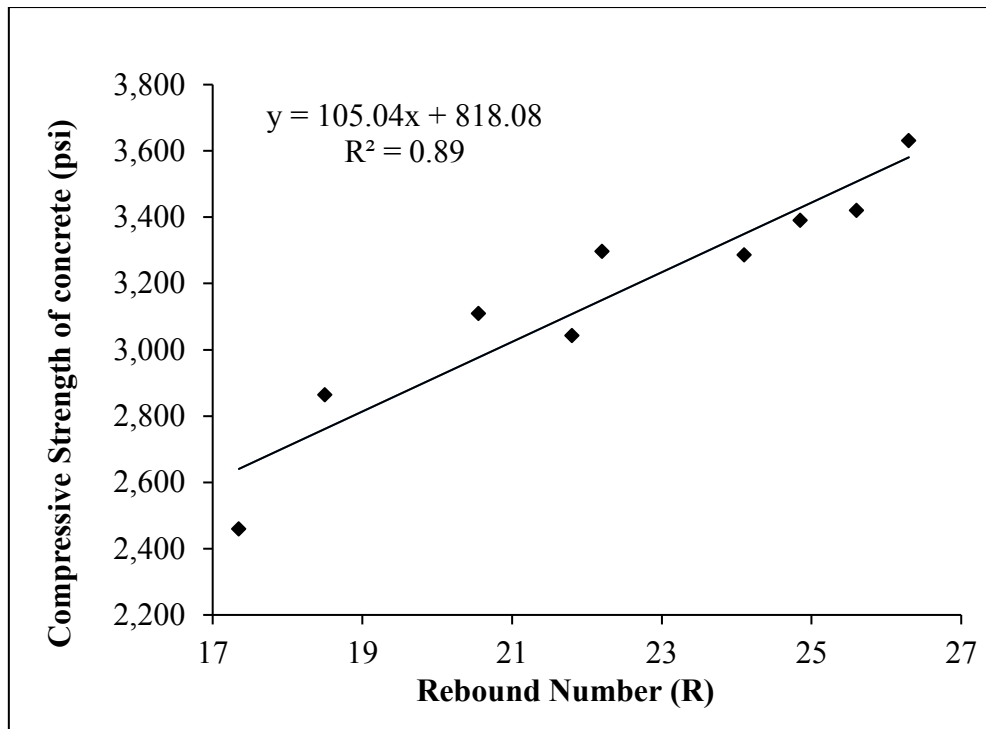


Figure 4-6: Plot of rebound number and compressive strength of test cylinders.

4.1.3.1.3 Field data and strength estimation

Following the ASTM C 805 (ASTM 1997), several rebound hammer tests were performed on the bottom face of the pier cap using a standard N-type Schmidt hammer.

Following steps were followed to find the rebound number of the concrete surface:

- a. Test surface was selected avoiding cracks and uneven surfaces. Also, concrete surface directly under reinforcement was avoided. Patched surface was also avoided for reliable results.
- b. Test surface was smoothed by abrasive stone.
- c. Hammer was held firmly in a vertically upward position and pushed towards the test surface until it impacted. A lock button was pressed to lock the plunger, and the nearest whole number was recorded from the scale as a rebound number.
- d. Two sets of 12 rebound numbers were recorded from two separate locations. None of the test points were closer than 1 in. in both locations.

- e. The highest and the lowest values were disregarded, and 10 readings were used to find the average value.

Fig. 4-7 shows a picture of the rebound hammer test performed on the bridge pier cap. From two locations, average rebound numbers of 10 readings were found to be 50.1 and 50.9. Necessary calculations including the application of the reduction and correction factors show the compressive strength of concrete used in pier cap to be 3.5 ksi (Appendix C).



Figure 4-7: On-site Rebound hammer test.

4.2 Structural analysis of pier cap

Using available construction drawings and appropriate dead and live loads, maximum factored bending moment on the structure was determined to be 1976.16 kip-ft at the cap-column interface of the hammerhead pier cap (Fig. 4-8). The moment capacity and stresses at various critical locations (Fig. 4-9) of the pier cap were determined for two

different conditions: (i) at the initial state without any corrosion in reinforcement and (ii) at the present state with reduced reinforcement area (Appendix E).

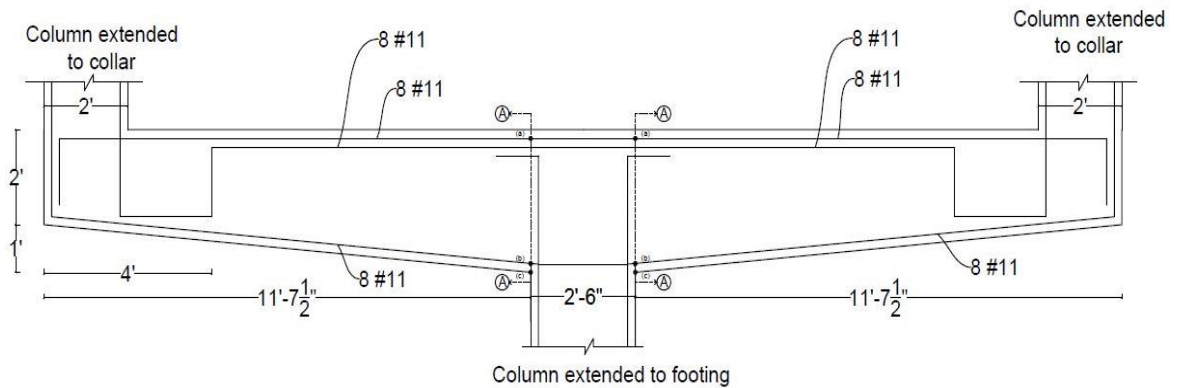


Figure 4-8: 2D elevation of pier cap.

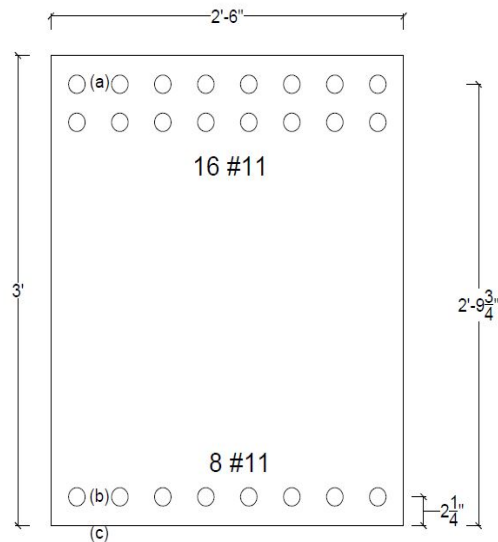


Figure 4-9: Section at cap-column interface (A-A) showing study locations.

Moment capacity and stresses at various locations of the pier cap were determined as given in Table 4-5.

Results from the structural analysis verify that the present capacity of the bridge pier cap is structurally sufficient to support the present loads on the pedestrian bridge. There is no need to strengthening the structure at the present condition. Instead, the ongoing corrosion problem should be addressed to prevent further deterioration of the pier cap so that it can function well until full replacement.

Table 4-5: Comparison of moment capacity and stresses at various locations in different conditions

i. Initial condition				
Due to loads		Strength of material		Remarks
Bending Moment	1976.16 k-ft	Moment Capacity	3315.69 k-ft	ok
Stress at (a)	34.94 ksi	Strength of Steel	60 ksi	ok
Stress at (b)	31.83 ksi	Strength of Steel	60 ksi	ok
Stress at (c)	2.43 ksi	Strength of Concrete	3.5 ksi	ok
ii. Current condition (with 5.52% rebar area loss)				
Due to loads		Strength of material		Remarks
Bending Moment	1976.16 k-ft	Moment Capacity	3132.33 k-ft	ok
Stress at (a)	37.04 ksi	Strength of Steel	60 ksi	ok
Stress at (b)	32.90 ksi	Strength of Steel	60 ksi	ok
Stress at (c)	2.52 ksi	Strength of Concrete	3.5 ksi	ok

4.3 Recommended repair

After structural analysis, it can be concluded that the pier cap is structurally sufficient to support present loads and does not need any strengthening. In order to control the ongoing corrosion in the pier cap reinforcement, which will further deteriorate with time, necessary repair should be done.

Considering the finding that the cause of ongoing corrosion in reinforcement is carbonation of concrete and inadequate concrete clear cover, a patch repair coupled with hydrophobic coating can be an effective way to address the current corrosion problem in the pier cap. Based on several factors, such as simplicity of application and effectiveness of the repair method, this approach was recommended. A more efficient method, such as cathodic prevention, will cost more to install.

A patch repair should follow three steps: i) removal of affected concrete ii) cleaning rust and dust from exposed/affected reinforcement, and iii) placing of concrete cover. Following the patch repair, silane or siloxane is recommended to be coated on the newly repaired concrete surface.

Spalled and delaminated concrete should be removed at least 1 in. or until sound concrete is exposed. At locations where reinforcement is exposed, at least 1 in. depth of concrete beyond the reinforcement or until sound concrete is found needs to be removed. Concrete can be removed using sawcutting or a light chipping hammer. If concrete has to be removed beyond the steel reinforcement, rebars should be cleaned by grit blasting or high-pressure water jet so that they are free of loose rust and dust.

In order to provide sufficient bond between original concrete substrate and repair patch, substrate concrete should then be prepared by sandblasting or grit blasting. The prepared surface should be rough and free from dust for sufficient adhesion. For improving adhesion, water-cement grout or epoxy systems should be sprayed on the prepared dry surface. Repair material should be placed on the prepared surface while the surface is wet to ensure proper bond.

For a patch repair, low-slump dense concrete is recommended because of its efficiency in extending service life of many bridge structures in the country. Although some popular concrete overlays, such as silica-fume and latex-modified overlays, work better, they are more expensive and do not have any significantly high service life compared to low-slump overlays. Previous studies have shown that low-slump overlays have potential service life of 30 to 50 years, if the patching procedures are followed correctly. Besides, it is easy to produce this type of concrete by altering the cement content and water-cement ratio in ordinary concrete mix design. Increasing the cement

content to more than 800 lb/yd³ of concrete with a water-cement ratio of as low as 0.30 to 0.32 can produce low-slump dense concrete. Superplasticizer can be used to ensure good workability.

A patch repair might solve the ongoing corrosion problem. In order to prevent further ingress of aggressive elements in future, the outer surface of the repaired concrete should be treated as well. The application of hydrophobic sealer will control the capillary action at the surface, thereby preventing water and aggressive ions from penetrating into the concrete surface. Studies have shown that the hydrophobic sealers, i.e., silane and siloxane, have performed significantly better than pore blockers, such as water-based and solvent-based epoxy. Besides, they are easy to apply and can be applied in any structure. The application of silane or siloxane should be carried out in a dry surface. In order to ensure good protection, it is recommended to apply one of these hydrophobic sealers in every five years.

Chapter 5 Summary, Conclusions and Recommendations

5.1 Summary

Corrosion in reinforced concrete structure is one of the most common problems in old structures as well as maritime structure. A case study was done on 40 years old RC pier caps experiencing corrosion to find the cause and solution to the problem. The study focuses on three hammerhead pier caps, supporting a pedestrian bridge that connects Moser and Cushwa Hall that have areas of spalling of the concrete cover leading to exposure of some bars on the bottom face.

An extensive literature review was performed on the possible causes and solution of corrosion in a RC structure. Corrosion are found to be of two types, carbonation induced corrosion and chloride induced corrosion. Furthermore, cracks formed by any physical deterioration provide pathways for aggressive elements to reach the steel rebars. Different repair options for corrosion problem in existing structures has been discussed. Proactive approach, which would prevent similar kind of problem in new construction has also been discussed.

A detailed structural analysis was done to investigate the load carrying capacity of pier cap in current condition for design loads. Documented construction drawings provided by the FMS were studied. The documentation was lacking some vital information of the structure, such as strength of the concrete used, grade of steel rebars and concrete cover dimension. Information on grade of steel rebars and concrete type used were obtained from construction specifications documented in ODOT. Besides, the structure was investigated with Schmidt Hammer and Profoscope to approximate concrete strength, rebar location and clear cover.

Finally, considering the state of structure and cause of the deterioration, a suitable repair approach was recommended.

5.2 Conclusions

After a detailed field investigation and comprehensive literature review, the reason for the corrosion, in the pier caps, was found to be carbonation. The principal cause of carbonation induced corrosion was shallow clear cover.

It was noted that the built structure varied in rebar arrangements from the original drawings. Smaller bars (dia 0.25 in.), which were not shown in drawings, existed in real structure and were the bars that got exposed. Researcher assumed the smaller bars were used as temperature and shrinkage reinforcement. Even though the constructor provided 1.5 in. clear cover to main reinforcement, adequate cover to temperature and shrinkage reinforcement were not offered. From exposed area, cover to a smaller steel bar was measured to be 0.30 in. on average. This cover thickness is less than the predicted carbonation depth of 1 in.: predicted analytically. For validation, this finding was compared to a case study that gave similar carbonation depth experimentally in similar environmental condition. Since the carbonation front has already passed beyond the steel rebars, it was predicted that corrosion occurred due to carbonation.

Chloride-induced corrosion is common in structures subject to deicing salts or saline water. Being a pedestrian bridge between two educational buildings with no access to deicing salts or saline water, chloride-induced phenomenon was ruled out as an active cause of corrosion in the substructure. Ingression of aggressive ions through cracks due to physical deterioration was not considered either. No signs of crack due to sulfate attack, alkali-silica reaction, freezing and thawing action, shrinkage or support settlement were seen on the structure.

After careful consideration of cause of corrosion and strength of structure, patch repair coupled with hydrophobic coating was recommended as a repair option. These two methodologies are expected to produce a synergic effect to solve the ongoing problem in structure. While the patch repair takes care of current corrosion process, coating of concrete will prevent possible corrosion in future. Low-slump dense concrete is recommended for patch repair, and silanes or siloxane for coating. If the repair is done following the standard procedure and if the coating is applied every five years, the service life of repair is expected to be 20 to 25 years.

From detailed structural analysis, it was seen that corrosion of reinforcement increases the stresses at steel and concrete. In this case study, reduction in steel rebar area shifted the neutral axis toward compression half, due to which stresses were increased in different locations of a section.

5.3 Limitations of the study

It is mostly a descriptive study, except the strength of concrete, location of the rebar and the clear cover thickness was found experimentally. A formula was used to calculate carbonation depth that was compared to a case study, of a structure with similar circumstances, for the validation. Instead, experimental approach may have been used by extracting a core and performing phenolphthalein spraying test. From the extracted cores, exact carbon and chloride content could have been determined to strengthen our findings. Moreover, corrosion condition of reinforcement was also estimated through descriptive study, which could have been determined by condition surveys like half-cell potential measurement, resistivity measurement or other standard methods. Determining the exact rate of corrosion and exact cost of rehabilitation could have been compared for different rehabilitation methods.

5.4 Recommendations for future work

There is a need for more studies to rationally estimate the area loss of reinforcement due to carbonation-induced corrosion. Considering various factors that affect the carbonation process, one can come up with a general relation to estimate area loss or loss in diameter. This relationship would predict the state of rebars at different point of time, which would give a rough idea on when to start extensive condition survey in a structure. Based on the factors affecting carbonation, the relation may include several parameters like concrete quality, cover thickness, environmental factors, and reinforcement type.

New approach of repair method should be explored. Bio-concrete is a new type of construction material that is still being studied. The use of bio-concrete, which is capable of healing cracks itself, can be advantageous to stop the degradation caused by other physical phenomenon. This type of concrete can be used as overlay in existing structures or may be used in new construction.

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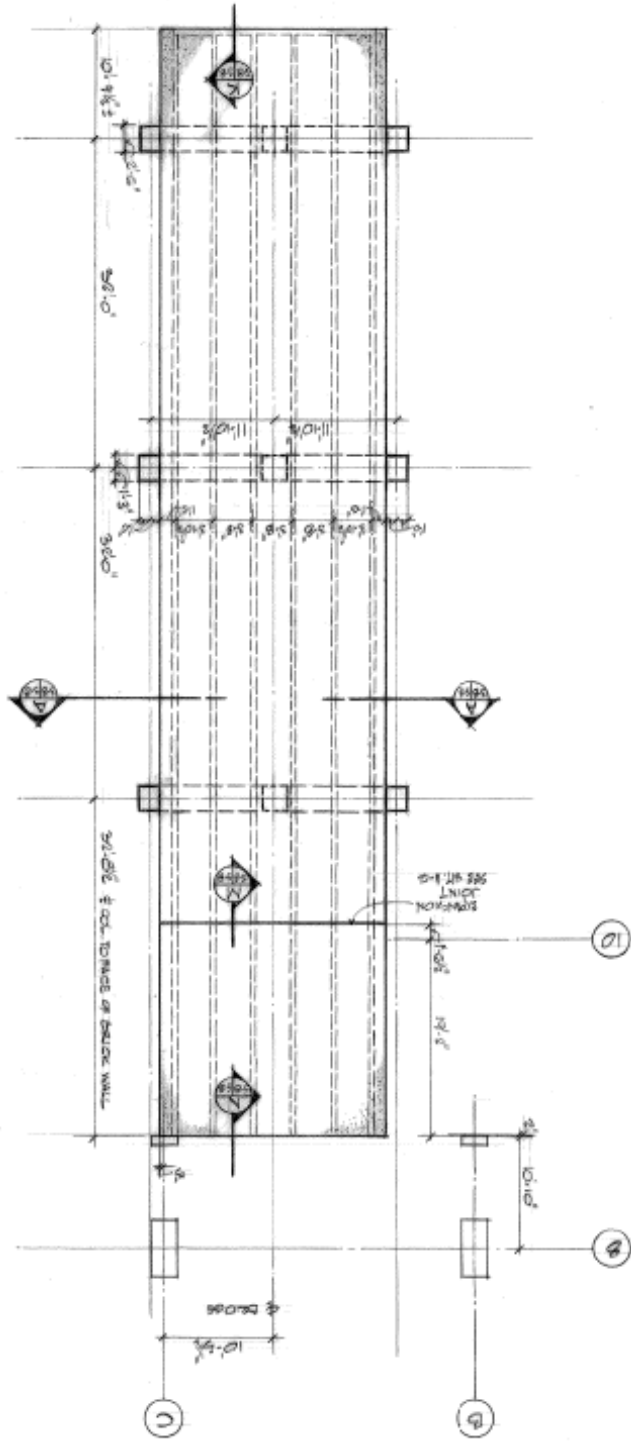
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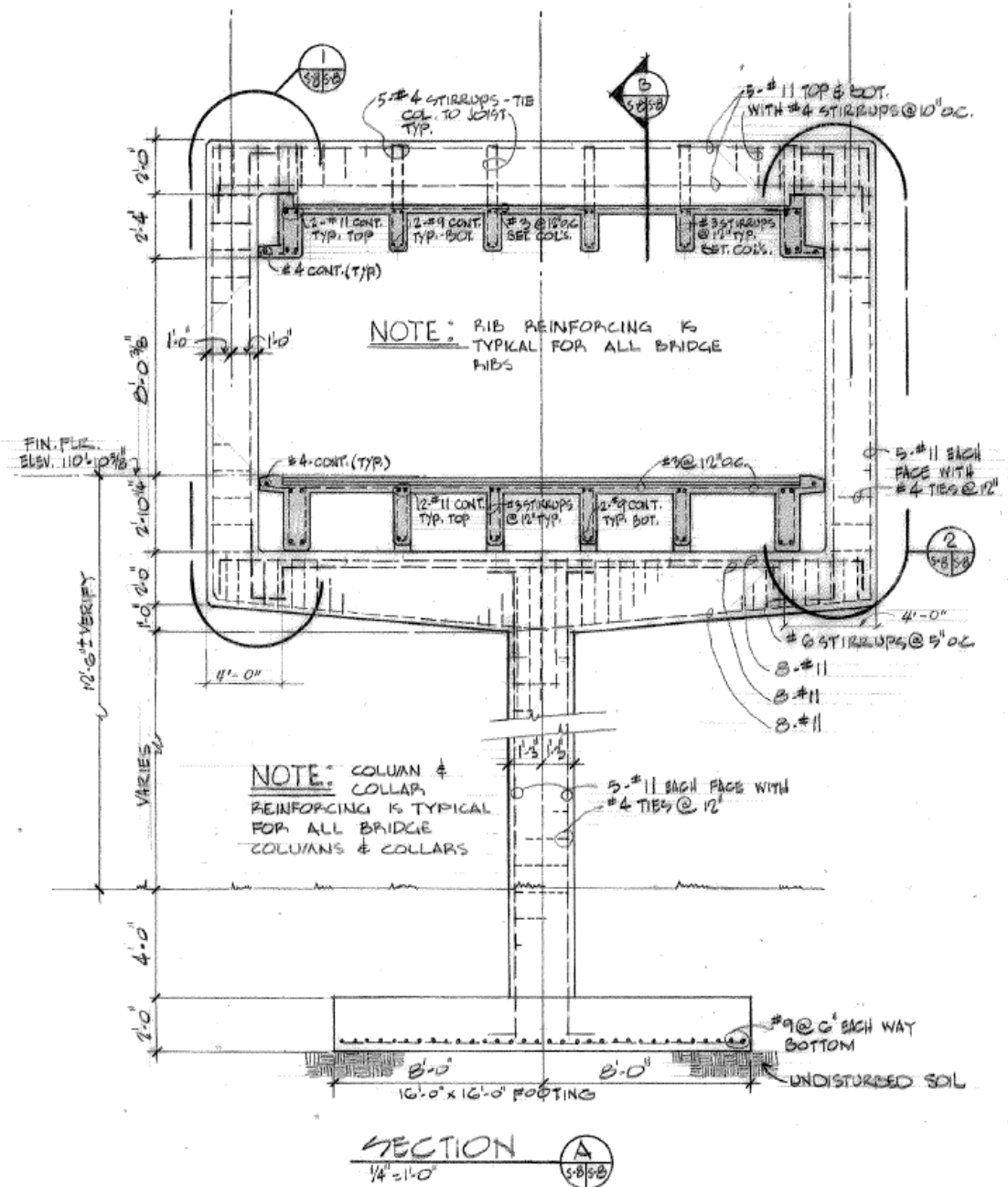
APPENDICES

Appendix A. Structural drawings of bridge provided by FMS at YSU

1. Bridge Plan



2. Bridge Cross-section



Appendix B. Correspondence with Daniel Miller, PE, ODOT

D Daniel.Miller@dot.ohio.gov
Mon 2/6, 3:16 PM
Bhishan Poudel; Lisa.Zigmund@dot.ohio.gov; Prasad.Kudlapur@dot.ohio.gov; Tim.Keller@dot.ohio.gov

Bhishan,

The trend for concrete between the 1970s and 1980s was a prescriptive mix with 600 lbs. of cement and a w/c ratio of 0.50. The aggregate weights are as follows:

	Fine Agg.	Coarse Agg.	Total
Gravel	1160	1735	2895
Limestone	1285	1630	2915
Slag	1350	1360	2710

Specific gravities were standard as well, they are as follows:

Natural Sand: 2.62
Manufactured Sand: 2.68
Limestone: 2.65
Gravel: 2.62

These are the design parameters that may have been used when the pedestrian bridge was constructed. If you have any other questions or concerns, please let me know.

Regards,

Dan Miller, PE
Cement and Concrete Engineer
Office of Materials Management, ODOT
[1600 W. Broad St.](#)
[Columbus, OH 43223](#)
Phone: (614) 275-1325
Cell: (614) 398-7748
Fax: (614) 887-4048
Email: daniel.miller@dot.ohio.gov
Website: <http://www.dot.state.oh.us/Divisions/ConstructionMgt/Materials/Pages/default.aspx>

D Daniel.Miller@dot.ohio.gov
Yesterday, 10:39 AM
Bhishan Poudel

Bhishan,

Looking through old spec books in my office, reinforcing steel was required to meet the requirements of ASTM A615, but at the time did not specify grade of steel. My assumption is that Grade 60 would've been the requirement as it is the current requirement at ODOT. If you have any other questions or concerns, please let me know.

Regards,

Dan Miller, PE
Cement and Concrete Engineer
Office of Materials Management, ODOT
[1600 W. Broad St.](#)
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Appendix C. Rebound number test and calculations:

Date: January 20, 2017

Position of Hammer: Vertically upward position

Reading No.	Location 1	Location 2
1	50	52
2	54	48
3	50	51
4	52	52
5	45	52
6	44	50
7	52	50
8	52	52
9	50	52
10	52	50
Average	50.1	50.9

$$\text{Average rebound number (x)} = \frac{50.1+50.9}{2} = 50.5$$

Correction due to position: -5.3

$$\text{Corrected value} = 50.5 - 5.3 = 45.2$$

Strength calculation from calibration equation:

$$y = 105.04x + 818.08$$

$$\text{or, } y = 105.04 * 45.2 + 818.08 = 5565.89 \text{psi}$$

Reduction factor due to age (for structure older than 3000 days) = 0.63

$$\text{Final result} = 0.63 * 5565.89 = 3506.51 \text{ psi} \approx \mathbf{3.5 \text{ ksi}}$$

Appendix D. Calculation of Area loss of rebars using data of Apostolopoulos and Papadakis (2007)

As mass (m) = density (ρ) x Volume (V)

$$\text{Percentage mass loss, } \Delta m = \frac{m_o - m_f}{m_o}$$

$$\text{or, } \Delta m = \frac{\rho \times V_o - \rho \times V_f}{\rho \times V_o}$$

$$\text{or, } \Delta m = \frac{V_o - V_f}{V_o}$$

But, Volume (V) = Area (A) x Length (l)

And, length was constant for all samples, 250 mm.

$$\text{So, } \Delta m = \frac{A_o - A_f}{A_o}$$

Hence, final area for each specimen, $A_f = A_o(1 - \Delta m)$

Sample	Initial Area A_o (mm ²)	Mass loss Δm (%)	Final Area A_f (mm ²)	Area Loss ΔA (mm ²)
B1	78.54	3.71	75.63	2.91
B2	78.54	5.69	74.07	4.47
B3	78.54	7.63	72.55	5.99
B4	78.54	4.42	75.07	3.47
B5	78.54	6.63	73.33	5.21
B6	78.54	8.88	71.57	6.97
B7	78.54	6.8	73.20	5.34
B9	78.54	4.81	74.76	3.78
B10	78.54	9.79	70.85	7.69
B13	78.54	2.77	76.36	2.18
B15	78.54	4.58	74.94	3.60
B16	78.54	9.74	70.89	7.65
B17	78.54	0.25	78.34	0.20
B18	78.54	3.3	75.95	2.59
B19	78.54	3.48	75.81	2.73
B20	78.54	5.8	73.98	4.56
Average	78.54		74.21	4.33

$$\begin{aligned}\text{Average area loss (\%)} &= \frac{A_o - A_f}{A_o} * 100\% \\ &= \frac{4.33}{78.54} * 100\% = \mathbf{5.52\%}\end{aligned}$$

Appendix E. Calculation of moment capacity and stresses in concrete and steel

Concrete Strength (f_c') = 3.5 ksi

Yielding Stress of Rebars (f_y) = 60 ksi

$$\text{Modular ratio } (\eta) = \frac{E_s}{E_c} = \frac{29000 \times 1000}{57000 \sqrt{f_c'}} = \frac{29000 \times 1000}{57000 \sqrt{3500}} = 8.59 \approx 8$$

Design Moment (M_u) = 1976.16 kip-ft

Analyzing the structure at location of maximum moment, Section A-A

1. Initial Condition

Area of #11 rebar (A) = 1.56 in²

Area of rebars in tension zone (16 #11) = 24.96 in²

Area of rebars in compression zone (8 #11) = 12.48 in²

a. Calculation of moment capacity:

Equating compression and tension forces,

$$A_s f_y = 0.85 f_c' \beta_1 c b + A_s' \left(\frac{c - d'}{c} \times .003 \times 29000 \right)$$

$$24.96 \times 60 = 0.85 \times 3.5 \times 0.85 \times c \times 30 + 12.48 \left(\frac{c - 1.5}{c} \times .003 \times 29000 \right)$$

On solving, $c = 8.08$ in.

$a = \beta_1 c = .85 \times 8.08 = 6.87$ in.

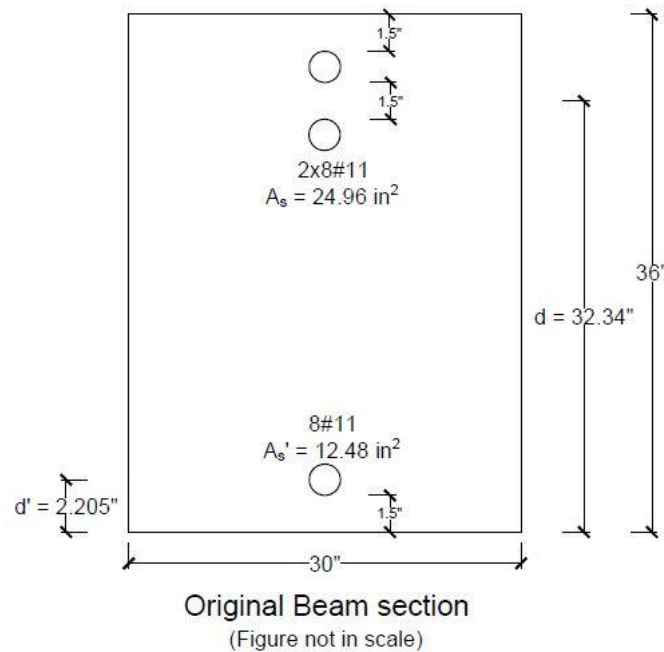
$$f_s' = \frac{8.08 - 1.5}{8.08} \times .003 \times 29000 = 70.85 \text{ ksi} > 60 \text{ ksi}$$

Check for tension controlled section

$$\varepsilon_t = \frac{d - c}{c} \times .003 = \frac{32.34 - 8.08}{8.08} \times .003 = .009 > .005 (TC)$$

So, the section is tension controlled and $\phi = 0.9$

$$\begin{aligned} \phi M_n &= \phi \left(A_s f_y \left(d - \frac{a}{2} \right) + A'_s f'_s (d - d') \right) \\ &= 0.9 \left(12.48 \times 60 \times \left(32.34 - \frac{6.87}{2} \right) + 12.48 \times 60 \times (32.34 - 2.205) \right) \\ &= \mathbf{3315.69 \text{ kip} - \text{ft}} \end{aligned}$$



b. Calculation of Stresses:

Location of neutral axis (NA)

$$\frac{by^2}{2} + (2n - 1)A_s(y - d')^2 = nA_s(d - y)^2$$

$$\text{or, } \frac{30 * y^2}{2} + (2 * 8 - 1) * 12.48 * (y - 2.205)^2 = 8 * 24.96 * (32.34 - y)^2$$

On solving, neutral axis depth, $y = 12.09$ inch.

Cracking moment of inertia

$$\begin{aligned} I_{cr} &= \frac{by^3}{3} + (2n - 1)A_s(d' - y)^2 + nA_s(d - y)^2 \\ &= \frac{30 * 12.09^3}{3} + (2 * 8 - 1) * 12.48 * (12.09 - 2.205)^2 \\ &\quad + 8 * 24.96 * (32.34 - 12.09)^2 \\ &= 117844.92 \text{ in}^4 \end{aligned}$$

i. Tensile stress in steel at (a):

$$f_s = \frac{n * M * (d_1 - y)}{I_{cr}} = \frac{8 * 1976.16 * 12 * 21.705}{117844.92} = \mathbf{34.94 \text{ ksi}} < 60 \text{ ksi (ok)}$$

ii. Compressive stress in steel at (b):

$$\begin{aligned} f_s' &= \frac{2n * M * (y - d')}{I_{cr}} = \frac{2 * 8 * 1976.16 * 12 * (12.09 - 2.205)}{117844.92} \\ &= \mathbf{31.83 \text{ ksi}} < 60 \text{ ksi (ok)} \end{aligned}$$

iii. Compressive stress in concrete at bottom face of beam, at (c):

$$f_c = \frac{M * y}{I_{cr}} = \frac{1976.16 * 12 * 12.09}{117844.92} = \mathbf{2.43 \text{ ksi}} < 3.5 \text{ ksi (ok)}$$

2. Current condition

Percentage loss of area due to corrosion = 5.52%

$$\text{Area of \#11 rebar (A)} = 1.56 * \frac{100 - 5.52}{100} = 1.47 \text{ in}^2$$

Area of rebars in tension zone (16 #11) = $16 \times 1.47 = 23.52 \text{ in}^2$

Area of rebars in compression zone (8 #11) = $8 \times 1.47 = 11.76 \text{ in}^2$

a. Calculation of moment capacity:

Equating compression and tension forces,

$$A_s f_y = 0.85 f'_c \beta_1 c b + A'_s \left(\frac{c - d'}{c} \times .003 \times 29000 \right)$$

$$23.52 \times 60 = 0.85 \times 3.5 \times 0.85 \times c \times 30 + 11.76 \left(\frac{c - 1.5}{c} \times .003 \times 29000 \right)$$

On solving, $c = 7.73 \text{ in}$.

$$a = \beta_1 c = .85 \times 7.73 = 6.57 \text{ in}.$$

$$f'_s = \frac{7.73 - 1.5}{7.73} \times .003 \times 29000 = 70.12 \text{ ksi} > 60 \text{ ksi}$$

Check for tension controlled section

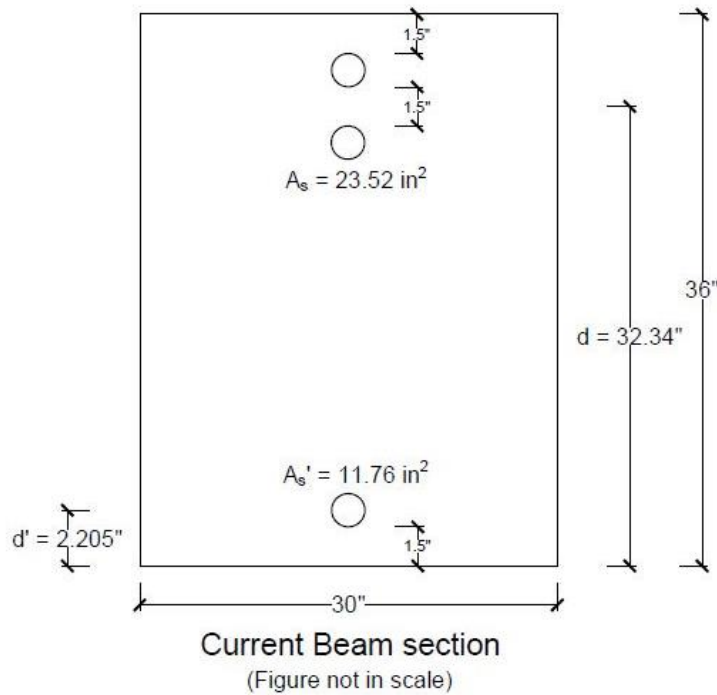
$$\epsilon_t = \frac{d - c}{c} \times .003 = \frac{32.34 - 7.73}{7.73} \times .003 = .0096 > .005 \text{ (TC)}$$

So, the section is tension controlled and $\phi = 0.9$

$$\phi M_n = \phi \left(A_s f_y \left(d - \frac{a}{2} \right) + A'_s f'_s (d - d') \right)$$

$$= 0.9 \left(11.76 \times 60 \times \left(32.34 - \frac{6.57}{2} \right) + 11.76 \times 60 \times (32.34 - 2.205) \right)$$

$$= \mathbf{3132.33 \text{ kip} - \text{ft}}$$



b. Location of neutral axis (NA)

$$\frac{by^2}{2} + (2n - 1)A_s(y - d')^2 = nA_s(d - y)^2$$

$$\frac{30 * y^2}{2} + (2 * 8 - 1) * 11.76 * (y - 2.205)^2 = 8 * 23.52 * (32.34 - y)^2$$

On solving, neutral axis depth, $y = 11.92$ inch.

Cracking moment of inertia

$$I_{cr} = \frac{by^3}{3} + (2n - 1)A_s(d' - y)^2 + nA_s(d - y)^2$$

$$= \frac{30 * 11.92^3}{3} + (2 * 8 - 1) * 11.76 * (11.92 - 2.205)^2$$

$$+ 8 * 23.52 * (32.34 - 11.92)^2$$

$$= 112043.83 \text{ in}^4$$

i. Tensile stress in steel at (a):

$$f_s = \frac{n * M * (d_1 - y)}{I_{cr}} = \frac{8 * 2074.18 * 12 * 21.875}{112043.83} = \mathbf{37.04 \text{ ksi}} < 60 \text{ ksi (ok)}$$

ii. Compressive stress in steel at (b):

$$f_s' = \frac{2n * M * (y - d')}{I_{cr}} = \frac{2 * 8 * 1976.16 * 12 * (11.92 - 2.205)}{112043.83}$$
$$= \mathbf{32.90 \text{ ksi}} < 60 \text{ ksi (ok)}$$

iii. Compressive stress in concrete at bottom face of beam, at (c):

$$f_c = \frac{M * y}{I_{cr}} = \frac{1976.16 * 12 * 11.92}{112043.83} = \mathbf{2.52 \text{ ksi}} < 3.5 \text{ ksi (ok)}$$