Assessment of Deteriorated Structural Concrete to Provide Durable Repairs

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Kentucky Transportation Center
College of Engineering, University of Kentucky, Lexington, Kentucky

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Research Report

KTC-21-32/SPR15-505-1F

Assessment of Deteriorated Structural Concrete to Provide Durable Repairs

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16. Abstract
Most of the structural elements on Kentucky bridges are made of reinforced concrete. Many of these elements deteriorate as a result of corrosion of the reinforcing steel caused by carbonation and — primarily — applications of chlorides by the Kentucky Transportation Cabinet (KYTC). Corrosion of reinforcing steel is reviewed along with assessment procedures that leverage nondestructive evaluation (NDE) methods and related laboratory and field tests. Several maintenance procedures are discussed, including the application of sealers and coatings, patch and major concrete repairs, and electrochemical methods. Incorporating NDE and test results into maintenance procedure selection is discussed. Recommendations are provided on expanding application of NDE and tests as this can enable better maintenance decision making related to corrosion of reinforced concrete.

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bridges, concrete, corrosion, deterioration, nondestructive evaluation, reinforcing steel

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Executive Summary

About 27,000 structural elements on Kentucky Transportation Cabinet (KYTC) bridges are made of reinforced/prestressed concrete. They pose the greatest maintenance challenge the agency faces — now and in the future. The most pressing issue is corrosion of reinforcing steel, which probably accounts for over 90 percent of all deterioration that occurs in structural bridge elements made of reinforced/prestressed concrete.

The primary causes of corrosion are carbonation and chloride penetration into concrete. Reinforcing steel is protected by a passive layer at its surface created by very alkaline (pH 12-13) concrete. Carbonation occurs when carbon dioxide in the atmosphere penetrates concrete pores and reacts with retained moisture to form carbonic acid. This reacts with the concrete and lowers the pH to about 9, which destroys the protective passive layer. Reinforcing steel can then corrode due to oxygen and moisture present in the concrete. Chlorides can be present due to construction materials acting as cast-in sources of attack on the reinforcing steel passive layer. More commonly, chlorides are present as the result of anti-icing and de-icing chemicals applied to bridge decks. Chlorides diffuse to other bridge elements via leaking joints, clogged drains, and aerosols kicked up by traffic during wet conditions.

Carbonation and chlorides can work together to accelerate corrosion. When steel corrodes, expansive corrosion products are produced that cause concrete to crack. Generally, the effects of this deterioration manifest over the time required for the damaging materials to enter the concrete, attack the passive layer, initiate corrosion, and accumulate corrosion products sufficient to fracture the concrete.

In 2016, KTC surveyed other state highway agencies (SHAs) and KYTC districts on their use of nondestructive evaluation (NDE) and tests used to assess reinforced concrete bridge elements. Survey responses indicated that SHAs do not use the full range of available NDE methods. KYTC district responses indicated the same thing. The two surveys demonstrated that, generally, KYTC is similar to the other SHAs in this respect.

KTYC's biennial bridge inspection process provides the means to assess the superficial condition of reinforced concrete and identify budding signs of deterioration, primarily by visual inspections. Simple NDE methods (e.g., sounding) and tests (e.g., carbonation indicators) can be incorporated into inspections of bridges that are likely candidates for some type of maintenance intervention. Based on inspection findings, a detailed investigation may be required, especially if biennial inspections indicate repairs may be warranted. This investigation includes in-depth field inspections that assess the condition of reinforced concrete bridge elements in sufficient detail to make a maintenance action determination. More detailed visual inspections than those typically carried out during the biennial inspections are usually performed as an initial step in the process.

Based on visual findings, a range of NDE methods and minimally invasive tests can be performed to provide information to support the need for and scoping of any repairs. NDE methods and tests are typically performed on a survey/mapping basis. They are used to assess concrete quality, detect concrete deterioration, evaluate concrete crack severity, detect corrosion in reinforcing steel, and identify attendant factors that can affect the corrosion/deterioration process (e.g., lack of concrete cover). In addition to determining what maintenance actions, if any, are required based upon the in-depth field investigation, an assessment of anticipated future deterioration if no actions were taken should be conducted.

Findings can be used both to support maintenance actions and scope them. That will help ensure that any follow-on work is sufficient to preserve structural elements (or any other maintenance action). Specific NDE methods and tests KYTC can employ to generate useful information on reinforced concrete conditions are discussed.

Both routine and preventive maintenance should be used to help preserve reinforced concrete and minimize future deterioration/repairs. Routine maintenance can be scheduled independent of biennial inspections. If the results of those inspections indicate a bridge is in good condition, low-cost preventive maintenance actions (e.g., application of surface treatments such as sealers and coatings) can be applied to provide the greatest benefits.

When in-depth inspections are performed and NDE/test findings provided, decision making about follow-on actions can result in several options (including replacement and do nothing). Repair options include: 1) the application of surface treatments; 2) physical repairs, including removal and replacement of distressed concrete and the removal and replacement damaged reinforcing steel or addition of supplemental reinforcement; and 3) the use of electrochemical treatments and repairs.

Concrete joints, particularly leaking deck seals, pose a major problem to the durability of reinforced concrete bridge elements, including beams, piers, and abutments. Those should be made a preventive maintenance focus area.

The report's conclusions are as follows:

- Available NDE methods and tests presented here can be used to assess most reinforced concrete conditions. They can provide data to help KYTC officials to make good choices when selecting and scoping maintenance work. An example of this is KTC's use of NDE and chloride testing on the I-65 JFK Bridge southbound offramp piers, which gathered information that supported KYTC's decision to retain the piers.
- KYTC should consider more widespread use of NDE and related tests prior to initiating repairs.
- KYTC should consider adopting routine and preventive maintenance programs to protect reinforced concrete bridge elements from corrosion.

The following actions are recommended:

- KYTC officials should develop a plan for protecting reinforced concrete elements on bridge members.
- KYTC officials should review the NDE and test methods discussed in this report. They should provide KYTC district personnel with the equipment and training needed to use these devices to evaluate reinforced concrete bridge elements other than decks.
- KYTC should develop training for district personnel on the selection of NDE methods and tests identified for potential use.
- KYTC should initiate a pilot program in one district to address routine and preventive maintenance focused on reinforced concrete bridge elements.

Chapter 1 Introduction

1.1 Background

The use of reinforced/prestressed concrete for bridge construction has increased continuously since the 1970s. One reason for this is that bridge owners are concerned about the maintenance burden due to the corrosion/painting of structural steel. While reinforced concrete may reduce maintenance requirements over the short term, a previous Kentucky Transportation Center (KTC) study noted that corrosion of reinforcing steel in structural concrete will likely become an increasing problem for the Kentucky Transportation Cabinet (KYTC) in the future (1). The magnitude of potential future reinforced concrete repair needs is enormous. KYTC has about 10,000 structures in its bridge inventory. Among these are 1,000 bridges with steel superstructures and a few timber bridges. Altogether there are approximately 27,000 reinforced major reinforced/prestressed concrete elements (substructures, superstructures and decks) in the KYTC bridge inventory that will need to be maintained.

The primary mechanism for the deterioration of reinforced concrete is the corrosion of reinforcing steel due to the application of deicing chemicals (chlorides) on bridge decks. Carbonation is another corrosion mechanism that acts solely, or in conjunction with, chloride-induced corrosion. Other concrete deterioration mechanisms include drying shrinkage cracking, freeze-thaw cracking/scaling, thermal cracking, internal or external restraint, expansive aggregate pop outs, alkali-silica reactivity (ASR) and external factors such as subgrade settlement and applied loads.

Several deterioration mechanisms may warrant in-depth assessments prior to determining the needs for follow-on actions, but reinforcing steel corrosion probably accounts for well over 90 percent of all distress found in reinforced/prestressed concrete on bridges. Deicing chemicals are commonly salts (chlorides) that dissolve in water and penetrate through pores in the concrete cover to the level of the steel reinforcement. They then corrode steel reinforcing bars. Expansion of the resulting corrosion products (i.e., rusts) results in concrete fractures. In addition, the loss-of-section of corroding reinforcing steel can further degrade and weaken reinforced concrete. Due to the prevalence of reinforcing steel corrosion as the primary deterioration mechanism on concrete highway bridges, it is the primary topic to be addressed when discussing NDE methods and related tests.

The average KYTC bridge is about 50 years old. An assessment performed by KYTC between 2001 and 2002 indicated that most bridge decks did not have excessive chloride contamination at the depth of reinforcing steel. However, follow-up testing of bridge concrete for chloride contamination conducted by KTC in 2013 indicated problematic chloride contamination (0.15 % by weight of concrete) of bridge decks at reinforcement depths and very high chloride contamination (0.40 % by weight of concrete) at rebar depth in other concrete structural elements (abutments and pier caps). The rapid increase in chloride contamination may be attributed to the application of more and different types of de-icing and anti-icing chemicals (liquid calcium chloride and brine) by KYTC beginning in the early 2000s. In fact, KYTC tripled the quantity of deicing chemicals used from 2004 to 2011 (2). During that period, KYTC adopted more frequent use of liquid anti-icing pretreatments for snow and ice control (Figure 1). Chloride-contaminated concrete bridge elements may not exhibit deterioration presently, but will do so eventually as chloride levels increase at the reinforcing mat level and the steel reinforcement begins to corrode. In epoxy-coated reinforcing steel, slower deterioration by corrosion is typically noted with its performance related to quality of the epoxy coating. Epoxy-coated reinforcing steel durability is typically related to its time-to-initiation for corrosion (3). Corrosion damage to epoxy-coated reinforcing steel can result in weakened structures that are prone to unexpected failures (4).

Deicing salts are applied directly to decks. In liquid form, they penetrate the concrete and cause the aforementioned types of visible distress (Figure 2). Similar behavior has been observed on other structural reinforced concrete — abutments, beams, and piers (Figures 3-6). In Kentucky, deterioration of those components is typically due to leaking/open joints or inadequate drains that leave them exposed to chloride-contaminated deck runoff. Another source of salts is aerosols laden with deicing salts that are kicked up by traffic. These aerosols are deposited on fascia girders, wing walls, abutments, and piers of overpass bridges located near roadways.

Decks typically employ concrete mixes that are more impermeable to chloride penetration than those used on the substructure elements. In a 2004 report on the performance of bridge elements, KTC researchers studied the performance of 312 deck girder bridges in central Kentucky (op. cit. 1). They reviewed the average change in condition of reinforced concrete components (decks, superstructures, and substructures) from NBI data for those bridges over a 6- to 7-year period from about 1995-1996 to 2001-2003. Data indicated that all three concrete components had about the same average condition ratings at the beginning and end of the inspection periods and that the average ratings of all of those components had decreased from *good* to *fair*. That information indicated that the primary catalyst of deterioration (i.e., salt-induced corrosion) had similar impacts on all reinforced concrete bridge elements. Most concerning was the indication that eventually the deterioration of reinforced concrete was going to be relatively consistent throughout bridges, especially those entirely made from reinforced/prestressed concrete.

It is unlikely that state highway agencies (SHAs) will decrease their use of deicing salts. Weather trends over the last 30 years do not indicate a significant change in winter temperatures for Kentucky (5). Past reviews found that clear roads policies enabled by their use had considerable benefits when compared to the direct costs of applications. No cost-effective salt alternatives have been identified despite extensive research and field trials. Application of deicing salts on bridges will likely continue with undesirable consequences for the structural integrity of bridges.

1.2 The Potential Role of Nondestructive Evaluation and Tests of Reinforced/Prestressed Concrete

From the preventive perspective, some SHAs are looking at alternative deicing methods (e.g., beet juice applications). To date, no low-cost alternatives to salt applications have emerged. To address that situation, a number of SHAs are using/investigating the application of corrosion-resistant reinforcing materials, impermeable asphalts, membranes, polymer laminates, and dense concrete to stop/limit the impacts of deicing salts. These technologies are employed on new structures or rehabilitation projects. For existing bridges, there are several pressing needs: 1) protective treatments to eliminate, slow, or mitigate the damaging impacts of deicing salt applications, 2) criteria for applying those protective treatments for preventive maintenance (PM) on existing bridges and 3) methods of nondestructive evaluation (NDE) and minimally invasive test methods. The latter are required to assess reinforced/prestressed concrete to promote the use of exacting maintenance needs criteria and provide data for scoping maintenance activities (PM or repairs). In this report, optical, chemical, mechanical and electrochemical tests used to evaluate the condition of reinforced concrete are considered along with methods (e.g., sounding, impact echo, thermography) commonly classified as nondestructive.

Visual inspection, typically performed by KYTC personnel during biennial bridge inspections, is the most common NDE method. The use of other NDE methods for PM applications on reinforced concrete structures is not commonplace. NDE is typically used to determine the extent of physical distress in reinforced concrete — primarily bridge decks — for repair assessments. On other bridge structural elements, simple methods are typically employed such as sounding with chains or hammers and probing with picks. Additional NDE/testing tools and procedures need to be identified along with directions on how they are used to properly assess concrete. Beyond that, guidance is needed on effective maintenance/repair decision making based upon those findings.

In 2013, KTC researchers helped KYTC perform NDE and tests on four concrete pier columns on the I-65 Kennedy Bridge Kentucky approach spans in Louisville. Those piers were under leaking joints that had severely damaged the pier caps and resulted in localized spalling in the supporting columns (41-50 ft. high). As part of the rehabilitation of I-65, the pier caps were to be replaced. KTC's work was done to determine whether the pier columns could be salvaged. Two evaluation methods were used: 1) ground penetrating radar (GPR) to detect delaminations and 2) chloride (concentration) measurements at several depths below the column surfaces (Figures 7, 8). Field testing produced results that were graphically identified in a memorandum report (6). Those findings were used to assist KYTC in making a determination to retain the columns.

Much research and literature have been devoted to the use of NDE on bridge decks. While reinforced/prestressed concrete deterioration due to chlorides and many of the NDE methods employed are similar for all bridge elements, this document focuses primarily on the use of NDE for preservation, evaluation, and repairs of other bridge structural concrete elements (piers, abutments, and barrier walls). As significant literature exists on NDE and bridge deck repair, issues with other bridge elements (beams, piers, abutments, barrier and retaining walls) are primarily addressed in this report. It focuses on specific equipment and methods the authors believe are potentially adoptable for use by KYTC in evaluating the aforementioned bridge elements.

1.3 Research Objectives

To investigate the benefits of using NDE and tests on bridge elements other than decks, KYTC awarded the KTC Bridge Preservation Group a research study with the following objectives:

1) Review current practices used by other SHAs and KYTC officials to assess structural concrete,
2) Examine current state-of-the-practice methods/procedures used to evaluate structural concrete for chloride contamination, rebar corrosion, concrete chemical assessment and deterioration by NDE/test methods,
3) Prepare guidance for the use of advanced structural concrete evaluation methods, analysis of test results, scoping of work, and proper reporting of findings to facilitate preventative maintenance or repairs, and
4) Prepare guidance for structural concrete repairs based upon assessments performed using the state-of-practice methods.

Note: Conventional, prestressed and post-tensioned reinforced concretes are complex materials. In addressing distressed concrete in bridges, generally, this report addresses conventional reinforced concrete bridges subject to common material problems. The emphasis is on reinforcing steel corrosion. Problems involving structural issues, subsidence, or constraint issues are not addressed. NDE methods and tests that are potentially viable to KYTC are reviewed. Repair methods involving electrochemical methods are discussed briefly, though KYTC has yet to employ that technology.

Chapter 2 Carbonation and Chloride-Induced Corrosion of Reinforced Concrete

2.1 Concrete Composition and Factors Related to Corrosion of Reinforcing Steel

Concrete has properties that both work to promote and hinder corrosion of the reinforcing steel. Some of these include porosity, permeability, water-cement (w/c) ratio, strength, water/moisture content in cured concrete, alkalinity, and resistivity. To some extent, they tend to be interrelated and all, directly or indirectly, are indicators of durability/susceptibility of reinforced concrete to deterioration by corrosion and other forms of degradation. Concrete consists of cement (a fine powder), water, fine and coarse aggregates, and admixtures. When cement is mixed with water, it forms a paste that sets and hardens due to hydration reactions yielding calcium silicate hydrate (C-S-H) gel and calcium hydroxide.

The cement paste formed by hydration reactions contains interconnected pores of different sizes. These can be include gel pores, capillary pores, and air voids. The ratio of the volume of concrete that consists of pores and voids to its total volume is termed *porosity* (expressed as percent). Gel pores are typically very small and do not allow significant transport of aggressive species. Capillary pores are voids within the hardened cement paste. They have dimensions in the range of 10 to 50 nm in good concrete, but can reach up to 3-5 µm in concrete with high water/cement (w/c) ratios or concrete that is not well hydrated. Poorly consolidated concrete can have pores up to several millimeters in size. Air voids are discrete porosity formed by trapped air in wet concrete. When concrete cures, voids are fixed in place (e.g., spherical voids). Air voids typically around .05 – 0.2 mm may be present as entrained air to produce freeze-thaw resistance. Typical concrete has a porosity of 9 to 10 percent. Porosity impacts concrete density. Generally, a denser concrete provides greater strength, fewer voids, and lower porosity.

Interconnected pores allow the penetration of gases like nitrogen, oxygen, and carbon dioxide from the atmosphere and water, which typically contains dissolved materials (ions). The susceptibility of concrete to penetration by gases and liquids is termed *permeability*. This allows concrete to transport aggressive materials (e.g., carbon dioxide, chlorides in solution) into the concrete to corrode reinforcing steel.

Water is a necessary component of concrete. It performs the following functions:

1. It acts as a lubricant for the fine and coarse aggregate that makes the mixture workable.
2. It acts chemically with cement to form the binding paste.
3. It is employed to damp the aggregate surface to prevent it from absorbing water vitally necessary for chemical action.
4. It facilitates the spreading of aggregate.
5. It helps to flux the cementing material over the aggregate surface.
6. It enables the concrete mix to flow into formwork (7).
7. It prevents excessive heat from being generated in curing that can lead to cracking (8).

The w/c ratio and curing have a major effect on capillary porosity. For complete hydration of the cement paste, a w/c ratio by weight of 0.38 is required. Higher w/c ratios necessary for practical concretes yield higher porosity in cured concrete. According to The American Concrete Institute (ACI) 318, *Building Code Requirements for Structural Concrete and Commentary*, and 322, *Code Requirements for Residential Concrete and Commentary*, the maximum w/c ratio is specified is 0.45. The volume of capillary pores increases with amount of water used in the paste (i.e., higher w/c ratios). With increased curing, by keeping the cement paste moist, hydration proceeds and the volume of capillary pores decreases, reaching a minimum when cement hydration is complete. The resistance of concrete to the transport of aggressive species depends not only on the volume of the capillaries but also their size, irregularity, and degree of interconnection. Larger capillary pore sizes tend to be more interconnected, making the concrete more permeable and more susceptible to transporting aggressive materials.

Concrete strength decreases with an increase in porosity. Higher w/c ratios typically yield lower-strength concretes. Cement formulation affects concrete porosity. Plain Portland cement concrete (PCC) is typically not waterproof. Modern cements are typically blends of Portland cement along with other materials such as fly ash, blast furnace slag, and silica fume. Compared to PCC, these produce a denser concrete with finer pores that reduce permeability and resist the penetration of aggressive species. They also yield higher-strength concretes. However, their use makes the concrete prone to cracking when curing and they have typically lower alkalinity than PCC and provide less passive protection to the reinforcing steel.

In cured concrete, water may be present in hydrated cement paste in several forms. The greatest amount of water in concrete is found in capillary pores. As discussed below, this water plays has the most influence over corrosion. Water in capillary pores larger than about 50 nm in diameter readily evaporates when the relative humidity of the environment decreases below 100 percent. Water in capillary pores smaller than 50 nm in diameter evaporates, but at lower relative humidity values, which decrease as pore size decreases. Some water is retained on the inner surfaces of the pores as a thin layer of adsorbed water. This water can be removed when external humidity falls below 30%. It contributes very little to transport phenomena. Other forms of water are retained in the paste are interlayer water and chemically combined water; these do not factor into the corrosion of reinforcement.

In the absence of wetting (e.g., surface exposure of the concrete to rain), concrete water content can be related to the relative humidity of the atmosphere. Atmospheric moisture enters the pores and condenses to fill smaller pores while larger pores remain filled with air. The transport of gases in water-filled pores is hindered, while the transport of dissolved chlorides (ions) is facilitated. Typically, when concrete is exposed to the atmosphere (i.e., bold exposure) it is periodically wetted by rain. When that occurs, it tends to absorb water more rapidly than it releases it when the external atmosphere adjacent to the concrete is dry. As a result, concrete moisture content at the depth of the reinforcement tends to be higher than that of equilibrium with relative humidity of the environment.

Ordinary PCC has commonly been the basis for most structural concretes used on Kentucky bridges, though KYTC specifications allow contractors to replace a certain percentage of Portland cement with pozzolans. In that case, the pH of the concrete may be reduced. However, that reduction may be somewhat offset by reductions in concrete permeability. This discussion addresses ordinary PCC.

Pores of hydrated cement paste contains some water. The amount depends upon the humidity of the surrounding environment. The paste contains high concentrations of soluble calcium, sodium, and potassium oxide around pores. When water is present, those form hydroxides which are very *alkaline*, giving concrete a pH of 12-13 (also given as 13-14 by some sources). This results in the formation of a thin, tightly adhering oxide film on the reinforcing steel surface. When this occurs, the reinforcing steel was *passivated* as the film protects the reinforcing steely by acting as a barrier to corrodents. Even if sufficient moisture and oxygen are available to promote corrosion, reinforcing steel encased in good concrete corrodes at a very low rate – approximately 0.000004 in./year (9). That rate does not persist if sufficient chloride ions from deicing salts have penetrated to the reinforcement or if the pH of the pore solution is reduced by carbonation. In those cases, the protective film is destroyed and the reinforcing steel is *depassivated*. Under these conditions, reinforcing steel can begin and continue to corrode, eventually yielding corrosion products that expand to about six times the volume of the reinforcing steel that has corroded. This creates tensile forces within the concrete that leads to cracking, delamination, and finally spalling.

Resistivity refers to the electrical resistivity of concrete. Electric resistivity affects the flow of ions and the rate at which corrosion can occur. When concrete has a low water content, resistivity is high. It is also high if the concrete is impermeable. Resistivity can be a measure of concrete quality in resisting chloride penetration. Recent studies have shown that there is a direct correlation between resistivity and chloride diffusion rate. Concrete resistivity also provides an indication of both the likelihood of corrosion and the corrosion rate.

2.2 Carbonation and Chloride Causes of Reinforcing Steel Corrosion

As noted, the high alkalinity in PCC promotes the generation and stability of a passive layer on the surface of reinforced/prestressed steel embedded in concrete. A requirement for significant corrosion of reinforcing/ prestressed steel is for that layer to be attacked. Epoxy-coated reinforcing steel isolates steel from aggressive materials that promote corrosion. It does not rely on the passive layer for protection of the reinforcing steel. Corrosion of epoxy coated reinforcing steel is related to a breakdown in the epoxy barrier.

2.2.1 Carbonation

Carbon dioxide (CO_2) exists in gaseous form and is primarily a product of combustion of fossil fuels. In rural areas, the airborne concentration of carbon dioxide is around 0.03%; in urban areas, it can be as much as 0.1%. Carbon dioxide can penetrate into the pores of the concrete and dissolve in water to form a mildly acidic solution of acetic acid. This reacts with alkaline calcium hydroxide in the pore water to form insoluble calcium carbonate that lines the pores.

$$CO_2 + Ca\,(OH)_2 \rightarrow CaCO_3 + H_2O$$

The reaction reduces the concrete's pH from 12 – 13 to about 9. When the pH drops to about 10.5, the passive layer that protects the reinforcing steel is destroyed. The reinforcing steel loses it protection (i.e., depassivation). Due to the presence of moisture and oxygen in the concrete, depassivated reinforcing steel can begin to corrode. Typically, reinforcing steel corrosion damage due to carbonation is negligible unless the reinforced concrete bridge elements are exposed to alternating periods of low and high humidity.

2.2.2 Chlorides

Chloride contamination is the most common cause of corrosion of reinforcing steel and subsequent damage to reinforced/prestressed concrete structures. The primary source of chlorides is surface-applied de-icing/anti-icing chemicals. Chlorides can also be provided in the mixing of concrete by constituents, including aggregates, water, and admixtures (e.g., cast-in chlorides). Typically, KYTC evaluates concrete materials to preclude that source of chlorides.

As surface-applied chlorides dissolve into ions in water, they penetrate primarily by diffusion into capillary pores and eventually reach the level of the reinforcement. Ions that are in solution, including any pre-existing in the pore solution, are termed free ions. They can locally depassivate steel and promote electrochemical corrosion. Some chlorides are bound in the concrete matrix. Bound chlorides that are acid-soluble are chemically bound and those that are water-soluble are physically bound. The relationship between free and bound chlorides varies with the type of binder. The combination of free ions in solution and bound chloride ions in the solid phases of the concrete comprise the total chloride content.

Typically, chloride contamination in new concrete is limited by specifying the use of potable water. There has been a case where high chloride levels have been detected in potable water in Kentucky. Some of these chlorides may be chemically bound and unavailable to promote corrosion. ACI Committee 222, *Protection of Metals in Concrete Against Corrosion*, recommends a maximum (acid- soluble) chloride content in new non-stressed concrete exposed to wet conditions of 0.20 % by weight of cement and 0.08% for prestressed concrete per ASTM A1152, *Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete* (10).

Chlorides depassivate steel by breaking down the passive layer. Thereafter, they act as catalysts and promote rapid corrosion. Depassivation occurs when the ratio of chloride ions to hydroxyl ions exceeds 0.6 Cl^-/OH, which is approximately 0.40% chloride by weight of cement in existing concrete (11). The chloride content at the rebar depth necessary for depassivation of the reinforcing steel is variously termed the *critical chloride content* or the *chloride threshold*. It is not a fixed value, but depends upon the composition of the concrete (resistivity), the amount of moisture present, and atmospheric conditions (temperature and humidity). There is considerable scatter in chloride threshold values for corrosion in concrete exposed to the atmosphere such as found in bridges (12).

2.3 Electrochemical Corrosion of Reinforcing Steel in Concrete

Corrosion of reinforcing steel in concrete is an electrochemical process. It involves the flow of electrons between anodic and cathodic sites in the reinforcing steel. Four basic components of that corrosion process are:

Anode — the site where steel ionizes (corrodes) and gives up electrons that flow towards the cathode.

Cathode — the site where corrosion is prevented by receiving electrons from the anode and water breaks down to form hydroxyl ions (OH^-).

Electrolyte — a medium capable of conducting electric current by ionic current flow (concrete). When concrete is subjected to wet and dry cycles due to bold exposure, it can act as an electrolyte.

Metallic path — connection between the anode and cathode which allows electron flow and completes the circuit (reinforcing steel).

If any one of the elements of the electrochemical cell is eliminated, corrosion can be prevented.

Electrochemical reactions for the corrosion of reinforcing steel in concrete are:

Anode for moisture and oxygen (carbonation) corrosion:

$$Fe^{++} + 2H_2O \rightarrow Fe\ (OH) + 2H^+$$

As corrosion progresses, the anode reaction progresses:

$$2\ Fe\ (OH)_2 + H_2O + 1/2O_2 \rightarrow 2Fe\ (OH)_3$$

Anode for chloride-induced corrosion:

$$Fe \rightarrow Fe^{++} + 2e^-$$
$$Fe^{++} + 2Cl^- \rightarrow FeCl_2$$

As corrosion progresses, the anode reaction progresses:

$$FeCl_2 + 2OH^- \rightarrow Fe\ (OH)_2 + 2Cl^-$$
$$Fe\ (OH)_2 + ½\ O_2 \rightarrow Fe_2O_3 + 2H_2O$$
$$Fe\ (OH)_2 + O_2 \rightarrow 4\ Y \cdot FeOOH + 2H_2O$$

Cathode:

$$½O_2 + H_2O + 2e^- \rightarrow 2OH^-$$

Two types of corrosion cells develop in concrete: microcells and macrocells. In microcell corrosion, the anode and cathode are on the same reinforcing bar and usually are in close proximity. Typically, corrosion caused by chloride ions can be due to a localized breakdown in the passive film, leading to microcell corrosion-induced pitting (Figure 9). Carbonation-induced corrosion is usually considered more of a uniform type of corrosion.

As with carbonation, chlorides from de-icing and anti-icing treatments enter concrete from the exposed surface. The penetration of chlorides and their resulting concentrations adjacent to reinforcing steel vary due to the heterogeneous nature of concrete. Differences in concentration result in the creation of closely-spaced anodes and cathodes along a reinforcing bar yielding microcell corrosion. The same can occur with carbonation-induced corrosion though the initial corrosion is usually more uniform than for chloride-induced corrosion (see below).

In macrocell corrosion, on bridges — usually related to chloride-induced corrosion — the anode and cathode are typically separated in distance either along the same reinforcing bar, in adjacent reinforcing bars on the mat, or between bars in different electrically-coupled reinforcing steel mats at different depths in the concrete, such as the upper and lower mats of a bridge deck, where the upper mat is exposed to higher chloride concentrations (Figure 10). Macrocell corrosion can also occur where different locations become anodic/cathodic to each other due to carbonation-induced corrosion. When large areas of reinforcing steel are electrically cathodic and tied to smaller areas that are anodic, macrocell corrosion can occur with the anodic area corroding at a rapid rate.

2.4 Transmission of Carbonation and Chlorides into Concrete

Penetration of carbonation or chlorides into concrete determines the initiation of corrosion/damage. This movement can be through macro pathways such as cracks or voids. In sound concrete, reactants move through pores in the cement binder (15).

A model of the corrosion process is shown in Figure 11. The *corrosion initiation phase* involves the movement of the damaging materials (e.g., CO_2 gas and chloride ions in solution) into undamaged concrete and the depassivation of the reinforcing steel. This starts at its surface and penetrates into the concrete through the cement binder. Permeation, migration, capillary suction (absorption) and diffusion are the methods of transport of corroding species into concrete through interconnected pores (op. cit. 12).

Permeation is the transmission of gases or liquids due to pressure gradients. It is usually applied to hydraulic structures for water and is not common on bridge elements, except for wheel tracks on bridge decks.

Migration is the movement of ions (e.g., hydroxide ions and chloride ions) in solution in concrete subject to an electric field. The rate of transmission is proportional to the strength of the electric field, along with the charge and size of the ion. Electrical currents by ion migration in concrete are important both for macrocell corrosion and electrochemical repairs.

For solutions, *capillary suction* can be active due to capillary effects in the cement paste that pull surface water into concrete. Usually, the top ½-inch or so of conventional concrete is more porous than its interior and suction can be the controlling means of transport in this region. Typically, this transport mechanism will not, by itself, bring chlorides to the level of the reinforcing steel unless the concrete is of extremely poor quality or the concrete cover over the reinforcing steel is thin. It does quickly bring chlorides to some depth in the concrete and reduces the distance that they must diffuse to reach the rebar (17) (Figure 12).

Diffusion is transport due to concentration gradients. It is the driving force for many aggressive materials such as O_2, CO_2, and chloride ions moving through pores from the surface of the concrete, where concentrations are higher, to inside the concrete, where concentrations are lower. Diffusion is relatively slow.

2.5 Diffusion of Carbonation And Chlorides in Concrete

Diffusion mechanisms are the most important means by which damaging species are transmitted into concrete. Modelling methods are describe below.

Carbonation penetrates as a low pH front. The depth of carbonation is given as:

$$d = K \cdot (t)^{1/n} \qquad (1)$$

where:
d = depth of carbonation (mm)
K = carbonation coefficient ~ 3-6 mm/year$^{1/2}$ for good-to-low quality concrete (18)
t = time (years)
$n \simeq 2$ (common value for conventional concrete)

The carbonation coefficient K depends on the diffusion coefficient D, atmospheric CO_2 concentration at the concrete surface, and the amount of alkaline components to be consumed by the CO_2. K is the measure of the rate of penetration for a specific concrete and set of environmental conditions. In hard or wet concrete, n increases and the rate of penetration of carbonation becomes negligible.

Carbon dioxide penetrates most effectively into concrete through open pores. It can penetrate to some extent through partially saturated pores but it will not travel through pores that are completely saturated. In those cases, carbonation process is hindered or prevented. For carbonation to occur, some moisture/humidity must be present in the pores of the concrete to form acetic acid. Carbonation is thought to occur on a wide front moving into the depth of the concrete. The reaction with calcium hydroxide also only occurs in solution, and so in very dry concrete carbonation is slow. In saturated concrete, moisture presents a barrier to the penetration of carbon dioxide; again, carbonation is slow. The most favorable condition for the carbonation reaction is when there is sufficient moisture for the reaction but not enough to act as a barrier. This is usually considered to occur at 60-70% ambient relative humidity.

Chloride transmission by diffusion can be modelled by Fick's second law:

$C(x) = C_x - (C_x - C_i)\mathrm{erf}\,[x/2(D_a t_{ex})^{1/2}]$ (2)

where:
C =Chloride content vs. depth (x) at time t
C_s = Surface chloride content
C_i = Initial chloride content in cement
erf = error function
t_{ex} = Exposure time (seconds)
D_a = Apparent chloride diffusion coefficient (10^{-10} to $10^{-13} m^2/sec$) (19, op. cit. 12).

Chloride ions only diffuse in solution and will transport more effectively in saturated pores rather than in partially saturated ones. A portion of the chloride ions react with the concrete matrix, becoming either chemically bound (with aluminates in the concrete) or physically bound (as absorbed water on pore walls). Binding reduces the rate of diffusion. If the diffusion coefficient is measured after steady-state conditions have been reached, this effect will not then be observed (op. cit. 17).

While carbonation lowers the concrete's pH to approximately 9, different chlorides can have different effects on pH (op. cit. 12). Some chlorides can block the diffusion process.

2.6 Impact of Cracks on Corrosion

Cracking is a common phenomenon in concrete. Cracks can develop during construction before the concrete is fully cured (plastic cracks). It can also occur after curing (hardened cracks). Concrete cracking can be due to shrinkage, expansive reactions (including reinforcing steel corrosion), settlement, thermal stresses, freezing and thawing, and loading (20). Cracking can provide a direct pathway for aggressive materials to penetrate the concrete cover and possibly directly access the reinforcing steel. This short-circuits protection provided by the concrete cover and accelerates both reinforcing/prestressing steel corrosion initiation and growth.

2.7 Corrosion Initiation

The rate of carbonation depends on environmental factors such as humidity, temperature and concentration of carbon dioxide, and factors related to the concrete (mainly alkalinity and permeability). As carbonation penetrates into concrete on a broad front, it is commonly thought to cause uniform corrosion in reinforcing steel. Moisture and oxygen are necessary for corrosion to occur once the passivating layer of the steel is eliminated. Concrete elements exposed to wet-dry cycles are usually considered the most susceptible to carbonation corrosion. Dry concrete lacks the moisture to promote carbonation reactions or corrosion. Constantly wet concrete prevents CO_2 penetration from driving carbonation and O_2 penetration from causing corrosion. Other factors that reduce initiation time for carbonation-induced corrosion are thin concrete cover, cracks in concrete, and high porosity associated with concretes that have a low cement factor or high water-cement ratio.

The threshold chloride concentration in concrete below which significant corrosion does not occur depends on several parameters, including pH and humidity. Some of these factors are presented in Figure 13 and show the effects of relative humidity and quality of concrete on the critical chloride threshold. In that figure, the critical threshold as determined by the CEB is 0.40% chloride ion by mass of cement (~2.4 lb./yd^3 of concrete). This is higher than the acid-soluble chloride threshold typically used in the U.S. (1.0-1.5 lb./yd^3 of concrete). Chlorides locally attack the passive layer, which initiates microcell corrosion and pitting. As the chloride level increases around reinforcing steel, the passive layer breaks down further and the corrosive becomes more uniform.

If carbonation and chloride ingress both occur, the chloride content at the carbonation front can be higher than in unaffected areas. In carbonated areas inside the concrete, chloride levels can be higher than those measured near the concrete surface. When the carbonation front reaches the reinforcing steel, the resulting decrease in pH can depassivate the steel and lead initiate corrosion at low chloride levels.

Once corrosion initiates, moisture and oxygen are needed to maintain the corrosive attack. The primary rate-controlling factors are available oxygen, electrochemical resistivity, and relative humidity, all of which are interrelated with the pH and temperature. Carbonation also initiates corrosion but does not influence the rate of corrosion.

If concrete porosity is low, the maximum corrosion rate can occur at a moisture content that is equivalent to equilibrium with an ambient relative humidity of about 95%. For more porous concrete, this occurs at a slightly higher relative humidity. Lower or higher relative humidities result in lower corrosion rates. Higher relative humidities reduce oxygen diffusion into the concrete, while lower ones result in higher concrete resistivities driven by lower moisture contents in drier concrete.

2.8 Corrosion Growth

Corrosion growth occurs after corrosion initiation. It depends upon the corrosion rate of the reinforcing steel. The corrosion rate is the rate at which corrosion occurs after depassivation of reinforcing steel. The corrosion rate is the key element in determining the time from corrosion initiation to concrete cracking and the progress of structural deterioration. It depends on the availability of oxygen and moisture at the cathode and on concrete resistivity, which is mainly affected by the internal moisture content and concrete porosity.

Factors that influence the corrosion rate of the reinforcing steel are:

- Availability of water and oxygen
- Ratio of the anode to cathode area
- Ion content of the pore water
- Resistivity of the concrete
- Temperature
- Internal and external relative humidity
- Concrete microstructure (op. cit. 16)

Other factors include the presence/types of cracks.

From a material aspect, concrete factors influencing both corrosion initiation and corrosion rate include:

- Depth of concrete cover over reinforcing steel
- w/c ratio
- Consolidation
- Type/quantity of cement
- Pore content and structure
- Presence of harmful cracks
- Degree of curing
- Environment exposure including anti-icing/deicing chemicals

2.9 Resulting Distress in Concrete

As corrosion of the reinforcing steel progresses, corrosion products are formed on the periphery of the reinforcing bars. Several more oxidation stages occur which form expansive corrosion products or rust capable of causing cracking and spalling of surrounding concrete. Ferric oxide (Fe_2O_3) has a volume about twice that of the uncorroded reinforcing steel it displaces. Ensuing chemical reactions create various hydrated ferrous oxides that swell up to six times the original volume of steel. These reactions create high tensile stresses in the concrete resulting in its fracture.

Epoxy coatings on reinforcing steel can contain film defects (holidays) that allow chloride-contaminated solutions to penetrate the coatings and cause corrosion. Prior to construction, epoxy coatings can be damaged by handling or bending and can be weakened by prolonged exposure to sunlight (UV deterioration) during storage. Those circumstances can weaken the coating or provide breaches, which leads to premature corrosion of the reinforcing steel.

An accelerated period of damage is where corrosion increases due to easy access of oxygen and water through cracks in the cover concrete. This promotes delaminations and eventually spalling of concrete (Figure 14). The deterioration rate is the rate that concrete distress (cracking, delaminating and spalling) progresses. The deterioration rate determines the length of time before repair or replacement is necessary. Several important factors are needed to determine the deterioration rate: cover depth, chloride profile, carbonation depth, concrete resistivity, and the environment (op. cit 19, 22). In addition to concrete cracking, reduction of the steel cross section, possible loss of steel ductility, and reduced bond strength are all possible consequences of reinforcement corrosion. These can result in loss of strength and, potentially, structural failures.

Chapter 3 Assessing Structural Concrete on Bridges and Follow-On Decision-Making

3.1 Role of the Assessment Process

Before deciding whether maintenance is needed (other than routine maintenance), structural concrete on a bridge should be assessed. That process provides the rationale and framework for using NDE and other tests on reinforced concrete. A preliminary investigation is needed to identify any distress in the concrete, determine its probable cause, establish the need for a follow-on detailed investigation, and provide sufficient information to scope it. The American Concrete Institute Committee 364 provides a detailed process for determining those actions, ACI 364.1R-07, *Guide for Evaluation of Concrete Structures before Rehabilitation*, [with the appropriate considerations for PM and minor repairs as discussed in this report]. The document contains a flowchart suitable for representing the potential steps for that work (Figure 15). KYTC biennial bridge safety inspections are represented by Step 2, *Visual condition survey, with documentation of deterioration and distress*, under the heading **Preliminary Investigation**. As shown in the ACI flowchart, a preliminary visual survey (field inspection) is a key component of the preliminary investigation.

3.2 Preliminary Field Inspections

The purpose of the preliminary field inspection is to obtain a general understanding of the condition of major concrete elements and determine the need for a more in-depth follow-up inspection (including the test procedures necessary for the follow-up in-depth inspection and the locations/quantities of testing required). Before performing a preliminary field inspection, all documentation should be reviewed, including past inspection reports, to assess whether there has been a significant decline in the condition of any bridge elements. Since these inspections are done on a regular basis, good historic records should be available. The current condition of concrete bridge elements help determine whether additional investigative steps are necessary when planning a biennial inspection. Plans can also show locations where epoxy-coated reinforcing steel is used, which impacts some testing and, potentially, repair options.

The preliminary field inspection can be incorporated into a scheduled biennial inspection, but it differs from the routine KYTC biennial inspections in that certain NDE methods and tests may be used to supplement the regular visual inspection. For practicality's sake, their use should be limited and performed by normal bridge inspection personnel. Typically, testing would use methods with low operator requirements that could be performed rapidly with minimum site preparation. That includes sounding (hammer/chain) and thickness of concrete over reinforcing steel depth/locations (covermeter). Rock picks and pry bars can be used to probe weak concrete, cracks, and spalls on a spot-testing basis. In addition, a few carbonation measurements, half-cell corrosion potential readings, or chloride measurements could be taken on a spot-testing basis. The latter could be performed at reinforcing steel depth in the concrete. Additionally, pieces of detached concrete can be retrieved for subsequent laboratory testing (if needed) to determine the cause of failure.

For both preliminary and in-depth field inspections, the weather may impact bridge conditions (i.e., rain saturating exposed concrete). That may limit the types of testing that can be performed on days when field inspections are scheduled and personnel available. For those reasons multiple test methods should be used and site conditions recorded just prior to and during the field tests. For instance, with saturated concrete, half-cell potential readings may not be possible, but concrete cover and carbonation/chloride readings might suffice (op. cit. 15).

In general, the visual inspection performed during the preliminary field inspection will identify bridge elements in good condition, locations of potential concern on specific elements, locations of previous concrete repairs, and any signs of concrete distress on specific elements. In areas of concrete with visible distress, spot tests can be performed by sounding along with spot cover meter readings to determine the thickness of concrete cover over the reinforcing steel. Signs of visible distress include concrete cracking, rust bleed, spalling, and corroding reinforcing steel where spalling has occurred. Readings can also be taken adjacent to areas of concrete distress to determine whether sufficient sound concrete cover is present over the reinforcing steel. ACI 201, *Guide for Conducting a Visual Inspection of Concrete in Service*, provides recommended steps for performing and reporting a visual inspection.

Locations of potential concern include areas with ponding water, water/seepage stains, efflorescence, leaks, and minor rust spotting on the surface of the concrete. The proposed NDE and test methods can also be applied in each of the localized areas of concern to determine their potential for subsurface problems and, in the absence of unfavorable readings, confirm that they are intact. Sounding will be needed to detect subsurface delaminations in the concrete. Areas with visible concrete distress should be measured and located on drawings for potential follow-up assessments.

Macro- and micro-environments around the bridge should be noted. Issues that contribute to actual or potential concrete distress should be documented for future maintenance actions such as crash damage, leaking joints, or splash zones. Leaks may be indicative of localized areas of contamination.

3.3 In-Depth Inspections for Detailed Investigations

Based upon the concrete distress identified in the preliminary investigation, a *detailed investigation* may be necessary, including a thorough review of plans, more in-depth field inspections (including field condition surveys using a variety of NDE/test methods) and, possibly follow-on laboratory tests of cores. Based on these tasks, a comprehensive assessment can be prepared outlining the need for follow-on maintenance actions when the bridge condition requires it or funding permits. The assessment can include scoping of repairs, projected costs, and preparation of contract documents. It also aids in determining the need for a structural evaluation required either by the existing condition of the structure or by follow-on repairs. The comprehensive assessment will determine the cause, extent ,and severity of deterioration. ICRI Guideline 210.4-2009, *Guideline for Nondestructive Evaluation Methods for Condition Assessment, Repair, and Performance Monitoring of Concrete Structures*, provides recommendations for selecting NDE and test methods for assessing specific concrete features, including potential flaws.

For the in-depth field inspection, thorough surveys can be performed, including mapping of the results of visual inspection and specific NDE/ tests conducted on the surface of reinforced concrete elements. This will provide information needed to locate where repairs are required and identify repair materials/quantities (also where preservation treatments are possible). Representative areas may be selected for surveys of cover thickness, carbonation depths, chloride contents or profiles, half-cell potentials, and other tests depending upon the presence/type of distress observed or anticipated. That work will include tests to reveal internal concrete distress, the likelihood/presence/intensity of corrosion activity, concrete strength (if necessary) and depths/rates of progression of chloride/carbonation penetration. In areas with significant visible cracking and spalling, deteriorated concrete cover should be removed and the cross-sectional areas of distressed reinforcing steel measured to determine loss-of-section/strength. As many tests can be impacted by temperature, ambient and concrete surface temperatures should be recorded throughout a field test.

For visual surveys, pictures can be taken of distressed locations. Alternatively, sketches can be made showing structural elements (usually simple block diagrams) portraying surface distress such as crack patterns. It is useful for pictures or sketches to show dimensions/measurements .

To perform surveys, the surface of concrete elements can be marked in a grid pattern and NDE/tests performed at grid points. A typical grid can be spaced at 2-4 feet depending on the surface area of the concrete element to be tested. For smaller surfaces such as beams, pier caps, or columns, readings can be listed. For large surfaces with many readings (e.g., bridge decks), readings can be displayed on a equipotential map. In some cases, where access time is limited, spot sampling methods can be used for large surfaces (24, 25).

In addition to determining the presence and amount of concrete distress, the cause(s) of distress need to be known to execute proper repairs. Besides corrosion, there are other forms of reinforced concrete deterioration, including ASR, sulfate attack, thaumasite attack, delayed ettringite formation, freeze-thaw, thermal movement, settlement, and any other movement that can lead to concrete damage. The cause of distress at each location should be included in the final surveys. Some of those types of concrete distress, if present, may require coring the concrete and laboratory petrographic analyses. Where concrete strength is an issue, cores may need to be extracted for follow-on compression tests or for use with comparative field strength tests (e.g., Schmidt hammer).

3.4 Concrete Assessment Methods

Concrete quality can be assessed by:

- Cores for measuring compressive strength
- Projectile penetration
- Lateral pull-off tests (26)
- Petrographic examinations
- Pulse velocity
- Rebound hammer
- Air content measurements
- Cement content determinations
- Permeability measurements
- Impact echo

Cracking, delamination, and spalling can be assessed by:

- Visual examination
- Hammer sounding, chain drag mapping, and other acoustic testing
- Infrared mapping
- Radar mapping
- Core sampling

Cracking can be assessed by:

- Crack mapping
- Crack width measurements
- Crack movement monitoring

Corrosion activity of reinforcing steel and other metallic embedments can be assessed by:

- Potential mapping
- Corrosion rate measurements
- Metal section loss measurements

Factors that can affect corrosion of steel and other metallic embedments include the following:

- Depth of concrete cover
- Concentrations of chloride and other aggressive ions at varying depths
- Concrete resistivity
- Concrete moisture
- Concrete permeability
- Concrete pH
- Depth of carbonation of the concrete
- Temperature (op. cit. 10)

The resulting data should indicate specific locations and types of distress/deterioration and provide sufficient information to make decisions on the need/type/extent of repairs. Once in-depth field inspection work is complete, condition data should be evaluated for the entire structure to identify the condition(s) of all reinforced concrete elements. That includes the locations/quantities/types of needed maintenance actions. If circumstances warrant, Equations (1) and (2) can be used to model/predict the onset of corrosion. Based upon current structural conditions and anticipated deterioration rates, predictions should be made on future reinforced concrete condition if no maintenance actions were performed. Options for addressing a bridge's condition can be identified and compared based upon first and life-cycle costs (op. cit. 21). The resulting decision can incorporate that cost data along with other considerations such as importance of bridge, potential for future deterioration, necessity for repairs, funding constraints, availability of KYTC personnel for project management, and the potential for grouping a project with similar work on other bridges (project bundling). If work needs to be deferred, it could be prioritized among other competing projects.

3.4.1 Concrete Quality

Bad concrete, whether weak, poorly consolidated, or susceptible to gradual distress by means other than corrosion (e.g., ASR, freeze-thaw) may need to be replaced. Sufficient information/data should be available to assess the condition of the concrete throughout the volume of a structural element. Surface hardness tests can provide comparative field strength results. Those can be compared to laboratory compression tests of concrete cores extracted from structural elements. Other sonic tests can be used to assess concrete strength and identify locations where existing concrete is distressed. Permeability and strength measurements can be used to assess concrete susceptibility to ingress of carbonation and chlorides. Petrography from bridge concrete can be used to identify concrete quality and the causes of concrete distress and can be useful for maintenance decision making.

3.4.2 Cracking

The most common manifestation of concrete distress, regardless of the cause, is concrete cracking. Primarily, cracks are caused by environmental exposure, improper construction, design inadequacy, and incorrect detailing. Cracks can be flexural cracks, where the tensile stress exceeds the tensile strength of concrete. Drying shrinkage is also a crack that occurs after concrete hardens. It happens due to the reduction of concrete volume when water evaporates. Thermal cracks are due to restrained contraction as concrete contracts and expands with changes in moisture and temperature. Plastic shrinkage cracks result from evaporation on the surface of concrete that is greater than the available rising bleed to fill the surface moisture; this triggers a volume change that causes cracks. Other types of cracks can be caused by alkali-aggregate reaction, sulfate attack, settlement cracking, freezing and thawing (D-cracking) of saturated concrete, or corrosion of reinforcing steel. Allowable crack widths are as follows: > 0.3 mm for mild exposure, > 0.2 mm for exposure to elements/moisture and > 0.1 mm for severe exposure (e.g., chlorides). (27)

Specific causes of cracking typically occur at various stages of life of the structural concrete after it is placed. Some types of cracks emerge during or shortly after construction; others (e.g., carbonation/chloride-induced cracking) occur years later. Approximate times for various types of cracking are provided in Table 1. Related crack patterns are described in Figure 16.

There are several opinions about the role (early) cracks play in reinforcing steel corrosion. One is that cracks shortcut the normally slow corrosion initiation process and allow damaging species to directly access the reinforcing steel, resulting in onset of early corrosion and corrosion propagation. Another opinion is that while cracks may allow for the onset of early corrosion, corrosion propagation is limited as the corrosion is localized. That opinion holds that there is little difference in the damage caused by early crack-induced corrosion and that which results from the normal corrosion initiation/propagation process. Cracks running along reinforcing steel may accelerate corrosion/propagation. Cracks transverse to the reinforcing steel may not have a significant role in causing large-scale corrosion of the reinforcing steel early on. Cracks less than approximately 0.3 mm (0.012 in.) wide have little influence on the corrosion of reinforcing steel (op. cit. 10). The Virginia DOT seals cracks over 0.2 mm (0.008 in.) in width on bridge decks after a drying age of six months (28). That is in general agreement with the ACI for cracks exposed to de-icing chemicals (29).

Corrosion can be measured through electrochemical evaluations using half-cell corrosion potential tests and linear polarization (corrosion rate) tests. The former reveal the probability of reinforcing steel corrosion activity and the latter reveal the corrosion rate of reinforcing steel. Where corrosion activity is occurring, maintenance decisions are generally limited to repairs or doing nothing. Where corrosion activity is present, the extent of section loss in reinforcing steel is probably negligeable until expansion of corrosion products results in concrete cracks, delaminations, and spalls. In that case, reinforcing steel, either naturally exposed by spalling or by manual excavation, can be sized to determine the amount of section loss. This will factor into assessing the structural capacity of the damaged element or into the need for reinforcing steel replacement/augmentation during a repair. Generally, corrosion and significant section loss such as shown in Figure 5 result in a repair decision.

3.4.3 Attendant Factors

Factors that affect corrosion of steel and other metallic embedments can be important for selecting/targeting maintenance activities or determining their necessity. Concrete cover thickness has a significant role in the susceptibility of bridge members to corrosion. High permeability or low resistivity of concrete are indicators of susceptibility to corrosion initiation, and probably high corrosion propagation once the reinforcing steel becomes depassivated and corrodes. Concrete pH can be an indicator of carbonation damage. The moisture level in concrete can be an indicator of susceptibility to corrosion. Carbonation and chloride ingress testing at various depths in the concrete cover on selected concrete elements can be used to determine the progress of corrosion initiation. Those data can also be used to determine whether preservation activities are feasible.

Factors to consider in making maintenance/repair decisions include:

- Determining the extent of damage
- Evaluating the consequences of that damage
- Establishing the appropriate time for a maintenance action
- Determining the remaining service life (with/without intervention)
- Identifying practical repair options, if any (op. cit. 12).

Selecting the best repair option based upon technical and cost considerations is a challenging task. There are a range of actionable measures that can be applied in whole, or part, depending upon the existing conditions of the structural concrete as determined by the assessment procedure. The do-nothing option is the non-actionable decision that can be justified by condition, structure importance, or inability to program work. Decision making after an assessment can also be affected by funding limitations and competition with alternate needs. It can also be affected by limitations on personnel to either perform in-house work or administer maintenance-related projects.

While having a range of options can result in the best maintenance decision, some level of agencywide standardization is desirable. The Utah DOT has a formal process to address bridge corrosion. It incorporates a plan for every structure that addresses:

- Preservation
- Rehabilitation
- Replacement

For bridge decks the Utah DOT has prepared *Concrete Bridge Deck Condition Assessment Guidelines* (Report No. UT-05.01), which lets agency personnel decide on preservation, rehabilitation, or replacement using standardized:

- Condition assessment methods
- Threshold values for decision making

That approach can be utilized for all reinforced/prestressed concrete bridge elements (30). It would benefit KYTC by providing consistent bridge maintenance practices in addressing structural concrete.

A representation of structural concrete deterioration is shown in Figure 17, with the concrete initially in *good* condition. The relative cost of maintenance actions is shown on the vertical axis and National Bridge Element (NBE) condition states and time on the horizontal axis. When a concrete bridge element is in *good* condition, there are typically no signs of reinforcing steel corrosion. However, the corrosion initiation process can be occurring with carbonation and chloride penetrating concrete cover and migrating toward the reinforcing steel. In this condition state, low-cost preservation treatments can prolong this condition state if the concrete is free of significant cracking and, optimally, the carbonation reduction of pH/chloride critical threshold concentration has not reached the depth of the reinforcing steel. These treatments include the application of sealants and protective coatings to halt further carbonation/chloride ingress. When the carbonation or chloride critical threshold level reach the reinforcing steel and initiate depassivation and corrosion, some electrochemical treatments can keep reinforced concrete in *good* condition. However, these are more expensive than a simple preservation treatment such as applying coatings or sealants. NDE and other tests are useful to ensure that the condition of reinforced concrete is suitable for applying preservation treatments. As reinforcing steel corrodes and causes more extensive distress, the concrete element condition drops from *good* to *fair*. Repairs may be warranted in areas where the reinforcing steel has become the depassivated. NDE and testing are useful for assessing the extent of internal concrete damage and determining the quantity of repair work required. When the concrete elements are in *fair* condition, portions of concrete may be in sufficiently good condition to benefit from preservation treatments, especially if localized repairs are performed. NDE and testing can indicate where reinforcing steel is intact, has not received sufficient exposure to problematic levels of carbonation or chloride attack, and can receive those treatments. In the *fair* and *poor* condition states, more extensive in-depth NDE and tests are desirable to properly assess the condition of the structure/elements and scope them for repairs/rehabilitation. When the structural elements deteriorate from the *poor* to *severe*, NDE and tests provide useful information for determining the scope of follow-on maintenance actions including repairs, rehabilitation, or replacement.

While preservation treatments are usually reserved for PM actions, combining them with repairs is a good approach to bridge maintenance. It allows the repaired element (and most other bridge elements) to age more uniformly and minimizes the need for additional repairs while providing many years of maintenance-free service.

The following chapter reviews NDE methods and tests for investigating structural concrete on bridges, with a focus on corrosion. The ensuing chapter addresses specific maintenance options and when they are viable.

Chapter 4 NDE Methods and Tests for Structural Concrete on Bridges

This report only addresses the use of NDE for conventional reinforced concrete. Some of the evaluation methods may relate to prestressed beams, but post-tensioning girders and decks require other methods that are beyond the scope of this report.

4.1 Preliminary Work to Identify NDE Methods for Assessing Bridge Reinforced Concrete

To determine how highway agencies use NDE on concrete bridge elements, KTC conducted surveys of selected SHAs. Most were in the Midwest. KTC conducted similar surveys of KYTC Districts in March 2016. KTC also performed a literature search on the use of NDE and other relatively non-invasive tests for assessing reinforced concrete, focusing primarily on bridge applications.

4.1.1 Highway Agency Surveys

In 2016, KTC surveyed 17 SHAs that had previously participated in the Midwest Bridge Working Group. Eleven agencies responded. Their responses are summarized in Appendix A.

Over half of the respondents (9 of 15) stated that they use NDE methods other than visual inspection to determine the extent of needed repairs. Nine respondents use NDE on bridge decks while five each use NDE on pier caps and abutments. The responses on who performs the work was evenly split between agency personnel, contractors, and both agency personnel and contractors (three responses for each). Eight of the respondents said that they use visual inspection and sounding to determine the extent of repairs on deteriorated structural concrete. Four use thermography. Three use pachometers (covermeters) and ground penetrating radar (GPR). Two respondents use half-cell corrosion potential testing. Three respondents use chipping to evaluate concrete. Four respondents said that they perform proactive NDE of structural concrete to assess future needs (PM treatments). Six respondents routinely use NDE to inspect bridge decks, while two each stated they routinely use NDE on pier caps and abutments. Parties performing routine NDE on concrete bridges were evenly split between in-house, contractors, and both (two each). Use of NDE for proactive assessments of concrete included chloride contamination (powder) measurements (three respondents), thermography (three respondents), GPR (two respondents), electrical potential testing (one respondent), and sounding (one respondent). Two respondents noted that they use protective (PM) treatments while two other respondents said that they do not perform PM treatments. PM treatments included sealing (two respondents) and coating (one respondent). The respondents noted that PM treatments are applied to decks (two respondents), pier caps (one respondent), and abutments (one respondent). One respondent uses NDE findings as criteria for applying PM treatments. One other respondent does not use NDE for that purpose. One responder noted that thin overlays are used on bridge decks with less than 10% repair area (presumably as determined by both visual inspection and NDE), otherwise decks receive a membrane seal with asphalt. Three respondents (one from Kentucky) noted that they had used GPR and thermography but not routinely. Five respondents stated that they typically employ chain drag and hammering for sounding. One respondent stated that NDE is used commonly for bridge decks and occasionally for piers.

4.1.2 KYTC District Surveys

KTC also sent the same survey form to all 12 KYTC districts. Responses were obtained from 10 districts (several were partial responses). Those are summarized in Appendix B.

Eight of the 10 respondents stated that they use NDE methods other than visual inspection to determine the extent of needed repairs. Eight respondents use NDE on bridge decks, while four each use NDE on pier caps and abutments and two use NDE on retaining walls. Seven responders said the NDE work is performed in-house while one stated that it is performed both in-house and by contractors. Eight respondents use visual inspection and sounding to determine the extent of repairs on deteriorated structural concrete. There were two responses each for pachometers (covermeters) and GPR. Five respondents use chipping to evaluate concrete. One respondent said that they perform proactive NDE of structural concrete to assess future needs (PM treatments) — seven respondents stated they do not. One respondent routinely uses NDE to inspect bridge decks, while one stated they routinely use NDE on pier caps. Both of those respondents said that work is performed in-house. The NDE methods used for those tests were not identified. None of the respondents said that they use proactive (PM) treatments on structural concrete. Two respondents stated that they typically employ chain drag and hammering for sounding. One respondent stated no funds were available to perform NDE on structural concrete.

4.1.3 Summary Of Surveys

One general takeaway from the surveys is that KYTC does not lag most other SHAs in the use of NDE on bridges. Some other SHAs had committed to bridge preservation practices for several years and were using NDE for assessment purposes related to PM treatments. After reviewing literature on reinforced concrete distress and methods to assess those, it was concluded that there is a range of NDE equipment available for that purpose, but it is probably not seeing widespread use by SHAs on reinforced/prestressed concrete bridge elements.

4.1.4 Literature Review

A literature review was performed, turning up 50 references related to reinforced concrete, including topics such as deterioration mechanisms, NDE and other evaluation test methods, PM, and repairs. That review provided the basis for identifying suitable NDE methods and their roles in assessing reinforced concrete bridges for a range of maintenance and repairs of concrete bridge elements. The literature review also revealed that there are several field and laboratory tests that could be categorized as minimally invasive that are useful for assessing the condition of reinforced concrete. While those were not addressed in the KTC NDE surveys, they are included in this report as they can be valuable for supplementing or, in some cases, supplanting NDE methods.

4.1.5 Field Demonstrations for KYTC Officials

On May 13 and June 7, 2016, field demonstrations were performed on concrete testing and NDE equipment from two equipment suppliers — Germann Instruments Inc. and Proceq. The demonstrations were conducted in Louisville on elevated portions of the I-64 Riverside Parkway near East Witherspoon Street on columns of the Parkway located in the Great Lawn City Park. At testing, the Parkway had been in service about 40 years and some of the columns under deck joints were beginning to spall, indicating active corrosion of the reinforcing steel. The Germann demonstration included use of their instruments for assessing corrosion rate, half-cell potentials, and electrical resistance of the concrete as well as testing the surface compressive strength of concrete, surface ultrasonic velocity tests to measure concrete uniformity, extent of severity of near-surface damage to concrete, and depth of surface-opening cracks in concrete, rapid field tests, *RCT* rapid test to measure chloride content in concrete, indicating spray to measure the depth of carbonation in concrete cover, impulse response testing for deteriorated concrete, and 3D tomographic testing to reveal internal defects in concrete. The Proceq demonstration included use of a cover meter to measure concrete cover depth, reinforcing steel location and reinforcing steel size, and a Wenner 4-point test to measure concrete electric resistivity. These demonstrations gave KYTC officials the opportunity familiarize themselves with the range of equipment that could be used to assess reinforced concrete.

All of the NDE and test methods demonstrated, and additional ones that were not, can provide detailed information on the condition of a reinforced/prestressed concrete element, what types of distress may be present, and the susceptibility of good-appearing reinforced concrete to deterioration through corrosion of the reinforcing steel. Also, information from some of those methods may enable models to predict future performance of the reinforced concrete in the presence of aggressive environments present on many bridges. Those methods and tests will be briefly explained, including their operating principles, the resulting test information/data, and its interpretation. While many of those can be used to assess concrete distress from other causes, the primary focus of this report is on reinforced concrete distress related to reinforcing steel corrosion.

4.2 Relevant NDE Methods and Tests to Evaluate Reinforced/Prestressed Concrete

The various NDE and tests impose different requirements on the users in terms of knowledge on how to select and employ them as well as how to interpret data and relate them to recommendations for maintenance actions. Their use also imposes costs for test equipment and, possibly consumables, along with potential costs for employing a skilled consultant to perform/interpret certain tests and field inspection costs for access equipment and time on a structure ,which may entail traffic control and user costs due to motorist delays. The decision to use those NDE methods and tests must balance their costs against the value of the information they can provide. Additionally, use of several complimentary NDE methods and tests may be desirable in case field problems arise in performing some of them.

Table 2 lists some of the proposed test methods along with the level of skill need to conduct and complexity of the methods (noted as general for simpler ones and specialist for those that entail special equipment and operator knowledge for performing/interpreting resulting data). Table 2 also provides estimates for rates of testing that may be useful in selecting methods based upon funding constraints or on time available onsite. By their overall role in the assessment process, the NDE methods and tests considered for use by KYTC are described below.

4.2.1 Test Methods to Determine the Quality of Concrete Include Field and Laboratory Strength Testing Methods

Penetration Resistance Methods

Penetration resistance methods measure the depth of penetration of probes (rods or pins) forced into concrete. This provides an indication of the concrete's resistance to penetration, which can be related to its strength. The most widely used is the Windsor probe test. That test works by using a gun-like driver that fires a probe into concrete using a calibrated charge. The concrete surface being tested must be flat (by grinding or brushing) and the driver must be placed perpendicularly to that surface.

ASTM C 803-97, *Standard Test Method for Penetration Resistance of Hardened Concrete*, addresses the use of the Windsor probe. A locating template is provided for the insertion of three probes into the concrete. The exposed lengths of the probes are measured using a depth gauge. The longer the exposed length of the probe, the higher the compressive strength of concrete. An average of the three probe length measurements is considered the test reading.

As with rebound tests, the penetration resistance test results are difficult to correlate with specific concrete compressive test values. They are near-surface tests and do not provide through-thickness strengths for concrete bridge elements. Good laboratory correlations have been achieved with compression tests (31). However, for in-situ concrete such results may be difficult to achieve. For field tests of concrete elements, they can be used as indicators of relatively high or relatively low strength concretes.

The Windsor probe method is simple and easy to operate, however, it should not be used within 6 to 8 inches of the edge of a structural member, and it should not be used within 4 inches of reinforcing steel. The test, while quick, leaves a small cone-shaped hole on the concrete surface. Those can be quickly filled using masonry sealant from a caulk gun.

Rebound Hammer

ASTM C805-97, *Standard Test for Rebound Numbers of Hardened Concrete*, addresses the use of the rebound hammer or Schmidt hammer test. This method is based upon the rebound principle. It consists of measuring the rebound of a spring-driven hammer mass after it impacts concrete. The unit consists of an outer body, a plunger, and a hammer mass in the mainspring. It has a locking mechanism that locks the hammer mass to the plunger rod and a sliding indicator to measure the rebound distance of the hammer mass. That distance is marked on a scale from 10 to 100 and provides a rebound number corresponding to the post-test position of the indicator on the scale.

To perform a test, the surface of the concrete must be flat and smooth. The hammer is held with a plunger normal to the concrete surface and pushed towards the concrete, stretching on the mainspring connected to the hammer mass. When the body is pushed to a specific distance the latch is automatically released and energy stored in the spring propels the hammer towards the plunger tip. The mass strikes the shoulder of the plunger rod and rebounds. During rebound the slide indicator travels with the hammer mass and records the rebound distance (i.e., rebound number) on the scale.

Each hammer is furnished with correlation curves used to correlate the rebound number with the compression test value. The rebound hammer is an inexpensive and rapid NDE field test of concrete.

The test reflects the near-surface properties of a concrete element, but results do not reflect those throughout its thickness. Researchers disagree on how accurately rebound readings measure concrete strength. The test is affected by flaws in the concrete (e.g., scaling, porosity), which can reduce the rebound numbers, and by carbonation, which can increase them. Its value is in determining concrete and structure uniformity.

Pullout Test

ASTM C 900-99, *Standard Test Method for Pullout Strength of Hardened Concrete*, addresses the lateral pullout test used to determine the in-situ compressive strength of a concrete bridge element. The lateral pullout test (e.g., Germann *CAPO-TEST*) enables pullout tests on existing structures without the need of pre-installed inserts. This test provides a pullout system similar to the embedment method used on fresh concrete (e.g., Germann *LOK-TEST*) system (op. cit. 26).

To perform the lateral pullout test, the surface of the concrete must be ground flat and the test location sited where reinforcing bars are not in the failure region. A 0.76-in. hole is cored perpendicular to the concrete surface approximately 2.75 in. deep. A recessed slot is routed in the hole to a diameter of 1 in. at a depth of 1 in. from the surface. A split ring is expanded in the recess and pulled out using a pull machine reacting against a 55-mm-diameter counter pressure ring (Figure 18). The concrete in the strut between the expanded ring and the counter pressure ring is in compression, making the pullout force related directly to compressive strength. The test is performed until the conic frustum between the expanded ring and the inner diameter of the counter pressure is extracted (Figure 19).

Several investigations have shown that the pullout strength measured by the lateral pullout test is essentially the same as the pullout strength measured by the Kaindl embedment test. While the lateral pullout test does not provide a full-depth assessment of concrete compressive strength, a recent KTC research report noted, "*Two series of tests — a laboratory test on freshly poured concrete slabs and an in-situ test on ready-to-demolish old bridges — were carried out. Both series of tests run the compressive strength cylindrical concrete test and lateral pull off test simultaneously to validate lateral pull off test as an acceptable and dependable method of determining concrete compressive strength. The test procedure and results look very promising based on the project's objective*" (op. cit. 26).

The test is semi-destructive in that it results in a conical cavity in the concrete. This cavity needs to be filled with a patching compound for appearance and to prevent potential durability problems (32).

Cores for Measuring Compressive Strength

When field inspections reveal conditions that raise questions about the strength of concrete, especially if a structural evaluation appears warranted, cores may be extracted from areas where knowledge of concrete compressive strength is needed. Nondestructive, in-situ tests of concrete, such as by probe penetration, impact hammer, ultrasonic pulse velocity, or lateral pullout may be useful in determining whether a portion of the structure actually contains low-strength concrete. Those tests are considered valuable for comparison purposes within the same bridge or structural element. ACI 214.4R-21, *Guide for Obtaining Cores and Interpreting Compressive Strength Results*, provides a detailed overview on coring concrete.

Three cores are usually extracted for each strength test. A covermeter should be used to prevent coring over reinforcing steel. The cores should be numbered and their locations identified. In addition, they should be photographed and their condition recorded (including any defects) for use in interpreting compression test results. A sketch can be made of each core that shows its dimensions and the presence of any defects that could impact test values (op. cit. 10).

Cores must be must be taken in accordance with ASTM C 42M-20, *Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*. Cores should be taken perpendicular to the concrete surface. They should have a diameter of at least 3.70 in. and be about twice that diameter, though ASTM C 42 provides correction factors for shorter cores. After the cores are cut, they should be wiped and allowed to surface dry (for no more than

1 hour) before being bagged to prevent moisture loss (33). The test requirements of ASTM C 42 must be followed explicitly to ensure proper test strengths.

Concrete strength in the area represented by core tests is considered adequate when the average compressive strength of the three cores equals at least 85% of the specified compressive strength and no single core strength is less than 75% of the specified compressive strength (per ACI CODE-318-19, *Building Code Requirements for Structural Concrete and Commentary*). In addition to verifying concrete compressive strengths for load capacity purposes, the tests can indicate if concrete has been weakened by ASR or freeze-thaw damage. Freeze-thaw damage can also result in horizontal cracking of the upper portion of the core.

4.2.2 Concrete Quality Can Also Be Assessed by Field and Laboratory Examinations

Petrographic examinations

Petrography is a powerful analytical tool for distressed concrete that has numerous applications in quality assurance and failure investigations. Petrography is a branch of geology applied to concrete and concrete raw materials. It is used to assess the optical and microstructural characteristics of concrete and applies a variety of microscopical techniques including stereo microscopy, polarizing microscopy, fluorescent microscopy, and metallurgical microscopy. It is often supplemented with chemical analysis, x-ray diffraction analysis, and scanning electron microscopy.

Petrographic analyses can be performed on concrete extracted from a bridge as cores or even on pieces of concrete that have broken loose from the concrete mass. As the method only requires small specimens, their selection as far as representing material of interest is important for effective use of the method. The appropriate number of specimens and their location are vital. Therefore, guidance on their selection should be given by the petrographer — or better yet, the petrographer can do the actual sample site selections (op. cit. 33).

Petrographic analysis for concrete includes the aggregate (ASTM C 295-19, *Standard Guide for Petrographic Examination of Aggregates for Concrete*) and the concrete (ASTM C 856-20, *Standard Practice for Petrographic Examination of Hardened Concrete*, and ASMT C 457-26, *Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete*). As petrography is a complex practice, the techniques employed are subject to some variances which occur as the analysis progresses. As a consequence, ASTM C856-20 provides guidance on the examination rather than being a fixed procedure. The number of petrographic methods used should be adequate for detailed evaluation of the original distressed concrete and its cause (34).

Information obtained during a petrographic analysis can include the following:

- Condition of material
- Causes of inferior quality
- Identification of distress or deterioration caused by chloride-induced corrosion, carbonation, alkali-aggregate reactions, and freezing and thawing cycles
- Probable future performance
- Compliance with project specifications
- Degree of cement hydration
- Estimation of w/cm and density (unit weight)
- Extent of paste carbonation
- Presence of fly ash and estimation of amount of fly ash
- Evidence of sulfate and other chemical attack
- Identification of potentially reactive aggregates
- Evidence of improper finishing
- Estimation of air content and how much of the air voids are entrained versus entrapped
- Evidence of early freezing
- Assessment of the cause of deterioration

The above information is vital for assessing the causes and mechanisms of deterioration and their effects on the concrete's existing condition. Petrography diagnoses the cause(s) and extent of a range of concrete deterioration mechanisms such as shrinkage, spalling, aggregate pop-outs, ASR, freeze-thaw damage and chemical attacks. Proper diagnosis of the cause(s) and mechanism(s) of deterioration and their effects on the existing and future condition are necessary for selecting the appropriate maintenance decision.

The following sonic/ultrasonic NDE methods can be used to assess concrete quality in terms of determining whether concrete is distressed and contains flaws/defects. They can also be grouped with methods to detect in-situ concrete distress (below).

Pulse Velocity Testing

Pulse velocity testing uses ultrasonic longitudinal stress waves generated in a material (concrete) to enable its evaluation. A test instrument creates a pulse of ultrasonic (> 20 kHz) longitudinal stress waves introduced into one surface of a concrete member by a transducer coupled to the surface with a coupling gel or grease. The pulse travels through the concrete and is received by a similar transducer coupled on the opposite surface (Direct method in Figure 20). The receiving transducer also is connected to the instrument. The transit time of the pulse is determined by the instrument. The distance between the transducers is divided by the transit time to obtain the pulse velocity, which is the acoustic velocity of sound in the material. The pulse velocity acoustic velocity, V, of stress waves through a concrete mass is related to its physical properties (per ASTM C 597-16, *Standard Test Method for Pulse Velocity Through Concrete*) is a function of Young's Modulus of Elasticity E, the mass density ρ, and Poisson's Ratio, *v*. The relevant equation for wave speed C_P is:

$$C_p = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)}} \qquad (3)$$

The acoustic velocity of concrete typically varies from 2,000 to 5,500 m/s. The pulse velocity in saturated concrete may be up to 5 % higher than in dry concrete. Transducers with resonant frequencies from 24 kHz to 150 kHz are recommended for concrete. Wheel probes are available for continuous testing in the field (35).

Ultrasonic pulse velocity (UPV) can be used for the following applications:

- Evaluating the uniformity of concrete within a structural member
- Locating internal voids and cracks
- Estimating the depth of surface-opening cracks
- Estimating severity of deterioration
- Estimating depth of fire-damaged concrete
- Evaluating effectiveness of crack repairs
- Identifying anomalous regions for invasive sampling with drilled cores
- Estimating early-age strength (with project-specific correlation)

Testing of good concrete will provide consistent velocity measurements (homogeneous concrete). Variations in pulse velocity are an indication of interior cracking, deterioration, honeycombing, and variations in mixture proportions (op. cit. 35). Deterioration reduces the pulse velocity. The test method is totally nondestructive, and it is possible to repeat the test at the same point at different times to monitor change of ultrasonic pulse velocity with time as a method of monitoring the progress of deterioration. Reinforcing steel can result in higher velocities than plain concrete. Pulse velocity tests have been used to detect the presence of reinforcing steel. Pulse velocity tests should be performed in locations where reinforcing steel will not interfere with the test.

Impact Echo

Impact echo is primarily used to locate flaws and determine the thickness of concrete elements. With the impact-echo method, a stress pulse is introduced into the concrete by impacting its surface using a steel ball. The pulse penetrates into the concrete as an expanding spherical wavefront of P- (pressure) and S- (shear) waves. The impact also creates a R- (Raleigh) wave that travels away from the impact point on the surface of the concrete. The P- and S-waves are reflected by internal interfaces or boundaries on the backside of the concrete.

The impact generates a surface pulse (P-waves) that reflects against a material with another acoustic impedance, a reflected pulse either an internal interface such as a void or crack or the back face of the concrete element (air). The reflected wave is converted into an electrical signal by a transducer mounted on the same surface and near to the point of impact. The signal is sent to a data acquisition system. The resulting electric signal (waveform) is complex due to multiple reflections of the stress waves inside the body being tested. The waveform is transformed into the frequency domain to produce an amplitude spectrum, which shows the predominant frequencies in the waveform. The frequency of P-wave arrival is determined as the frequency with a high peak in the amplitude spectrum.

For a typical test, where the wave hits the back face of a concrete element and is reflected, the period is equal to the path length, $2\boldsymbol{T}$, divided by the pressure wave's speed. As frequency is the inverse of the period, the frequency $\boldsymbol{f}$ is equal to C_P divided by $2\boldsymbol{T}$. Knowing the frequency of the waves generated in the speed of sound in the material, the thickness can be determined by frequency analysis by performing a Fast Fourier Transform (FFT) analysis of the recorded signals using the equation:

$$\boldsymbol{T} = \boldsymbol{C}p/2\boldsymbol{f} \qquad (4)$$

Also, the depth of any significant reflector can be determined.

A typical data acquisition system performs FFT analysis on the recorded signals, which gives the relative amplitude of the component frequencies in the waveform. The frequency of P-wave arrival is determined as the frequency with a high peak in the amplitude spectrum (Figure 21). Accurate measurement of thickness requires knowledge of the in-place P-wave speed (ASTM C1383-15, *Test Method for Measuring the P-Wave Speed and the Thickness of Concrete Plates Using the Impact-Echo Method*.) One method is by determining the thickness frequency and then measuring the actual plate thickness at that point using Equation (4). Alternatively, *Cp* may be determined by measuring the time for the P-wave to travel between two transducers with a known separation (36).

In performing field inspections, a grid can be laid out on the surface and tests performed at specific points to provide a range of frequency amplitude responses which can be compared to each other after signal processing to detect the component thickness and the presence of defects.

The impact-echo method can be used for the following applications:

- Measure the thickness of pavements, asphalt overlays, slabs-on-ground, and walls
- Detect the presence and depth of voids and honeycombing
- Detect voids below slabs-on-ground
- Evaluate the quality of grout injection in post-tensioning cable ducts
- Integrity of a membrane below an asphalt overlay protecting structural concrete
- Delamination surveys of bridge decks, piers, cooling towers, and chimneystacks
- Detect debonding of overlays and patches
- Detect ASR damage and freezing-and-thawing damage
- Measure the depth of surface-opening cracks
- Estimate early-age strength development (with proper correlation) (op. cit. 36)

Impact-echo testing has had good success in locating flaws and defects in highway structures. It can be used to measure the thickness of concrete elements and internal defects with an accuracy of a few percent. However, while it can identify defects, it generally cannot classify them. It also can be used to detect the depth of surface- opening cracks. It can be used to assess areas with visible or suspected distress and can detect a range of defects. It requires specialized equipment and trained operators. Typically, this equipment is used where the need for repairs is likely.

Impulse Response Method

With this method a stress pulse is generated in a concrete element by mechanical impact using an instrumented rubber hammer (Figure 22). A transducer located near the impact point monitors surface vibrations in response to the impact. The impact causes the element to vibrate in a bending mode. A velocity transducer, placed adjacent to the impact point, measures the amplitude of the response of the tested material to the impact, which is relatively low (below 1kHz). The time records for the hammer force and the velocity response from the transducer are received by the computer as voltage-time signals. They are processed using FFT, producing velocity and force spectra. A mobility transfer function is obtained by dividing the velocity spectrum by the force spectrum (37). Mobility is expressed in units of velocity per unit force, such as (m/S)/N. When defects are present in the concrete matrix, there will be reductions in the damping and stability of the mobility plots over the test frequency range (Figure 23).

The impulse response method can be used for:

- Detecting voids beneath concrete slabs in highways, spillways, and floors
- Detecting curling of slabs-on-ground
- Evaluating the integrity of anchoring systems of wall panels
- Locating delaminations and honeycombing in bridge decks, slabs, walls, and large structures such as dams, chimney stacks, and silos
- Detecting the presence of damage due to freezing and thawing
- Detecting the presence of ASR
- Detecting debonding of asphalt or concrete overlays and repair patches from concrete substrates
- Evaluating the effectiveness of the load transfer system in transmitting forces across joints in concrete structures (op. cit.37)

The impulse-response method provides a rapid test that can be used for mapping a concrete element to detect possible areas containing defects. As with impact-echo testing, a grid can be laid out on the surface of the concrete and tests can be conducted at the grid points. The resulting signal data can be stored on a portable computer and post-processed to evaluate the element and locate any potentially rejectable areas. While the method allows rapid screening of structures for defects, other methods (impact echo, pulse velocity, ultrasonic tomography or coring/petrography) are necessary for more in-depth analyses.

The application of impulse-response to plate-like structures is governed by ASTM C1740-16, *Standard Practice for Evaluating the Condition of Concrete Plates Using the Impulse-Response Method*.

4.2.3 Test Methods For Assessing In-Situ Concrete Distress, Including Cracking, Delaminations And Spalls

Visual inspection

Visual inspection is used to determine what areas of reinforced concrete are superficially good and which show signs of distress. While it is limited to detecting only apparent distress, it has the ability to detect the presence of defects (e.g., occurrence of rebar corrosion resulting in concrete spalling), categorize defect types (e.g., crack patterns, rust stains) and assess them (size, location and, for cracks, width). During normal inspections, visual inspections can be supplemented by NDE methods. Visual findings may factor into what NDE methods or tests are used. Visual inspection can also aid in scoping more in-depth assessments, particularly if repairs appear warranted.

Similar to dye penetrant testing of metals for cracks, concrete can be wetted to highlight fine cracks during the evaporation process, making them easier to detect.

Findings of visual inspections should be documented with written descriptions indicating the location, type, and extent of any apparent distress (op. cit 19). Written condition records can be supplemented by sketches, pictures, or video (38). Crack gages can be used, where applicable, to denote crack widths. Gaged locations can be photographed to provide a permanent record.

ACI 201.1R, *Guide for Conducting a Visual Inspection of Concrete in Service*, provides guidelines for conducting visual inspection surveys on all types of reinforced concrete structures along with photographic examples of typical concrete defects (op. cit. 10).

Hammer Sounding, Chain Drag Mapping, and Other Acoustic Testing

Acoustic sounding is a qualitative evaluation method used on concrete surfaces. It uses energy induced into a concrete element by application of a force to generate an audible sound from the concrete. The inspector performing the test listens for the sound of the impact. If the concrete is solid, the resultant sound is a tinny, ringing sound. If significant defects, typically delaminations, are present in the underlying concrete, the resulting sound is hollow or drum-like. Delaminations can be a result of poor concrete quality, debonding of overlays or applied composites, corrosion of reinforcement, or global softening.

Two methods are used for sounding: hammer tapping and chain dragging (or sweeping). Hammer tapping is usually used for smaller concrete elements such as pier caps. It can be used on both horizontal and vertical surfaces. The surface is tapped with a steel rod or hammer to produce the audible signature. The concrete surface being tested can be laid out in a grid and the grid points tested and delamination-sounding locations highlighted with spray paint for subsequent recording. The chain drag uses multiple short chains attached along a bar mounted on a handle. The handle is swung back and forth like a flail whipping the chains across the surface of the concrete. This method can test more area in a given time than the hammer tapping method.

Sounding has several advantages. First, the test equipment is inexpensive, allowing it to be widely used. The second advantage is that testing is rapid. A third advantage is that it provides an immediate indication of the test result, enabling follow-up evaluations using other methods while still at the bridge site. While it does not require a significant amount of training, an inspector must have some experience and show the ability to properly interpret sounds emitted by the test. Sounding can be used to supplement/confirm other forms of NDE including visual, half-cell corrosion potential, and carbonation/chloride penetration tests. Background noise can be an issue in interpreting test results. The method is limited in the types/sizes of defects it can detect and is usually not effective in detecting subsurface defects with much concrete cover (op. cit. 30). Delamination surveys using sounding may underestimate the amount of internal cracking and should be supplemented with other methods (op. cit. 19).

Probing by Chipping

A rock pick hammer can be used to manually strike the concrete surface. This can be done to detect delaminations and weakened concrete. If the concrete is weak, it will readily fracture. The depth of the weakened concrete can be determined by excavating it with continued blows by the hammer until sound concrete is reached that will not readily fracture when hammered. Probing with hand tools can also be used to evaluate crack depth.

Infrared Mapping

IR thermography is commonly used to identify delaminations and voids in reinforced concrete structures, principally decks, by detecting temperature differentials between sound and delaminated reinforced concrete.

The air/moisture in the pocket formed by a delamination or void provide different thermal conductivities than solid concrete. This difference shows up when there are temperature gradients within the concrete and thermal radiation during temperature changes in the concrete element. Sensitive thermographic cameras can be used to detect/record images of the element, showing thermal gradients that can reveal the presence of a delamination or void. Interpretation of the images may be difficult due to variations in exposure to sunlight throughout the structure. Some bridge elements are typically shaded from sunlight and obtaining suitable temperature gradients between defects and solid concrete may be difficult (op. cit. 10). In the past, KTC experimented with the use of thermography for NDE on bridge elements other than decks but did not achieve consistent results. IR thermography has been successfully used to evaluate bridge decks. That application is addressed by ASTM D 4788-03, *Standard Test Method for Detecting Delaminations in Bridge Decks Using Infrared Thermography*.

Radar Mapping

GPR is analogous to sonic or ultrasonic test methods except electromagnetic waves (i.e., microwaves) rather than stress waves are used to probe concrete in a similar manner for indications. When they hit the interface between materials with different dialect or conductive characteristics, a portion of the wave energy is reflected. Diagnosis of those reflections enables the measurement of concrete thickness, detection of delaminations and voids, and significant deterioration of rebar (op. cit 10).

A GPR unit uses an antenna, either placed on the surface of the concrete element or sometimes offset with an airgap. The beam is usually driven at 1 GHz for use in concrete. A contact (dipole) antenna is commonly employed. It sends out the electromagnetic waves as a diverging beam. A non-contact (horn) antenna sends out a focused beam (Ref. Figure 7). The antenna contains both a transmitter and a receiver. The transmitter emits a short burst or pulse of microwaves aimed at the concrete surface. The receiver also detects that pulse, whose duration depends upon the antenna frequency. For a contact antenna, as the microwaves penetrate into the concrete, some of the microwave pulse is reflected and detected by the receiver. The balance penetrates into the concrete, and a portion of the pulse is reflected by an interface with a different material (air/water inside delaminations or voids or reinforcing steel). The reflected pulse is picked up by the receiver that generates a voltage signal. The signal is shown on a display as a function of time (waveform) between when the pulse was generated and when the reflection was received. The signal from the receiver is plotted as a function of time. By multiplying one-half of the round-trip travel time by the propagation speed in the material, the vertical axis on the display shows the depth of the reflecting interface or the target.

Modern computer-based GPR systems display the waveform output from the antenna in real time, allowing for prompt evaluation of the concrete element. As the antenna is rolled continuously along the surface, a 2D image is created (Figure 24). Reflections from reinforcing bars result in an inverted V pattern in the image. This pattern occurs because the antenna has a characteristic influence zone and it is capable of detecting a reinforcing bar when the center of the antenna is not directly over the bar. The image represents the cross section of the test object along the scan line based on the antenna signal. The bands at the top of the image are due to the pulse being received directly by the receiver as the pulse is emitted. In the scanning mode, it can be used to accurately detect the concrete cover over reinforcing steel while also evaluating the concrete element for other defects. GPR has been shown to be more effective in mapping reinforcing steel than covermeters (39). ASTM standards related to the use of GPR on concrete include ASTM 4748-10, *Standard Test Method for Determining the Thickness of Bound Pavement Layers Using Short-Pulse Radar*, and ASTM 6087-08, *Standard Test Method for Evaluating Asphalt-Covered Concrete Bridge Decks Using Ground Penetrating Radar*.

GPR is useful for:

- Detecting internal voids and deterioration
- Locating reinforcing steel, metallic tendon ducts, and other metallic embedments
- Measuring concrete cover over reinforcement and other embedments
- Measuring the thickness of slabs and pavements
- Detecting embedded cables carrying electrical current

While GPR has been used to detect internal concrete defects, its ability to do so depends on several factors which, at a minimum, require significant expertise in testing and evaluation (40).

Ultrasonic Surface Waves

This relates to a special case of ultrasonic pulse velocity testing (reference Figure 20 – c) Indirect method). When a crack is present in concrete and where the wave speed is known, the time-of-flight of ultrasonic waves can be used to calculate crack depth. When the two transducers are separated by a crack at a known distance, the crack will cause the path length of the ultrasonic pulse to become longer than for the same transducer spacing in unflawed concrete. Comparing the two transit times (crack and unflawed) and knowing the speed of ultrasound, the depth of the crack can be computed (as long as the crack width is sufficiently large to prevent the ultrasound from bridging the crack (op. cit. 30). A simple instrument is available that contains transducers at a fixed spacing (Figure 25). The unit can be first tested in unflawed concrete to obtain the speed of sound and then tested with the two transducers spanning the crack. The instrument is designed to provide the resulting crack depth. The device can be used to:

- Assess concrete uniformity
- Estimate the extent and severity of deterioration of near-surface concrete
- Evaluate flexural strength of stone panels using correlations
- Evaluate damage to test specimens during durability testing (freezing and thawing, sulfate attack, ASR)
- Estimate depth of surface-breaking cracks
- Estimate early-age strength development (with project-specific correlation)

3D Tomography
3D (dimension)-tomography is an advanced method of evaluating concrete for flaws such as delaminations and voids and measuring concrete thickness. These units have arrays of dry, point-contact transducers operated in an ultrasonic pitch-catch method mounted on the back face of the instrument. The operator places the backside of the instrument on the face of the concrete element (Figure 26a). The transducers create shear waves which are reflected off of internal defects and back surfaces of a concrete element. The reflected waves hit the transducers which are clocked to operate as receivers of the reflected waves. The device subsequently produced a 2D image of the reflected surfaces that is presented on a screen in B-scan format with the image colorized to indicate the intensity of the reflector (and potentially the severity of a defect). He can index the unit as needed over the concrete surface (or as needed) to test the element. The series of 2D images obtained from test are transferred to a computer with imaging software that compiles the 2D slices into a 3D image of the test object (Figure 26b). The 3D image can then be manipulated for interpretation of test results. The test is very rapid and the operator can see the 2D scans in real time to adjust his test pattern if necessary.

The 3D tomography test can be used to:

- Locate and possibly categorize subsurface concrete defects (voids, delaminations, internal honeycombing)
- Measure the thickness of concrete elements
- Determine concrete pulse velocity to estimate homogeneity and strength

ACI 228.2R-13, *Report on Nondestructive Test Methods for Evaluation of Concrete in Structures*, addresses the use of 3D tomography.

4.2.3 Cracking Type/Severity Characterization
Methods for crack characterization include:

- Crack mapping
- Crack width measurements
- Crack movement monitoring

As noted, cracks and their causes can be correlated with crack location, size, and pattern (see Table 1 and Figure 16). As part of a visual survey maps can be prepared that indicate where cracks are located along with descriptions of their dimensions and causes. Along data on length and orientation, information on crack width is useful for assessing severity. The concrete's ambient temperature and surface temperature should be recorded. Many cracks are essentially dormant, but their widths may vary with reinforced concrete temperature. Working cracks, driven by traffic loads, can be encountered in negative moment areas on bridge decks, and in some cases on girders. Because these can impact the type of repair employed, they need to be identified. Working cracks can be usually identified by location, width, and orientation. Girder cracks probably need to be evaluated by monitoring with strain gages — preferably during load tests. Cracks in beams may warrant structural analyses.

4.2.5 Assessment of corrosion activity of reinforcing steel and other metallic embedments

Corrosion of reinforcing steel is an electrochemical process. Therefore, electrochemical tests (corrosion potential and corrosion rate) are used to characterize the location and severity of corrosion activity. Corrosion activity can also be evaluated directly by the measuring section loss of reinforcing steel.

Electrical Continuity and Potential Testing

Electrical continuity tests are used as a preliminary method to evaluate whether reinforcing steel and other metallic embedments are in electrical contact. The test should be performed prior to half-cell corrosion potential tests to verify suitable electrical continuity. It can reveal where epoxy coated reinforcing steel is present, which usually limits corrosion potential testing to individual reinforcing bars. When electrochemical protection measures are considered for reinforcing steel, the electrical continuity must be known.

Uncoated (black) reinforcing steel is typically intimately connected through contact with transverse bars, couplers, welding, chairs, or tie wires, enabling performance of the potential tests. Additionally, steel may be welded to conductive embedments such as steel scuppers, guardrails, armored edges of deck joints, or finger dams. When the electrical continuity of reinforcing steel mat is good, half-cell corrosion potential measurements can be performed. Also, exposed steel embedments can potentially be used as contact points for attaching voltmeters employed for half-cell corrosion potential tests, eliminating the need to excavate the concrete to make a direct electric connection to reinforcing steel.

Prior to field testing, corrosion potential testing grids should be identified and laid out in field notes to facilitate data collection. Grids should be electrically isolated from one another (e.g., decks in adjacent spans, columns on piers). Concrete must be excavated at several points on the grid to expose underlying reinforcing bars. A covermeter can be used to locate the bars. Four tests can be used to check for proper continuity:

- *DC resistance* measures the resistance between the two rebar locations within a grid, with the leads firmly clamped to the reinforcing bars. The resistance between the two locations is measured with a high-impedance multimeter (>10 MΩ) with lead polarity normal and reversed.
- *AC resistance* is measured with the same field attachments*.* AC resistance is determined using an AC bridge null resistance meter. For proper continuity, resistance should be less than one ohm.
- *DC voltage difference* measures the potential difference between the two locations with a high-impedance multimeter. For continuity, potential differences should be less than one mV.
- *Half-cell potential* is measured with the same field attachments The potential is measured against a reference cell at a fixed point on the concrete within the grid, with alternating attachments to the two rebar locations. For continuity, potential measurements should be less than 3 mV (op. cit. 10).

Potential mapping

After completing the visual inspection of concrete elements, half-cell corrosion potential testing can be performed to assess the potential presence and aggressiveness of corrosion of the reinforcing steel under visually sound concrete. To perform the tests, lead wires are required to connect a high impedance voltmeter (>10 MΩ) to a half-cell reference electrode. In the past, copper/copper sulfate electrodes were used. However, they are prone to several issues and have commonly been replaced by silver/silver chloride electrodes.

To perform the test, a grid is placed on the concrete's surface. For smaller concrete elements such as walls, pier caps, and columns, the grid spacing can be 1-2 feet. For larger surfaces such as decks, a 4-ft. spacing may be acceptable. As noted for electrical continuity testing, once the underlying reinforcing steel has been exposed and checked for continuity, the reference electrode is connected to the negative terminal of the voltmeter. The lead from the reinforcing bar is connected to the positive terminal (Figure 27). The electrode may use a wetted sponge for coupling with the concrete. If the concrete appears dry, a small patch of concrete about the grid marks can be wetted with potable (tap) water. Then, the reference electrode is placed on the concrete surface. Shortly thereafter, when the voltmeter reading is stabilized, the reading can be recorded. All potentials should be rounded to the nearest 0.01 V and entered (for convenience) on a data sheet laid out in a grid pattern to an approximate scale similar to the bridge element being tested (41). Some corrosion potential test devices have the ability to electronically store test data and subsequently download it to a computer for display, such as equipotential or 3D plots (test readings vs. surface reading locations).

Different probe types are available. For smaller test areas the rod-type probe will suffice. For larger areas, single- and multiple-wheeled probes used with data logging devices provide faster tests (Figure 28).

ASTM criteria are normally used to interpret readings. For silver/silver chloride reference electrodes, readings are categorized as follows:

1. Potentials less than -100 mV generally indicate a 90% or higher probability of no corrosion taking place at the time of measurement.
2. Potentials in the range of -100 mV to -250 mV indicate an intermediate corrosion risk
3. Potentials less than -250 mV generally indicate a 90% or higher probability of active corrosion in the area at the time of testing
4. Potentials less than -400 mV indicate severe corrosion
5. Positive potentials typically indicate insufficient moisture in the concrete and should not be considered valid. Stray DC currents may also cause potential measurements and, therefore, careful review of the data is necessary.

The resulting data can be depicted graphically as an equipotential contour map with all potentials plotted and contours drawn through points of equal or interpolated equal values. Maximum contour intervals should be 100 mV. The report should include the method used to pre-wet the concrete along with attachment locations/methods and estimates of percentages of readings in each corrosion category. Several problems can affect the test results. Half-cell corrosion potential tests are not good in carbonated concrete or on water-saturated (or in near-saturated conditions) and submerged concrete. Delaminations may disrupt the potential field, producing false readings. Readings can also be affected by temperature, pH, and oxygen availability.

Locations of probable/severe corrosion activity indicated by corrosion potential testing (i.e., hot spots) can be further evaluated using the corrosion rate tests (below). They can also be compared to concrete cover maps to correlate corrosion with thin concrete cover. They can also be excavated to expose the reinforcing steel for visual inspection of corrosion activity. Also, chloride measurements can be taken in the concrete to determine if the critical concentration threshold has been reached at the depth of the reinforcing steel. Half-cell corrosion potential tests are addressed in ASTM C876-15, *Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete*.

Corrosion Rate Measurements

Rate-of-corrosion equipment consists of a measuring device and a suitable probe. The linear polarization resistance (LPR) technique provides a reliable and simple method for determining instantaneous corrosion rates.

In addition to half-cell corrosion potential tests, other methods have been developed to directly measure the instantaneous corrosion rate, including transient or steady-state techniques. Examples of steady-state techniques include Tafel extrapolation (E-log I) and LPR. Potential step, small-amplitude cyclic voltammetry, electrochemical noise, and AC impedance measurements are classified as transient methods.

Corrosion rate equipment consists of a measuring device and a suitable probe. The measuring device controls the test process and the probe applies current into the concrete. One lead to the measuring device is attached directly to reinforcing steel to complete the circuit. The LPR test is a commonly used DC corrosion rate test method. In performing this test, the measuring device applies steps of current ($\Delta \mathbf{i}$) or potentials (ΔE) are applied using a counter electrode for a set time interval. At the end of the interval, $\mathbf{R_{ct}}$ (resistance of the concrete) is calculated by taking the potential shift (if ΔE is the response variable) and dividing by the applied current. The resulting corrosion current density, $\mathbf{i_{corr}}$, is calculated by dividing the total corrosion current, $\mathbf{I_{corr}}$, by the surface area of the steel being polarized. The resulting changes in current or potential — whichever is the response variable — are measured and those measurements allow estimates to be made of the steel reinforcement corrosion rate. The readings are made in timed steps, because reactant transport in the concrete is slow, and the steps allow the current and potential to reach a steady state condition (op. cit. 15). Typical corrosion rates from those tests are provided in Table 3.

While LPR is used by several instruments, a similar instrument demonstrated for KYTC officials uses a different principle (Figure 29). That device evaluates the corrosion rate of reinforcement by measuring polarization resistance using the galvanostatic pulse technique (43). A current, **I,** is pulsed on the reinforcement from a counter electrode placed on the concrete surface (Figure 30). A metal guard ring confines the current to an area, **A,** of the reinforcement below a central counter electrode. The applied current is usually in the range of 5 to 400 μA and is applied for 5 to 10 seconds. The reinforcement is polarized in the anodic (+) direction compared to its free corrosion potential, $\mathbf{E_{corr}}$. The resulting change in the electrochemical potential of the reinforcement is recorded as a function of time using a silver chloride (Ag/AgCl) reference electrode. A typical potential response for reinforcement actively corroding is shown in the right of Figure 30. When the current is applied, there is an ohmic potential drop, **IRo**, as well as change in potential due to polarization of the reinforcement, **IRp**. Assuming the Randles circuit model, the polarization resistance of the reinforcement **Rp** is calculated by curve fitting to the transient portion of the potential data. By means of the Stern-Geary equation for active corrosion ($\mathbf{I_{corr}}$ = (26 **A**)/**Rp**) and Faraday's law of electrochemical equivalence, the corrosion rate is estimated as: Corrosion Rate (μm/year) = 11.6 $\mathbf{I_{corr}}$ /**A** where **A** is the confined area (in cm^2) of the reinforcement below the central counter electrode. The factor 11.6 is for black steel. The value of $\mathbf{R_o}$, the electrical resistance of the concrete between the counter electrode and the reinforcement, is also determined.

That particular instrument can be used to measure half-cell potentials, electrical resistance, and the corrosion rate of reinforcement in concrete. For structural assessment purposes it can be used for monitoring corrosion activity in reinforced concrete structures (especially structures in wet environments where half-cell corrosion potential measurements cannot be used effectively) and estimating remaining service life. Post-repair applications include evaluating the efficiency of corrosion-arresting measures (e.g., application of inhibitors, membranes, electrochemical removal of chlorides, concrete realkalization) and measuring corrosion activity in repaired areas (op. cit. 36). Corrosion rate tests have been proven to be relatively accurate, though they can be affected by ambient conditions (op. cit. 10).

Corrosion mapping of half-cell corrosion potentials can, with minimal invasiveness, identify areas of probable corrosion. While they are relatively easy to prepare, they don't provide quantifiable information about corrosion activity. Corrosion rate measurements provide better information for predicting future deterioration. The two methods are complimentary and permit assessment of reinforcing steel corrosion with the need for extensive concrete removal. At this time, there is no standard for corrosion rate measurements.

Metal Section Loss Measurements

Cross-section measurement of reinforcing steel bars can be used to determine the amount of corrosion that has taken place since the concrete element was put in service. To perform this measurement, the concrete cover must be removed, though in some cases, it has previously been dislodged by spalling (Ref. Figures 4-6). The surface rust on the reinforcing steel bars must be removed by abrasive blasting or wire brushing. The in-situ diameter of reinforcing steel can be measured using a caliper and the section loss can be determined by comparing the in-situ cross-sectional area from the field measurements with the nominal area of the specified reinforcing bars. The corrosion rate can be estimated by dividing the difference between the two diameters by the number of years the bridge has been in service (μm/year). The measured corrosion rate can be helpful for predicting the future deterioration rate (along with corrosion potential and corrosion rate electrochemical test results). It can also be useful for determining structural integrity (op. cit.10).

4.2.6 Attendant Factors Impacting Corrosion Initiation and Subsequent Propagation of Reinforcing Steel and Other Metallic Embedments

Depth of Concrete Cover; Magnetic Testing/Covermeter

Concrete covermeters are also referred to as a pachometers. Most covermeters operate on the pulse-induction method. They employ a measuring device containing the power source, amplifier, and meter/display with a separate probe. The probe is placed, slid, or rolled upon the reinforced concrete surface (Figure 31). A repetitive current pulse is sent to coils in the search head (Figure 32). The termination of the pulse causes the magnetic field produced by the coils to collapse. That induces eddy currents in a steel reinforcing bar located within the coils' influence zone. As the eddy currents decay, a decaying magnetic field induces a secondary current in the coils. The instrument measures the amplitude of the induced current, which depends on the orientation, depth, and size of the bar. The closer the reinforcing bar to the probe and the larger the size of the reinforcing bar, the greater the amplitude of signal created by the induced current in the bar. The concrete industry primarily uses covermeters to detect the presence, size, and depth of rebars (op. cit.37).

The pulse-induction technique is uniquely stable, is not affected by moisture in concrete or magnetic aggregates, and is immune to temperature variations and electrical interference. However, it has a limited detection range and may generate inaccurate results when the depth of concrete cover is equal to or close to the spacing of the reinforcing bars. During this test, steel or other metals must not be present close to the area under examination. Some results from tests must be calibrated by chipping at concrete to confirm concrete cover and bar size.

Some more elaborate units incorporate advanced signal processing, enabling them to locate rebars, determine the concrete cover, and estimate the bar size. Some units can perform line scans using probes equipped with rollers, enabling the unit to be rapidly moved across the concrete surface while providing a display showing the bar sizes and cover thickness.

Compared to GPR, covermeters only locate a single layer (or cross layer) of rebars. They can only detect conductive materials. GPR can locate all embedded steel and other embedded objects in concrete. Covermeters are limited to about 140 mm in depth. GPR can locate to a depth of 500-700 mm using common concrete scanning models. Providing the reinforcing bar size is known, a covermeter can locate its depth within 1 mm. GPR can accurately locate the lateral position of reinforcing bars and provide an estimate of depth. A calibration of the concrete's dielectric constant will improve the accuracy of its depth measurements. For data display, covermeters can record either a table of rebar cover depth values or a diagram of reinforcement position and depth. Basic GPR data can be presented as images of the scan in a line scan or area scan format. Covermeters need correctly set adjustment factors and a skilled operator following good procedure to correctly measure the precise depth of closely spaced reinforcing bars (44).

Concrete Resistivity

As corrosion of reinforcing steel is an electrochemical process, concrete resistivity directly impacts the corrosion process. Concrete resistivity affects flow of ions in between the anode and cathode sites in the reinforcing steel. It depends on the size and volume of the pores, the pore water composition, and the concrete moisture content (op. cit. 12). Dry concrete or concrete with low w/c ratios have high resistivity (up to 1MΩ-cm).

There are a number of one- and two-probe methods used to provide in-situ measurement of concrete resistivity. The most common method uses the four-point probe (the Wenner method). The test uses an instrument with four equally spaced probes that are typically wetted and placed on the surface of the concrete (Figure 33). The outer two electrodes produce an AC current and the two inner probes measure the resulting potential drop in the electric field (Figure 34). The resistance, $\boldsymbol{R}$, is equal to $\Delta \boldsymbol{E}/\Delta I$. The resistivity of the concrete, $\boldsymbol{\rho}$, is given as:

$$\boldsymbol{\rho} = 2\,\boldsymbol{\pi} \cdot \boldsymbol{R}$$

In testing reinforced concrete, a covermeter should be used to properly locate the Wenner unit so that the probes are not located over concrete. The optimum orientation is to measure diagonally to the reinforcing bar mat (Figure 35a). This is possible if the probe span is less than the rebar grid spacing. If the reinforcing bar mat spacing is too small to be avoided, the influence of the steel can be minimized by measuring perpendicular to the rebars (Figure 35b).

The probe spacing, ***a***, should be larger than the maximum aggregate size. If the concrete is relatively homogeneous, differences in resistivity mapping can pinpoint wet (low resistance) and dry areas (high resistance), which can be used to locate potential problem areas (op. cit. 12). Changes in resistance can also indicate differences in the w/c ratio. Carbonation increases the resistivity of concrete if its depth is greater than the probe spacing.

Resistivity is directly linked to both the likelihood of corrosion due to chloride diffusion and to the corrosion rate once depassivation of the steel has taken place. Resistivity measurements can be used to estimate the likelihood of corrosion. The likelihood of corrosion increases when concrete resistivity is low. When electrical resistivity is high, the likelihood of corrosion decreases. Tests have shown the following criteria for corrosion of PCC at 20°C (68°F). When resistivity ≥ 1000 Ω m, there is negligible risk of corrosion. When resistivity = 500-1000 Ω m, there is a low risk of corrosion. When resistivity = 100-500 Ω m, there is a moderate risk of corrosion. When resistivity ≤ 100 Ω m, there is a high risk of corrosion.

For corrosion rates, the following relationships have been shown for depassivated steel. When resistivity > 200 Ω m, there is a low corrosion rate. When resistivity is in the range of 100-200 Ω m, there is low-to-moderate corrosion rate. When resistivity is in the range of 50-100 Ω m, there is a high corrosion rate. For resistivity < 50 Ω m, there is a very high corrosion rate.

Currently, the ASTM resistivity test is covered in ASTM C1876-19, *Standard Test Method for Bulk Electrical Resistivity or Bulk Conductivity of Concrete*, which can be performed on cores extracted from bridges. There have been some problems encountered by others with the field use of the Wenner test method (45).

Concrete Permeability

The permeability of existing concrete can influence both corrosion initiation and propagation. Where there is no visual indication of concrete distress, permeability can be a determining factor in deciding whether to add further protection to the concrete by coating or sealing or to do nothing. Where corrosion damage has begun, it can be a factor in estimating how rapidly it will progress. A rapid laboratory test can be performed to provide an indication of chloride ion permeability. The test requires extracting a 4-in.-diameter core from the in-situ concrete. The core is then taken to a laboratory for testing. A 50 mm (2 in.)-thick slice is cut from the core. The side of the cylindrical specimen is coated with epoxy. After preliminary treatments, the slice is placed in a test cell between two chambers (Figure 36). One chamber given a negative charge is filled with a 3% NaCl solution. The chamber given a positive charge is filled with 0.3N NaOH solution. The system is then connected and a 60-volt potential is applied across the chambers for 6 hours. Readings are taken every 30 minutes. At the end of the test, the sample is removed from the cell and the total charge (coulombs) passed through the specimen is calculated. That charge is a function of the concrete's initial conductivity and the change in conductivity during the test. Those are affected, in part, by the pore structure of the paste, providing a relative indication of the resistance to chloride ion penetration. Table 4 compares typical test results for various concretes.

Several standards address the performance of this test, including AASHTO T277, *Standard Method of Test for Rapid Determination of the Chloride Permeability of Concrete*, and ASTM C1202-19, *Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*.

Test results are affected by a range of factors, including curing of the concrete and the presence of salts in admixtures. Other factors include cement type, air content, w/c ratio, sample curing, and aggregate type and curing of the test sample aggregate source or type (46).

Depth of Carbonation in Concrete

As noted, carbonation attack reduces the pH of concrete to about 9. At that point, the protective oxide coating is destroyed and, in the presence of moisture and oxygen, the reinforcing steel corrodes. Carbonation testing can be performed on a spot basis on structures during routine biennial inspections or in-depth assessments, especially when reinforced concrete distress is evident. Testing can be performed in conjunction with chloride tests to get a complete picture of the situation. The test requires the dry extraction of a small (~ 1.5-in.-diameter core) to a depth below that of the concrete cover thickness (~ 1 in. below the lower reinforcing steel layer in the mat). Then, the fresh core should be immediately tested by spraying it with an indicator solution to keep atmospheric CO_2 from interacting with the surface of the core and reducing the pH. Smaller surface cores can be taken to begin with just to find out whether any carbonation has occurred, thereby eliminating the need to take deeper cores.

The depth of carbonation is measured by exposing a fresh concrete surface and applying a solution of phenolphthalein in ethanol. Phenolphthalein is a clear pH indicator that turns magenta/purple (or a pink tint) when in contact with concrete at a pH of approximately 9 or above. Magenta areas observed on the core indicate uncarbonated concrete; the colorless areas indicate carbonated concrete.

A commercially available indicating solution for testing the pH on freshly exposed concrete surfaces provides a range of pH indications ranging from 5 to 13 in steps of two pH. Each pH value corresponds to specific pH: orange – 5 pH; yellow – 7 pH; green – 9 pH; purple – 11 pH; and dark blue – 13 pH. This test is suitable for assessing concrete pH as well as investigating the presence/depth of carbonation. The firm also provides a single-color (phenolphthalein) indicator. Using an indicator, on normal concrete the depth of the carbonation front can be determined with an accuracy of ± 10 % to ± 15 %.

Smaller cores with shorter lengths can be quickly cut from concrete using hammer drills (Figure 37). It they do not show any carbonation, deeper cores are not required. The resulting holes can be quickly patched with mortar repair sealant using a caulk gun. A covermeter can be used to determine the appropriate drilling depth to determine whether carbonation has reached the reinforcing steel and to also avoid drilling where it is present.

Knowing the carbonation depth, the concrete cover thickness over the reinforcing steel, the age of the concrete, and estimating the carbonation coefficient based upon the quality of the concrete (e.g., w/c ratio), the time to initiation of corrosion of reinforcing steel due to carbonation can be estimated using Equation 1.

Currently, there is no U.S. standard for field determination of concrete pH.

Concentrations of Chloride Ions at Varying Depths

As noted, chlorides in sufficient concentrations in concrete can attack the passive protective film and promote corrosion of reinforcing steel. To determine the susceptibility of reinforced concrete to chloride attack, the chloride content in a concrete element can be determined. Several approaches to testing are available. One involves coring to remove a concrete specimen. The core is taken to a laboratory and sliced at several representative depths from the concrete surface. The resulting slices can be ground to a powder and then tested using either an AASHTO standard wet chemistry test or one of several available commercial tests. The other approach is to extract powder samples from various depths in the concrete by drilling (Figure 38a). While the first method is considered more accurate from a depth standpoint, KTC researchers have found that with care, the drilling method can produce accurate results as well. Taking powder samples enables faster all around processing, but care must be taken when obtaining the powder samples at specific depths.

The test depth should be to at least the level of the reinforcing steel. As that can vary, covermeter tests can be performed first (as should be done for the carbonation tests above) to identify the best locations to carry out the tests. Those should include sites where the cover depth is the thinnest. Test frequency and location can also be guided by half-cell corrosion potential tests. Chlorides are generally present in all concrete-making materials to some degree. A few test samples can be extracted at deeper-than-rebar depths to determine the background (cast-in) chloride content versus shallower test sampling, which may also include chlorides from diffusion. As the top 1/2” or so may experience capillary suction rather than chloride diffusion (Ref. Figure 12), the concentration of chlorides just below the surface of the concrete may be higher than the surface concentration as the surface concentration may be reduced by periodic washing from maintenance cleaning operations or rain. When taking powder samples, care should be taken to prevent cross-contamination from dust generated by previous sampling.

The typical commercial test employs a pre-measured extraction fluid (typically a mild acid) that is mixed with a pre-weighed sample of the concrete powder. The extraction process removes sulphide and chloride ions from the sample, typically for a limited amount of time (usually a few minutes). Thereafter, the sample can be filtered and the extract tested with titration tubes or an analytical instrument with a probe to measure the electrochemical reaction of the digested sample, providing a temperature-compensated direct reading of the percent of chlorides in the concrete on a digital display.

KTC researchers have frequently used a third commercial test similar to the other two commercial tests. Again, a fixed amount of powdered concrete sample is mixed with a pre-measured weak (nitric) acid for a minimum of five minutes. Typically, KTC would allow the reaction to occur over a 24-hour period to ensure complete extraction of the chlorides from the concrete into the solution. The test uses a calibrated electrode immersed in the solution to provide a determination of the amount of chloride ion in the concrete expressed as a weight percentage (Figure 38b). The firm also provides a water-soluble test kit requiring a longer (24-hour) extraction period. The standard acid-extraction test results from this manufacturer have been well correlated with AASHTO - T 260, *Standard Method of Test for Sampling and Testing for Chloride Ion in Concrete and Concrete Raw Materials*, by several laboratories in the U.S. and Europe. The accuracy of the results of this method compared with the known amount of chlorides is as good as with the AASHTO T 260 potentiometric titration method. The average deviation of the results of this test from the known amount of chlorides is ± 4 %. For repeated testing with this method on the same concrete powder, the coefficient of variation of test results is on average 5 % (op. cit. 36).

The Nebraska Department of Roads performed a laboratory analysis of the AASHTO - T 260 test using several titration methods, various sample sizes, and solvents (water and acid) for sample preparation. The results favored the acid-soluble sample preparation method (47).

The use of the water-soluble test is intended to extract only free chlorides in concrete that are readily available to promote depassivation of the steel. The acid-soluble test measures the total chlorides in the concrete, including both free and bound (physically and chemically). Total chloride content analysis is accepted because bound chlorides in the concrete can become unbound as a result of chemical reactions within the concrete and participate in the corrosion process. Also, the acid-soluble test is more reproducible and quicker than water-soluble chloride analysis procedures and has become more accepted (op. cit. 10). There are various recommendations for considering a critical chloride threshold for corrosion based on chloride tests.

As noted, values for the chloride threshold for corrosion at the rebar depth vary. However, it is useful to rely on a fixed value for decision making. In a past report (48), KTC referenced the following values:

- 0.03 percent chloride to weight of concrete = initiation of corrosion
- 0.08 percent chloride to weight of concrete = accelerated corrosion
- 0.18 percent chloride to weight of concrete = major section loss of steel (49).

For acid-soluble tests, a chloride threshold of 0.05 percent chloride to weight of concrete has been recommended as a value for the onset of corrosion (op. cit. 45). Knowing the chloride threshold, the concrete cover thickness over the reinforcing steel, the age of the concrete, the chloride content at the depth of the reinforcing steel, the surface chloride content, the time-of-service exposure/age of the concrete, and the apparent chloride diffusion coefficient (based on the type and quality of the concrete), the time to initiation of corrosion of the reinforcing steel due to chloride penetration can be estimated using Equation 2.

Test results can be used for:

- Establishing the chloride ion profile for service life estimation
- Establishing the depth of removal of a chloride ion contaminated surface layer
- Diagnosing a structure for corrosion activity in combination with other test methods, including half-cell corrosion potentialhttp://germann.org/products-by-application/half-cell-potential/minigreatdane, corrosion rate, and a carbonation indicator (pH)
- Monitoring the chloride ion content during electrochemical removal of chlorides
- Measuring the chloride ion content of fresh concrete or its constituents (op. cit. 36)

Concrete Moisture and Resistivity Measurements

The moisture content in concrete affects many deterioration processes, including ASR, freeze and thaw damage, sulfate attack, and corrosion. Concrete resistivity impacts corrosion. For less dense concretes, the maximum corrosion rate occurs when the relative humidity of the concrete is in equilibrium with that of the atmosphere and is slightly above 95%. Concrete moisture testing can be conducted along with resistivity tests. The resistivity of the concrete, which is a function of the moisture and electrolyte content, impacts the corrosion rate of the reinforcing steel. Therefore, both of those properties can be measured. The method contained in ASTM F 2170 – 02, *Standard Test Method for Determining Relative Humidity in Concrete Floor Slabs Using in situ Probes*, may be adaptable for this purpose. The method involves drilling a hole in the concrete and inserting a probe that measures relative humidity. The test takes several days to complete.

Other Tests

There are several other NDE and test methods involving ultrasound, radiography, acoustic emission, and resonant frequency that fall outside of the normal methods that might be considered by KYTC for evaluating bridges for repairs. Those are beyond the focus of this report and not addressed.

Stray Currents

Reinforcing steel in concrete bridges may be susceptible to corrosion due to electric currents that stray from their intended path and flow into the reinforced concrete. While the most common sources of this are electrified railways or cathodic protection systems, in Kentucky the most likely source of stray currents is high-voltage power lines. In the 1980s, KTC researchers were attempting to monitor sensors on the I-64 westbound bridge over the Tennessee river. To do that, they installed sensors on the main span and ran signal cables several hundred feet along the gutter line of the approach span intending to monitor the sensors by connecting them with the cables to instruments at the end of the approach. A large cluster of power lines was located parallel to the bridge, with the power lines spanning the river. The power lines were several hundred yards away from the bridge. When the researchers attempted to operate the instruments, a voltage was induced into the signal cables by the power lines, preventing the researchers from using their equipment. That indicates the power lines may be able to induce voltages into the reinforcing bars running parallel to them. However, that has never been investigated. Other bridges may be located near power lines or carry them as well. At this point, when medium- and high-voltage power lines are nearby or on bridges and corrosion damage is present, the possibility of stray currents should be considered (50, 51). The presence of stray currents may be detected by using low-impedance voltage meters connected from the reinforcing steel to the ground or by non-contact sampling of electric fields about reinforcing steel using electric field sensors. As this issue has not been fully addressed for bridges, no further inspection methodology or repair suggestions are provided. This issue should be addressed by specialists experienced in working on stray current problems in reinforced concrete structures.

Chapter 5 Maintenance and Repair of Structural Concrete

5.1 Scope of Maintenance and Repair Actions

Decisions making for follow-on bridge maintenance actions depends on the current operational practice of KYTC. Funding and personnel constraints can impose limits on the scope of maintenance actions. Typically, such limitations result in an emphasis on repairs/rehabilitation, replacement (bridge), or do nothing (funding-constrained or condition-driven). Those limitations generally result in an emphasis on reactive maintenance, leading to repairs/rehabilitation. In those instances, NDE and testing of structural concrete can help KYTC assess a structure, determine the best maintenance option, and scope the work. The latter includes identifying what action is warranted and (if applicable) how extensive that work needs to be. It may not be appropriate to apply a single treatment or repair on the whole structure. In planning treatments or repairs, it is best to use targeted treatments so the entire structure will provide similar service performance in the future — with the possible exception of high-wear items like bridge decks and expansion joints.

5.2 Proactive Maintenance Actions

Past KTC research indicated that reliance on decision making that emphasizes reactive maintenance is not sustainable in the long-term. It needs to be supplemented with proactive maintenance measures that are low-cost and can limit gradual structural deterioration typically caused by reinforcing steel corrosion. A focus on the application of proactive measures to concrete bridge elements is warranted as they are the most common elements on KYTC bridges. The annual implementation of this practice, on even a small percentage of bridges in the KYTC inventory, should have noticeable benefits in preserving KYTC bridges systemwide within a very few years.

Routine maintenance and PM are the two proactive maintenance actions. They are the lowest-cost actions that can be taken on bridges. Typically, when they are performed, they do not increase the condition rating of bridge elements. However, they are very effective in arresting or reducing the rate of deterioration of bridges, including reinforced concrete, thereby preserving structures.

5.2.1 Routine Maintenance

Routine maintenance can include tasks such as removing debris, washing bridge elements, opening drains, cleaning gutter lines, cutting brush abutting the bridge, or stream upkeep (Figure 39). Many times, this work can be done with in-house forces. Bridge cleaning entails collecting debris on bridge elements and washing the structure. Washing operations should probably include low-volume (4 – 5 GPM) pressure washers to move debris along gutters and clear it from around joint seals. It can also be used to eliminate visible soiling on vertical surfaces. High-volume/low-pressure (firehose) washing is probably more suitable for washing away accumulated chlorides on the surface of steel and concrete members. That approach has been shown to be effective in reducing near-surface chloride contamination in deck concrete after several years of treatment (52).

Clogged drains should be cleared for proper function and the operation of drainage systems should be evaluated for potential follow-up modifications if necessary. Drainage problems can lead to ponding of rain water on decks, causing a motorist hazard, and can also retain water contaminated with deicing/anti-icing chlorides on decks and promote reinforcing steel corrosion. Improperly functioning drainage systems can deposit chloride-contaminated runoff on beams and substructure elements. Cleaning joints can keep seals from being damaged by debris build-up. Leaking deck joints are one of the major causes of corrosion in reinforced concrete substructure elements. Routine maintenance is typically performed independently of structural assessments. However, biennial inspections can help identify how frequently routine maintenance should be performed. An example of routine maintenance from the *Iowa DOT Maintenance Manual* specifies annual washing of bridge beam seats as part of joint and deck cleaning activities. The operation includes preliminary manual cleanup of debris using shovels, brooms and other hand tools. The washing operation is to clean off salt contaminants, dirt, and other foreign matter using pressure washing with fresh water at 5 gpm (min.) and 1,000 psi (53). Effective routine maintenance can prevent/minimize reinforcing steel corrosion by eliminating or reducing the environmental exposure of bridge elements to chlorides and moisture.

5.2.2 Preventive Maintenance Actions

PM actions can be targeted by structural assessments. Assessments can determine if the condition of concrete elements is sufficiently good to allow the use of PM. When the visual inspection reveals no cracks (or cracking that is not typical of reinforcing steel corrosion), NDE and testing can be used to determine whether conditions at the reinforcing steel depth in the concrete are suitable for PM actions. In many instances, the condition of concrete elements may be such that repairs (e.g., minor concrete patching) are required on a portion of the element while PM actions can be performed elsewhere. Such assessments should be sufficiently detailed to indicate which actions should be performed at various locations.

PM actions include surface treatments for reinforced concrete, including sealing and painting, application of penetrating corrosion inhibitors, and crack sealing. Electrochemical PM actions include chloride extraction and realkalization of the concrete.

5.3 Procedures for Repairing and Addressing Corrosion on Reinforced Concrete Bridge Elements

There are a variety of methods to address 1) overall concrete deterioration and 2) corrosion of reinforcing steel. This section addresses some of the more common methods. There are basically three approaches to these methods: 1) Surface treatments are intended primarily as corrosion prevention/minimization measures. They are intended to make the environment less aggressive by eliminating or diminishing conditions that prompt corrosion. 2) Concrete repair methods fix distressed concrete, typically by local removal and replacement. These should address corrosion, especially if it is a source of concrete distress to be addressed in the repair process. 3) Electrochemical treatments and repairs attempt to arrest corrosion or significantly reduce the corrosion rate by affecting the corrosion process (op. cit. 10). There are a range of options for repairs/strengthening of reinforced concrete bridge elements. Most methods, such as steel jacketing, are not be addressed in this report. FRP wraps are only be discussed in conjunction with reinforced/prestressed concrete repairs.

5.3.1 Surface Treatments

The typical bridge environment is aggressive to reinforced/prestressed concrete. Surface treatments act as barriers to reduce or eliminate the damaging environmental components (e.g., moisture, oxygen, aggressive contaminants) from entering the concrete. These treatments act as barriers.

Ideally, surface treatments are applied on reinforced concrete bridge elements that are free from visible distress, do not yield corrosion potential or corrosion rate readings indicating possible corrosion, and do not have carbonation or critical chloride threshold test results at the level of the reinforcing steel. Surface treatments are very effective and provide the maximum extension of service life if applied under these circumstances. Some sources state that after corrosion initiation, surface treatments are relatively ineffective (54). ACI PRC-222R-01 notes that barrier systems used after corrosion initiation hinder its progress (op. cit. 21). Surface defects in the concrete (e.g., delaminations, spalls) must be repaired prior to their application (op. cit. 10). Corrosion may continue, especially if the chloride ion content is high at the reinforcing steel depth, in the concrete until it may impact the structural integrity. Barrier systems may only be effective for 10 to 20 years and must be maintained. But they are usually effective in extending the service life of a structure. Surface preparations for typical liquid-applied products are addressed by ICRI Technical Guideline 310.2, *Guideline for Selecting and Specifying Concrete Sealers, Coatings and Polymer Overlays*.

Some of the surface treatments include bridge deck overlays (e.g., laminates, polymer concrete, low-slump concrete, silica-fume concrete, latex-modified concrete, rubberized asphalt, epoxy asphalt). These are not addressed in this report. Other methods that can be used on bridge elements include coatings and sealers, alkaline concrete and dense concrete overlays for horizontal surfaces, and various fiber wraps. The latter can be used for restoration of cracked reinforced concrete and for strengthening.

Several types of surface-applied treatments are available for liquid applications, allowing for rapid placement on concrete elements of various shapes and surface orientations. Those can be classified as pore liners, pore blockers and film formers (coatings).

Pore Liners

Pore liners and pore blockers penetrate into the concrete and do not impact its surface appearance or texture. As these products penetrate concrete, they alter the pore structure to limit the ingress of moisture and contaminants that promote corrosion (55). They impregnate the surface of the concrete and line the pores. They are hydrophobic and repel moisture, preventing it from penetrating into the concrete. At the same time, they allow concrete to breathe by expelling water vapor. Penetrating sealers need to be applied to clean, dry surfaces of bare concrete. If pressure washing is used for cleaning, the concrete surface should be allowed to dry for at least 24 hours or as specified by the product manufacturer. These include silanes, siloxanes, and siliconates. A number of proprietary products are available within each type.

Silanes are a molecule with one silicon atom and four attachments (inorganic for sealers). Silanes work best on dense concrete due to their ability to penetrate well (related to their small molecular structure and low surface tension). While the concrete surface needs to be dry during silane application, inside the concrete pores some moisture is necessary for the them to react by hydrolysis. Silanes form covalent bonds with the concrete inside the pores and provide a high surface tension which is hydrophobic. They prevent moisture from travelling through the pores and greatly reduce ingress of dissolved chlorides. In addition, they can help prevent freeze-thaw damage. The concrete being treated needs to be cured for at least 28 days before applying silanes. There are various proprietary silanes available to address applications on horizontal, vertical, or overhead surfaces. They are usually applied by spraying and rolling (Figure 40).

Siloxanes are larger molecules which have Si-O-Si linkages. They are best used in highly porous concrete and on vertical surfaces. Unlike silanes they do not react well with the concrete inside the pores. Due to their molecular size, they do not penetrate deeply into concrete. They are sometimes combined with silanes to provide both sealing of large pores and better penetration. Like silanes, they are hydrophobic and breathable to water vapors. As with silanes, siloxanes prevent moisture from travelling through the pores and greatly reduce the ingress of dissolved chlorides. They also can help prevent freeze-thaw damage. Like silanes, concrete being treated needs to be cured for at least 28 days before applying siloxanes.

Siliconates are generally similar in function and performance to silanes and siloxanes. In addition to being hydrophobic, they are oleophobic and resist oils. They are medium-sized molecules and can be used on both dense and porous concrete. Their application requirements are less restrictive than the other two sealants. They can be used as a curing agent and applied to freshly placed concrete. They can be used in conjunction with silicates to provide both densification of concrete and hydrophobic resistance to moisture penetration.

Pore Blockers

Pore blockers enter concrete and react with it to form products that block pores. These include small molecules — sodium, potassium, magnesium, and lithium silicates. When placed on the concrete surface, they penetrate into pores and react with calcium hydroxide (portlandite), a by-product of cement hydration. The reaction forms crystalline structures that fill the pores and strengthen the concrete. They are commonly termed concrete densifiers or hardeners. Pore blockers restrict contaminants (e.g., chlorides) from penetrating the concrete. To be effective, pore blockers must be used with concrete containing sufficient Portland cement to provide calcium hydroxide for the silicate to react with. Lithium silicate is the best option for treating bridge concrete. Prior to application, the concrete should be cleaned, similar to the requirements for the sealers. It can be applied without surface wetting and penetrates and reacts better than the other silicates. It also does not raise the pH of the concrete and pose a potential ASR problem. One treatment should provide concrete protection for the life of the structure. One issue to be addressed with densifiers is their potential reduction of skid resistance when used on bridge decks.

Film Formers

Film formers are inorganic or organic coatings that form protective films on the concrete surface without chemically reacting with the concrete. They can change the color, gloss, and texture appearance of concrete and can be both protective and aesthetic. Some coatings are flexible and can bridge working cracks.

For coatings applied on concrete, surface preparation entails solvent cleaning (if grease or oil stains are present) followed by either pressure washing or abrasive blasting (Figures 41, 42). Typically, pressure washing is used followed by a 24-hour hold to allow the surface to dry. Coatings are applied by brushing, rolling, or spraying (Figure 43).

In 2005, KYTC experimented with various concrete coatings including an test of various manufacturers coatings on barrier walls of the KY 676 connector over the Kentucky River at Frankfort. Several of these coatings continue to perform well after 15 years of exposure. Epoxy coatings work well directly on concrete and are effective in stopping chloride penetration. However, they are subject to weathering when exposed to direct sunlight. Acrylic coatings are effective at resisting penetration of gases such as carbon dioxide, which promotes carbonation. A two-coat system consisting of an epoxy primer and an acrylic topcoat should provide good resistance to the two aggressive materials that promote corrosion (Figure 44). However, epoxies have limited re-coat windows, especially for successive coatings, such as acrylics, that have a limited “bite” into base coats. To prevent eventual disbonding of the acrylic topcoat, the epoxy recoat window has to be observed. If not, the epoxy surface must be abraded to provide a suitable substrate that the acrylic topcoat can adhere to. Textured masonry coatings were used for years as an aesthetic treatment over concrete. However, KTC tests have shown that these products typically have poor chloride penetration resistance.

Pore liners can be used on all structural concrete corrosion protection. Pore blockers can be used on all structural elements, with the possible exception of bridge decks. Film formers — protective coatings — can be used on all structural concrete except decks (which can receive liquid-applied films such as spray-on membranes and laminates) All of those methods can be used in conjunction with other maintenance activities, including steel repair, joint replacement, and steel painting. According to ACI, there are potential benefits in service life extension even when some reinforcing steel corrosion or conditions favorable to corrosion are present. Certainly, when repairs are necessary on a portion of a reinforced concrete element, the intact portion could receive a surface treatment that would enable the refurbished element to provide additional years of service without requiring continual maintenance attention as the untreated portions of an element began to fail. The *Iowa DOT Maintenance Manual* contains prescribed maintenance actions using sealers, including sealing of barrier rails, sealing of concrete beam ends, and sealing of bridge beam seats (op. cit. 53). Those treatments typically involve repair of any distressed concrete and surface preparation by light abrasive blasting.

Currently, the performance of surface-applied corrosion inhibitors does not appear to warrant their use. However, the ACI lists surface inhibitors as a treatment for reducing the corrosion of reinforcing steel (op. cit. 21). Therefore, they probably should be considered candidates for future experimental evaluation.

5.3.2 Concrete Replacement

Distressed reinforced concrete that is cracked or spalled is readily detected on bridges by visual inspection. The NDE methods and tests discussed in the previous chapter can assist in determining the scope of that distress that is not readily observable (e.g., delaminations). Electrochemical tests such as half-cell corrosion potential or corrosion rate, along with carbonation and chloride-depth tests. can reveal compromised reinforced concrete where the effects of reinforcing steel corrosion have not manifested via concrete distress. Damaged or corrosion-compromised reinforced concrete is an obvious candidate for removal and replacement. Concrete replacement can also be considered for concrete elements that exhibit partial damage and are likely to continue to degrade due to corrosion advancing toward the reinforcing steel (op. cit. 10).

In a previous KTC report related to concrete removal it was noted that, “Estimating the quantity of concrete that should be removed before executing repairs is a challenging task, especially if plans call for removing only unsound concrete. Errors made during the estimating process can be minimized by thoroughly surveying concrete condition as close as possible to when the work will be executed. Before starting concrete removal on a structural member, analyze the member to determine whether shoring or formwork is required. After the concrete is removed, the remaining section must support its weight, any superimposed dead load, live load (if the bridge is to be repaired under traffic), formwork, equipment, and the weight of plastic concrete. On a flexural member, the final dead load deflection must be compatible with the other members in the unit” (56). Removing concrete can impact the structure and, except for minor repairs, an engineer should review the proposed work.

Patch Repairs

Patch repairs are the most common method of repairing localized concrete damage. Applying patches enables concrete elements to continue functioning properly and can eliminate or minimize spread of distress by corrosion. The following discussion generally pertains to both patch and major concrete repairs.

To perform patch repairs, the perimeter and depth of distress need to be determined. Typically, the surface perimeter of the repair is marked off in rectangular areas with straight lines. That boundary can extend several inches beyond the distressed area. Generally, fewer larger patches save time compared to numerous smaller ones (57). The resulting boundary should be saw cut perpendicular to the concrete surface to a depth of about 1 in. The cut should avoid reinforcing steel. This will eliminate issues associated with feathering the repair material, which should be avoided. The damaged/carbonated/chloride-contaminated concrete should be removed to a minimum distance of 1 to 2 in. from the reinforcing steel. Generally, about 2 in. of non-rusted steel should be exposed (58). Bridge decks or other concrete elements that are completely contaminated with chlorides and/or carbonates should probably undergo full-depth concrete replacement.

All damaged, deteriorated, loosened, or unbonded portions of existing concrete should be removed by hydrodemolition, bush hammering, or jackhammering (Figure 45). Proper removal and preparation can be the most important factor in determining the repair's longevity, irrespective of the material or technique used. Whenever concrete is removed using impact tools, the potential exists for micro-cracking to damage the concrete. To minimize this, SHAs usually specify the use of low-impact tools (e.g., Iowa DOT specifies 30-lb. rated jack hammers and 15-lb. rated chipping hammers). If the remaining concrete substrate is cracked, the replacement material may suffer what appears to be a bond failure (op. cit. 53). To prevent that, the micro-cracked concrete needs to be removed by shotblasting, wet abrasive blasting, or high-pressure water blasting (5,000 to 10,000 psi). Concrete removal should be extended to the saw cut line by jackhammering. In some cases, deteriorated concrete removal will result in the need for a full-depth patch. In those cases, partial forming may be needed to contain the repair material (Figure 46). ACI RAP Bulletin 14, *Concrete Removal by Hydrodemolition*, addresses the use of this method for concrete repairs.

Exposed reinforcing steel must be cleaned to remove all rust, remaining coating, and residual chloride contamination. It is desirable to clean the reinforcing steel to a white metal finish. If necessary, corrosion-deteriorated reinforcing steel should be replaced or supplemented with additional bars wired to the existing reinforcement. ICRI Guideline 310.1R-2008, *Guide for Surface Preparation for the Repair of Deteriorated Concrete Resulting from Reinforcing Steel Corrosion*, along with ACI *TechNotes* 364.6T-02(11), *Concrete Removal in Repairs Involving Corroded Reinforcing Steel*, and 364.7T-02(11), *Evaluation and Minimization of Bruising (Microcracking) in Concrete Repair*, address the proper removal of concrete when making repairs. ICRI Guideline 310.3-2004, *Guide for the Preparation of Concrete Surfaces for Repair Using Hydrodemolition Methods*, addresses the use of hydrodemolition of horizontal, vertical, and overhead surfaces. If a patch involved epoxy coated reinforcing steel, an epoxy coating can be applied to restore the barrier protection along the reinforcing bars. ACI *TechNote* 354.3T-15, *Treatment of Exposed Epoxy-Coated Reinforcement in Repair*, provides guidance for addressing exposed epoxy reinforcement in different repair circumstances.

Prior to patching, the concrete surface should be kept clean and dry unless surface saturation is required by the patch material. The following are properties of repair materials used for patching:

- Properties of the patch material should correspond to those of the existing concrete (especially modulus of expansion)
- The repair material should be as strong as the existing concrete
- The repair material should have low shrinkage, low permeability, and a low w/c ratio
- The repair material should adhere to the existing concrete either by use of a rich cement mix or an epoxy bonding compound
- If the repair mixture uses Portland cement, it should have low shrinkage properties and be properly cured after application.

Common repair materials include latex modified concrete and mortar, epoxy patching compounds, polyester resin, acrylic concrete and mortar, polymer-modified cement-based materials, pozzolanic modified concrete, high-alumina cement compounds, magnesium phosphates, molten sulfur, calcium sulfate-based materials, non-shrink quick setting mortar cement-based polymer concrete, and pneumatically applied mortar/shotcrete (59). ICRI Guideline No. 320.2R-2013, *Guide for Selecting and Specifying Materials for Repair of Concrete Surfaces*, and ACI Guideline 546.3R-14, *Guide for the Selection of Materials for the Repair of Concrete*, address choosing the appropriate concrete material(s) based on owner requirements, type of deterioration, service requirements and application conditions.

The Iowa DOT has separate concrete patch materials and placement requirements for shallow patches (.75 in to 1.5 in. deep) and regular patches (1.5 in. deep minimum) (op. cit. 53). A bonding grout is employed for the shallow patches and a curing compound is used for the regular ones. A KTC-developed specification for repair of damaged concrete recommends the application of an epoxy bonding agent (op. cit. 58). There are proprietary products that can be applied to the concrete and reinforcing steel. Proper application and curing are necessary to achieve durable concrete repairs, including patches.

Application methods for concrete repairs are addressed by ICRI Guideline 320.1-1996, *Guide for Selecting Application Methods for the Repair of Concrete Surfaces*. Application methods include troweling, dry packing, forming, and casting-in-place, both partial and full-depth (ACI RAP Bulletin 4), forming and pumping (ACI Bulletin Rap 5), preplacing aggregate (ACI Bulletin RAP 9), dry mix shotcreting and wet mix shotcreting. Troweling is commonly used for small patch repairs. ACI RAP Bulletin 7, *Spall Repair of Horizontal Concrete Surface*, addresses installing patches on horizontal surfaces. ACI RAP Bulletin 6, *Vertical and Overhead Spall Repair by Hand Application* addresses troweling where repair orientation can potentially pose problems.

Major Repairs

Major repairs, such as broken pier caps or bearing seats on abutments, usually require reforming. Conventional repairs may require the partial removal of a concrete element or at least the concrete cover of a substantial portion (Figure 47). The resulting repair may also incorporate strengthening of the element (Figure 48). Typically, major repairs require the use of formwork to contain the repair material. However, in some cases, shotcreting can be an effective substitute, especially on vertical surfaces. In some cases, additional concrete or mortar cover can be applied to provide additional protection covering both patched and intact concrete.

The Iowa DOT has standard reinforced concrete maintenance procedures for patching bridge decks; patching concrete barrier walls and curbs; repairing tops of abutment backwalls; repairing wingwalls, abutments and piers; and repairing bridge seats under bearings (op. cit. 53).

Shotcreting uses a mixture of Portland cement, sand, and water (sometimes including coarse aggregate and admixtures) which are pneumatically thrown on a repair area. The method can be used for a range of material replacement repairs, from small random patches to large areas (jacket repairs). The thickness of the repair is usually limited to about 6 in. It can be used for both vertical and overhead repairs. Its advantage is that it can be used without the need for forms. There are two types of shotcreting — wet shotcrete, where the pumped material is completely mixed with water, and dry shotcreting, where a dry or damp mixture is used with a spray nozzle equipped with a water ring to properly moisten the mixture as is sprayed. The quality of a shotcrete repair depends a lot on the operator. If properly applied, it can provide durable concrete repairs. Shotcreting is addressed in ACI 506R, *Guide to Shotcrete*; ACI 506.2, *Specification for Shotcrete*; and RAP Bulletin 12, *Concrete Repair by Shotcrete Application*. A similar method (low-pressure spraying) uses a prepackaged mortar applied at a lower pressure than shotcreting. It can be used for vertical and overhead repairs. ACI RAP Bulletin 3, *Spall Repair by Low-Pressure Spraying*, addresses this method.

Halo ring corrosion of concrete patches is a common problem encountered when applying patches, probably more prevalent when several small patches are applied to a bridge element. Concrete patch material typically is more alkaline than the original concrete. When a concrete patch material is placed on exposed reinforcing steel, that steel becomes passivated and acts as a cathode. The portions of the reinforcing steel immediately outside the patch which are less alkaline act as anodes. This creates an electrochemical imbalance and the anode sites immediately outside the patch begin to corrode. Corrosion deposits form in those areas creating a ring or halo of spalling concrete around the intact patch. Typically, this is mistaken for an undersized repair.

There are several means of addressing this phenomenon, including application of chemical treatments, employing lower pH patching materials, and using galvanic anode embedments (discussed below).

5.3.3 Repair of Cracks

Visual inspection and the NDE sonic methods can be used to detect cracks and determine their depth. It is also important to understand their underlying cause, especially for structural cracks.

To determine a crack repair method, the following questions need to be addressed:

- What is the function of the structural element?
- Is the crack active or dormant?
- Is the crack vertical or horizontal?
- Is the crack structural or non-structural?
- How many cracks are present?
- Is the crack isolated or part of a pattern?
- Where is the crack located?
- How deep is the crack?
- What is the crack orientation relative to the bridge (transverse or longitudinal)?
- Is weather resistance required?
- Should the repair be waterproof? (60)

Structural integrity must be provided by any repair where it relates to existing cracks. Selection of crack repair procedures can address restoration of strength or element stiffness, improvement of functional performance and durability, as well as resistance of reinforcing steel corrosion. However, strengthening structural elements may not bear directly on the crack (e.g., adding reinforcement or prestressing the element) (61). Structural crack repair methods include epoxy injection, routing and sealing, drilling and plugging, stitching, doweling, bandaging, gravity filling, post-tensioning, and FRP wrapping (op. cit. 60).

Epoxy Injection

Epoxy injection can be used to seal dormant cracks. It can restore structural strength of cracked concrete. It can be used on crack widths down to 0.002 in. ACI 224 notes the tolerable crack width for exposure to deicing chemicals is 0.007 in., making epoxy injection suitable for sealing those cracks to prevent moisture intrusion and corrosion of reinforcing steel. It can also be used to repair subsurface delaminations detected by sounding or other methods to prevent follow-on spalling. Epoxy injection can be used on cracks of any orientation. ACI Repair Application Procedure (RAP) Bulletin 1, *Structural Crack Repair by Epoxy Injection*, addresses selection and use of epoxy injection for crack repairs in concrete.

The method involves sealing surface-breaking cracks and installing injection/vent ports at intervals along a crack. A KTC-developed specification directs those be placed at 6-in. to 12-in. intervals (62). Epoxy is pumped into the injection port until it is seen coming out of adjacent ports. The injection port is capped and the injection operation is moved to the next port. For vertical cracks, the lowest port is used for injection and the operation moves upward. For exposed repairs or for a follow-on FRP wrap installation, injection ports are ground off, otherwise they can be left in place. The Iowa DOT uses epoxy injection to repair concrete beams damaged by vehicular impact and to repair cracked wingwalls, abutments, and piers (op. cit. 53).

Epoxy injection is addressed by ACI SPEC-548.15-20, *Specification for Crack Repair by Epoxy Injection*.

Routing and Sealing
The most common and simple method of crack repair is routing a surface groove in the concrete along the crack. The width of the groove should be about the same dimension as its depth. If the crack is dormant, it can be sealed with an epoxy. If the crack is active, a flexible sealant such as a polyurethane can be used. In that case, a bond breaker or backer rod should be employed under the sealant to limit sealant adherence to the sides of the groove.

Drilling and Plugging
Vertical cracks that run in relatively straight lines and are accessible at one end may be repaired by drilling into them parallel to the plane of the crack. This method is primarily used to repair retaining walls. This technique is only applicable when cracks run in reasonable straight lines and are accessible at one end. For drilling and plugging, vertical holes, typically 2-3 in. diameter, are drilled down the length of the crack. The holes are subsequently filled with grout to form keys that prevent transverse movements of a wall and prevent or limit any leakage across it

Stitching
Cross-stitching involves drilling a series of holes at a prescribed spacing in the concrete at a 45-degree angle to a crack. Alternatively, vertical holes can be drilled on either side of a crack and connected with slots. Reinforcing bars are inserted in the holes and anchored using either an epoxy or grout (63). Reinforcing bar staples can be placed in the slots to bridge a crack. The staples are then anchored using either an epoxy or grout.

Doweling
Dowel bar retrofits involve placing slots approximately perpendicular to a crack at a prescribed spacing. The slot may be parallel to the surface of the concrete element. Then, smooth or deformed dowels are placed in the slots bridging the crack. The dowels are then anchored using either an epoxy or concrete repair material.

Bandaging
A surface bandage can be applied over a surface crack in concrete to prevent the entry of moisture and aggressive materials. For application, the surface would need to be prepared by brushing to remove all surface debris. The concrete surface will need to be dry. The bonding of the bandage to the concrete will depend on the material used. A proprietary visco-elastic resin sheet was used successfully on KYTC steel bridges steel beam ends and structural gaps in a large arch bridge to protect structural elements from moisture entry. This technology should also work well as bandage over cracked concrete to prevent moisture intrusion.

Gravity Filling
When numerous fine cracks are present on horizontal surfaces such as bridge decks, low viscosity liquid-applied resins called healer-sealers can be applied by broom, squeegee, or low-pressure sprayer over the surface (after surface cleaning). They penetrate by gravity into the hairline cracks and surface pores of concrete to seal against moisture and chloride penetration. If applied on a traffic surface, an abrasive is added for traction prior to the resin setting. Excess material is broomed off before the material solidifies. These materials can be used in small applicators such as squeeze bottles to treat individual cracks or cold joints (Figure 49). ACI RAP Bulletin 2, *Crack Repair by Gravity Feed with Resin*, addresses the selection and use of this method of crack repair. ACI RAP Bulletin 13, *Methacrylate Flood Coat*, addresses the use of methacrylates on horizontal surfaces.

Post-Tensioning
A range of proprietary external post-tensioning systems incorporating steel cables and bars and FRP are available and can be used in conjunction with other repair technologies to address cracking in concrete beams and cantilever piers. Application of this technology requires specialized engineering expertise.

FRP Wrapping
Fractured concrete beams and other structural members can be repaired using FRP fabrics and polymer rod panels. Typically, cracks or other concrete damage will need to be repaired (e.g., epoxy injection, patching). Concrete surfaces receiving the wrap/panel should be mechanically cleaned (e.g., blast cleaned) to create a surface profile with out-of-plane variations less than 1/32 in. (64). The surface should be clean, dry, and free of contaminants. The wrap/panel needs to be cut to fit and the concrete surface primed with an epoxy resin by spatula. The border around

the repair should be primed about 0.5 in. beyond the edges of the FRP wrap/panel. Generally, the wrap/panel will receive a coat of epoxy on the surface to be bonded with the concrete. The wrap/panel is applied to the concrete and a roller is used to press the two epoxy-coated surfaces together. Some panels may be applied directly to the epoxy-coated concrete and receive an additional surface coating of epoxy to bond the panel fibers/rods on both faces of the panel. Typically, the repair is allowed to cure for 24 hours. Thereafter, the wrap/panel can be painted to prevent UV degradation. The Iowa DOT uses wraps to repair concrete beams damaged by vehicle impacts (op. cit. 53). FRP wrapping can be used for a variety of strengthening and safeguarding (e.g., earthquake-proofing) procedures. It can also be used to jacket concrete columns that require repairs due to corrosion damage.

5.3.4 Electrochemical Treatments

Several electrochemical treatments are available to address carbonation and corrosion-threshold-level chloride contamination at the reinforcing steel depth in concrete as indicated by carbonation and chloride testing at depth in the concrete cover. Corrosion can be occurring, as indicated by half-cell corrosion potential and corrosion rate tests, but it is usually applied when there is little or is no visible distress in the concrete cover. Any cracking, delaminations, or spalls should be repaired before the treatment is used. These methods do not require significant concrete removal. They can provide cost-effective corrosion protection over reinforced concrete elements such as girders and piers.

Electrochemical Chloride Extraction (ECE)

This method involves installing a sprayed on or wrapped conductive media on the concrete surface being treated. The conductive media typically consists of cellulose fiber (e.g., ground up newspapers) mixed with water (Figure 50). A conductive metal mesh is placed over the media and a circuit is made between the reinforcing steel and the mesh with a DC power source (Figures 51, 52). The mesh is given a positive charge and the reinforcing steel a negative charge (Figure 53). The negatively charged chloride ions migrate away from the reinforcing steel toward the positively charged mesh. Ions travelling to the surface of the concrete are absorbed into the conductive media (op. cit. 54). The treatment time is from 10 to 50 days. The process removes from 20 to 50 percent of the chloride ions. Remaining chlorides have migrated away from the reinforcing steel; to pose a corrosion threat, they have to migrate back toward the reinforcing steel. The treatment results in restoration of the passivity of the reinforcing steel. Sealing the concrete surface (or painting it with an epoxy coating) after this process to prevent the further ingress of chlorides promotes additional durability of this treatment, which should last 10 or more years (op. cit. 21).

Electrochemical Realkalization (ER)

This treatment is similar to ECE in that it uses a temporarily applied electric field between the surface of the concrete and reinforcing steel. For ER, a conductive mesh is placed on the surface of the concrete and a highly alkaline electrolyte, such as a sodium or potassium carbonate, is used to wet and buffer the structure. The applied charge uses electro-osmosis to pull the electrolyte into the structure, which increases the pH in the cover around the reinforcing steel and restores its passivity. This method provides long-term corrosion protection where carbonation has been a problem. Sealing the concrete surface (or painting it with an acrylic coating) will provide additional protection against the possibility of future carbonation problems.

5.3.5 Cathodic Protection Systems

Cathodic protection systems make reinforcing steel cathodic by providing it with a source of electrons. The reinforced concrete can be subject to carbonation or critical threshold levels of chlorides at the reinforcing steel depth. Half-cell corrosion potential and corrosion rate measurements can indicate active corrosion. Covermeter readings can indicate inadequate concrete cover over the reinforcing steel. The concrete can have some cracking, delaminations, and spalling which need to be repaired prior to application of the cathodic protection system. Some patch-type repairs may be necessary as well. Sonic NDE methods may be used to determine the extent of those repairs. The anode source can either be on the surface of the concrete or embedded in it. There are two methods for supplying electrons by impressed cathodic current or galvanic anodes.

Impressed Cathodic Current (ICCP)

This method uses an external power source to provide DC current to the reinforcing steel. The current flows from permanently installed anodes to the reinforcing steel which acts as a cathode. The DC power source is applied to the anode (positive charge) and the reinforcing steel (negative charge), with the concrete acting as the electrolyte (Figure 54). The constant operation of the DC power source provides the reinforcing steel with a minimum level of cathodic protection necessary to eliminate or minimize corrosion. Specific current and voltage settings must be established to achieve the desired effect (op. cit. 10). During the system's life, it needs periodic monitoring and maintenance to assure it operates properly. Problems are experienced with ICCP systems that are not maintained. However, they are effective in preventing corrosion in even high chloride contaminated concrete if properly maintained (65). Potential hydrogen embrittlement problems may occur when ICCP is used with prestressed or post-tensioned wires (op. cit.21).

Galvanic Anode Cathodic Protection (GACP) Systems

These systems employ a sacrificial anode that has a higher (more negative) corrosion potential than reinforcing steel. When the anode is connected to the reinforcing steel and the concrete, the anode corrodes preferentially to the reinforcing steel. The steel acts as the cathode and is protected by the current generated by the corroding anode. Zinc or zinc-aluminum alloys are generally used as anodes. As the anodes corrode, they have a finite lifespan (10-30 years). They can be eventually replaced to provide continued protection to reinforcing steel. There are two types of galvanic anode systems — embedded anodes and surface-applied anodes.

Embedded galvanic anodes (EGAs) are the most common type. There are two types of EGAs. Discrete EGAs are electrically tied to reinforcing steel in an area to be repaired (Figure 55). They are available in a range of shapes and sizes to address various corrosion, steel density and uses (66). Tables are available for anode sizes and spacings (67). Type 1 discrete anodes are attached to reinforcing steel between the existing and patch concrete to prevent halo ring corrosion. Type 2 discrete anodes are placed in holes drilled into concrete to suppress corrosion locations identified by half-cell corrosion potential or corrosion rate surveys, chloride-depth tests, or covermeter measurements. They are connected to reinforcing steel either individually or by a header wire that can connect multiple anodes. Distributed EGAs are linear anodes, up to about 6 ft. in length, used for jacketing, joint, or overlay repairs (Figure 56). Both types of anodes are usually proprietary products and contain special coverings to activate the zinc and provide proper function. When properly applied, these systems should provide 5-15 years of protection. After they are expended, they can be replaced to give additional years of service. ACI RAP Bulletin 8, *Installation of Embedded Galvanic Anodes*, provides guidance for their installation.

Surface-applied galvanic anodes come in two types. With the first type, metallized anodes use zinc or zinc/aluminum molten sprayed on concrete surfaces at locations to receive galvanic protection. The spray forms a thin metallic film that is electrically connected to the reinforcing steel (Figure 57). A humectant activator solution is applied to attract moisture and provide sufficient current output and protection. The second type is preformed zinc sheets/tape attached to the concrete using a conductive adhesive. Surface-applied galvanic anodes are generally used to mitigate corrosion in larger non-traffic bearing areas (op. cit. 65). No estimates of service life for these systems were found, but they should be similar to embedded anodes and are probably easier to replace.

Galvanic protection systems are a low-maintenance, renewable means of providing corrosion protection and are generally compatible with prestressed and post-tensioned steel. They are an attractive choice for providing corrosion protection for many applications (op. cit. 54).

5.3.6 Special Case of Deck Joint Seals

Leaking or missing joint seals account for a significant amount of corrosion damage to beams and substructure elements on KYTC bridges. Leaking joints can be dealt with by replacement, either due to their loss of function or by scheduled periodic replacement. At least one SHA replaces joint seals on a periodic basis. If the overall joint condition requires a total rebuild, consideration should be given to eliminating the joint or replacing it with a better-performing type (or supplementing the replacement joint with a flexible trough). Troughs can be used with closed joints and prevent subsequent leakage from damaging the underlying bridge elements. Joint seals typically leak before their condition worsens to the point that replacement is apparent. Addressing joints can be part of the repair assessment process, or not, if scheduled replacement is employed.

5.3.7 Follow-Up Condition Tracking of Maintenance Actions

All maintenance actions on a bridge should be documented, including all treatments and repairs (with pictures). This will allow successive biennial inspections to evaluate them and compare how they are performing with the balance of the structural elements. The resulting service performance of these maintenance actions will help in planning for similar future work on this and other structures and predict the service life performance of specific actions.

Chapter 6 Summary/Conclusions

6.1 Summary

Reinforced concrete constitutes the majority of structural elements on KYTC bridges (probably well in excess of 90 percent). Past performance has shown that concrete bridge elements will pose a majority of the bridge maintenance issues facing KYTC moving forward. It is likely that most of the deterioration of reinforced concrete bridge elements is related to the corrosion of reinforcing steel due either to carbonation, chloride ingress, or a combination thereof.

Chlorides appear to be the most common cause of reinforcing steel corrosion. Several years ago, KTC researchers determined that the average age of KYTC bridges was about 50 years. This provides sufficient time for carbonation to become a factor where conditions favor it. It also provides ample time for chloride anti-icing and deicing chemicals used by KYTC to reach high levels in the surface of bridge decks. Issues with drains and joints result in other bridge elements (beams, piers, and abutments) not directly treated with chlorides being exposed to sufficient amounts of chlorides to cause corrosion problems. Chloride-contaminated aerosols kicked up by traffic in wet conditions provide a means of transferring chlorides to retaining walls, plinths, and concrete substructure elements adjacent to roadways and fascia girders of overpass structures. If 90+ percent of all concrete deterioration problems are related to reinforcing steel corrosion, roughly 80+ percent of bridge maintenance issues facing KYTC are due to this issue. Therefore, it deserves to be a (if not the) primary focus issue for KYTC bridge maintenance.

Carbonation and chloride penetration into concrete cover over reinforcing steel, their presence at the depth of the reinforcing steel, the initiation of corrosion of reinforcing steel, the subsequent loss-of-section of reinforcing steel due to corrosion, and the fracture of concrete due to expansive reinforcing steel corrosion products are the factors that need to be understood, evaluated, and addressed. Other factors, including design, material, construction-related factors, curing, environmental issues, settlement and service loads, can also trigger concrete cracking and facilitate corrosion-induced concrete deterioration. Some steps in evaluating those and resolving them do fall out of those needed to address corrosion, but the repair process provides an opportunity to alleviate existing concrete problems and to enhance the resistance of the intact reinforced concrete to future deterioration. Addressing both concrete distress and reinforcing steel corrosion gives KYTC the ability to address most current maintenance issues and those anticipated in the future. The need for this focus extends beyond bridge decks that are a major wear item and require continual repairs by overlays and laminates and periodic deck replacements.

This report reviewed many of the issues related to the assessment, decision making and repair of damaged reinforced concrete incorporating the use of specific NDE methods along with field and laboratory tests, and evaluation methods. It focused on KYTC needs and capabilities. The range of issues in this report have been addressed in-depth by technical and standards associations (e.g., ACI, ICRI, NACE International, ASTM International and AASHTO). The ACI two-volume *Concrete Repair Manual* (Fourth Ed.) provides an exhaustive set of definitions and guides along with references to specific standards used for testing and interpreting results.

KYTC officials can use information in this report to determine what methods to consider for specific evaluations and to guide decision making. The NDE and test methods and tests discussed in this report have distinct capabilities for addressing evaluations of reinforced concrete and providing data to support KYTC decisions for specific follow-on maintenance actions. They range from simple testing and evaluation methods (e.g., sounding) to complex techniques (e.g., impact echo) that require expensive equipment and knowledgeable operators. Some methods (e.g., covermeters, carbonation indications/field chloride tests) should be available in every district. Other NDE methods and tests such as GPR or corrosion rate can be contracted out to test firms/organizations. KYTC has the expertise and probably the equipment necessary to support any laboratory testing or analysis of concrete. KTC's survey of SHAs and KYTC district indicated the limited breadth of NDE tests used, typically related to low-cost, low-training methods. For the most part, the two surveys did not reveal that KYTC is far behind most SHAs in access to or use of NDE for evaluating reinforced concrete.

Biennial bridge inspections provide a good opportunity to conduct inspections to assess the condition of reinforced concrete. In some cases, inspections can be supplemented by simple NDE tests such as covermeter, sounding, and corrosion potential and field/laboratory testing for carbonation or chloride penetration. Those tests can be done where there are no/minimal visible indications of concrete distress. They can indicate the opportunity to employ PM treatments such as sealants and gain maximum benefit from their use. They can also locate corrosion hot spots where corrosion initiation is occurring so that suitable repairs can be made. Routine maintenance and PM activities can be used to extend the service lives of reinforced concrete bridge elements in good condition and minimize the need for future repairs.

Where concrete distress is occurring and repairs are imminent, more in-depth inspections can be performed using a wider range of NDE methods and supporting tests. Those tests can be employed on a survey basis to provide KYTC officials with a comprehensive understanding of the condition of the concrete bridge elements that cannot be derived from visual inspection and simple tests. A range of tests should be employed to identify which bridge elements warrant repairs and which can receive protective treatments. Some reinforced concrete elements requiring only partial repairs may be given treatments to preclude or minimize corrosion damage. In addition to PM and repairs, assessment decisions include partial or complete bridge replacement and the do-nothing option. The latter can be used for reinforced concrete bridge elements in good condition and elements in poor condition which eventually will require major repairs or replacement.

Three types of maintenance actions are performed on reinforced concrete: 1) applications of protective treatments (e.g., barriers) in the form of liquid-applied concrete sealers and coatings; 2) patch or other major repairs entailing concrete removal and replacement, possibly coupled with reinforcing steel replacement/augmentation, crack sealing, and wrapping; and 3) electrochemical methods, including electrochemical chloride extraction, realkalization, impressed current protection and galvanic protection. KYTC uses sealing on bridge decks, but can extend it to other elements including beams, piers, abutments, and retaining walls. KYTC has used concrete coatings on projects and should extend this practice moving forward as part of general concrete repair and steel bridge painting projects. KYTC has performed several notable concrete bridge repair projects, including I-65 in Louisville. Currently, KYTC would probably benefit by the use of galvanic methods for electrochemical protection against reinforcing steel corrosion. These methods are relatively low cost with little or no maintenance, which should warrant their consideration.

6.2 Conclusions

1. The available NDE methods and tests presented in this report can be used to assess most reinforced concrete conditions.
2. The methods and tests can provide data to help KYTC personnel make good choices when selecting and scoping maintenance work. An example of this is KTC's use of NDE and chloride testing on the I-65 JFK Bridge southbound offramp piers. This work generated information that supported a KYTC decision to retain the piers.
3. KYTC should consider more widespread use of NDE and related tests prior to initiating repairs.
4. KYTC should consider routine and PM programs to protect reinforced concrete bridge elements from corrosion.

Chapter 7 Recommendations

1. KYTC officials should develop a plan for protecting reinforced/prestressed concrete elements on bridge members. The plan would address standardized repair procedures for reinforced concrete, routine maintenance tasks, and maintenance schedules for each bridge. It would include PM tasks with the requirement that PM actions should be made part of any concrete repair work or steel bridge painting project. As part of the plan, new bridges would be prioritized to receive concrete protective treatments on all reinforced concrete bridge elements within two years of construction.

2. KYTC officials should review the NDE and test methods discussed in this report. At a minimum, they should select methods for measuring carbonation and chloride penetration, measuring concrete cover (covermeters), detecting delaminations, and determine corrosion activity (corrosion potential). They should provide KYTC district personnel with equipment and training in the use of these devices so the agency has the in-house competency needed to evaluate reinforced concrete bridge elements other than decks. KYTC officials should establish standard methods for testing reinforced concrete and develop standard procedures for performing survey tests. The agency will also benefit from developing requirements for when these methods should be employed during routine biennial inspections based on previous biennial inspection findings.

3. KYTC should develop training for district personnel on selecting NDE methods/tests identified for potential use.

4. KYTC should initiate a pilot program in one district to address routine and PM focused on reinforced concrete bridge elements. The district would receive necessary instructions and training related to recommendations 1-3 and necessary equipment as part of the pilot program. It would carry out the program for a period of one year. KYTC officials could review the results and modify the program as necessary. Once desired results were achieved in the pilot program, it could gradually be expanded to other districts.

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Tables

Table 1 Classification of intrinsic cracks – See Figure 16. (Ref. 11; Table 3.1)

1 hour 1 day 1 week 1 month 1 year 50 years

Time from placing of concrete

Type of cracking	Position on Fig. 3.3	Subdivision	Most common location	Primary cause (excluding restraint)	Secondary causes/ factors	Remedy (assuming basic redesign is impossible). In all cases reduce restraint		Time of appearance
Plastic settlement	A	Over reinforcement	Deep sections	Excess bleeding	Rapid early drying conditions	Reduce bleeding (air entrainment) or revibrate		10 minutes to 3 hours
	B	Arching	Top of columns					
	C	Change of depth	Trough and waffle slabs					
Plastic shrinkage	D	Diagonal	Roads and slabs	Rapid early drying	Low rate of bleeding	Improve early curing		30 minutes to 6 hours
	E	Random	Reinforced concrete slabs					
	F	Over reinforcement	Reinforced concrete slabs	Rapid early drying, steel near surface				
Early thermal contraction	G	External restraint	Thick walls	Excess heat generation	Rapid cooling	Reduce heat and/or insulate		1 day to 2–3 weeks
	H	Internal restraint	Thick slabs	Excess temperature gradients				
Long-term drying shrinkage	I		Thin slabs (and walls)	Inefficient joints	Excess shrinkage, inefficient curing	Reduce water content, improve curing		Several weeks or months
Crazing	J	Against formwork	'Fair-faced' concrete	Impermeable formwork	Rich mixes	Improve curing and finishing		1–7 days, sometimes much later
	K	Floated concrete	Slabs	Over-trowelling	Poor curing			
Corrosion of reinforcement	L	Natural	Columns and beams	Lack of cover	Poor quality concrete	Eliminate causes listed		More than 2 years
	M	Calcium chloride	Precast concrete	Excess calcium chloride				
Alkali-aggregate reaction	N		(Damp locations)	Reactive aggregate plus high-alkali cement		Eliminate causes listed		More than 5 years

Table 2 NDE methods and tests for evaluating reinforced concrete on bridges (Ref. 13; Table 4.1).

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

Table 3 Typical corrosion rates from linear polarization tests (Ref. 42; Table 1)

Corrosion Rate	R_{ct} ($\Omega \cdot cm^2$)	i_{corr} ($\mu A \cdot cm^{-2}$)
High	0.25-2.5	10-100
Medium	2.5-25	1-10
Low	25-250	0.1-1
Passive Steel	Above 250	Below 0.1

Table 4 Chloride permeability based on charge passed for AASHTO T 277 test (46)

Charge Passed (Coulombs)	Chloride Permeability	Characteristic of Typical Concretes
> 4,000	High	High W/C ratio (>0.60) conventional PCC
2,000 – 4,000	Moderate	Moderate W/C ratio (0.40-0.50) conventional PCC
1,000 – 2,000	Low	Low W/C ratio (<0.40) conventional PFF
100 – 1,000	Very Low	Latex-modified or internally sealed concrete
<100	Negligible	Polymer-impregnated concrete, polymer concrete

Figures

Figure 1 Tank truck applying liquid deicing chemicals to roadway as a pre-treatment

Figure 2 Spalled bridge deck concrete that has been patched

Figure 3 Deck joint failure with concrete spalling on the abutment backwall. note detached joint seal (arrow)

Figure 4 Spalling concrete on prestressed beam end under joint showing corroded reinforcing steel

Figure 5 Spalling concrete on pier cap with corroded reinforcing steel

Figure 6 Concrete spalling on pier column with corroded reinforcing steel on I-65 approach span

Figure 7 Ground penetrating radar being used to inspect for delaminations on I-65 pier columns (2013)

Figure 8 Extracting powder samples from column on I-65 to perform laboratory chloride tests (2013)

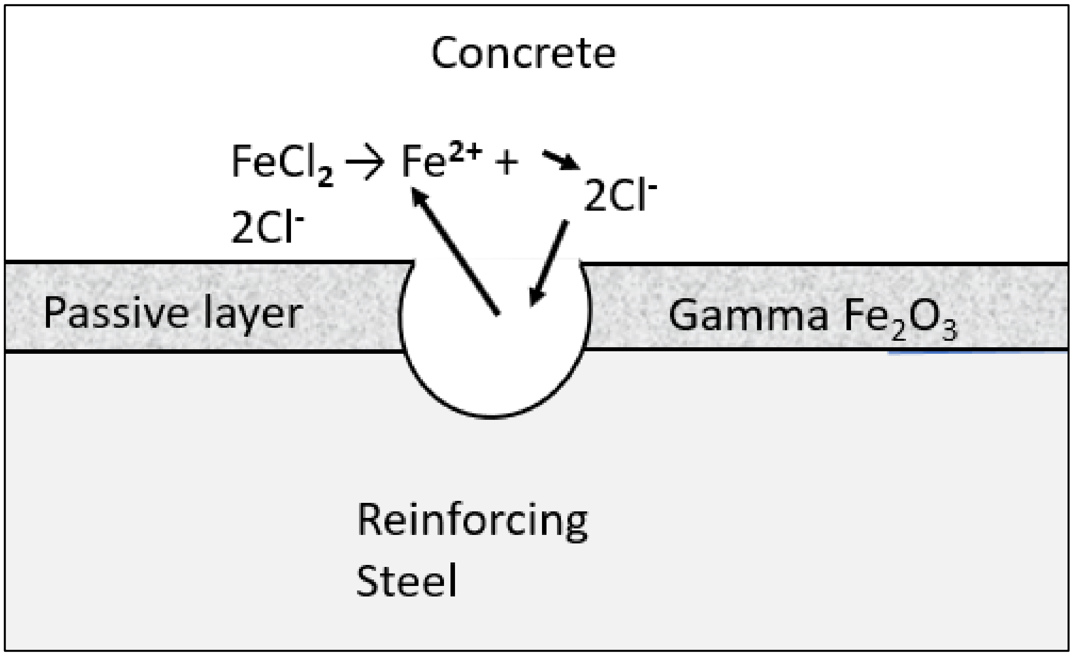


Figure 9 Initiation of micro corrosion by breakdown of passive layer (13)

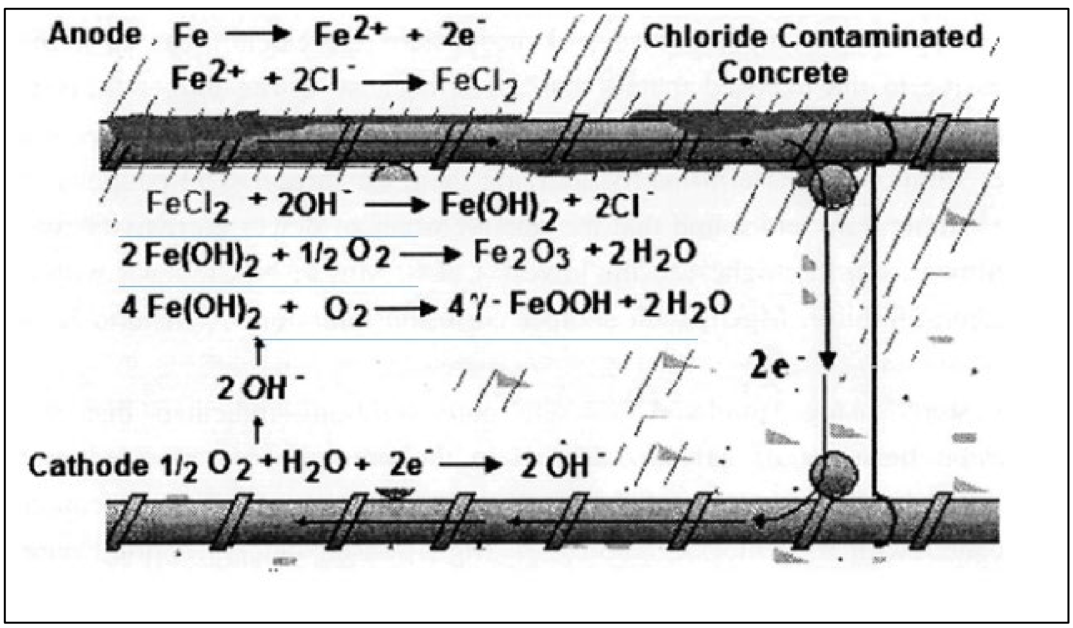


Figure 10 A typical macro corrosion cell in salt-contaminated concrete (14)

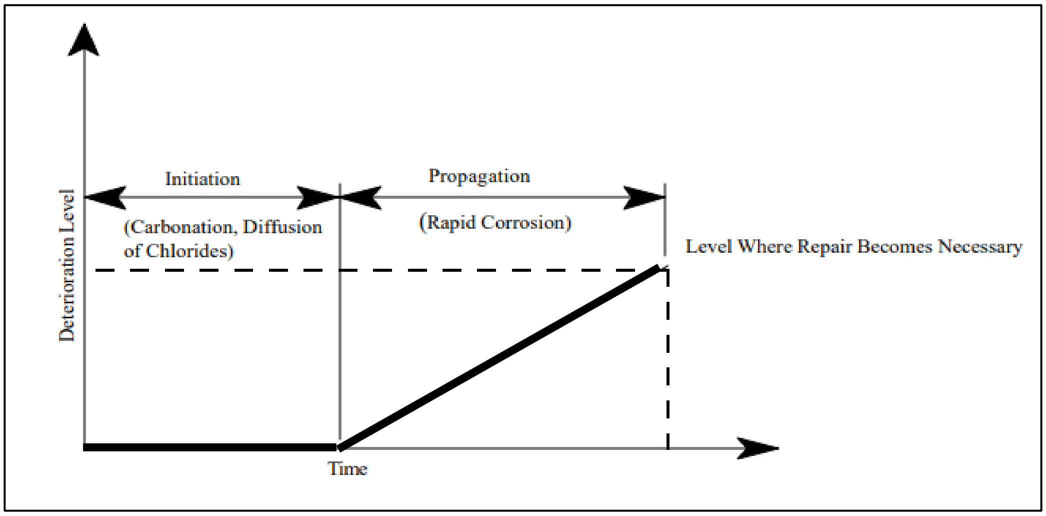


Figure 11 Simple deterioration model showing corrosion of steel in concrete (16)

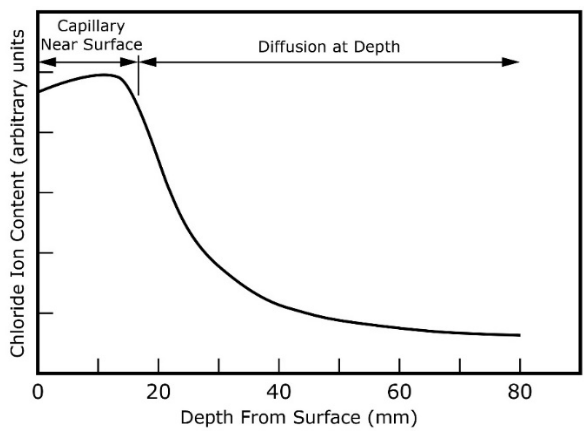


Figure 12 Chloride transport zones (op. cit. 15)

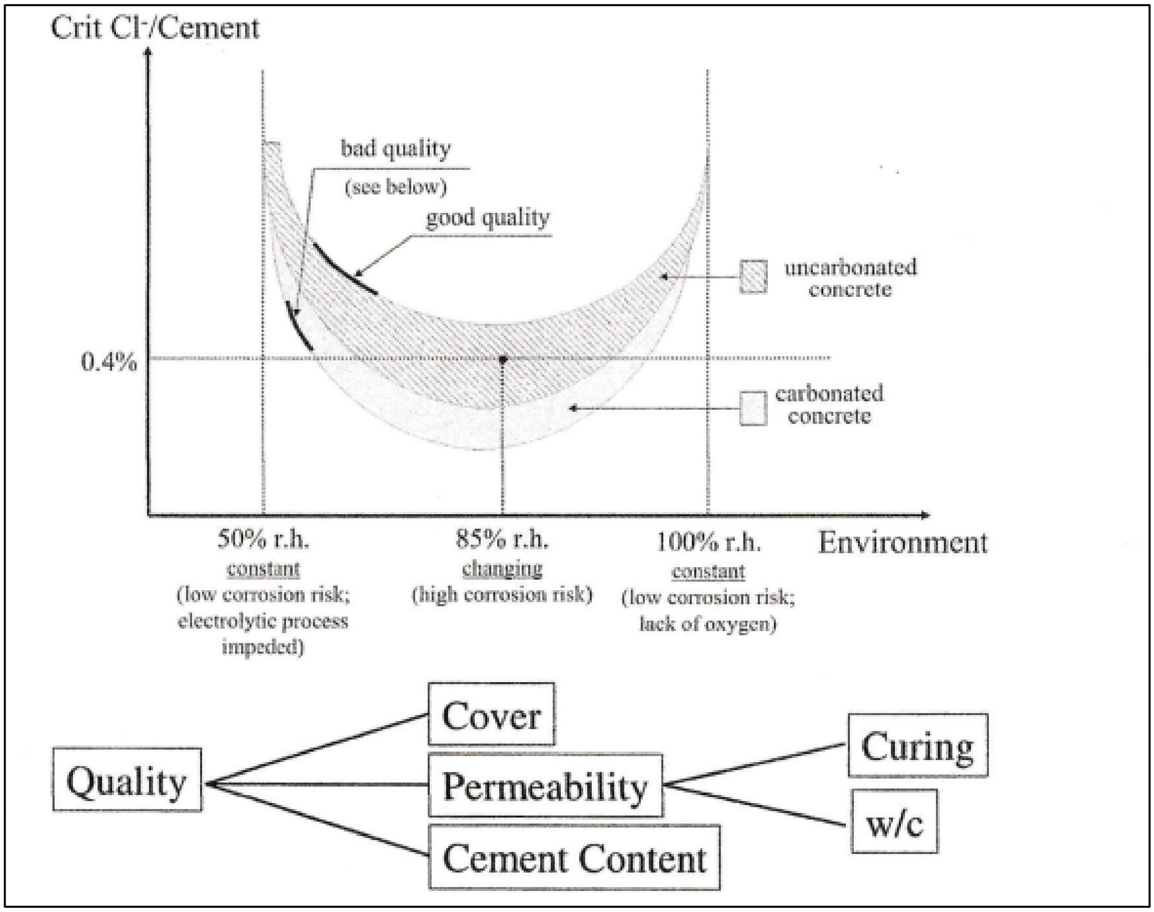


Figure 13 The critical chloride content according to CEB recommendations (op. cit. 11, 21)

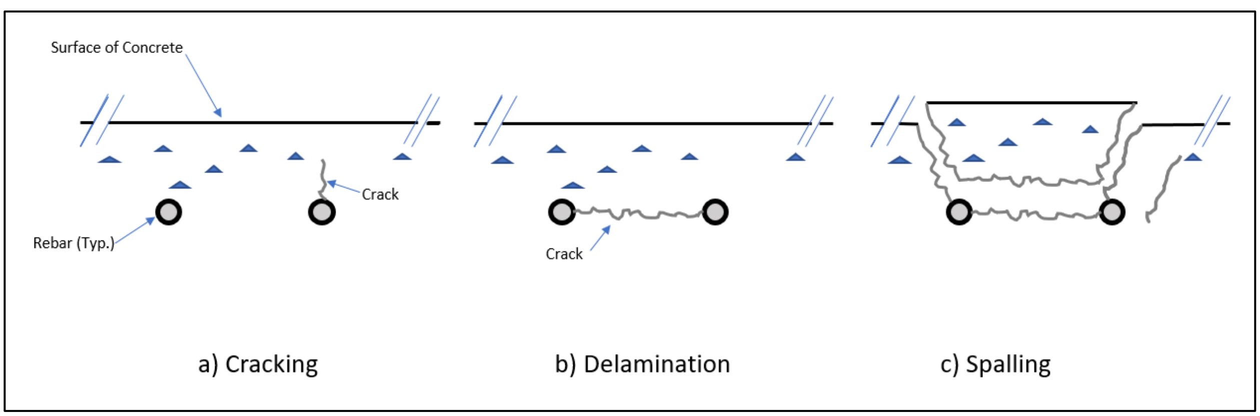


Figure 14 Progression of distress in reinforced concrete due to corrosion of reinforcing steel

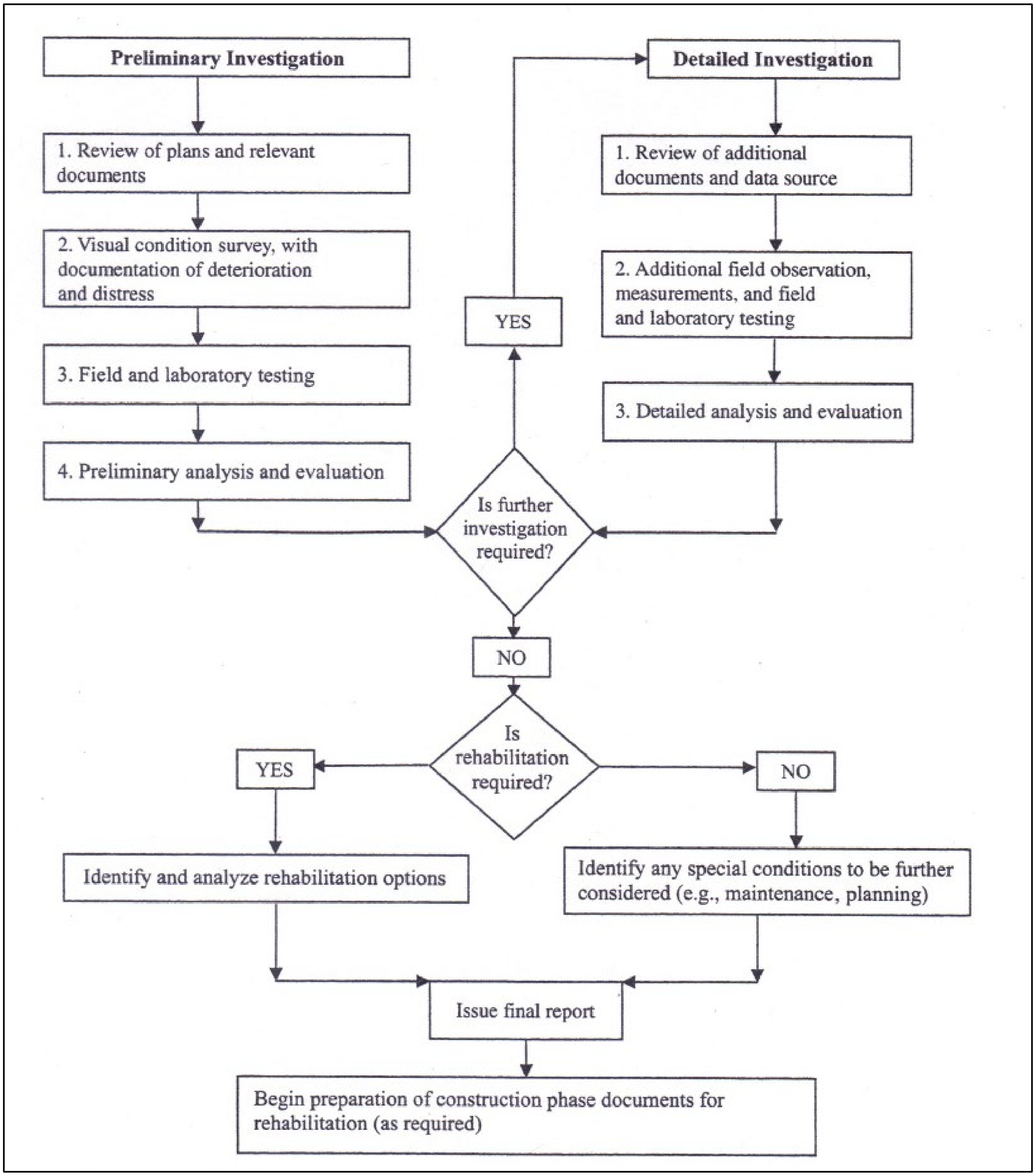


Figure 15 Decision-making process for structural concrete maintenance (23)

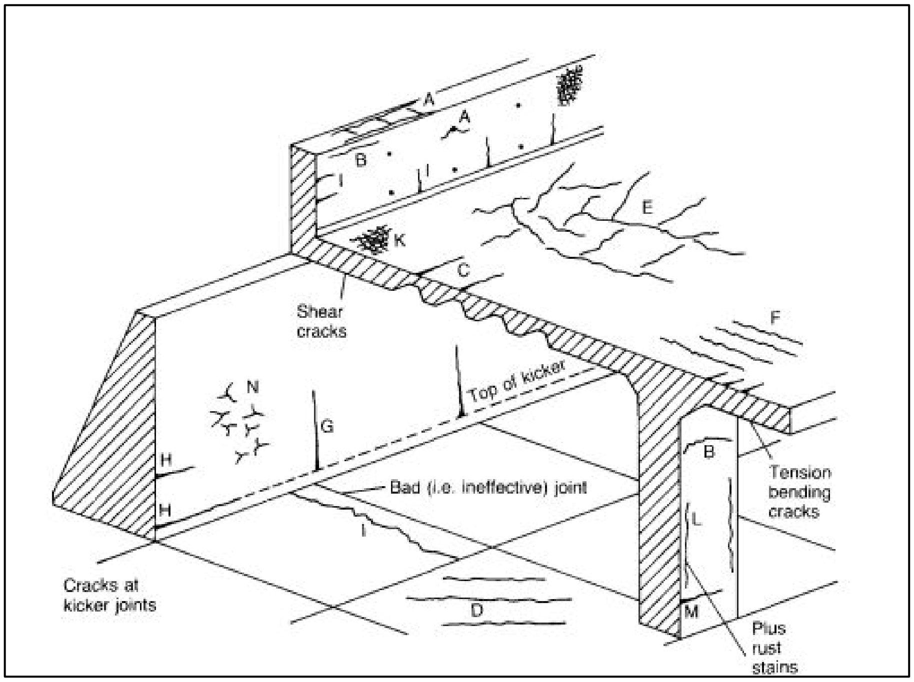


Figure 16 Examples of intrinsic crack patterns in hypothetical concrete – See Table 1 (op. cit. 11)

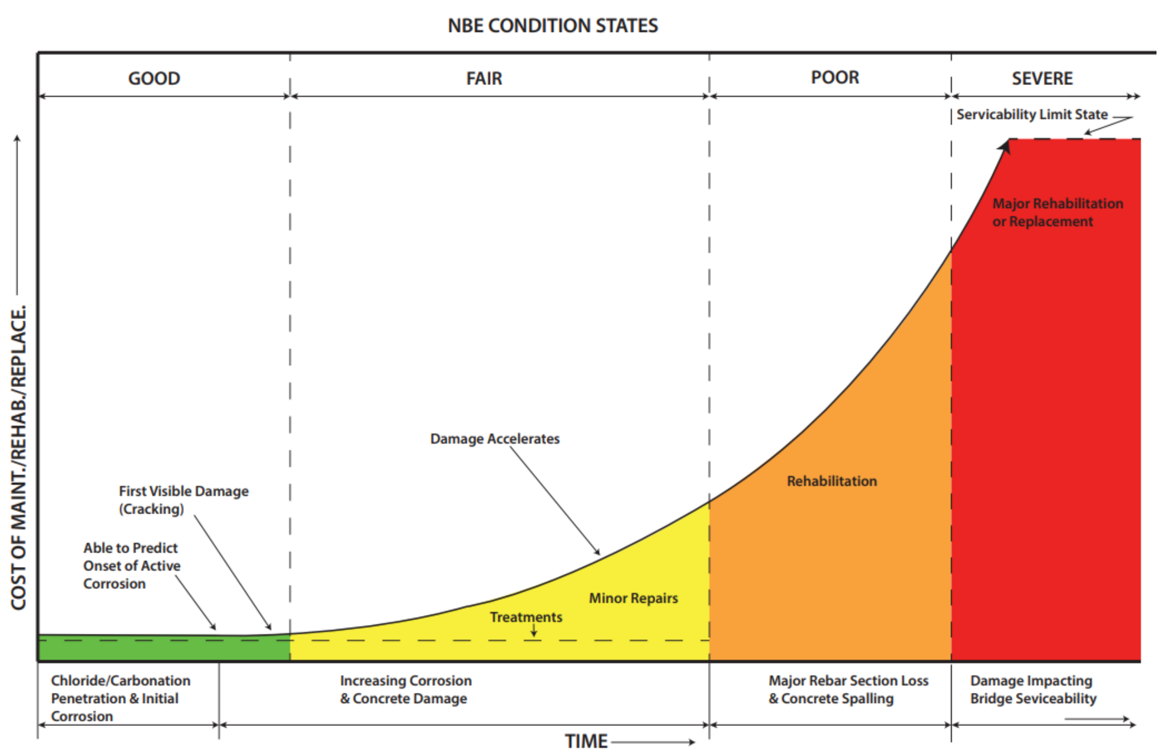


Figure 17 Idealized Cost of Bridge Actions versus Time/NBE Condition States

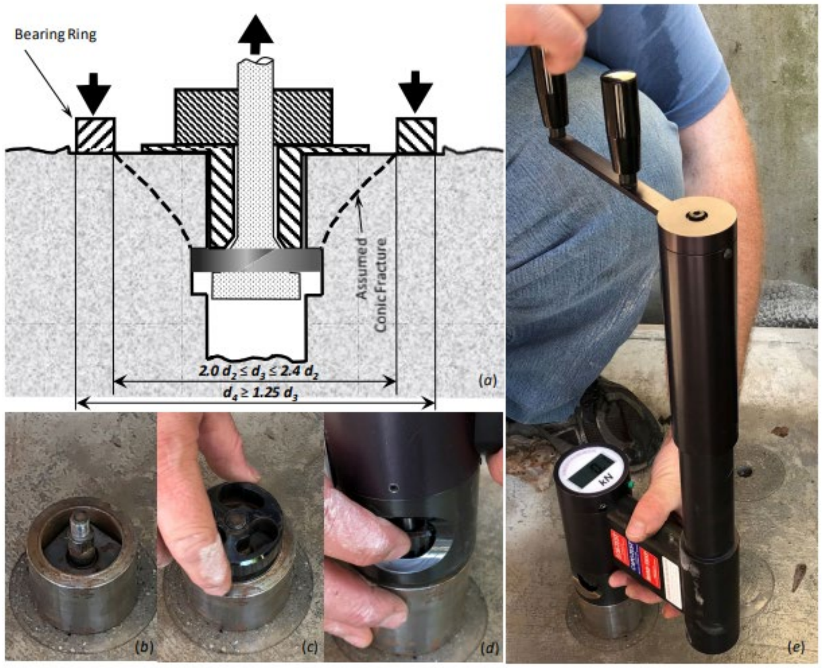


Figure 18 After fully expanding the expandable ring in the routed recess the counter pressure is installed on the surface and the coupling threaded to the center pull bolt. The hydraulic lateral pullout instrument is coupled to the coupling and the slack removed.

Figure 19 Measuring concrete strength using the lateral pullout test

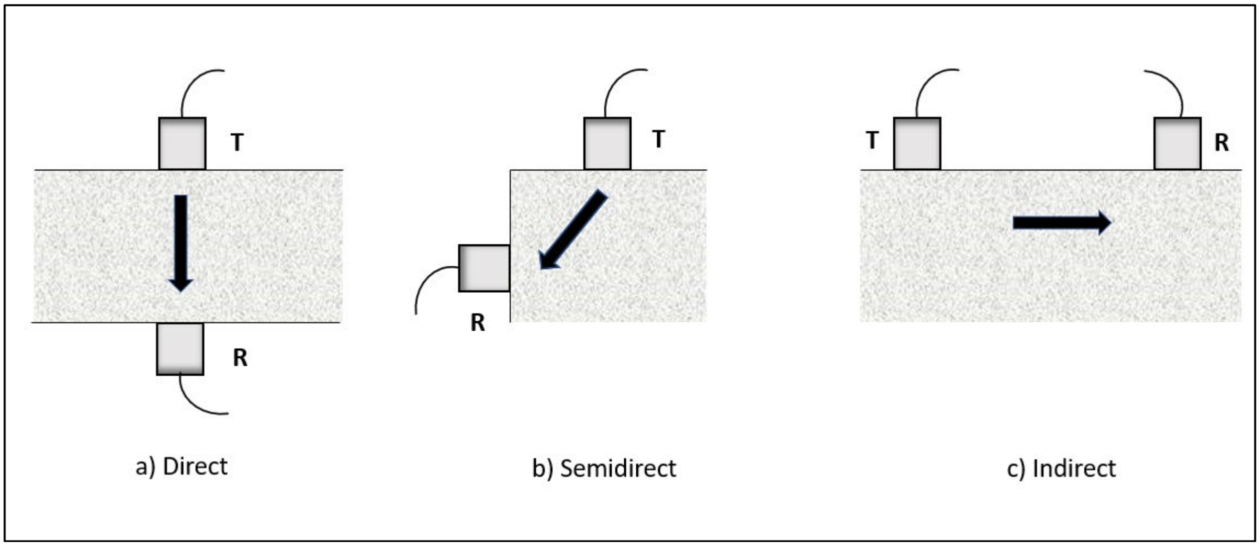


Figure 20 Pulse velocity measurement configurations. a) Direct method, b) Semidirect method, and c) Indirect method

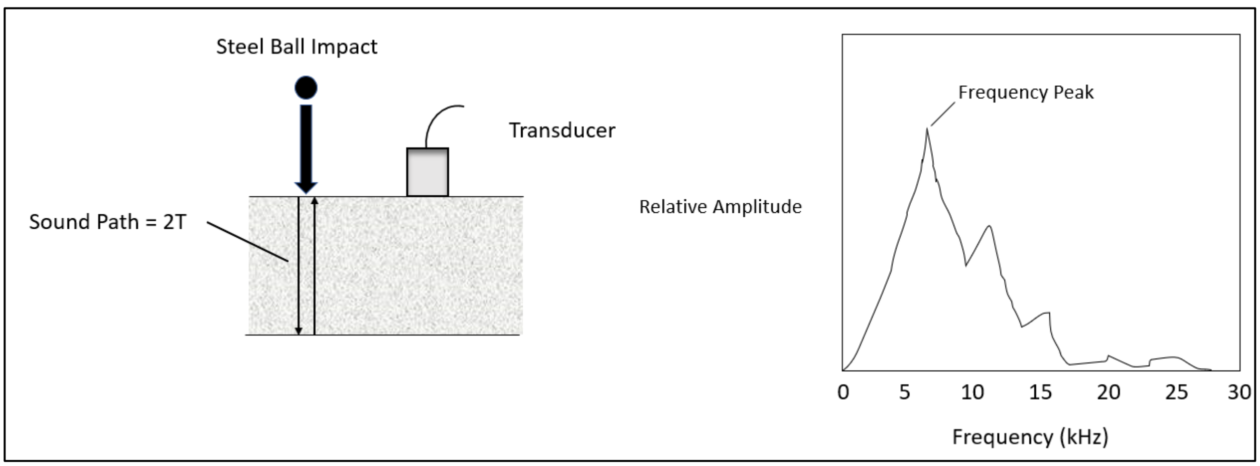


Figure 21 Amplitude spectrum from impact-echo test to detect thickness of concrete

Figure 22 Testing of a concrete column using the impulse response method. The hammer operator is holding the transducer on the column with his left hand while hitting it with the instrumented hammer held in his right hand.

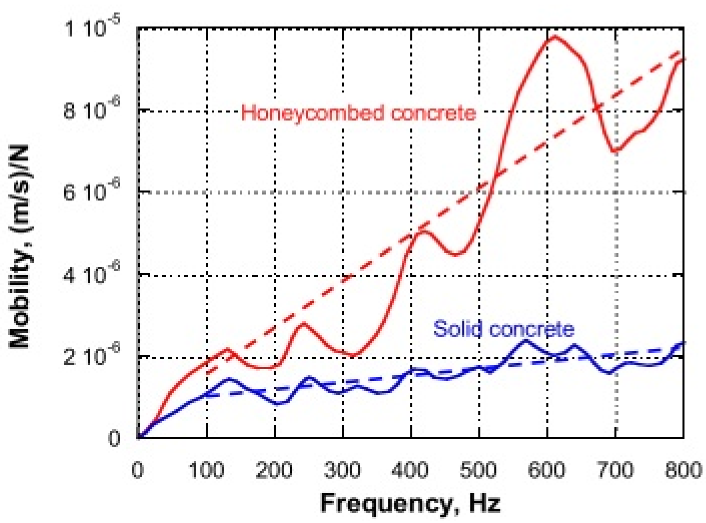


Figure 23 Mobility vs. Frequency graph comparing responses from solid and defective concrete (35).

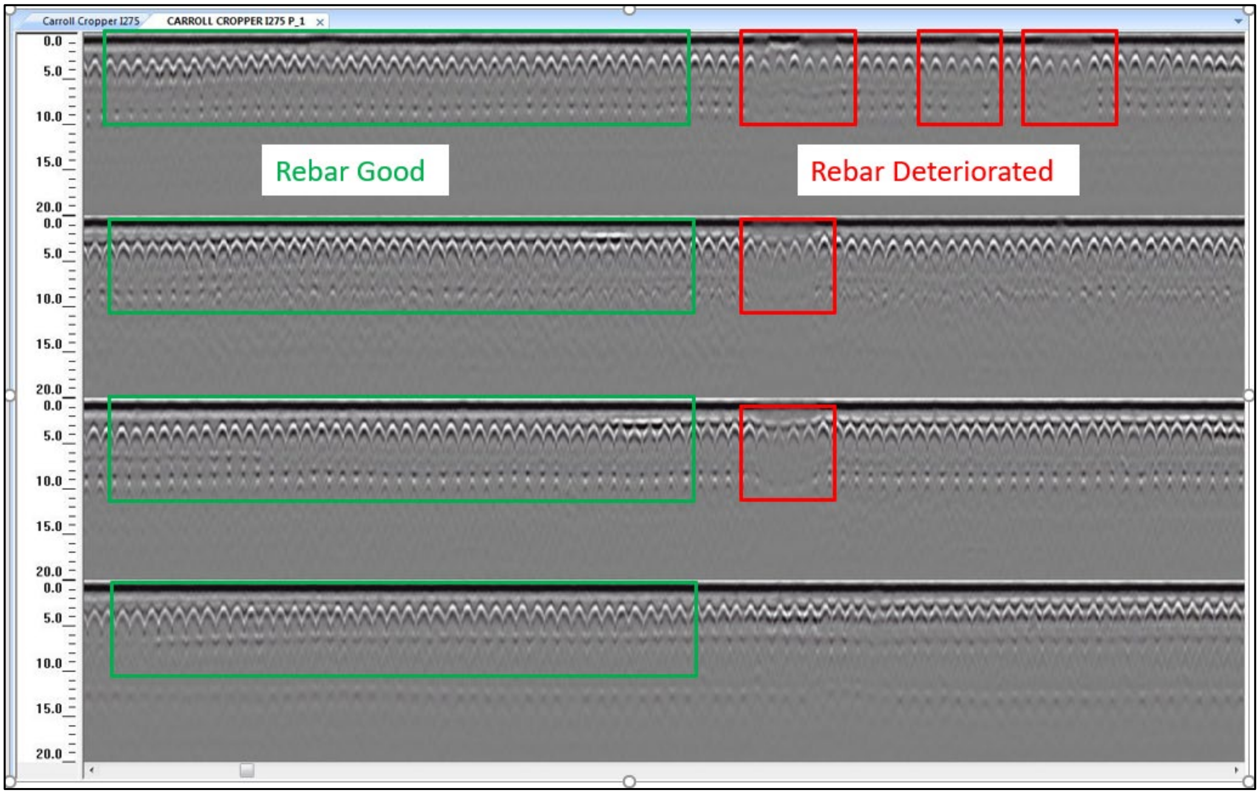


Figure 24 Screen shot of GPR test showing indications of good and corroded reinforcing steel in a bridge deck

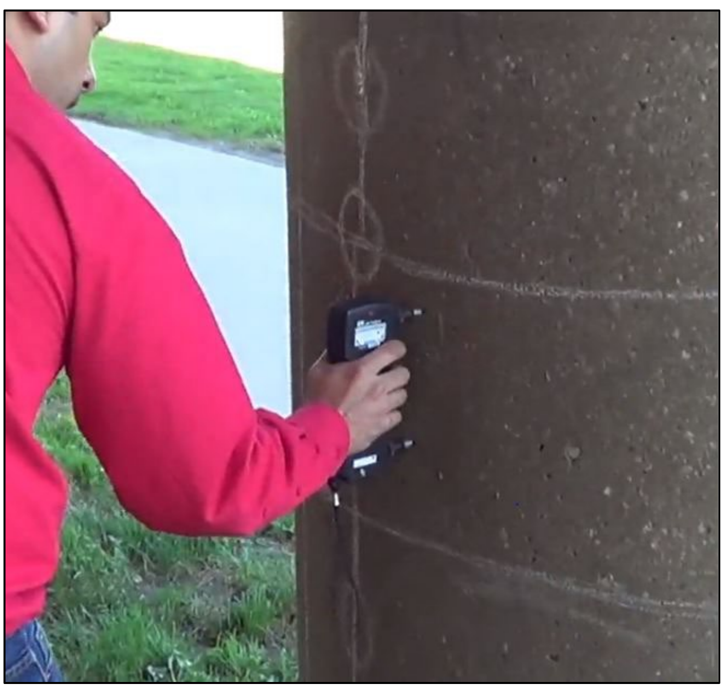
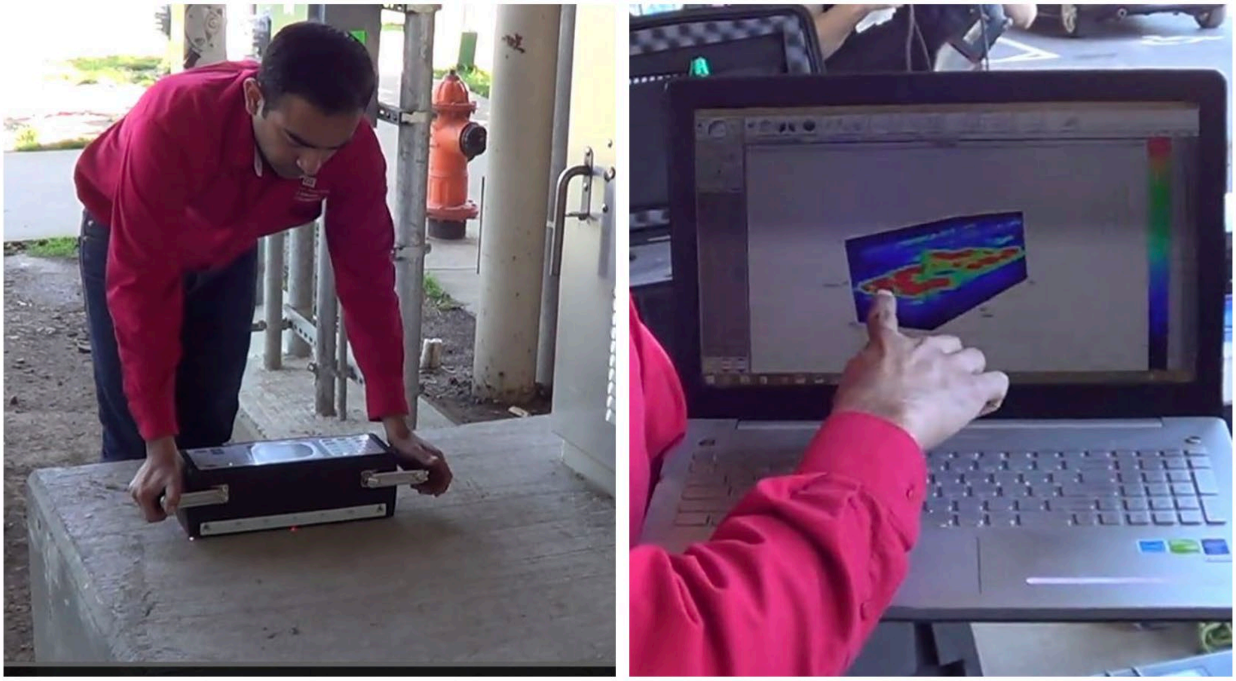


Figure 25 Measuring crack depth using the pulse velocity (time-of-flight) indirect method



(a) (b)

Figure 26 (a) Performing test on concrete slab with a 3D tomography device. (b) Resulting 3D display of internal defect in slab.

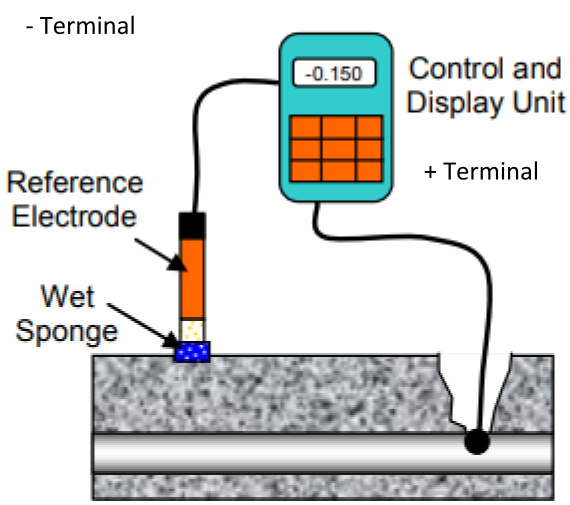


Figure 27 Diagram of the half-cell corrosion potential test (36).

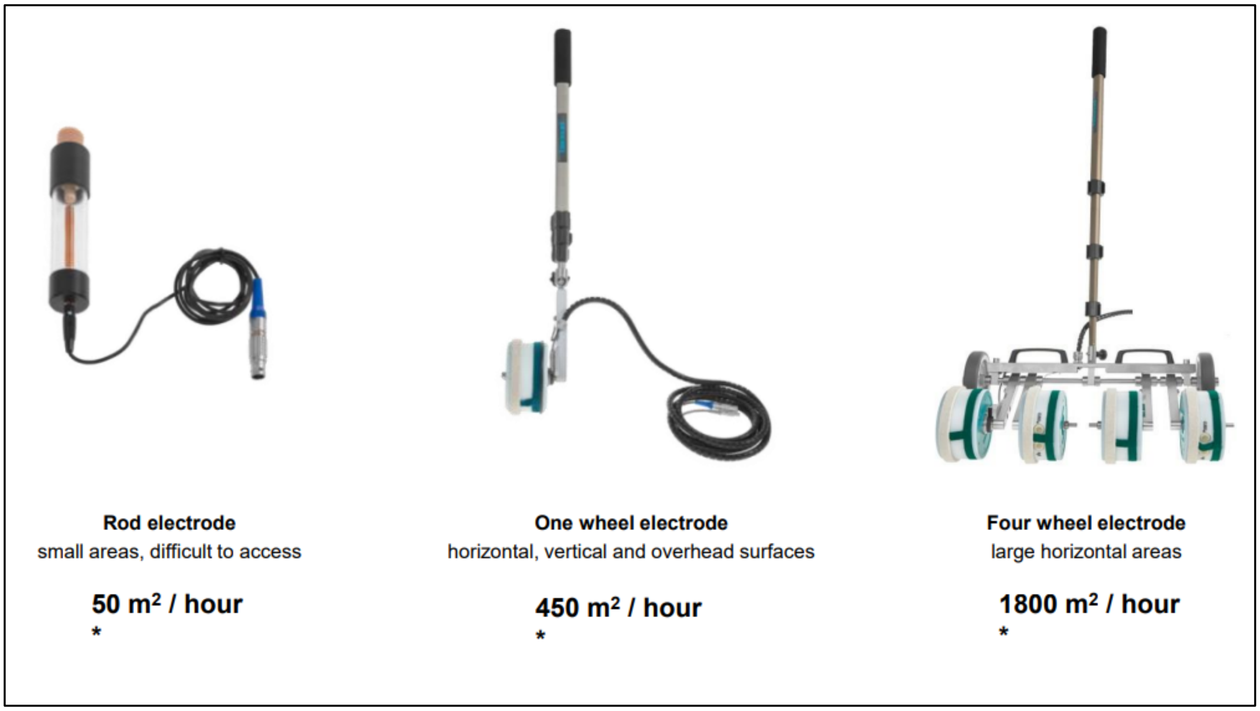


Figure 28 Various probes for performing half-cell corrosion potential tests. Note various test rates given in m^2 / hour (42)

Figure 29 Galvastatic pulse tester used for corrosion current, half-cell corrosion potential and concrete resistivity. The operator is applying the probe to the surface of the concrete with his right hand and reading the instrument held in his left hand.

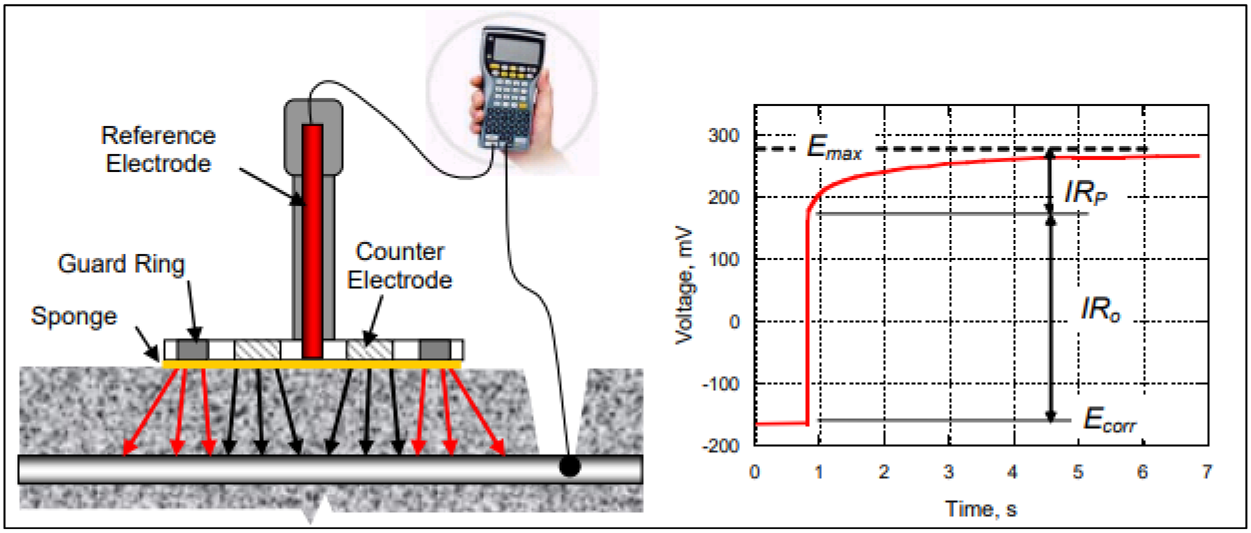


Figure 30 The galvastatic pulse method to measure corrosion current for unit shown in use in Figure 10 (op. cit. 36)

Figure 31 Detecting reinforcing steel and measuring cover depth using a covermeter. The inspector (left) has placed the probe is placed on the column and the measuring unit is being held by the person on the right.

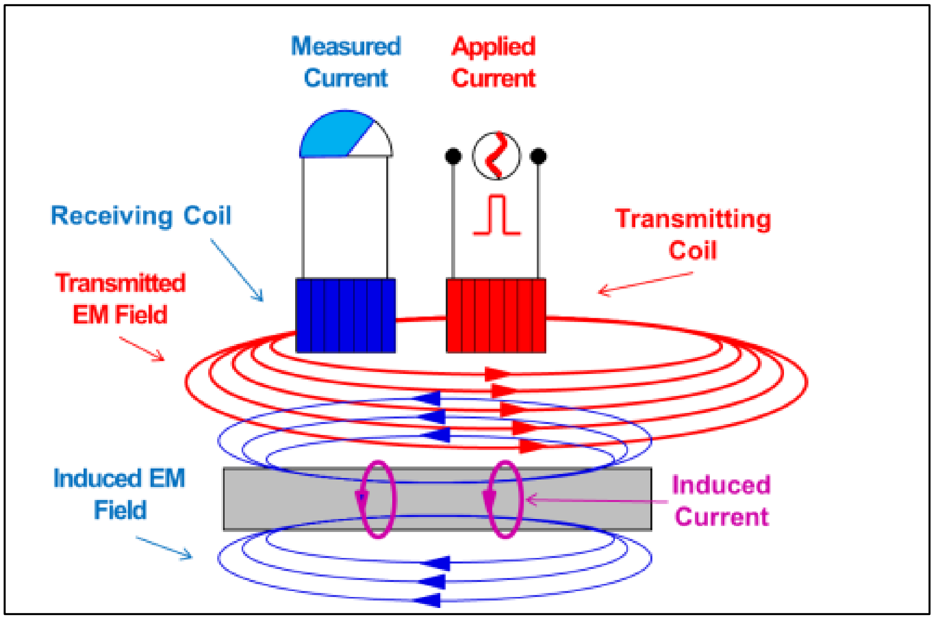


Figure 32 Pulsed induction method used by covermeters (44)

Figure 33 Measuring concrete resistivity using a Wenner four-point probe instrument

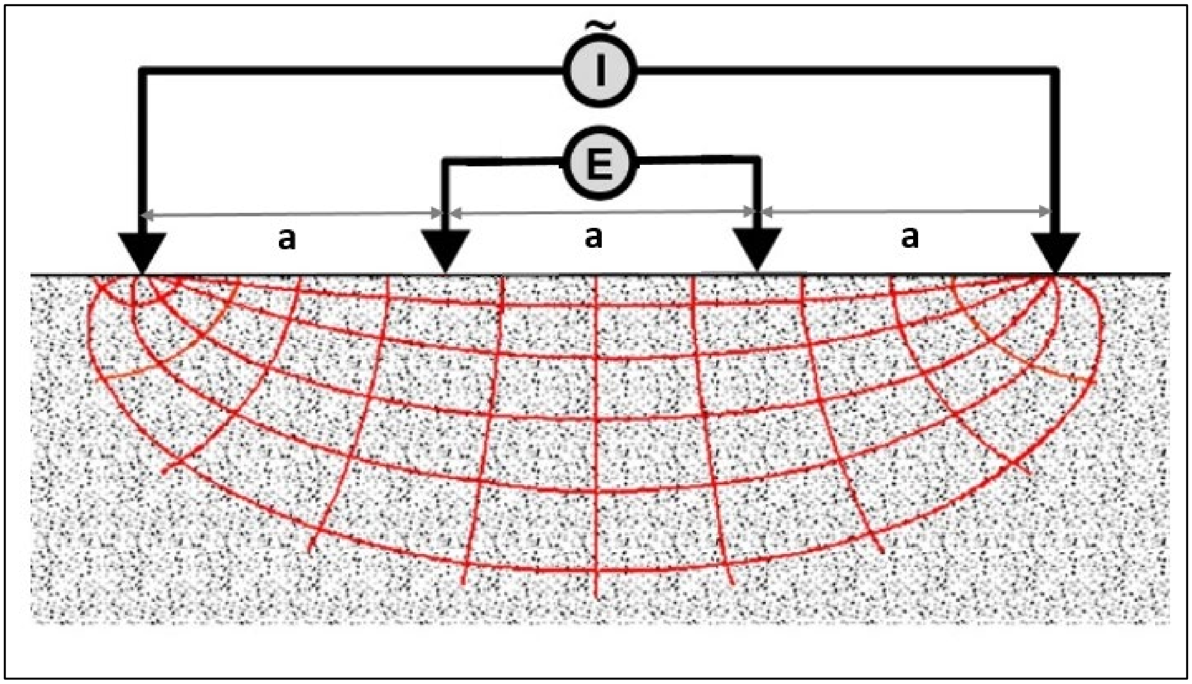
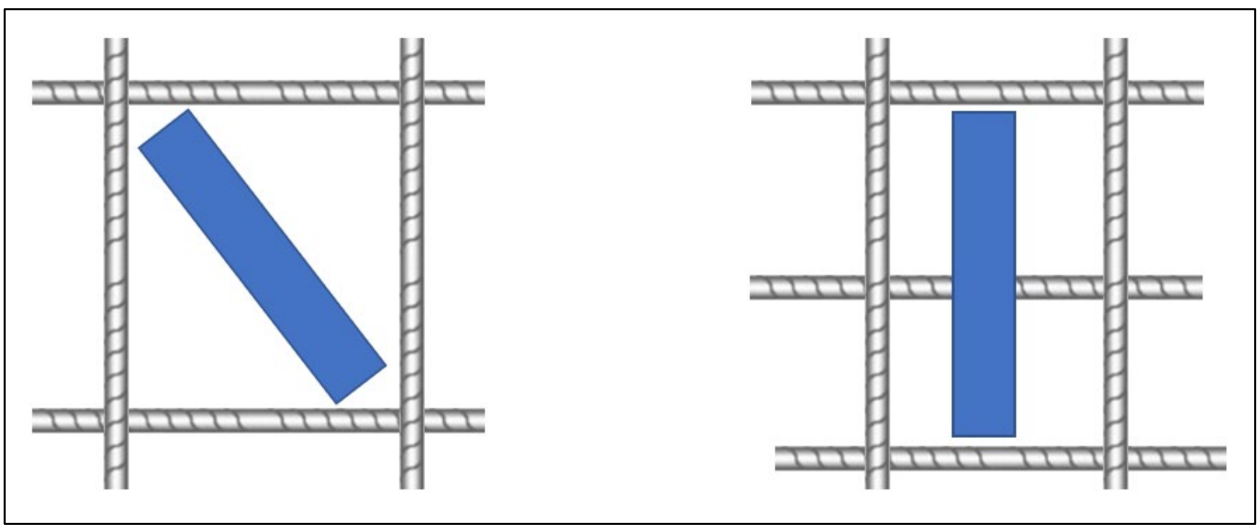


Figure 34 Schematic of the Wenner four-probe surface method to determine electrical resistivity of the concrete (op.cit. 15)



a) b)

Figure 35 Orientation of Wenner unit (blue rectangle) a) when the probe span is less than the reinforcing steel grid spacing; b) when reinforcing steel grid spacing is too small to be avoided.

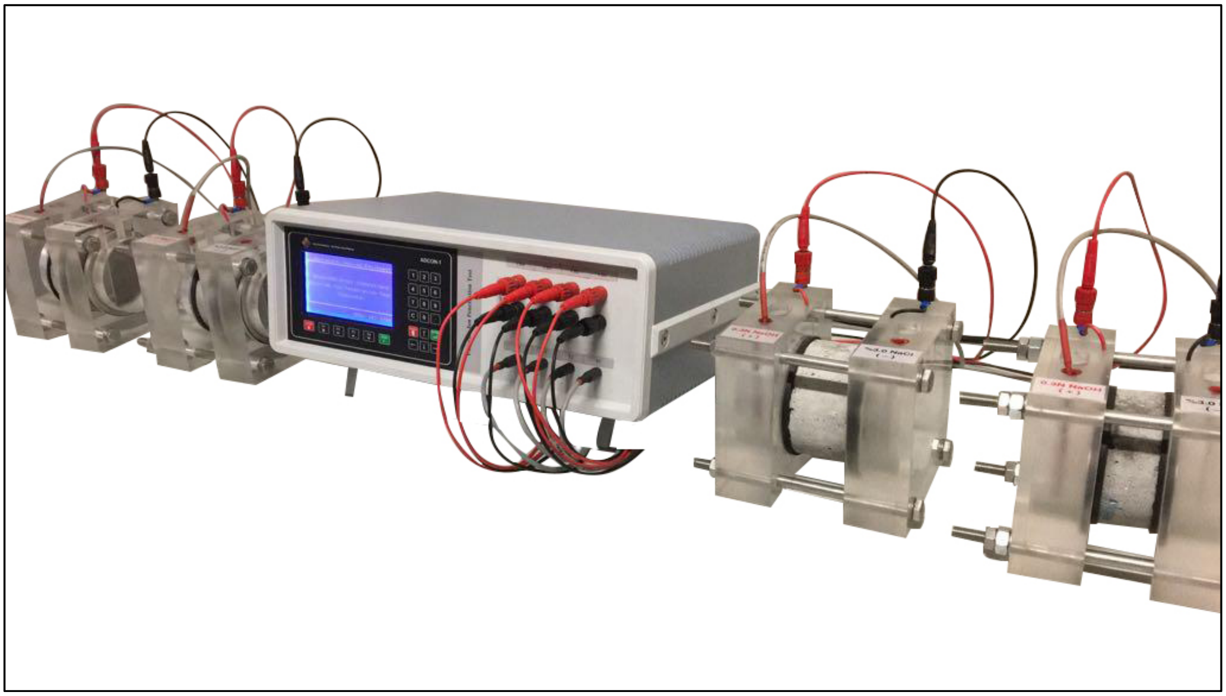


Figure 36 Test apparatus for AASHTO T 277 rapid permeability test (Courtesy Geotechnical Test Equipment Ltd.)

Figure 37 Use of indicator solution to measure depth of carbonation in small concrete core sample

(a) (b)

Figure 38 A field chloride test demonstration. (a) Extracting a concrete powder sample from column. (b) Using an electrode to measure chloride content from sample extract solution.

Figure 39 Cleaning gutter line of debris with a blower

Figure 40 Hand sprayer application of a silane sealer on a barrier wall

Figure 41 Pressure washing of a concrete barrier wall prior to painting

Figure 42 Abrasive blasting of a barrier wall prior to painting

Figure 43 Spray painting of an acrylic coating on a barrier wall

Figure 44 Painted concrete barrier wall with two-coat (epoxy-acrylic) paint system

Figure 45 Mechanical removal of distressed concrete on a pier column

Figure 46 Formwork being installed on a deck overlay repair to accommodate a full depth patch

Figure 47 A pier with concrete removed for a jacketing repair


Figure 48 Repaired concrete pier with blister applied to strengthen a column (arrow)


Figure 49 Sealing a cold joint in a bridge deck with methyl methacrylate

Figure 50 Applying conductive media wrap for electrochemical chloride extraction treatment (courtesy Vector Corrosion Technologies)

Figure 51 Metallic conductive mesh applied over conductive media for electrochemical extraction treatment (courtesy Vector Corrosion Technologies)

Figure 52 Wrapped girder undergoing electrochemical extraction treatment (courtesy Vector Corrosion Technologies)

Figure 53 DC power control panel for electrochemical chloride extraction (courtesy Vector Corrosion Technologies)

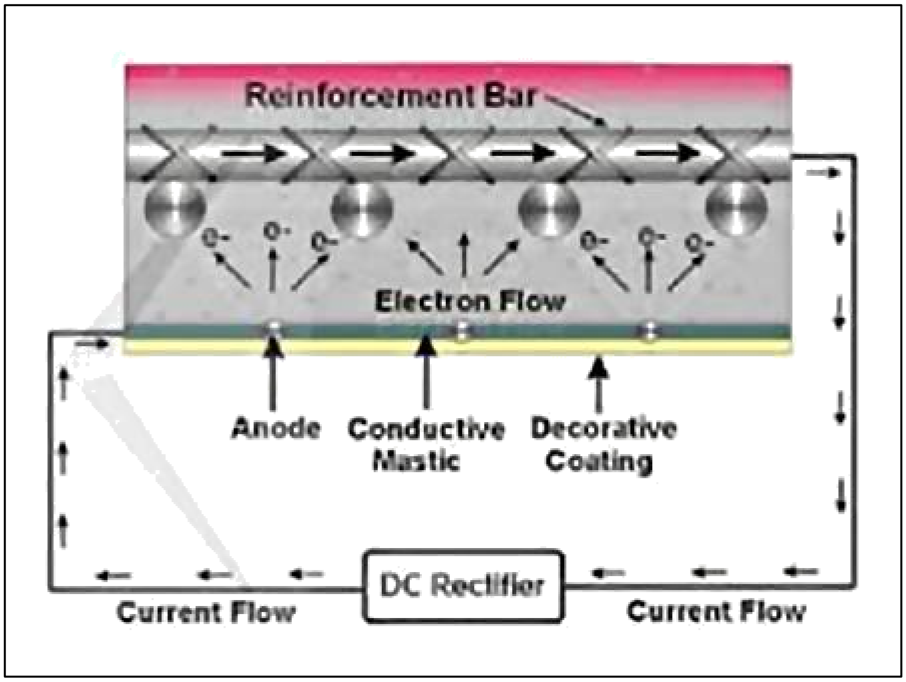


Figure 54 Impressed Current Cathodic Protection System (courtesy Vector Corrosion Technologies)

Figure 55 Four discrete galvanic anodes used in a patch repair (courtesy Vector Corrosion Technologies)

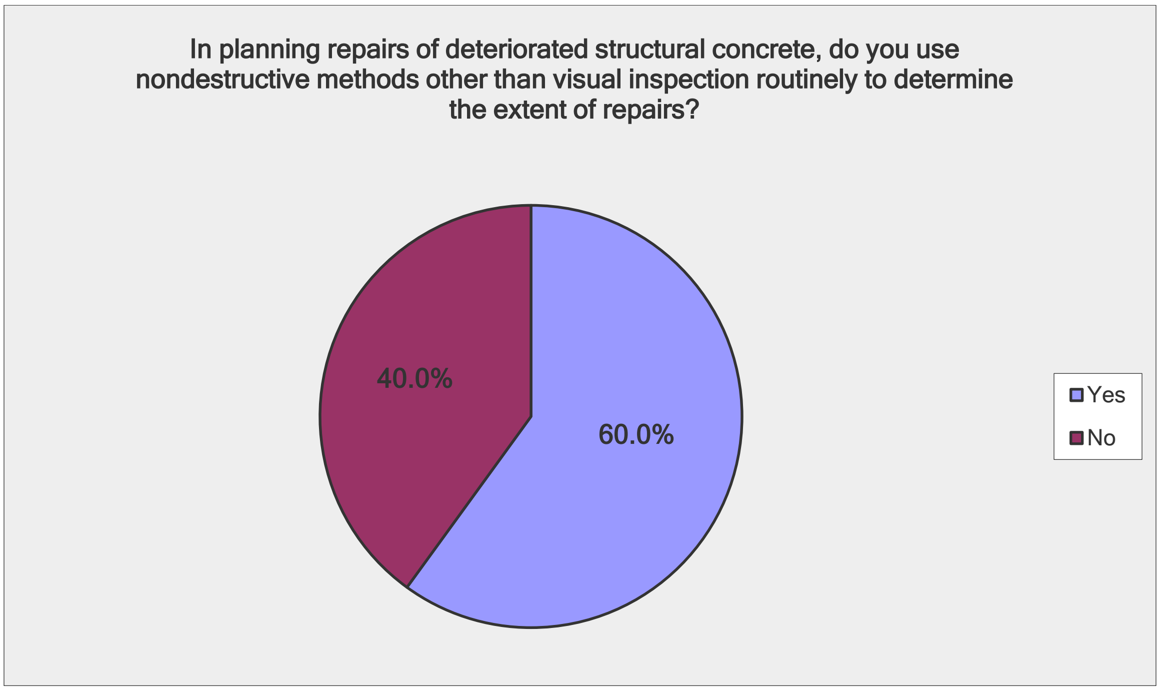
Figure 56 Distributed anodes (red arrows) employed on jacket repair (courtesy Vector Corrosion Technologies)

Figure 57 Sheet zinc galvanic anode being applied by metallizing concrete (courtesy Vector Corrosion Technologies)

Appendix A Summary of Survey on NDE of Concrete Bridges (Other State Highway Agencies)

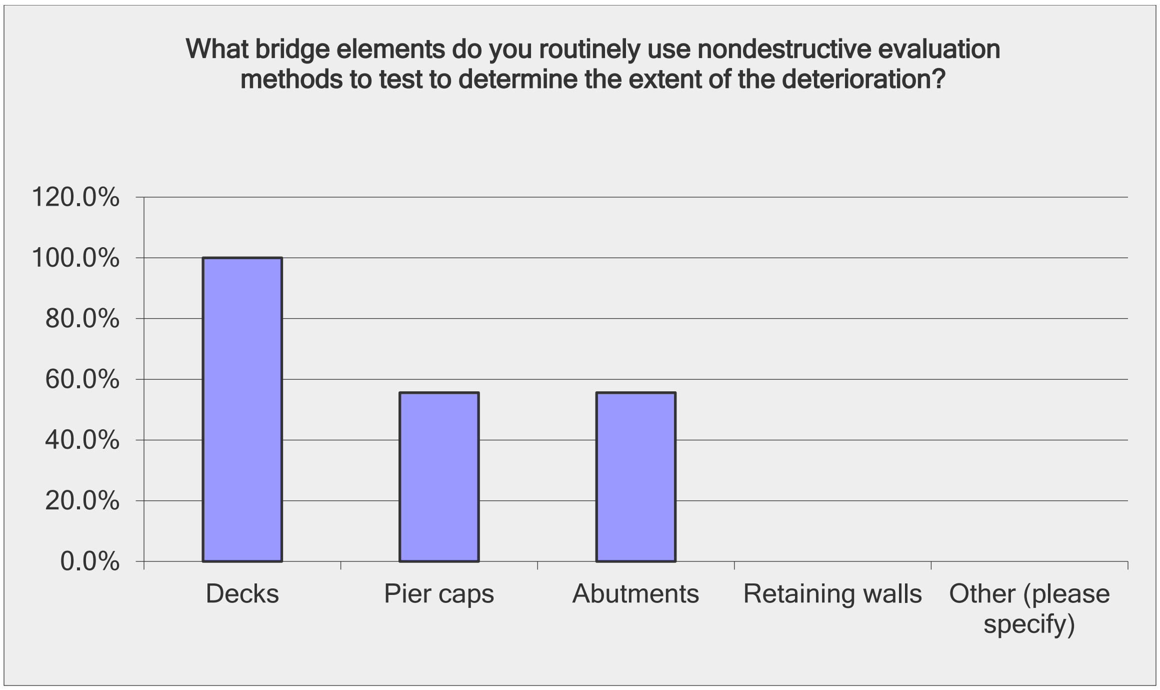
Question 1

In planning repairs of deteriorated structural concrete, do you use nondestructive methods other than visual inspection routinely to determine the extent of repairs?		
Answer Options	Response Percent	Response Count
Yes	60.0%	9
No	40.0%	6
answered question		15
skipped question		0



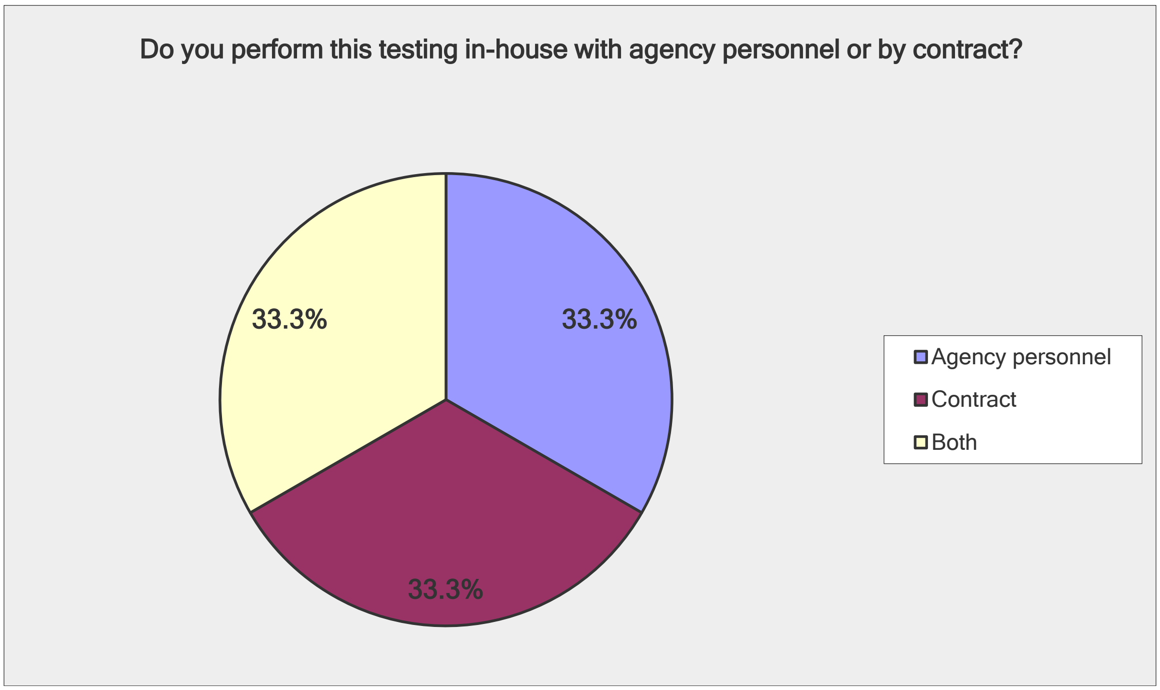
Question 2

What bridge elements do you routinely use nondestructive evaluation methods to test to determine the extent of the deterioration?		
Answer Options	Response Percent	Response Count
Decks	100.0%	9
Pier caps	55.6%	5
Abutments	55.6%	5
Retaining walls	0.0%	0
Other (please specify)	0.0%	0
answered question		9
skipped question		6



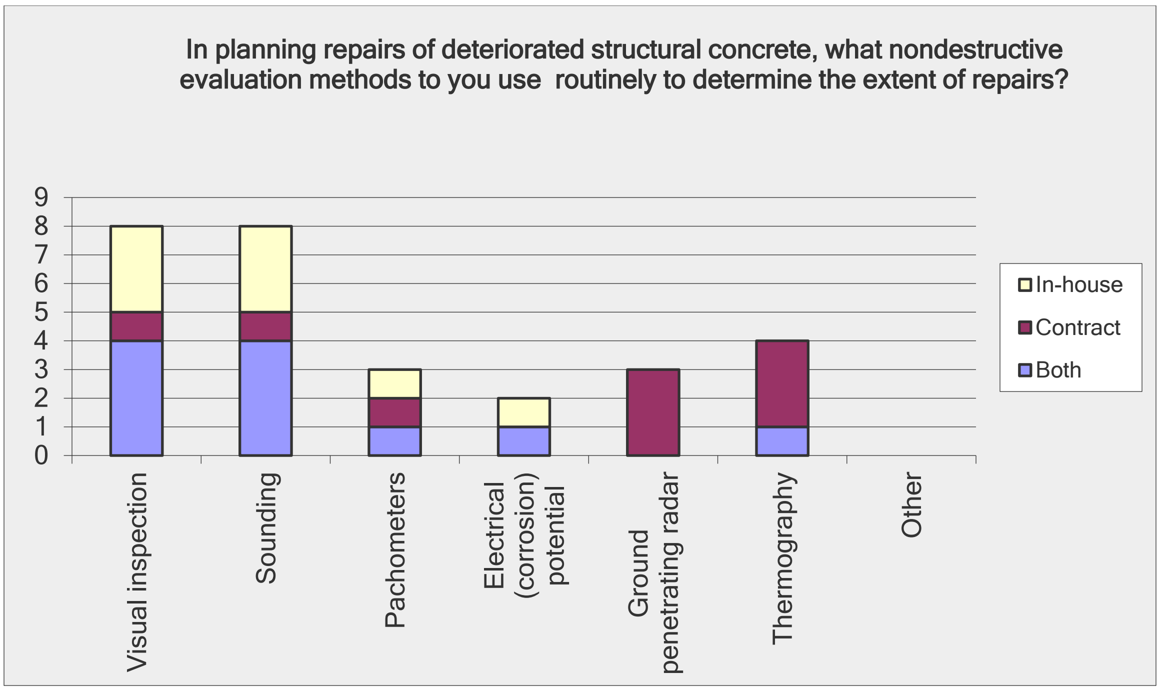
Question 3

Do you perform this testing in-house with agency personnel or by contract?		
Answer Options	Response Percent	Response Count
Agency personnel	33.3%	3
Contract	33.3%	3
Both	33.3%	3
answered question		9
skipped question		6



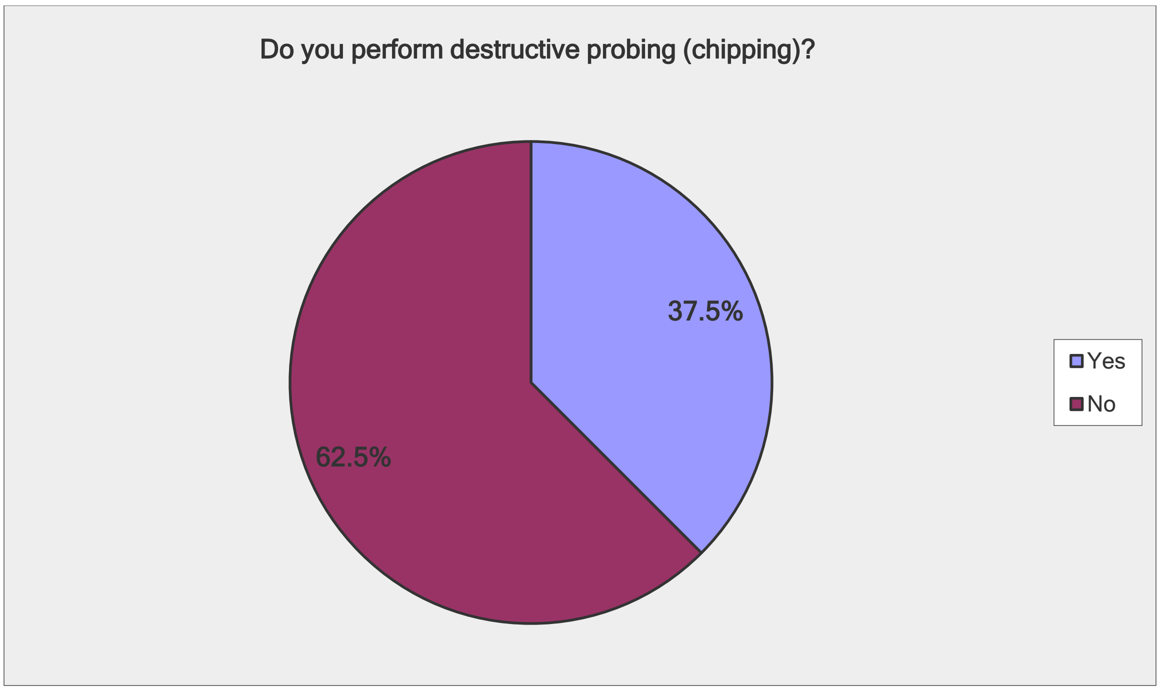
Question 4

In planning repairs of deteriorated structural concrete, what nondestructive evaluation methods to you use routinely to determine the extent of repairs?				
Answer Options	In-house	Contract	Both	Response Count
Visual inspection	3	1	4	8
Sounding	3	1	4	8
Pachometers	1	1	1	3
Electrical (corrosion) potential	1	0	1	2
Ground penetrating radar	0	3	0	3
Thermography	0	3	1	4
Other	0	0	0	0
Other (please specify)				0
answered question				8
skipped question				7



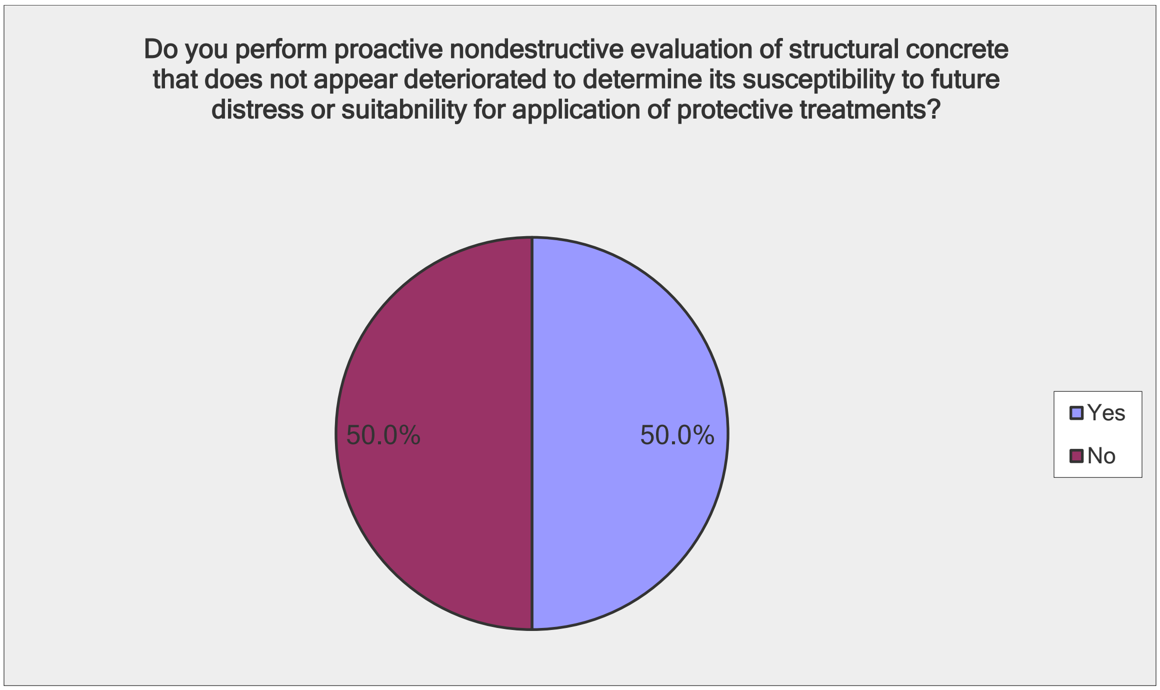
Question 5

Do you perform destructive probing (chipping)?		
Answer Options	Response Percent	Response Count
Yes	37.5%	3
No	62.5%	5
answered question		8
skipped question		7



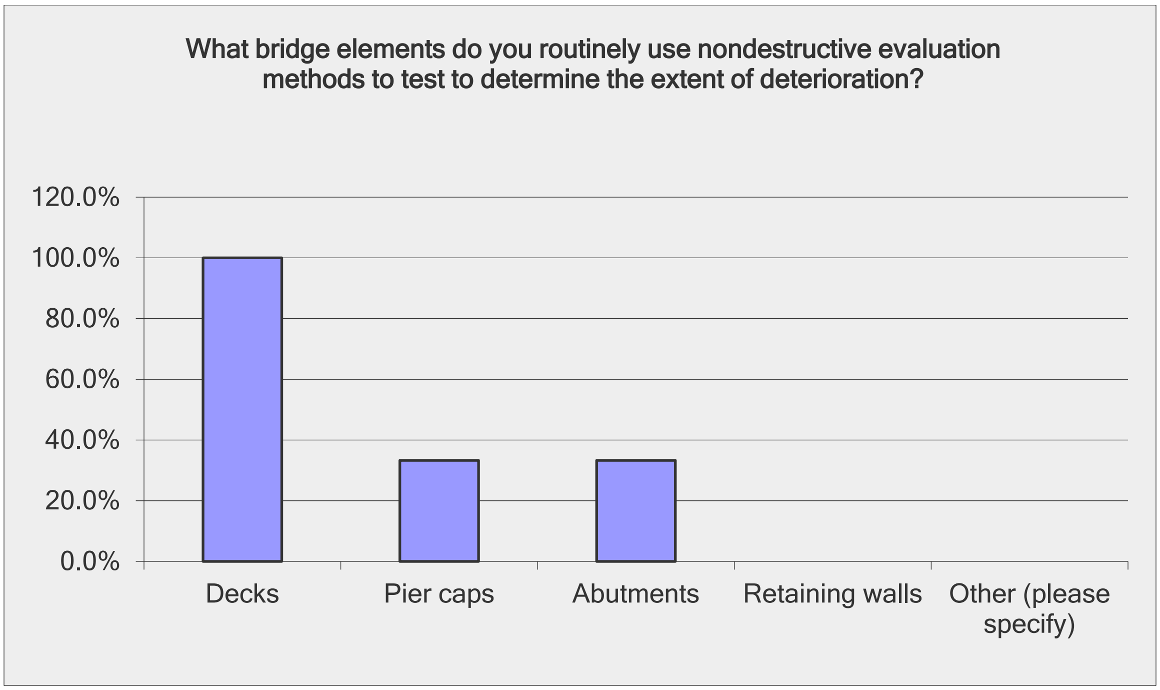
Question 6

Do you perform proactive nondestructive evaluation of structural concrete that does not appear deteriorated to determine its susceptibility to future distress or suitability for application of protective treatments?		
Answer Options	Response Percent	Response Count
Yes	50.0%	4
No	50.0%	4
answered question		8
skipped question		7



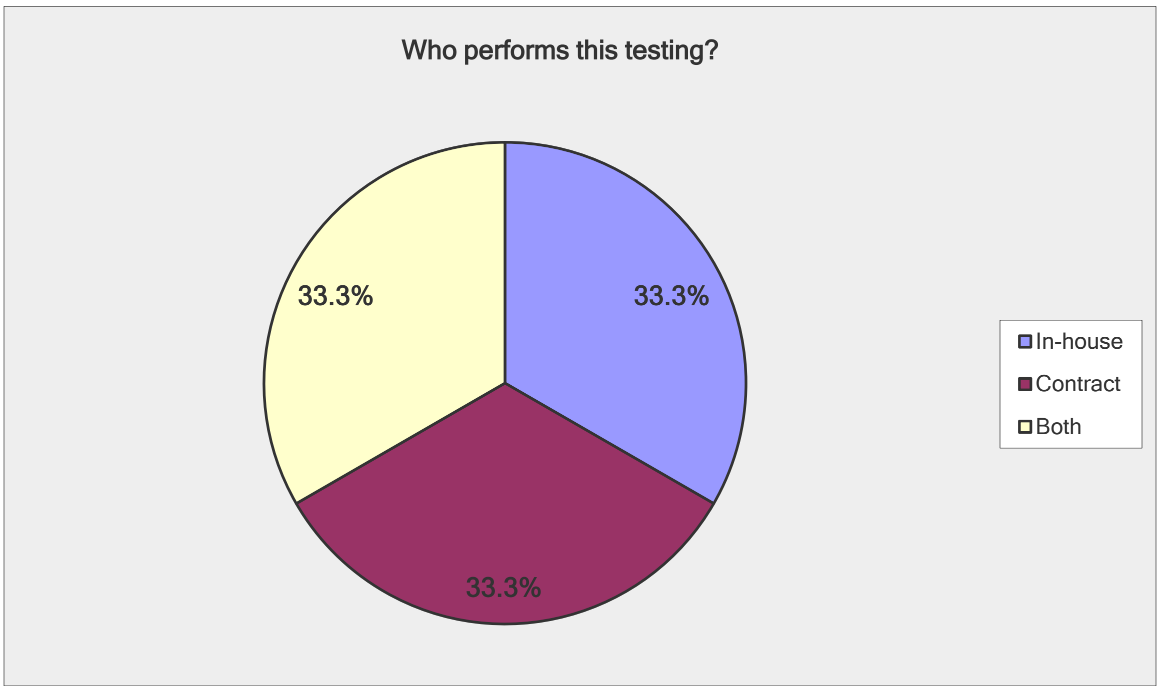
Question 7

What bridge elements do you routinely use nondestructive evaluation methods to test to determine the extent of deterioration?		
Answer Options	Response Percent	Response Count
Decks	100.0%	6
Pier caps	33.3%	2
Abutments	33.3%	2
Retaining walls	0.0%	0
Other (please specify)	0.0%	0
answered question		6
skipped question		9



Question 8

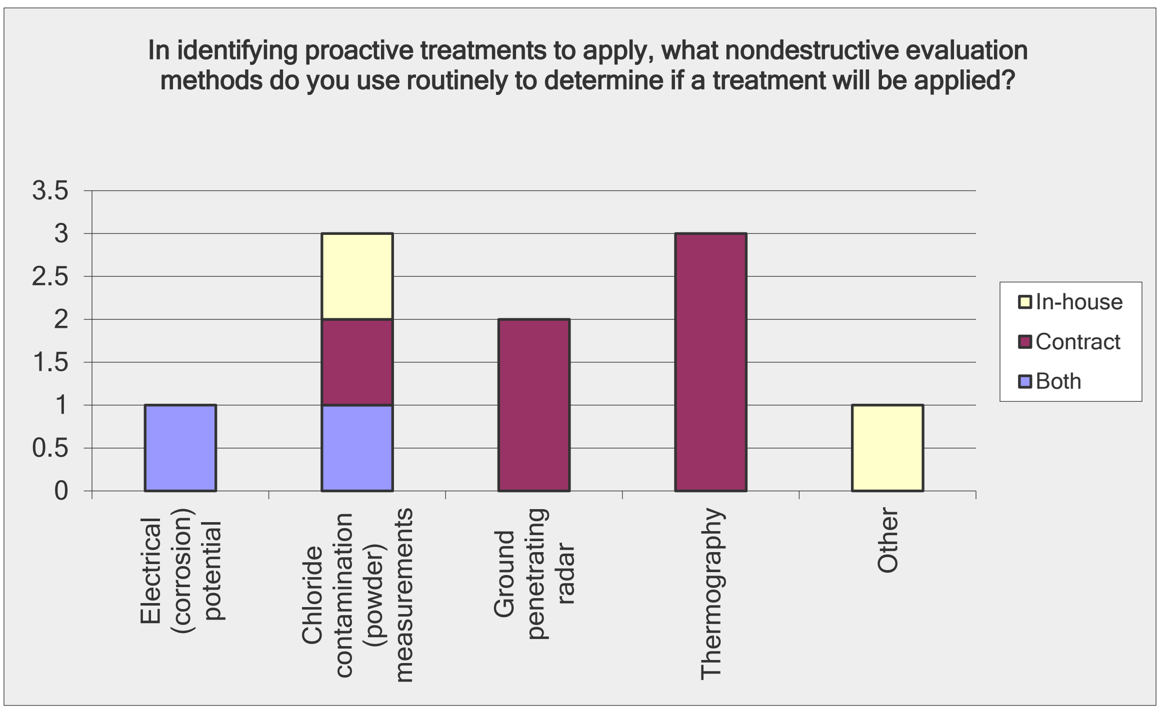
Who performs this testing?		
Answer Options	Response Percent	Response Count
In-house	33.3%	2
Contract	33.3%	2
Both	33.3%	2
answered question		6
skipped question		9



Question 9

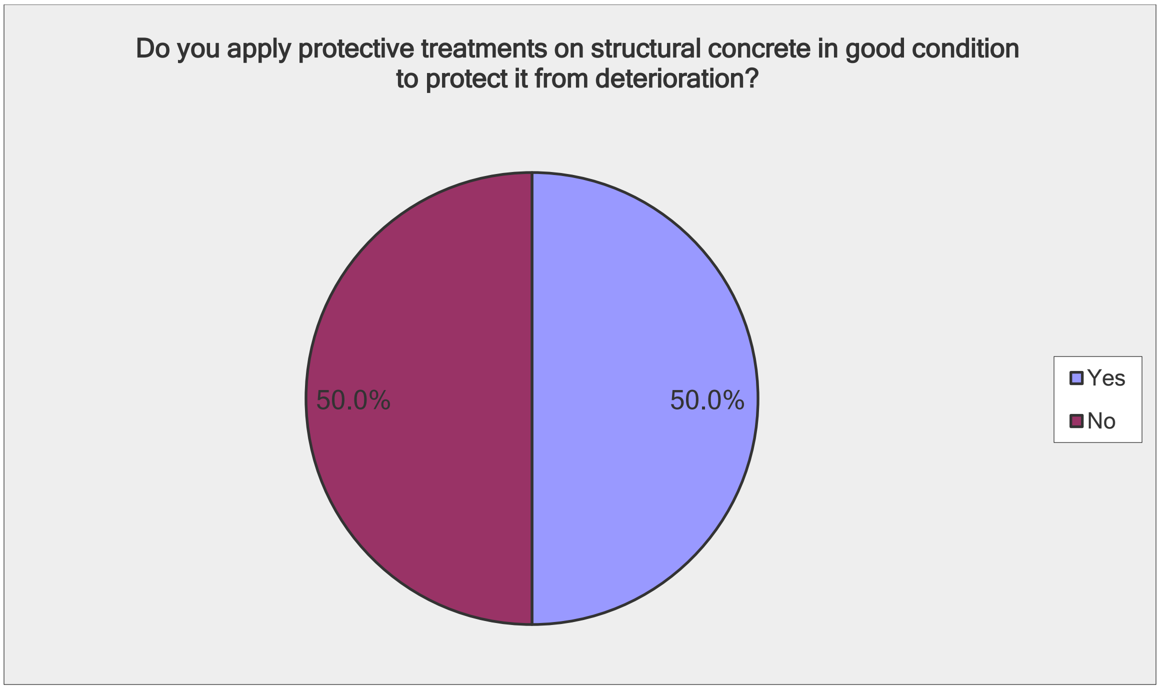
In identifying proactive treatments to apply, what nondestructive evaluation methods do you use routinely to determine if a treatment will be applied?				
Answer Options	In-house	Contract	Both	Response Count
Electrical (corrosion) potential	0	0	1	1
Chloride contamination (powder) measurements	1	1	1	3
Ground penetrating radar	0	2	0	2
Thermography	0	3	0	3
Other	1	0	0	1
Other (please specify)				1
answered question				4
skipped question				11

Number	Response Date	Other (please specify)
1	Feb 24, 2016 2:39 PM	sounding



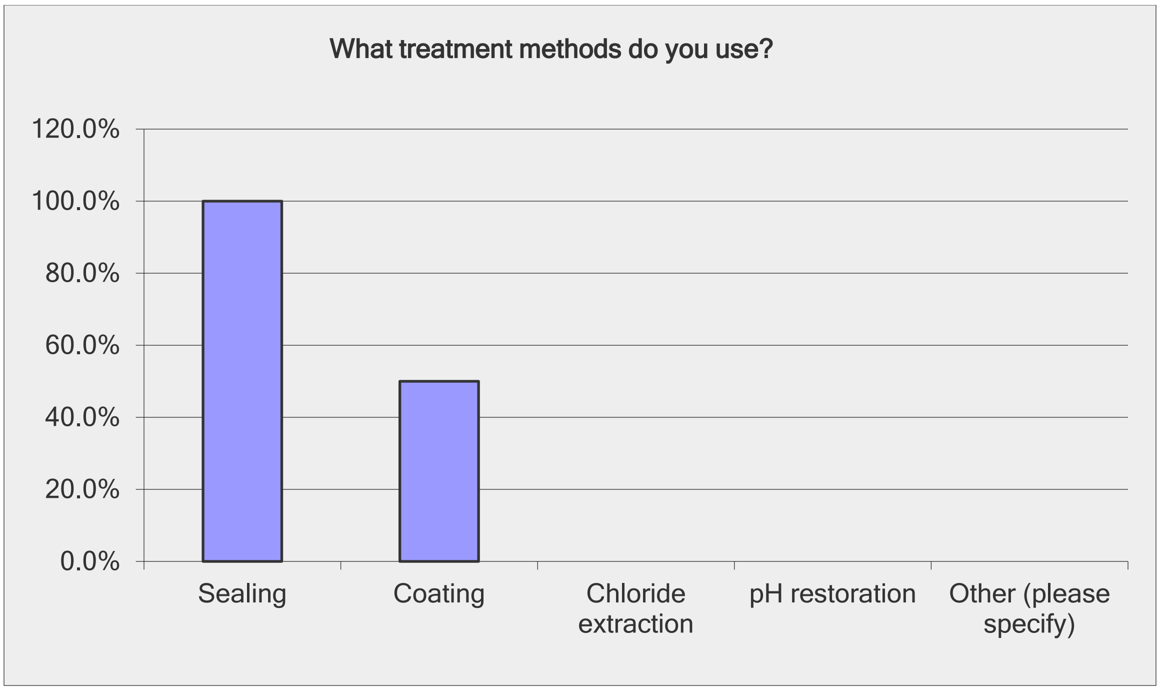
Question 10

Do you apply protective treatments on structural concrete in good condition to protect it from deterioration?		
Answer Options	Response Percent	Response Count
Yes	50.0%	2
No	50.0%	2
answered question		4
skipped question		11



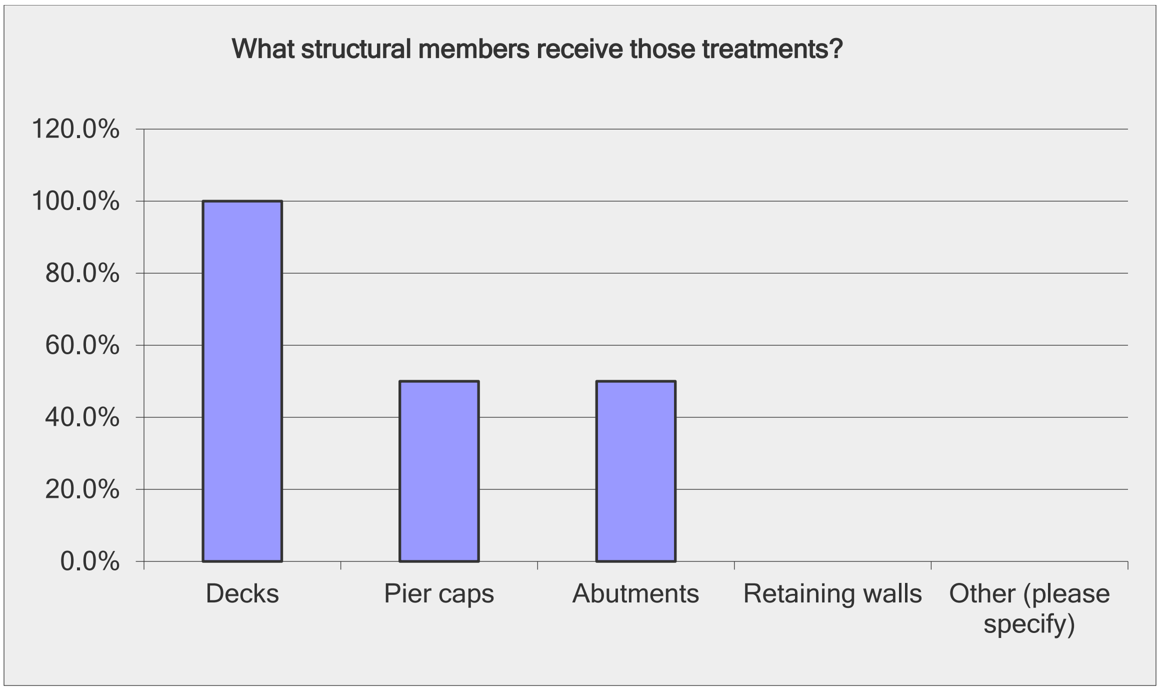
Question 11

What treatment methods do you use?		
Answer Options	Response Percent	Response Count
Sealing	100.0%	2
Coating	50.0%	1
Chloride extraction	0.0%	0
pH restoration	0.0%	0
Other (please specify)	0.0%	0
answered question		2
skipped question		13



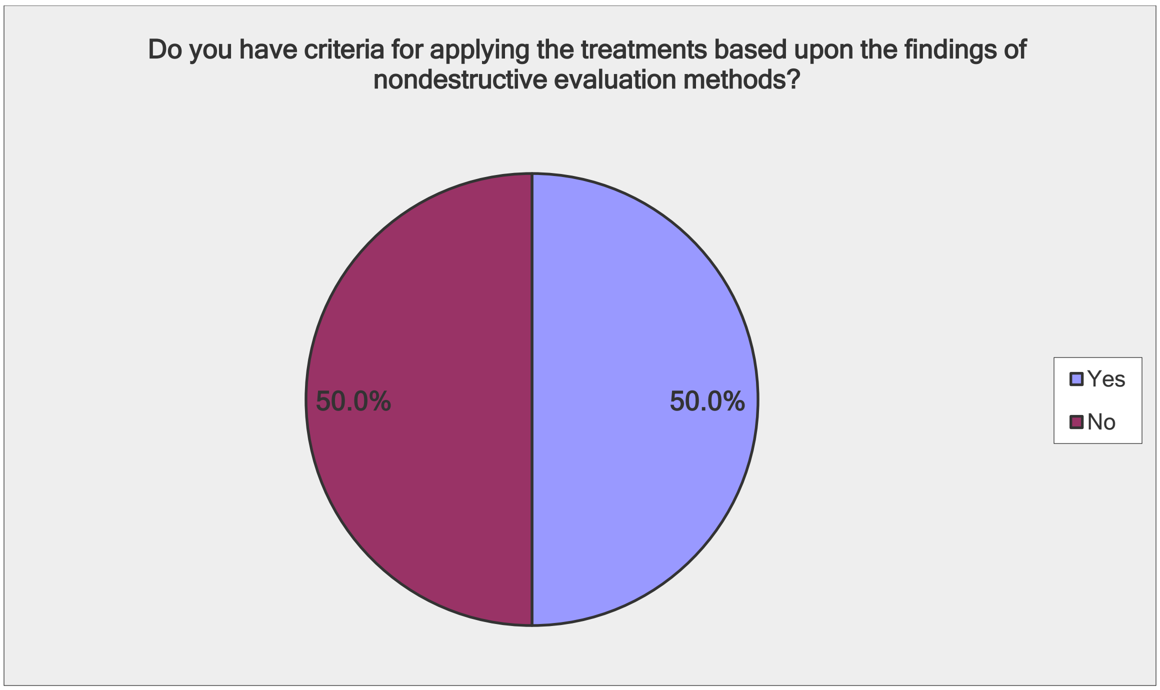
Question 12

What structural members receive those treatments?		
Answer Options	Response Percent	Response Count
Decks	100.0%	2
Pier caps	50.0%	1
Abutments	50.0%	1
Retaining walls	0.0%	0
Other (please specify)	0.0%	0
answered question		2
skipped question		13



Question 13

Do you have criteria for applying the treatments based upon the findings of nondestructive evaluation methods?		
Answer Options	Response Percent	Response Count
Yes	50.0%	1
No	50.0%	1
answered question		2
skipped question		13



Question 14

What criteria is used by your agency?	
Answer Options	Response Count
	1
answered question	1
skipped question	14

1	Feb 24, 2016 2:42 PM	Do not have hard rules but typically for decks we want less than 10% repair area for thin overlays. Others will get membrane seal with asphalt.

Question 15

Provide any comments that would further explain your agency's use on nondestructive evaluation on structural concrete.	
Answer Options	Response Count
	10
answered question	10
skipped question	5

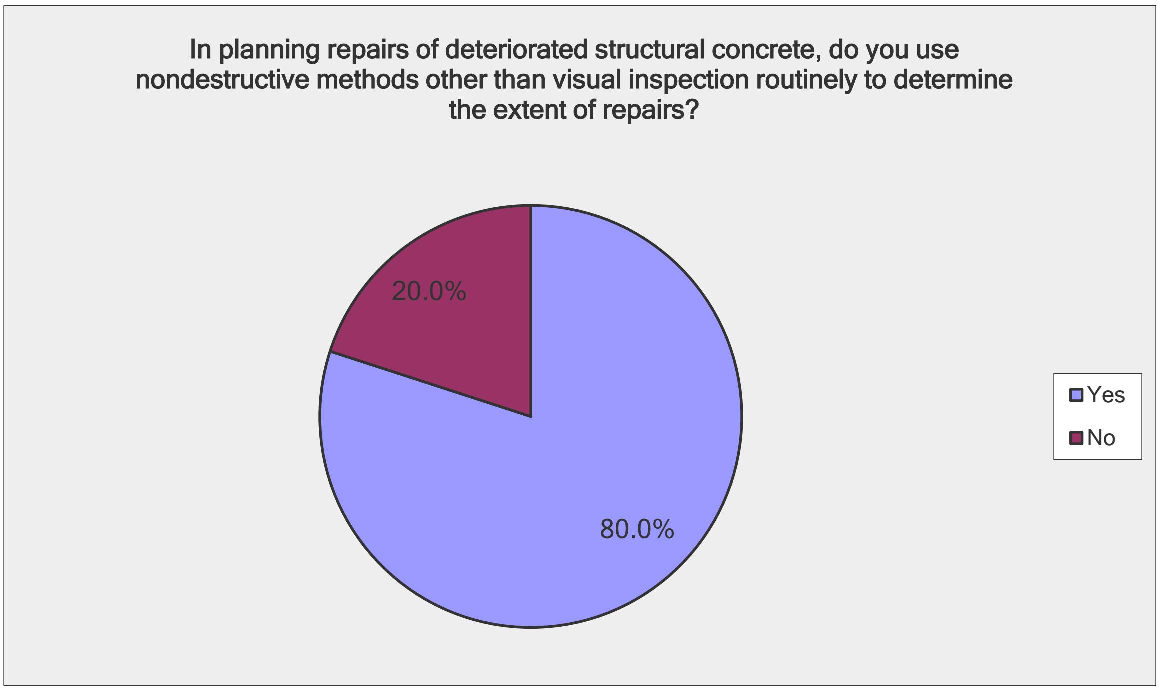
Number	Response Date	Response Text
1	Feb 26, 2016 7:48 PM	Although we contracted out work using these methods (GPR, and Thermography), I would not say we use them "routinely". We recently supplemented some of our routine inspections in high traffic areas by contracting this work out to get a better assessment of our decks (our inspectors could not get out and chain the deck). The cost for this benefit will only allow us to do this on limited routes in the large metro areas, and at limited intervals. So far, we have done this in one heavily traveled corridor in one large metro area in the state. We have plans to do other corridors in large metro areas in the future. The goal is to do these evaluations on a set schedule; however, funding may prevent our intentions from becoming reality.
2	Feb 26, 2016 5:44 PM	The only test I would do if I thought it of value would be half-cell testing
3	Feb 25, 2016 7:05 PM	Kentucky occasionally uses ground penetrating radar and infrared cameras.
4	Feb 24, 2016 1:34 PM	Mostly chain drag and hammer soundings
5	Feb 22, 2016 9:02 PM	We do use some GPR and infrared inspection techniques. We are capable of performing both within MnDOT but usually contract out the service.

Number	Response Date	Response Text
6	Feb 19, 2016 4:45 PM	Structural concrete sounded with hammer to determine concrete soundness.
7	Feb 19, 2016 3:17 PM	We use chain drag on bridge decks, but we rely primarily on visual inspections.
8	Feb 19, 2016 3:13 PM	Generally, we only use nondestructive evaluation for decks. We have done some testing on piers and on post-tension ducts and strands. We had 2 crews that completed extensive in-house deck reports, these people were let go. We still have some in-house equipment and expertise to do some. We now contract out most of this work and complete less & less as we go due to cost. We have completed one consultant project on thermography technology to check the deck condition on one major corridor in an urban area with good results.
9	Feb 18, 2016 8:13 PM	Routinely use visual and sounding. Advanced methods used for special investigations.
10	Feb 18, 2016 8:09 PM	Only hammer and chain sounding routinely used.

Appendix B Summary of Survey on NDE of Concrete Bridges (KYTC Districts)

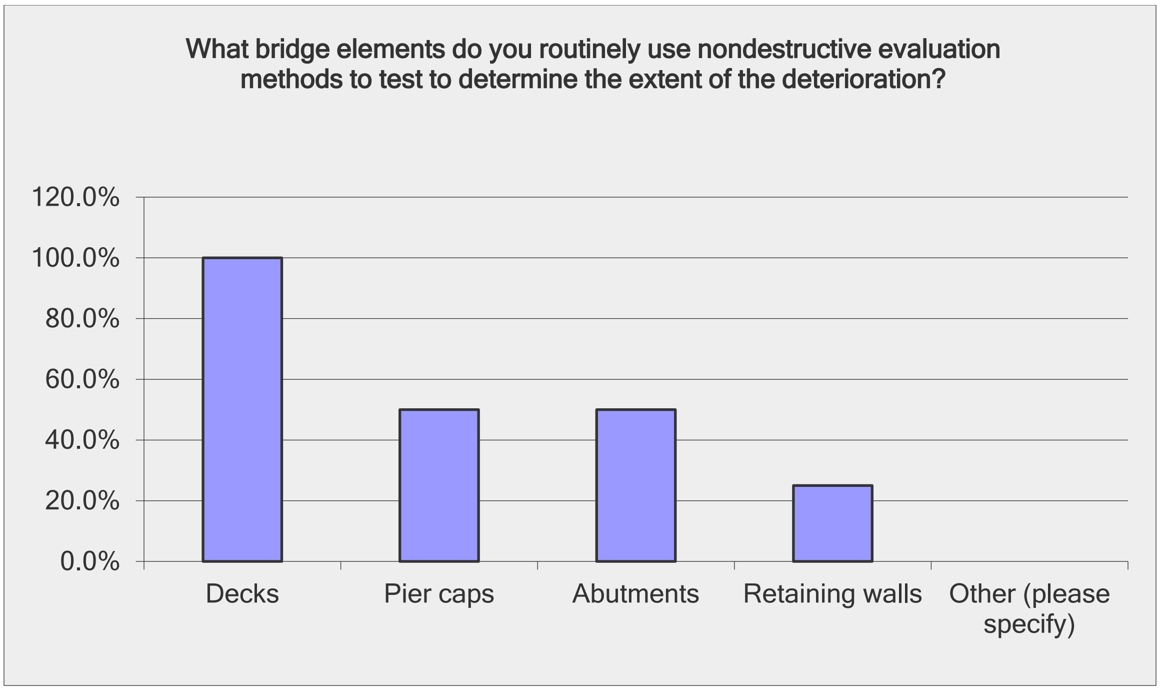
Question 1

In planning repairs of deteriorated structural concrete, do you use nondestructive methods other than visual inspection routinely to determine the extent of repairs?		
Answer Options	Response Percent	Response Count
Yes	80.0%	8
No	20.0%	2
answered question		10
skipped question		0



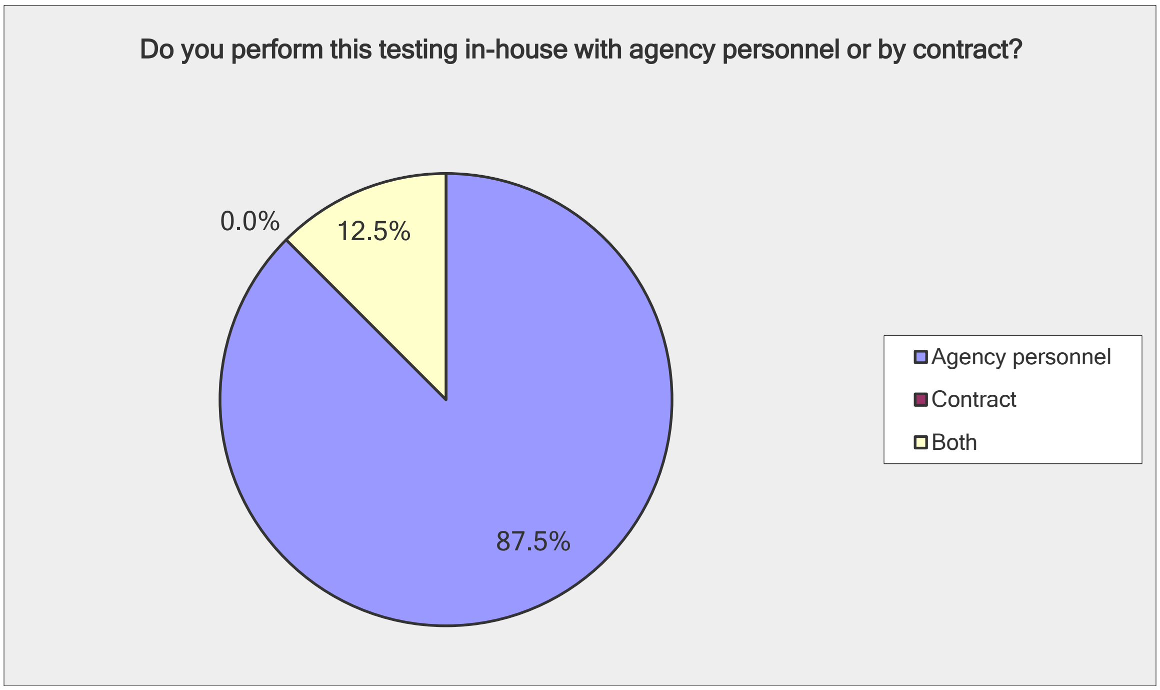
Question 2

What bridge elements do you routinely use nondestructive evaluation methods to test to determine the extent of the deterioration?		
Answer Options	Response Percent	Response Count
Decks	100.0%	8
Pier caps	50.0%	4
Abutments	50.0%	4
Retaining walls	25.0%	2
Other (please specify)	0.0%	0
answered question		8
skipped question		2



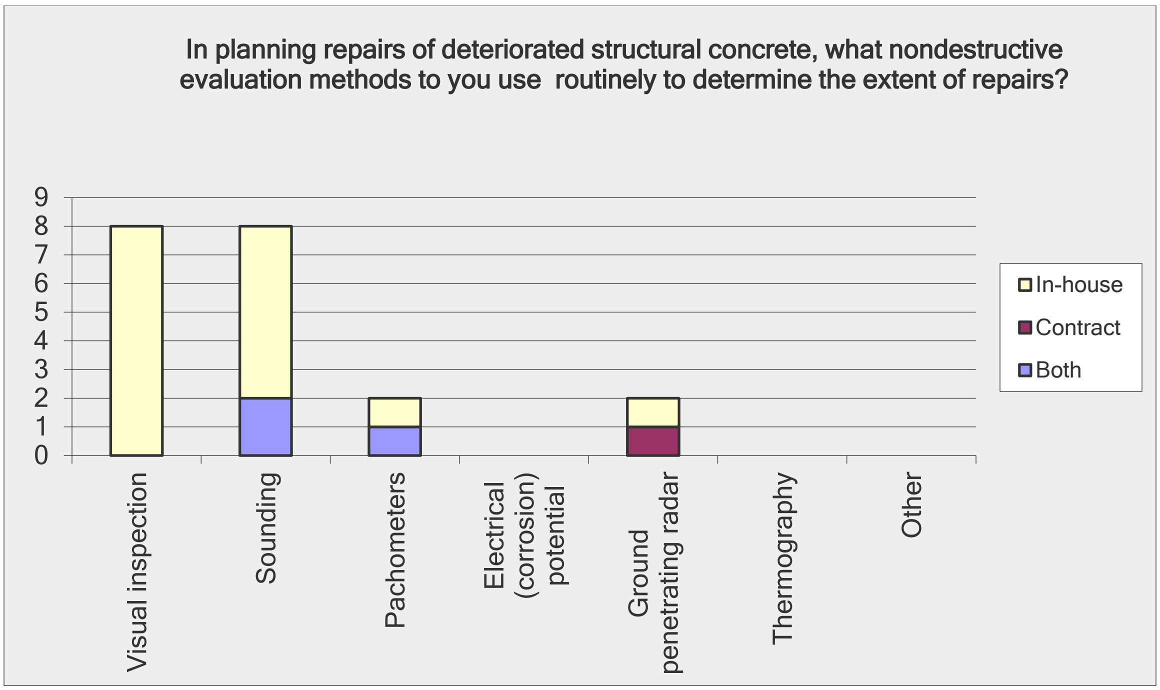
Question 3

Do you perform this testing in-house with agency personnel or by contract?		
Answer Options	Response Percent	Response Count
Agency personnel	87.5%	7
Contract	0.0%	0
Both	12.5%	1
answered question		8
skipped question		2



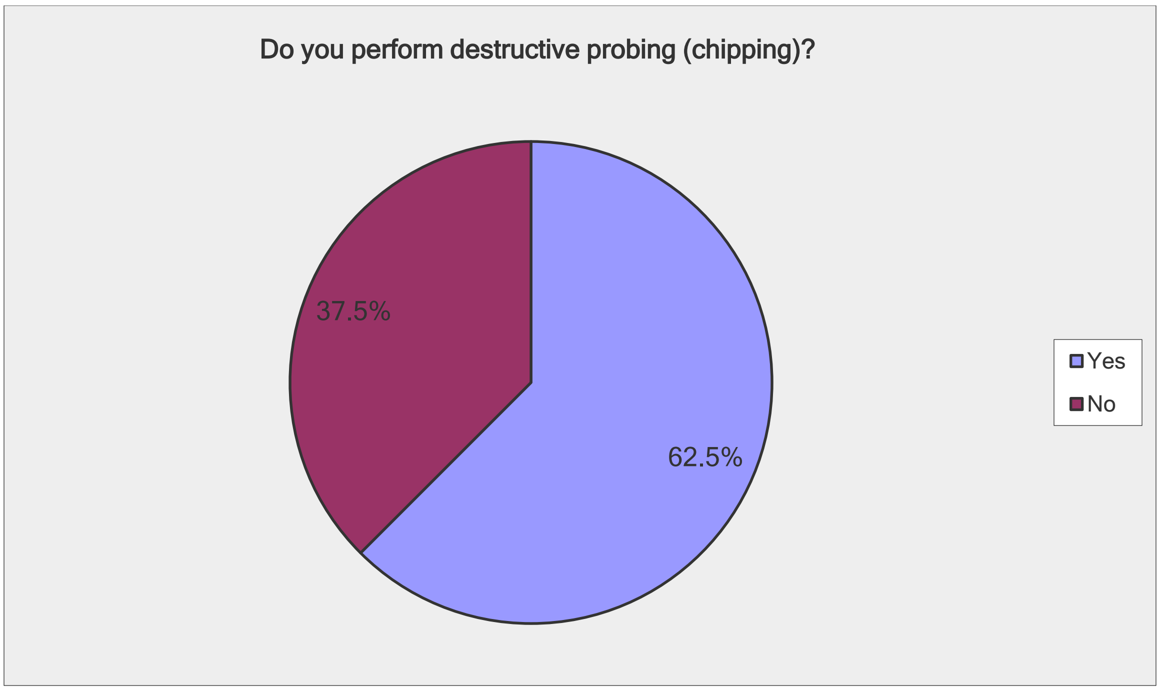
Question 4

In planning repairs of deteriorated structural concrete, what nondestructive evaluation methods to you use routinely to determine the extent of repairs?				
Answer Options	In-house	Contract	Both	Response Count
Visual inspection	8	0	0	8
Sounding	6	0	2	8
Pachometers	1	0	1	2
Electrical (corrosion) potential	0	0	0	0
Ground penetrating radar	1	1	0	2
Thermography	0	0	0	0
Other	0	0	0	0
Other (please specify)				0
answered question				8
skipped question				2



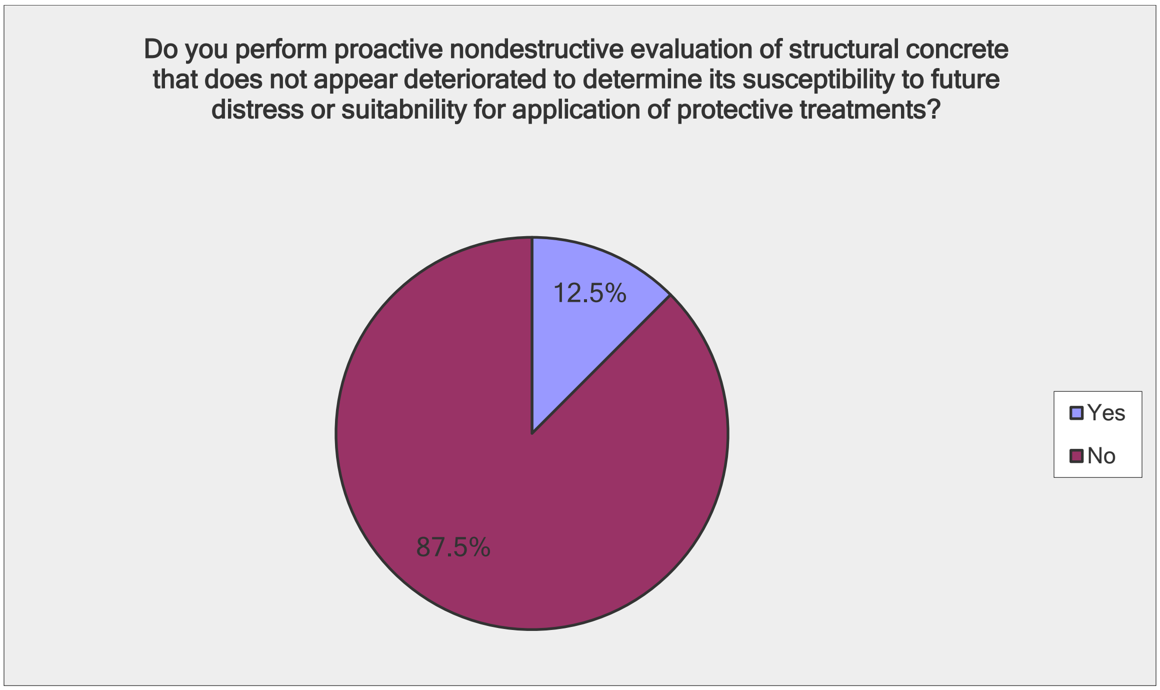
Question 5

Do you perform destructive probing (chipping)?		
Answer Options	Response Percent	Response Count
Yes	62.5%	5
No	37.5%	3
answered question		8
skipped question		2



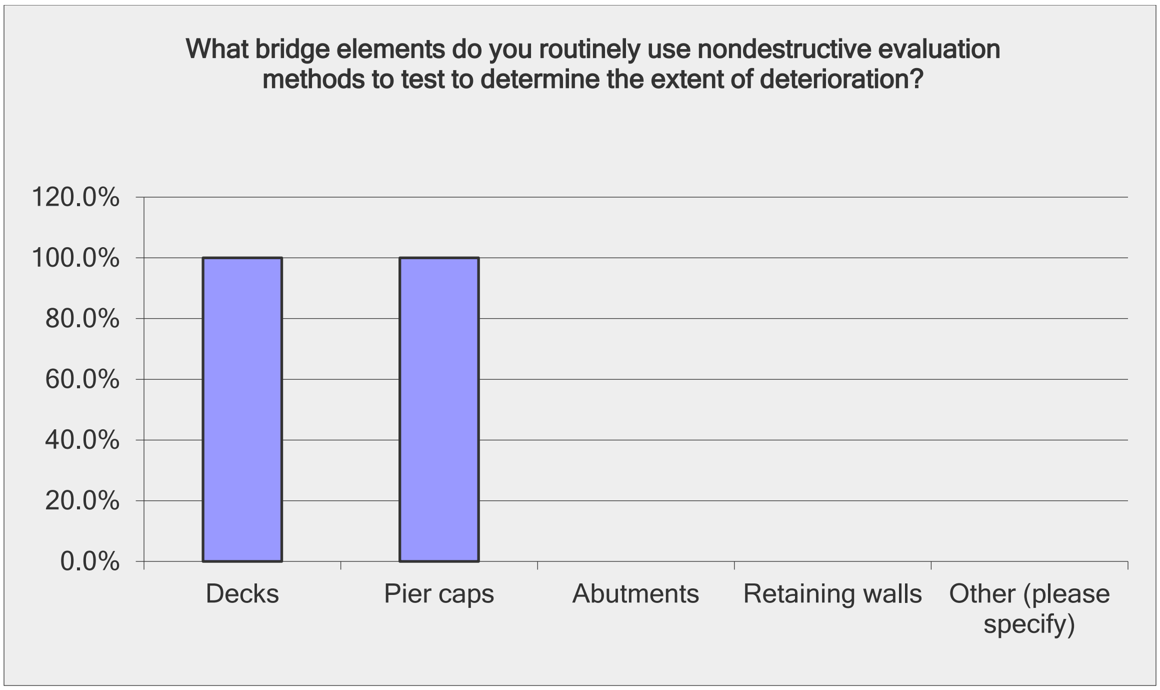
Question 6

Do you perform proactive nondestructive evaluation of structural concrete that does not appear deteriorated to determine its susceptibility to future distress or suitably for application of protective treatments?		
Answer Options	Response Percent	Response Count
Yes	12.5%	1
No	87.5%	7
answered question		8
skipped question		2



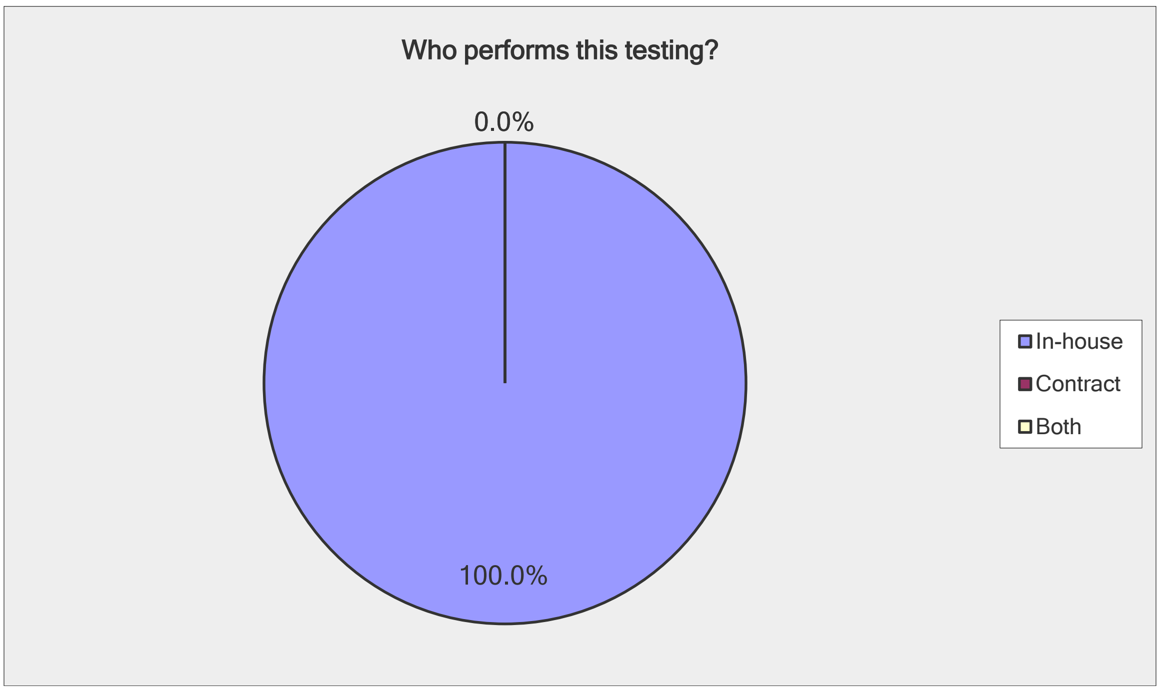
Question 7

What bridge elements do you routinely use nondestructive evaluation methods to test to determine the extent of deterioration?		
Answer Options	Response Percent	Response Count
Decks	100.0%	1
Pier caps	100.0%	1
Abutments	0.0%	0
Retaining walls	0.0%	0
Other (please specify)	0.0%	0
answered question		1
skipped question		9



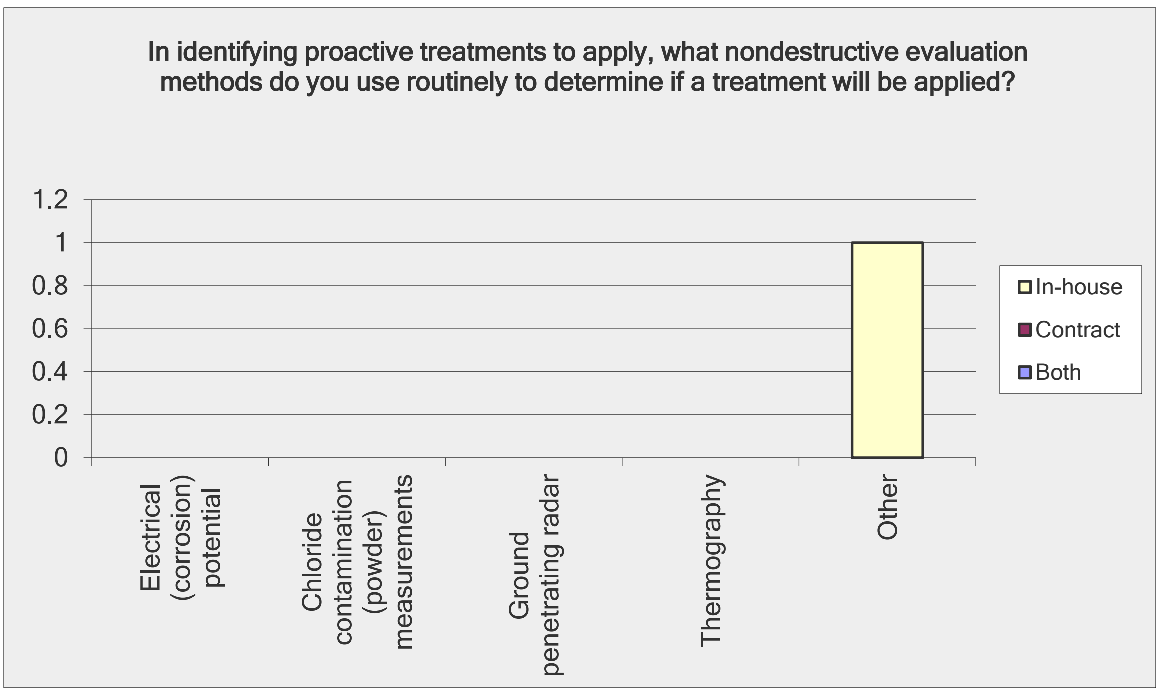
Question 8

Who performs this testing?		
Answer Options	Response Percent	Response Count
In-house	100.0%	2
Contract	0.0%	0
Both	0.0%	0
answered question		2
skipped question		8



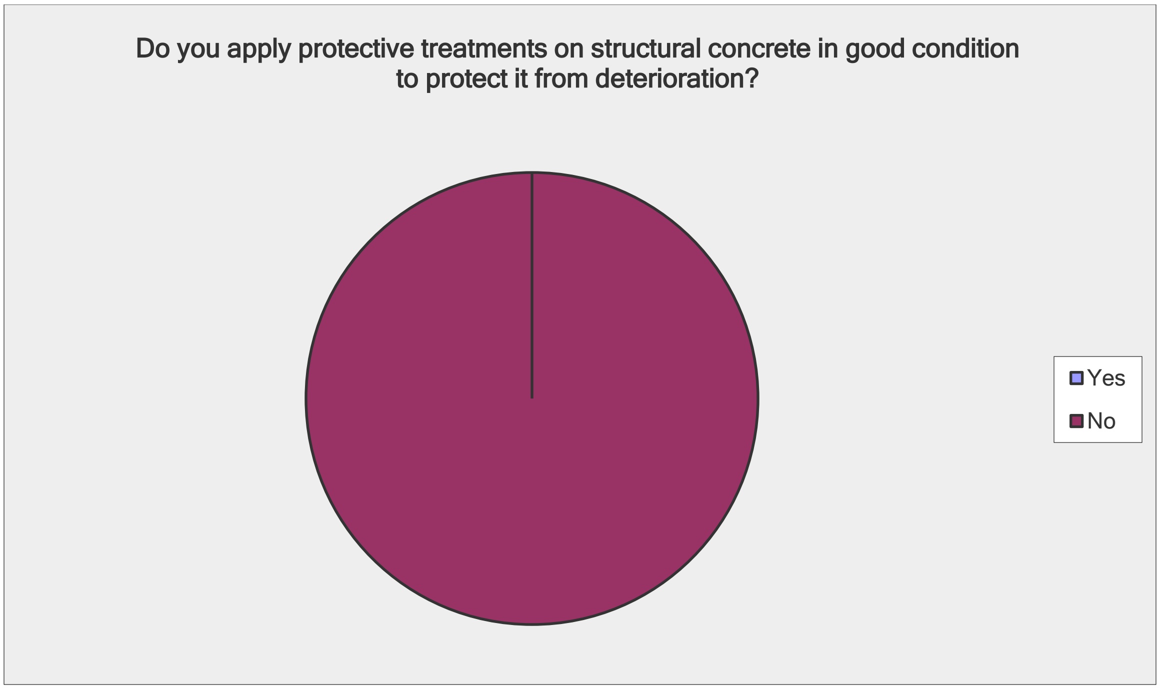
Question 9

In identifying proactive treatments to apply, what nondestructive evaluation methods do you use routinely to determine if a treatment will be applied?				
Answer Options	In-house	Contract	Both	Response Count
Electrical (corrosion) potential	0	0	0	0
Chloride contamination (powder) measurements	0	0	0	0
Ground penetrating radar	0	0	0	0
Thermography	0	0	0	0
Other	1	0	0	1
Other (please specify)				0
answered question				1
skipped question				9



Question 10

Do you apply protective treatments on structural concrete in good condition to protect it from deterioration?		
Answer Options	Response Percent	Response Count
Yes	0.0%	0
No	100.0%	1
answered question		1
skipped question		9



Question 11

What treatment methods do you use?		
Answer Options	Response Percent	Response Count
Sealing	0.0%	0
Coating	0.0%	0
Chloride extraction	0.0%	0
pH restoration	0.0%	0
Other (please specify)	0.0%	0
answered question		0
skipped question		10

Question 12

What structural members receive those treatments?		
Answer Options	Response Percent	Response Count
Decks	0.0%	0
Pier caps	0.0%	0
Abutments	0.0%	0
Retaining walls	0.0%	0
Other (please specify)	0.0%	0
answered question		0
skipped question		10

Question 13

Do you have criteria for applying the treatments based upon the findings of nondestructive evaluation methods?		
Answer Options	Response Percent	Response Count
Yes	0.0%	0
No	0.0%	0
answered question		0
skipped question		10

Question 14

What criteria is used by your agency?	
Answer Options	Response Count
	0
answered question	0
skipped question	10

Question 15

Provide any comments that would further explain your agency's use on nondestructive evaluation on structural concrete.	
Answer Options	Response Count
	3
answered question	3
skipped question	7

Number	Response Date	Response Text
1	Mar 11, 2016 6:59 PM	Primarily we use sounding to determine concrete damage.
2	Mar 10, 2016 3:05 PM	It would cost money, and we have none at our disposal.
3	Feb 26, 2016 1:03 AM	Use chain sounding (preferred) or sometimes hammer sounding to evaluate bridge deck delamination.