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CONCRETE BRIDGE DECK CONDITION ASSESSMENT AND IMPROVEMENT STRATEGIES

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ABSTRACT

The aging and deterioration of bridges in Utah mandates increasingly cost-effective strategies for bridge maintenance, rehabilitation, and repair (MR&R). Although the substructures and superstructures of bridges in Utah are in relatively good structural condition, the bridge decks are deteriorating more rapidly due to the routine application of deicing salts, repeated freeze-thaw cycles, and other damaging effects. Therefore, the Utah Department of Transportation (UDOT) initiated this research to ultimately develop a protocol offering guidance about when and how a bridge deck should be rehabilitated or when it should be replaced. The research specifically focused on concrete bridge deck performance issues, condition assessment techniques, rehabilitation methods, and bridge management system (BMS) concepts.

An extensive literature review was conducted to identify condition assessment methods used to detect concrete bridge deck deterioration, as well as to identify rehabilitation methods for deck repairs. A questionnaire survey was also conducted to identify the state of the practice for bridge deck management by state departments of transportation (DOTs) throughout the United States. The survey addressed issues such as climate and traffic information, new deck construction, winter deck maintenance, deck deterioration, deck condition assessment, and deck rehabilitation.

The results of the research show that a bridge deck management system is essential in maintaining the integrity of reinforced concrete bridge decks. Developing a successful system requires routine inspection and monitoring to enable prioritization of MR&R strategies for individual bridges. Many types of technologies are available for assessing the condition of concrete bridge decks, but the survey results suggest that only five methods are frequently used to monitor and detect bridge deck deterioration. These methods are visual inspection, chaining, chloride concentration testing, coring, and half-cell potential testing.

The survey identified chloride-induced corrosion, freeze-thaw cycling, and poor construction practices as the most common sources of bridge deck deterioration. Results also indicate that distress is most frequently manifested as cracking, delaminations, spalling, potholes, and scaling. Generally, the extent of deterioration that constitutes a full-deck replacement was given by survey respondents as 30 to 50 percent of the deck area.

One of the major contributors to bridge deck deterioration is winter deck maintenance. To mitigate the negative effects of deicing chemicals, DOTs are employing preventative measures such as increasing concrete cover over the reinforcement, using epoxy-coated reinforcement, including appropriate admixtures in the concrete mixture, and facilitating proper curing of the concrete. Rehabilitation options listed for bridge decks include electrochemical rehabilitation, concrete removal and patching, surface treatments, and epoxy injections.

UDOT should develop and implement a formal BMS with a searchable database containing information about the types of distress manifested on individual bridges, causes for the distress, values of measured test parameters, types of rehabilitation methods performed on the bridge deck, costs for rehabilitation methods, and service life extensions as a result of particular rehabilitation methods. Supporting data should be regularly collected through inspection and monitoring programs to facilitate prioritization of MR&R strategies for individual bridges and to evaluate the impact of such strategies on the overall condition of the network. Performance indices based on selected condition assessment parameters should be developed for use in BMS analyses, and mathematical deterioration models should be calibrated for forecasting network condition and predicting funding requirements for various possible MR&R strategies.

DISCLAIMER

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CHAPTER 1
INTRODUCTION

1.1 PROBLEM STATEMENT

The aging and deterioration of bridges in Utah mandates increasingly cost-effective strategies for bridge maintenance, rehabilitation, and replacement (MR&R). The Utah Department of Transportation (UDOT) is responsible for 1,700 bridges throughout the state, of which 46 percent are older than 30 years as shown in Figure 1.1. Although the substructures and superstructures of bridges in Utah are in relatively good structural condition, the bridge decks are deteriorating more rapidly due to the routine application of deicing salts, repeated freeze-thaw cycles, and other damaging effects. Therefore, UDOT initiated this research to ultimately develop a protocol offering guidance about when and how a bridge deck should be rehabilitated or when it should be replaced. Development of a decision-making protocol that utilizes bridge deck condition assessment information in combination with life-cycle costs is especially important, since the costs associated with replacing every bridge deck in Utah are extremely high.

Concrete is an important material in highway construction since the majority of highway structures are composed of concrete. Most bridge structures use concrete in their foundations, wing walls, abutments, piers, or bridge decks (*1*). During its service life, concrete will inevitably crack. For bridge decks, this is especially noteworthy since the presence of cracks provides an avenue for the infiltration of chlorides and other harmful elements (*2*). These chemicals cause corrosion of the reinforcing steel, which compromises the integrity of the bridge structure. Identification of typical damage mechanisms and test methods for determining the extent of damage sustained by concrete bridge decks was therefore an important element of this research. In particular, the utility of non-destructive testing to accurately and rapidly assess bridge deck condition was investigated. Additionally, information was sought on specifications for new concrete mixture designs and new deck construction implemented by transportation agencies to alleviate deterioration that may be caused by poor mixture designs or improper curing techniques, for example.

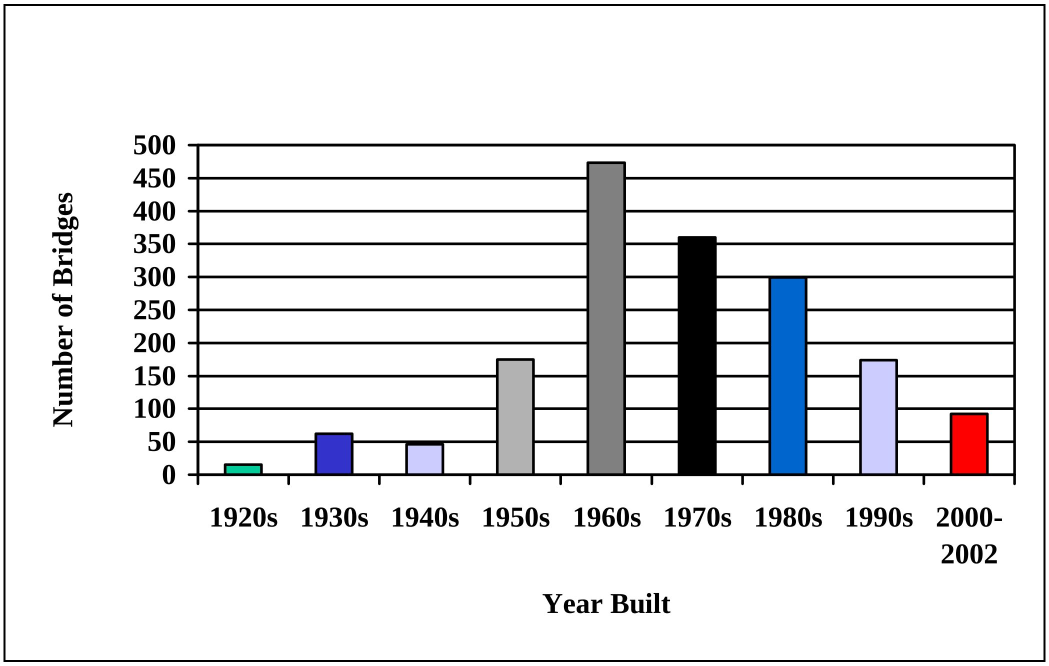

FIGURE 1.1 Bridge construction in Utah from 1920 to present.

A literature review was conducted to provide background information on the problem and to define target areas for the research. In addition, a questionnaire survey was conducted of state departments of transportation (DOTs) nationwide to determine the state of the practice for concrete bridge deck condition assessment, causes and types of distress typically governing bridge deck service life, preventative measures that efficiently mitigate deterioration, and decision-making protocols that are followed to determine whether a bridge deck should be replaced or rehabilitated. Information in this report covers deterioration mechanisms, condition assessment, rehabilitation methods, bridge management system (BMS) concepts, survey results, and conclusions.

CHAPTER 2
DETERIORATION MECHANISMS

2.1 CONCRETE DETERIORATION

The deterioration of reinforced concrete is caused by a combination of physical and chemical processes. The original concrete quality is often the major factor that affects the rate at which a reinforced concrete bridge deck deteriorates. Low quality concrete ultimately results in significant cracking, which allows for the ingress of moisture and harmful chemicals that accelerate the deterioration process. Therefore, control of the concrete mixture is essential in producing a reinforced concrete bridge deck that will meet or surpass its intended service life. This chapter discusses concrete composition and durability, corrosion of reinforcing steel, and concrete deterioration mechanisms.

2.2 CONCRETE COMPOSITION AND DURABILITY

Concrete is a mixture of aggregates and paste. The paste is comprised of cement and water. In properly mixed concrete, the paste should coat each aggregate particle and fill the void spaces between the particles. Together, the cement and water act as the "glue" that binds the aggregates into a solid, rock-like mass (*3*).

Aggregates used in a concrete mixture are divided into two categories: fine and coarse. Fine aggregates are either natural sands or crushed gravel with a particle size smaller than 0.2 inch. Coarse aggregates are comprised of particle sizes that are predominantly larger than 0.2 inch and generally between 0.375 and 1.50 inches. These aggregates are either obtained from natural sources in the desired particle size distribution, or they are crushed to finer sizes. Careful consideration must be given to the selection of aggregates since concrete quality significantly depends upon aggregate quality. Fine and coarse aggregates strongly influence the properties of hardened concrete since they comprise 60 to 75 percent of the concrete volume (*3*).

Understanding the specific causes of concrete deterioration is important to the development of sound and effective rehabilitation practices. Concrete mixture design, construction practices, and environmental factors may all affect the performance and long-term durability of concrete as discussed below (*3, 4*).

2.2.1 Concrete Mixture Design

Important properties of concrete mixtures include the water-cement ratio, cement content, and air content. The water-cement ratio of the cement paste is a key parameter controlling the quality of concrete (*1*). Simply stated, the water-cement ratio is the weight of mixing water divided by the weight of cement. Excessive amounts of water decrease the durability and compressive strength of the concrete mixture by increasing permeability and reducing bulk density. A deficiency in water content, however, can result in concrete that is not workable and in cement that is not completely hydrated (*1*). Unhydrated cement weakens concrete since the mixture depends on the complete hydration of the cement in order to produce sufficient "glue" to bind the materials together (*3*). Therefore, in certain areas of the finished concrete mass, significant strength variations can occur due to the lack of hardened paste.

Cement content is often controlled by the water-cement ratio and water content, although minimum cement requirements are often specified. The minimum cement requirement ensures that the concrete is adequately resistant to wearing, has satisfactory durability and finishability, and provides suitable appearance for vertical surfaces (*3*).

Air content provides release valves for excessive pore water pressure that may develop in the concrete due to the expansion of water upon freezing. However, only entrained air, as opposed to entrapped air, is effective in reducing the internal stress within concrete. Entrained air bubbles measure approximately 0.001 to 0.003 inch in diameter and are uniformly distributed throughout the paste (*5*). These bubbles provide a rapid escape path for liquid water displaced during the formation of ice. Entrapped air bubbles measure greater than 0.003 inch in diameter. These types of bubbles are too large and spaced too irregularly to be of any benefit in reducing freeze-thaw damage.

2.2.2 Construction Practices

Concrete construction activities such as placement, finishing, and curing practices have a strong influence on concrete durability, strength, permeability, abrasion resistance, and resistance to freezing and thawing and deicing salts. Even when the concrete mixture has been properly designed, poor construction practices can lead to poor performance.

2.2.3 Environmental Factors

Environmental effects such as temperature changes can cause cracking due to the thermal expansion and contraction of concrete. In addition, the freezing of pore water can lead to further cracking. Moisture changes cause concrete to expand and contract with gains and losses in moisture, respectively. High moisture levels can also cause greater amounts of damage during repeated freeze-thaw cycles.

2.3 CORROSION OF REINFORCING STEEL

Concrete is highly alkaline in nature and typically provides reinforcing steel with excellent protection from harmful corrosive substances. When reinforcing steel is placed in fresh concrete, a chemical reaction occurs between the steel and water in the concrete mixture that initiates corrosion of the reinforcement. This reaction causes the formation of a passive oxide layer on the surface of the reinforcement that remains stable only at pH levels above 11.5 (*6*). The ingress of chlorides and other chemicals can activate the oxide layer by causing the pH of the surrounding concrete to fall below this critical threshold, thus commencing the corrosion process.

Corrosion is an electrochemical process that is related to the flow of electrons. Individual elements may gain or lose electrons during this process. Elements that lose electrons are said to be oxidized, and those that gain electrons are said to be reduced. In the case of bridge decks, some of the elements that are involved during oxidation and reduction are iron, water, and oxygen. Together, these elements create an electrochemical cell whereby electrons are transferred and new compounds are formed (*6*).

In order for an electrochemical cell to form, two electrodes and an electrolyte solution are required (*6*). The positive electrode is called the anode, and the negative electrode is called the cathode. For steel reinforcement, the cathode and anode may form on separate bars or on the same bar. When they form on the same bar, that bar constitutes a mixed electrode. This condition may occur as a result of several possible conditions: the passive oxide layer on the reinforcement is destroyed locally, the concrete is sufficiently moist to act as the electrolyte, or the concrete cover is permeable to oxygen. Oxidation of the steel occurs at the anode. At this location, electrons are freed and flow through the steel by metallic conduction to the cathode. The electrons are then consumed in a reduction process that causes the formation of hydroxyl ions, which travel through the pore water in the concrete to the anode and combine with iron ions to form rust.

The flow of electrons between anodic and cathodic sites depends on the electrical resistivity of the concrete and the reinforcement. As the moisture content of the concrete increases, the electrical resistance of the concrete decreases. The increase in moisture content allows for completion of the electrical circuit between the anode and the cathode, consequently allowing an easier transfer of electrons. Generally, a lower concrete resistivity caused by higher moisture contents increases the probability of corrosion. The permeability of the concrete cover controls the ingress of moisture, which in sufficient quantities provides for an interconnected pore water system that can function as an electrolyte and enhance corrosion processes. In addition, excessive concrete permeability may permit the infiltration of chloride ions, which are especially detrimental in the corrosion process. Oxygen, which is an essential ingredient in the formation of rust, may also readily penetrate a pervious concrete cover (*6*).

Persistence of an electrochemical cell on reinforcing steel results in the eventual formation of rust. The density of rust is significantly lower than the steel from which it was formed. Consequently, a volume increase occurs that may be two to five times that of the parent steel (*6*). The expanding rust causes pressures that may exceed the low tensile strength of the concrete. Cracking inevitably results if the concrete cannot accommodate the excess pressures. Additionally, the structural strength of the reinforced concrete element is compromised due to the loss of cross-sectional area of the corroding reinforcement (*6*).

The rate at which corrosion occurs is influenced by the moisture content of the concrete and the rate at which oxygen migrates through the concrete to the steel (*7*). Corrosion is also heavily influenced by the degree of contact between the reinforcing bars and the surrounding concrete. Uncoated reinforcement, for example, is in direct contact with the surrounding concrete and therefore allows electrical current to be readily transmitted across the boundary. Thus, the use of uncoated reinforcement does not restrict the rate of corrosion, consequently reducing the life span of the bridge deck. However, epoxy and other reinforcement coatings disrupt the direct contact that would otherwise exist between the steel and the concrete, which weakens the corrosion current present in the deck. To maintain its efficacy, epoxy-coated reinforcement must be handled with care during construction to avoid exposure of the underlying steel. Once the coating is damaged, its efficacy is diminished, as even small defects can become gateways for damaging corrosion currents.

2.4 CONCRETE DETERIORATION MECHANISMS

Distresses on bridge decks are manifested in numerous ways and can often be used to determine the cause of the problem. The following are common types of distresses observed on bridge decks: cracking, scaling, popouts, honeycombing and air pockets, alkali-aggregate reactivity (AAR), carbonation, sulfate attack, and chloride-induced corrosion. Each type of distress is discussed in detail in this section.

2.4.1 Cracking

A crack is defined as a break without a complete separation of parts. In bridge decks, cracks are the precursors to more significant problems since they allow for the infiltration of harmful chemicals and substances. The severity of a crack is based on its length, width, and susceptibility to propagate (*1*). Several different types of cracks occur in concrete structures, including plastic shrinkage cracks, drying shrinkage cracks, settlement cracks, structural cracks, map cracks, corrosion-induced cracks, and temperature cracks (*1*).

Plastic shrinkage cracks are a result of the concrete drying too quickly in its plastic state. These types of cracks are typically wide and shallow. Also, they are in a well-defined pattern, occurring in regularly spaced intervals.

Drying shrinkage cracks are a result of the water migrating upward to the concrete surface and evaporating after the concrete has hardened. The loss of moisture causes the concrete to contract, or shrink. These types of cracks are finer and deeper than plastic shrinkage cracks. They are also randomly orientated.

Settlement cracks are a result of the settlement of hardened concrete. They can be of any orientation and width, varying from fine cracks near the reinforcement due to formwork settlement to wide cracks in supporting members caused by the settlement of the foundation.

Structural cracks are a result of discrepancies between calculated and actual stress intensities. The widths of the cracks may vary, but the orientation is well defined.

Map cracks are a closely spaced network of cracks caused by reactions between the cement paste and the aggregate. Over time, the width and quantity of the cracks increase. Two examples of reactions that cause this type of cracking are alkali-silica and alkali-carbonate reactions, which cause abnormal expansion and loss of strength in the concrete. Corrosion-induced cracks result from the corrosion of the reinforcement. The corrosion causes the steel to rust and expand. As stated earlier, the internal pressures caused by the formation of rust may exceed the comparatively low tensile strength of the concrete and cause it to crack. Typically, these cracks are manifested directly along the reinforcement and may be marked with rust stains in the vicinity. As corrosion continues, the widths of the cracks increase.

Temperature cracks are caused by the expansion and contraction of concrete as a result of temperature fluctuations. When the thermal stress exceeds the tensile strength of the concrete, cracks are initiated.

As a general rule, cracks in concrete cannot be avoided, only controlled. The most effective method of controlling cracks in a bridge deck is the use of joints. Concrete randomly cracks if appropriately spaced joints are not provided for shrinkage. The joints help isolate cracks to particular areas so they do not propagate throughout the entire concrete deck. Isolation joints, otherwise known as expansion joints, extend the full depth of the concrete section and are filled with a pre-molded filler material (*3*). They reduce cracking by permitting horizontal and vertical movement of two adjoining concrete parts. One example of an isolation joint is at the interface of a floor slab and a column; the joint separates the concrete floor from the concrete column, allowing each to move independent of the other.

Control joints, or contraction joints, are used to induce cracking in pre-determined locations as a result of shrinkage and thermal contraction (*3*). This type of joint allows for movement in the plane of the concrete section while still permitting the transfer of loads perpendicular to the plane of the section. Control joints can be constructed by grooving the concrete while it is still fresh, using forms that will allow for the formation of a joint, or sawing the concrete when it has hardened enough to endure tearing and other damage that may be caused by the saw blade. Construction joints are interfaces between concrete sections that were placed at different times (*3*). Typically, construction joints are designed and built to serve as both construction and isolation joints.

2.4.2 Scaling

Scaling is the deterioration of the upper concrete deck surface, typically the top 0.125 to 0.5 inch, and is characterized by the peeling off or flaking away of surface concrete (*1, 3, 8*). For the most part, it is a result of repeated freezing and thawing of concrete at moisture levels near saturation. Upon freezing, water held in the cement paste and the aggregate particles expands and can cause deterioration of the concrete. The effects of scaling are accentuated by the presence of deicers and salts.

Freezing and thawing is a mechanical process that requires water and cyclic cold temperatures to initiate concrete deterioration. Hydraulic and osmotic pressures in the capillaries and pores of the concrete increase as the water held in the cement paste and aggregates freezes (*3*). Hydraulic pressures are created when liquid water in concrete pores is attracted toward capillary pores where ice has started to form (*9*). When the water reaches the capillary pores, the water freezes, and the ice crystals increase in size. The capillaries dilate as a result of the increased pressure. Dilation of the capillaries continues until the tensile strength of the aggregate or paste is exceeded, at which point rupture of the voids occurs. Repeated freeze-thaw cycles lead to the eventual scaling of the concrete.

Osmotic pressures in concrete are the result of dissolved chemicals in the pore water, typically stemming from deicing salts applied to the roadway during winter maintenance operations (*9*). Increasing concentrations of salts decrease the freezing point of the pore water. Thus, at a given temperature below 32°F, part of the pore water is frozen while part remains liquid. Due to the fact that ice forms in a pure state, the salt concentration in the liquid water increases as freezing progresses. The increase in salt concentration causes a state of imbalance where water in other pores becomes attracted to the highly concentrated unfrozen water. In order to reestablish equilibrium, a subsequent migration of water begins. The pressure due to the movement of water from one pore to another increases the internal stress in the paste. The arrival of additional water in the highly concentrated pore lowers the concentration of dissolved chemicals in that pore, allowing more ice to form. The formation of ice causes an increase in stress in the capillary pores in addition to the pressure increase caused by water movement (*9*).

The use of deicing chemicals further aggravates surface scaling. First, the melting of ice by the application of salts requires energy, which is at least partially drawn out of the concrete (*6*). This loss of energy causes a rapid temperature drop at the surface of the concrete (*6*). The thermal shock from the temperature drop can lead to cracking and eventual scaling. Second, melted runoff containing deicing salts can infiltrate the surface of the concrete, lowering the freezing point of the water near the surface. Consequently, given a sustained freezing temperature, water in the uppermost section of the concrete may not freeze until after the water in underlying sections freezes. This causes differential expansion of the surface layers, which leads to scaling (*6*).

One of the greatest advances in concrete technology was the development of air-entrained concrete. It is produced by using an air-entraining agent or air-entraining cement. These bubbles are uniformly distributed in the cement paste and become part of the hardened concrete matrix. Entrained air cavities serve as pressure relief valves for migrating water by providing closely spaced, empty chambers in which water may accumulate, thus relieving excessive hydraulic and osmotic pressures and preventing deterioration of the concrete (*3*). When the temperature rises enough for the ice to thaw, the water returns to the pores due to capillary action and pressure from compressed air bubbles. The air-entrained bubbles are then available for the next freezing and thawing cycle.

A surface treatment may be applied to protect concrete surfaces from further deterioration if scaling has already caused damage to the concrete surface. For example, breathable, penetrating sealants include boiled linseed oil or methacrylate (*3*). The surface treatment retards further deterioration by sealing avenues that would otherwise permit the ingress of water and harmful agents.

2.4.3 Popouts

Popouts are conically shaped depressions that are associated with the removal or rupturing of aggregate particles near the concrete surface. Popouts are particularly related to frost-susceptible aggregates. Typically, fragments of a shattered aggregate particle will be found at the bottom of the cone while a piece of the aggregate particle adheres to the popout cone (*1*).

Popouts are caused by coarse aggregates that are susceptible to freezing and thawing conditions (*3*). Freeze-thaw resistance of aggregates is based on porosity, absorption, permeability, and pore structure. In some cases, aggregates absorb so much water that they become critically saturated. As water in the aggregate freezes, hydraulic and osmotic pressures escalate as described in the previous section. The formation of ice crystals in the aggregate pores may then generate excess pore water pressures. If the excess water is not readily transmitted to the surrounding cement paste, the aggregate ruptures when the osmotic and hydraulic pressures exceed its tensile strength.

Resistance to popouts can be achieved through the use of air-entraining agents, low water-cement ratios, and high cement contents (*3*). As discussed in the previous section, the use of entrained air aids in dispersing capillary water from the aggregate to the cement paste. Low water-cement ratios reduce the probability that aggregate particles will approach moisture contents near saturation by decreasing the permeability of the concrete mixture. Finally, higher cement contents generally yield greater concrete durability and increase resistance to freeze-thaw effects by increasing the concrete strength.

Coarse aggregates are more likely to cause popouts and deterioration because their porosity values and pore sizes are greater than those in fine aggregates (*3*). Medium-sized pores are more susceptible to becoming saturated and causing popouts. Large pore sizes do not usually become saturated, and water in aggregates with fine pore sizes does not readily freeze.

2.4.4 Honeycombing and Air Pockets

Honeycombing and air pockets are problems that can be avoided by proper consolidation of the concrete in the forms at the time of construction (*1, 3*). Consolidation ensures that fresh concrete is properly compacted so that the reinforcement and other embedded items are completely encased. In addition, consolidation minimizes the presence of entrapped air voids, thereby reducing settlement and subsequent cracking.

Consolidation is typically accomplished through the use of internal vibrators, which cause the friction between aggregate particles to be temporarily reduced to the point that the concrete mixture is liquefied (*3*). Since deck slabs are heavily reinforced, internal vibrators ensure that the fresh concrete completely envelops the reinforcement and allows air voids to flow more easily to the surface (*3*).

2.4.5 Alkali-Aggregate Reactivity

AAR mainly includes alkali-silica and alkali-carbonate reactions. Alkali-silica reaction (ASR) is a result of alkalis, such as sodium, potassium and lithium, reacting with the silica in concrete (*10*). Alkalis are weakly bonded to hydroxyl ions and are introduced into concrete by way of aggregate, cement, mix water, admixtures, or pozzolans, with cement being the most common source. The weak bonding allows alkali compounds to quickly dissolve in solution, where they become free ions (*10*). The increasing concentration of free hydroxl ions causes the pH of the solution to increase. For example, low alkali concrete has a pH of 12.7 to 13.1 compared to high alkali concrete, which has a pH between 13.5 and 13.9 (*10*). Silica in chemically unstable aggregates becomes soluble in alkaline solutions and reacts with the alkalis to form an alkali-metal-ion hydrous silicate gel in aggregate cracks (*10*). The gel absorbs water from the surrounding concrete, which causes it to expand, leading to map cracking or popouts in advanced cases (*5*).

ASR depends primarily on alkali concentration and aggregate reactivity (*10*). When aggregate particles are chemically stable, they do not react negatively with the alkalis that may be present in the cement (*3*). However, chemically unstable aggregate particles containing silica and carbonates may react with the alkali ions in the cement. A warm, moist environment further accelerates the reaction. Sufficiently high alkali concentrations are necessary to increase the pH of the concrete above the threshold required to initiate the reaction. The pH threshold is determined by the reactivity level of the aggregate. For example, an aggregate with low reactivity requires a higher pH than an aggregate with a higher level of reactivity to cause ASR. Once ASR has been initiated, it will continue until the reactive silica has been exhausted, the solution pH has been reduced below the threshold value, or the moisture supply has been depleted (*3*).

Alkali-carbonate reactivity (ACR) is caused by cement alkalis reacting with aggregate particles containing carbonates. Most carbonate rocks react with cement products, but very rarely do these reactions form expansive products. Expansion caused by ACR is a result of fine-grained dolomitic limestones that contain calcite, clay, silt, or dolomite rhombs (*3, 6*). Since fine-grained dolomitic limestones are very rarely used in concrete mixtures, expansion due to ACR is of little concern.

Studies have shown that increasing entrained air content reduces concrete deterioration due to AAR (*3*). The air bubbles help to alleviate the expansive stresses caused by the reaction and thus retard the deterioration process. In addition, certain mineral admixtures, such as silica fume, fly ash, and blast-furnace slag, can combine with alkalis in a pozzolanic reaction, thereby reducing the amount of alkalis that are available for harmful reactions (*3*).

In summary, the extent of deterioration caused by AAR is influenced by the amount, type, and particle size of the reactive aggregate. Additionally, keeping the concrete as dry as possible can significantly reduce AAR. In fact, reactivity can be essentially stopped if the relative humidity of the concrete is held below 80 percent (*3*). Minimizing moisture contents in concrete bridge decks can be achieved through the use of proper drainage.

2.4.6 Carbonation

Carbonation is caused by the penetration of air into concrete. In the presence of moisture, carbon dioxide from the air reacts with hydroxides, such as calcium hydroxide, to form calcium carbonate and carbonic acid (*3*). Both of these products can lower the alkalinity of normal concrete below the pH level at which the protective oxide film on the reinforcing steel is stable. Without the protective oxide film, the reinforcing steel begins to corrode. As discussed earlier, the resulting rust causes expansion, cracking, and spalling. Carbonation also increases drying shrinkage in concrete, thereby initiating cracking that may permit the intrusion of harmful chemicals and other substances (*3*). Carbonation usually does not penetrate more than 0.04 inch per year in high quality concrete but can occur more quickly in dry environments (*1*).

Carbonation in concrete is exacerbated by a number of factors, including high water-cement ratios, low cement contents, short curing periods, low strengths, and highly permeable paste (*3*). The degree of carbonation can be determined by measuring the amount of calcium carbonate through a petrographic analysis described in American Society for Testing and Materials (ASTM) C 856, Standard Practice for Petrographic Examination of Hardened Concrete. Depth of carbonation is measured with a phenolphthalein color test, which determines the pH of concrete (*3*). In poor-quality, carbonated concrete (pH of 9.0 to 9.5), the phenolphthalein solution remains colorless. In high-quality, non-carbonated concrete (pH of 12.5 or greater), the solution turns red or purple. Generally, the depth of carbonation is of no significance in high-quality, well-cured concrete.

2.4.7 Sulfate Attack

Sulfate attack is a common form of concrete deterioration where sulfate ions from an external source attack components of the cement paste (*11, 12*). The attack occurs when sulfate-contaminated water comes in contact with concrete (*11, 12*). Sulfates are naturally occurring mineral salt compounds and may exist, for example, in seawater, sewage water, swamp water, or ground water (*11, 13*). In particular, previous inundation of soil deposits by seawater can yield high levels of gypsum, a common form of calcium sulfate that is found in many areas throughout the Unites States and around the world (*11, 13*). In addition, soil deposits of marine origin are rich with other sulfates harmful to concrete, such as sodium and magnesium sulfate (*13*). When these soils become saturated by way of irrigation or rainfall, the sulfates dissolve into the pore water and may eventually contaminate adjacent infrastructure elements (*13*).

Distress caused by sulfate attack can occur when concrete is exposed to salts in solution above a critical concentration (*6*). Failure of the concrete occurs through expansion and deterioration of the cement paste. Tricalcium aluminate in the cement can react with sulfate ions to form an expansive crystalline product called ettringite (*6, 11, 12*). Ettringite crystals are extremely expansive and can develop pressures up to 35 ksi within the concrete (*14*). Expansion of the concrete due to the formation of ettringite is usually accompanied by strength loss resulting from chemical deterioration of the cement paste and damage to the aggregate-paste interface bond. Deterioration caused by sulfate attack is often manifested by hairline cracks, scaling, or by a white, powdery stain (*11, 12*).

2.4.8 Chloride-Induced Corrosion

Reinforced concrete bridge decks exposed to deicing salts and seawater are vulnerable to the ingress of chloride ions (*15*). High chloride content in concrete encourages corrosion of steel reinforcement by destroying the passive protective layer on the reinforcement (*3, 16*). Deterioration is usually manifested by pitting in the reinforcing bar and a significant reduction in cross-sectional area at the same location (*16*). This process is therefore known as pitting corrosion (*16*). Occasionally, a reinforcing bar may be deteriorated through its entire cross-section (*16*).

Chloride ions in concrete exist in two forms, combined chlorides and free chlorides (*16*). Combined chlorides result from the participation of chloride ions in the cement hydration process. Free chlorides are chemically unbound and can be found in the capillary pore water of concrete. If chlorides are introduced into concrete at the time of mixing, approximately 90 percent combine to form harmless complexes, while the remaining 10 percent remain as free chloride ions (*16*). However, the ratio of free chlorides to combined chlorides increases as the concrete cures. The application of typical deicing salts, such as sodium, calcium, or magnesium chloride, causes ice and snow to melt, creating a highly concentrated chloride solution. The solution can infiltrate the surface of the roadway and introduce additional chlorides into the hardened concrete. Approximately 50 percent of the chlorides combine with other elements, while the other 50 percent remain free (*16*). The free chlorides are the principal cause of breakdown in the passive oxide film on the reinforcement. Therefore, the presence of free chlorides has a large influence on the corrosion of reinforcing steel.

Although many studies have investigated the threshold value of chloride concentration that initiates corrosion, no definitive value has been widely accepted (*15*). Typically, chloride levels are classified in terms of corrosion risk, since even low levels of chlorides may initiate corrosion under certain conditions (*16*). Conversely, high levels of chloride concentrations may not initiate corrosion (*16*). For example, even if large amounts of chlorides are present in the concrete, the reinforcing steel will not corrode unless sufficient moisture is present (*17*). Once corrosion of the steel begins, the extent to which the reinforcement corrodes depends heavily on time and polarization effects (*18*).

Polarization can be measured using a potential difference electrode. In the case of corroding reinforcement, potential differences occur between the anode and the electrolyte, as well as between the cathode and the electrolyte (*6*). The potential values for the anodic and cathodic sites are separated by an intermediate value, which is exactly halfway between the anodic and cathodic potential values. Corrosion of the reinforcing steel occurs when the rates of anodic and cathodic reactions are equal (*6*). Steel will not corrode as long as the anodic and cathodic potentials remain distant from the intermediate value, which can be accomplished in part by preventing the intrusion of chlorides into concrete. Chlorides cause the potentials of anodic and cathodic sites to draw closer to the intermediate value.

The effects of chlorides on reinforcing steel may be minimized through the use of a corrosion mitigation method. These methods include cathodic protection, metallic coatings, epoxy coatings, alternative cementitious materials (pozzolans), inhibitors, and increased concrete cover (*19, 20*). Additional information about these methods is given in Chapter 4.

CHAPTER 3
CONDITION ASSESSMENT

3.1 ASSESSMENT METHODS

A condition survey of a concrete bridge deck provides critical information about the types and extent of distress present on the deck. Additionally, a survey greatly aids in the development of bridge deck preservation strategies, as well as in predicting the remaining service life of a bridge deck. The following is a list of condition assessment methods and equipment that will be discussed in this chapter: visual inspection, coring, chain dragging, hammer sounding, ground-penetrating radar (GPR), infrared thermography, resistivity testing, impact-echo testing, ultrasonic testing, chloride concentration testing, petrographic analysis, penetration dyes, Schmidt rebound hammer, half-cell potential testing, rapid chloride permeability, skid resistance testing, and corrosion sensors.

3.2 VISUAL INSPECTION

Visual inspection is the first step in assessing the condition of a bridge deck (*1, 21*). During inspection, the type and extent of deterioration are documented, and photographs may be taken to document any significant damage. Figure 3.1 displays the use of heavy equipment occasionally required to conduct thorough visual inspections (*22*). For the most part, however, the top surfaces and undersides of bridge decks are easily accessible so a satisfactory condition assessment can be readily performed.

For bridge decks overlaid with an asphalt wearing surface, the condition of the overlay surface may not give an adequate representation of the actual deck. For example, when a waterproofing membrane is used, the concrete deck may be in excellent condition while the wearing surface may exhibit extensive deterioration (*1*). Conversely, when bridge decks are not lined with a waterproofing membrane, the asphalt-wearing surface may be in good condition while the concrete deck is heavily deteriorated (*1*). With respect to visual inspections, the removal of the wearing surface is essential to avoid inaccurate inferences about the concrete bridge deck. However, other forms of non-destructive testing may not require the overall removal of the wearing surface.

When conducting a visual inspection, the inspector may identify concrete stains, radial cracks, or localized depressions in the deck as possible factors for the presence of deck deterioration (*1*). Another good indicator of deck deterioration is the distress manifested on the underside of the deck. The bridge should also be inspected for damage caused by collisions, excessive deflections, vibrations, or deformations because the deck near or at the location of these occurrences may have suffered accelerated deterioration.

Cracks are the precursors of deck deterioration and are the most important feature to document when conducting a visual bridge deck assessment. Cracks should be identified by their size, location, and orientation (*1*). The depth of a crack is also important, especially if the crack intersects the reinforcing steel, because such a case accentuates the risk of chloride infiltration, sulfate attack, and freeze-thaw deterioration. However, the depth of a crack cannot be measured unless cores are taken or the crack propagates through the entire deck cross-section. The orientation of a deck crack is identified as longitudinal, transverse, diagonal, or random (*1*).

FIGURE 3.1 Visual inspection of a reinforced concrete bridge deck (*22*).

The measurement of crack widths is not typically performed. However, individual circumstances may require a measurement to be performed, which can be accomplished with any of several pieces of equipment. Cracks that are easily detected with the naked eye can be measured with a crack width comparator card (*19*). For extremely small cracks, a small, hand-held crack comparator microscope can be used to measure to a resolution of 0.001 inch (*1*). Crack widths are categorized into four general groups: hairline, narrow, medium, or wide (*1*). Hairline cracks are those that are less than 0.004 inch wide. Narrow cracks have a width between 0.004 and 0.01 inch. Medium cracks have a width range of 0.01 to 0.03 inch, and wide cracks are those with a width greater than 0.03 inch.

Crack propagation can be monitored with several different instruments. One instrument is called a crack movement indicator, which attaches directly to both sides of a crack and measures translation and rotation (*1*). A crack movement indicator is fairly inexpensive and appropriate for long-term use. Linear variable differential transformers (LVDTs) and extensometers are also used to monitor crack propagation (*1*). However, these instruments only measure movement perpendicular to the crack and must be attached to a data-collecting unit.

Scaling is another distress that needs to be considered in visual inspections and should be reported with respect to its location and severity (*1*). There are four general categories of scaling: light, medium, heavy, and severe (*1*). Light scaling is when the top 0 to 0.25 inch of surface concrete has flaked off without exposing any coarse aggregate. Medium scaling is characterized by the flaking off of the top 0.25 to 0.5 inch of surface concrete along with exposure of coarse aggregates. Scaling is considered heavy when the top 0.5 to 1 inch of surface concrete has flaked off with coarse aggregates projecting from the surface. Severe scaling is distinguished by the flaking off of over 1 inch of concrete with a loss of coarse aggregate particles.

Rust stains are often good indicators of reinforcing steel corrosion (*1*). Sometimes, however, ferrous sulfide inclusions in the aggregate or the rusting of form ties may be mistaken for the corrosion of reinforcing steel.

The exhibition of spalls and delaminations on bridge decks is especially problematic. Spalls are easily detected since the deteriorated concrete and exposed reinforcing bars are difficult to hide, but delaminations are not easily detected through visual inspection unless they are relatively shallow and accompanied by a discoloration of the bridge deck (*1*).

One disadvantage of the visual inspection method is that it is subjective and may not provide an accurate assessment of the bridge deck condition (*1, 21*). Furthermore, the method is slow, qualitative, and potentially hazardous for the inspector (*23*).

3.3 CORING

When areas of distress are evident on a concrete bridge deck, investigation of the cause of the distress may be necessary to decide the proper course for repairs. Extracting a concrete core and performing a simple visual inspection or a petrographic analysis is an excellent way of determining the condition of the concrete. Core drilling is performed to extract cylindrical samples from concrete elements, as shown in Figure 3.2 (*24*). The procedure is described in ASTM C 42, Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete. Holes up to 18 inches in diameter can be drilled to virtually any depth using diamond-impregnated bits attached to a core barrel (*25*). Core specimens are taken perpendicular to the concrete surface in the area of the manifested distress (*25*). For visual inspection, core specimens of any size can be obtained so long as they are large enough to allow a thorough analysis with the naked eye. If taken for petrographic analyses, cores should be sampled in accordance with ASTM C 856.

Core analysis is a reliable way of determining the condition of in-place concrete. No other method can assess the condition of a bridge deck with more accuracy than extracting cores and performing a few simple laboratory tests on them. However, the laboratory tests require trained experts in addition to road crews to extract the cores from the bridge deck. Lane closures are required during the extraction process and while the replacement concrete cures. Also, the road crew members are at risk while they labor in the vicinity of moving vehicles.

FIGURE 3.2 Concrete core extraction (24).

3.4 CHAIN DRAGGING

Chain dragging is the process of dragging a steel chain across a bridge deck surface and listening to changes in the acoustic response, as shown in Figure 3.3 (*26, 27*). Good quality concrete will produce a clear ringing sound (*28*). When delaminations are present, the acoustic response is a dull, hollow sound (*17, 28*). The operator should not automatically assume that the presence of delaminated concrete is a manifestation of corroding reinforcement, however, as delaminations can be a result of different deterioration processes, such as freezing and thawing (*17*). Chain dragging can locate delaminations but is not a reliable method for directly identifying areas of corroding reinforcement (*17*).

FIGURE 3.3 Chain dragging on a reinforced concrete slab (*27*).

After visual inspection, chain dragging is the most widely used method in the United States for assessing the condition of bridge decks. Several factors can be attributed to the popularity of this method. Chain dragging is relatively simple, economical, and quick to perform (*1*). However, chain dragging does require an experienced technician to detect meaningful changes in the acoustic response, thereby introducing subjectivity into the test, as different operators will hear the same sound differently (*26, 28*). To one technician the acoustic response may sound clear, while to another the sound may seem dull. Oftentimes, the accuracy of chain-drag mapping is affected by technician fatigue since the constant noise tends to reduce the technician's sensitivity to changes in the acoustic response (*1*). The fatigue of the technician is thus an important source of variability in these tests (*26, 28*). Although chain dragging provides valuable information about the presence and location of delaminations, it can only do so after delaminations have progressed to the point where major rehabilitation is required (*29*).

The accuracy of chain-drag surveys on asphalt-covered decks is unsatisfactory (*1*). However, the relatively low cost of chain dragging and the speed at which it can be performed make it useful as a preliminary method to identify irregular areas that may be more thoroughly investigated by other techniques (*1*).

Two steps that may be taken to increase the accuracy of chain-drag surveys are personal training and data documentation. Technicians need training to become more attuned to the different types of acoustic responses they may encounter, while more accurate and detailed documentation will help eliminate some subjectivity and facilitate more standardized test results (*26*). The documentation should include a detailed procedure of the testing method, as well as specific information obtained during testing (*26*).

In one study, a chain-drag instrument was developed to facilitate real-time analysis of recorded responses, collection of processed data, and post-processing of the data for closer evaluation of the signals (*28*). The instrument consisted of a hand-pushed cart on which several devices were mounted to record acoustical readings, as shown in Figure 3.4 (*30*). The devices included a microphone, an amplifier, a data collector, a computer to process the data, a power source, a set of headphones for the technician to use in monitoring signal responses, and several chains. The chains were mounted to the underside of the cart for dragging along the concrete surface. A microphone closely attached to the chains recorded the acoustic response of the chains on the concrete. The measured response from the microphone was sent to the data acquisition system, where the signal was then forwarded to the technician's headphones for monitoring.

FIGURE 3.4 Chain-drag cart (*30*).

3.5 HAMMER SOUNDING

Hammer sounding is an acoustic impact method that technicians can use to detect delaminations in concrete by striking the concrete with a hammer and listening to the response (*31*). An example is given in Figure 3.5 (*32*). The method is similar to chain dragging in that it uses acoustics to assess the condition of a bridge deck (*28*). The same limitations that apply to chain dragging apply to hammer sounding. The results are affected by the subjective judgment and hearing sense of the technician (*1*, *31*). However, hammer sounding is slower and more tedious since only small areas of concrete can be analyzed at one time (*1*). In some cases, technicians have been known to use an iron bar dropped on its end in order to remain upright while performing the test (*1*). The iron bar serves as a wave-conducting device to project acoustic responses up the bar into the vicinity of the technician's ear.

FIGURE 3.5 Hammer sounding inspection of a concrete bridge (*32*).

3.6 GROUND-PENETRATING RADAR

GPR is a geophysical method that can be used to locate corroded reinforcing steel, contaminated concrete, inadequate concrete cover, delaminations, or separating cracks (*33*). This method has proven effective on concrete bridge decks since GPR can be utilized to map subsurface features at relatively shallow depths (*34*).

The GPR system uses electromagnetic (EM) waves in the frequency band of 1.0 to 2.5 GHz to map subsurface characteristics of bridge decks (*35*). Radar waves are emitted into the bridge deck from a surface antenna that may be pulled along the ground or mounted to a vehicle (*21, 36, 37*). When radar waves traveling through the bridge deck come in contact with an electrical interface, such as a boundary between air and concrete, for example, they are transmitted or reflected to various degrees depending on the dielectric contrast between the two materials forming the interface. Typically, a small portion of the radar wave is reflected back to the surface antenna, while the remainder of the wave continues through the bridge deck. The travel times and amplitudes of reflected radar waves are recorded in a digital data collector for post-processing (*21, 36, 37*). Figure 3.6 is a schematic of a typical GPR system (*1*).

The depths to which a GPR unit can be used effectively and accurately vary with the electrical conductivity of the surface and subsurface material (*34*). GPR waves can reach depths of 100 feet in low-conductivity materials such as dry sand or granite (*38*). However, materials with a high conductivity such as clays or shale attenuate GPR waves, therefore reducing the depth of penetration to typically less than 3 feet (*38*). Generally, the depth of penetration decreases with increasing electrical conductivity (*34*).

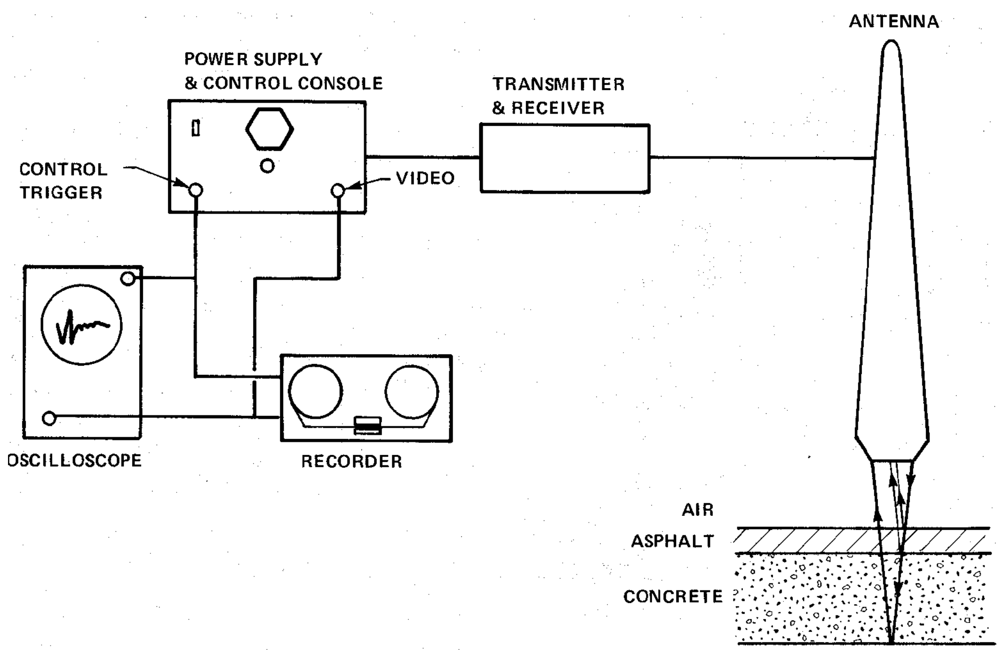

FIGURE 3.6 Schematic of a GPR system (*1*).

Since concrete is primarily composed of sand and gravel, which both have low electrical conductivity values, a concrete bridge deck can be an ideal environment for GPR surveys (*34*). Contaminated and damaged concrete causes an attenuation of the radar signal as the signal travels through the bridge deck and is reflected back from the damaged areas (*37*). Bridge decks overlaid with asphalt do not necessarily pose a problem for GPR mapping since asphalt consists principally of aggregate. GPR units are also capable of identifying concrete with high chloride and moisture contents since these factors produce highly variable reflections of the radar wave at the overlay-deck boundary (*37*).

The type of antenna used also affects the quality of GPR surveys, since the resolution of a GPR unit is directly proportional to the operating frequency (*38*). Antennas with low frequencies, 25 to 200 MHz, can reach depths of 100 feet but with low resolution (*38*). These types of antennas are generally used to locate large objects in the ground, such as sinkholes. Antennas with high frequencies, 300 MHz to 1 GHz, have a higher resolution but are limited to depths of about 30 feet in ideal conditions (*38*). In a bridge deck survey, higher frequencies are necessary to achieve the increased resolution required to identify smaller objects such as reinforcing steel. A study conducted on a bridge deck in Finland demonstrated that the use of a GPR antenna with a frequency of 1.0 GHz resulted in good radar images without any data corrections (*35*).

GPR offers many appealing advantages for assessing the condition of bridge decks (*34*). First, a GPR survey can reasonably provide an evaluation of the entire bridge deck with accuracy to within 3 to 15 percent of conventional core samples (*39*). Second, GPR decreases the safety risk to bridge inspectors by minimizing their exposure to traffic (*39*). In particular, the use of vehicle-mounted GPR units eliminates the need for lane closures and core patching and thus reduces traffic congestion as well. GPR units pulled or dragged by hand may necessitate a lane closure for relatively short periods of time, but the time that road crews are exposed to dangerous traffic conditions is still less than with other condition assessment methods. Third, a GPR survey is fast and efficient (*39*). Vehicle-mounted units can travel at highway speeds and survey up to 200 miles per day. Manually dragged units require more time to survey a bridge deck than vehicle-mounted units but require significantly less time than taking cores. Last, the method is non-destructive (*34*). GPR is an external unit that does not require preliminary surface preparation or any finishing reparations; therefore, labor and material costs to prepare and repair the deck are not required.

Two GPR systems that assess the condition of bridge decks with more accuracy and rapidity than previous systems were recently developed at Lawrence Livermore National Laboratory. The first system is called HERMES (High-Speed Electromagnetic Roadway Mapping and Evaluation System) (*26, 40, 41*). The distinguishing feature of HERMES is an array of 64 antenna modules mounted on a portable trailer, as shown in Figure 3.7 (*42*). The trailer houses a computer workstation, a data collector, and an antenna array. As the trailer is towed down the roadway, the antenna array captures data at freeway speeds and reproduces it in three-dimensional images.

The other system is called PERES (Precision Electromagnetic Roadway Evaluation System) (*43, 44*). Unlike HERMES, PERES consists of a single transceiver mounted on a robotic cart, as shown in Figure 3.8, and the cart moves significantly slower than freeway speeds (*45*). PERES is also capable of producing high quality three-dimensional images.

FIGURE 3.7 Antenna array for the HERMES GPR unit (*42*).

FIGURE 3.8 PERES GPR unit (*45*).

HERMES and PERES have the potential to improve the quality of bridge inspections, notwithstanding some limitations. For example, both systems have the capability of identifying reinforcing steel but lack the ability to produce images of typical delamination cracks (*46*). Several states have recently reviewed the limitations of the HERMES system and have made recommendations on how to improve its effectiveness. These recommendations are being incorporated into a HERMES II system by researchers at Lawrence Livermore Laboratory.

3.7 INFRARED THERMOGRAPHY

Infrared thermography uses temperature differentials, typically caused by solar radiation, to detect delaminations (*37*). Concrete is a good conductor of heat compared to air, and as a result substantial thermal gradients form as delaminated concrete experiences heating and cooling cycles (*1*). Delaminations minimize heat transfer through the deck because of the insulating air space between the separated layers of concrete, causing the concrete layer above the delamination to become hotter than intact concrete, in which heat is transferred throughout the entire cross-section (*1, 37*). Thus, during times of heating, the surface temperature of a delamination is higher than that of surrounding intact concrete, as shown in Figure 3.9 (*1, 37, 47*). Similarly, in the evening when heat is being discharged from the concrete, the surface temperature of delaminations is lower than that of the adjacent concrete (*1, 37*).

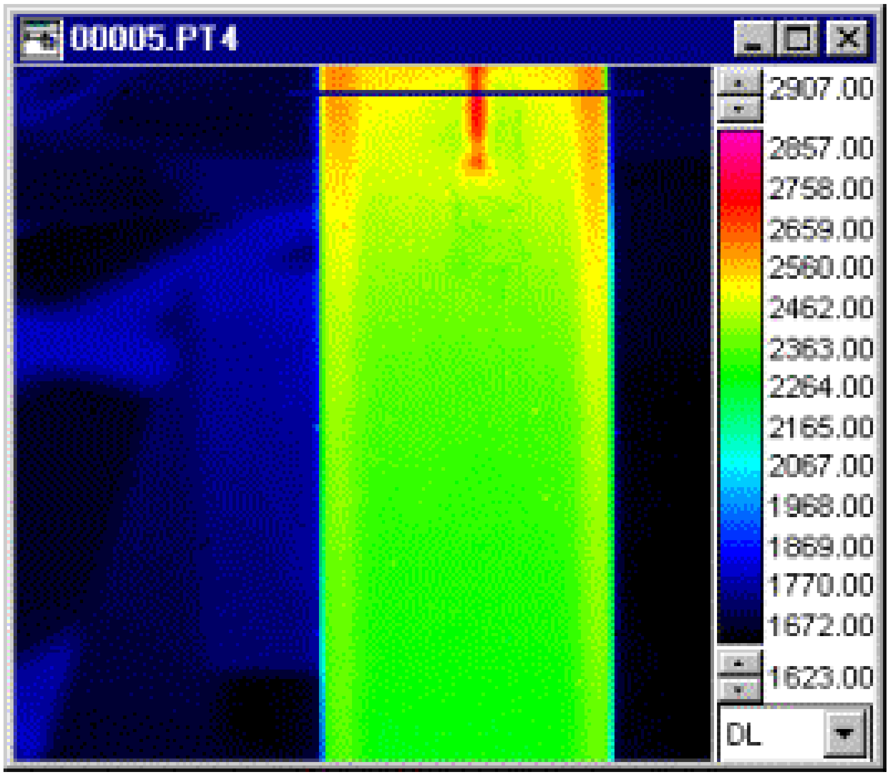

FIGURE 3.9 Infrared scan of a cracked surface (*47*).

Additionally, researchers have determined that larger temperature differences are associated with shallow delaminations than with deeper ones (*1*). Such temperature differences are the basis for detecting delaminations using infrared thermography. For example, studies have shown that the surface temperature difference between delaminated and intact sections of deck can be as large as 8.1°F under the summer sun and without the presence of wind or clouds (*1*).

A thermographic scan is performed with sensitive infrared equipment. The components of a thermographic system include an infrared scanner, a control unit, a battery pack, and a display screen, as shown in Figure 3.10 (*1*, *48*). The system receives infrared data from the scanner and produces a two-dimensional image on the display screen (*37*). Images can be designated to appear in black and white or in color. Black and white images are much easier to interpret and are more suitable for investigations of structural integrity (*1*). These images can be documented on a photographic plate or videotape (*1*).

FIGURE 3.10 Infrared thermograpy camera (*48*).

Like other methods, infrared thermography does have a few limitations. For example, the scanner is sensitive to both infrared radiation emitted from the deck and solar radiation reflected onto or diverted from the deck (*37, 49*). Therefore, the scanner is sensitive to glare from passing vehicles, shadows from fixed overhead structures, cloud cover, and other objects that may either reflect or divert solar radiation. In addition, wind can influence the results of a thermographic scan since it causes momentary variations in deck surface temperature (*1, 49*). Also, when moisture is present on the bridge deck, a thermographic scan should not be performed because of the high emissivity of water, where emissivity is the relative power of a surface to emit heat by radiation (*1, 49*).

The results of a thermographic scan can oftentimes be inconclusive. For instance, a positive result from a thermographic scan, which is manifest as a temperature difference between delaminated and intact sections of concrete, implies that deterioration is evident in the bridge deck (*1*). A negative result, however, suggests one of two possibilities: either there is no deterioration in the deck or the thermographic scan was unable to detect the deterioration. Although infrared thermography can be used on asphalt-covered decks, increasing asphalt overlay thickness leads to decreasing sensitivity of the method to the presence of delaminations in the underlying concrete (*1*).

Several different types of infrared thermography systems are available. They range from hand-held scanners to truck- and helicopter-mounted systems (*1*). For a typical bridge deck, truck-mounted systems yield the most encouraging results based on speed, accuracy, and definition (*1*). The system is mounted on a truck platform that is 13 to 20 feet above the bridge deck and enables lane-width scans in a single pass. Although the speed of the truck-mounted system is quite reasonable, the accuracy is no better than chain dragging (*1*).

Despite the limitations infrared thermography exhibits, the method is still ideal for rapid deck scanning. Infrared thermography can be used to quickly assess the condition of the slab and then determine if a more detailed evaluation is necessary (*1*).

3.8 RESISTIVITY TESTING

Resistivity testing is a method that uses electrical resistance to evaluate the quality of reinforced concrete. The technique is different than methods already discussed because it measures the likelihood of the reinforcing steel to corrode rather than the amount of distress that has already occurred due to corrosion. The method assumes that the electrical resistance of a dielectric material is a measure of its watertightness (*1*). In a porous media such as concrete, measurements of electrical conductivity can be used to determine resistivity, as shown in Equation 1 (*1*):

$$\rho = \frac{1}{\sigma} \tag{1}$$

where ρ = resistivity, ohm-in

σ = electrical conductivity, Siemen/in

Electrical conductivity is a measure of the ability of a material to sustain long-term current flow and depends on both the porosity and water content of the medium. For example, very porous concrete at high degrees of saturation has a higher hydraulic conductivity than denser concrete at lower water contents. Higher hydraulic conductivity allows for soluble ions from deicing salts and other sources to more easily infiltrate the porous concrete. Consequently, the rate of corrosion especially increases as chloride ions migrate at faster rates toward the reinforcing steel and accumulate in higher concentrations within the concrete. Because electrical conductivity is a direct measure of the flow of ions in a material, it may be used to estimate the corrosion potential of reinforcing steel in a concrete bridge deck.

The resistivity of concrete is commonly determined using the four-electrode method, as shown in Figure 3.11 (*1*). To conduct the test, four contact points (electrodes) are placed at a depth of 0.25 inch into the concrete. Care must be taken to ensure that the contact points (electrodes) are equally spaced and in a straight line. The test is performed by passing an alternating current between the outer electrodes. The inner electrodes detect the resulting current flow in the concrete and measure the potential difference between them. If the electrode spacing is too small, the entire current will flow through the surface layers and yield data that are not representative of the entire slab thickness. To avoid this problem, the electrodes should be inserted deeper into the concrete or spaced farther apart. The resistivity is then calculated using Equation 2 (*1*):

$$\rho = \frac{2 \cdot \pi \cdot a \cdot E}{I} \tag{2}$$

where ρ = resistivity, ohm-in

a = electrode spacing, in

E = potential difference between inner electrodes, V

I = current flow between the outer electrodes, A

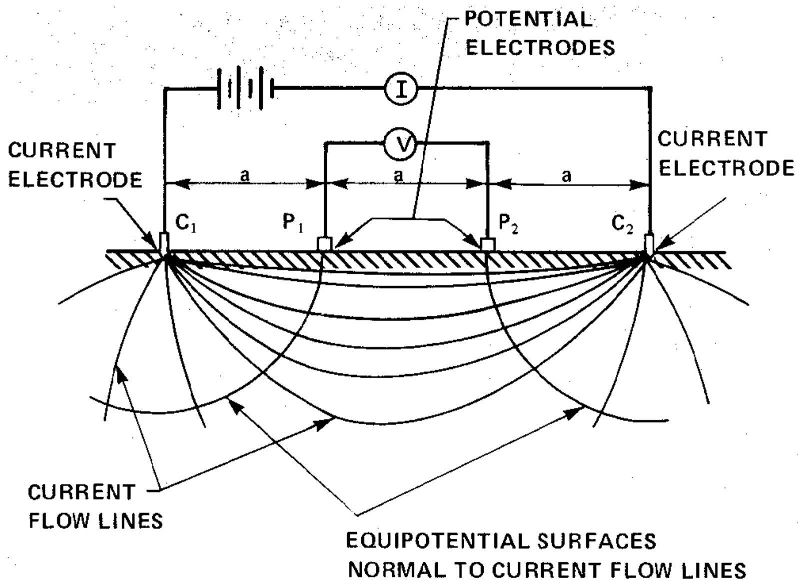

FIGURE 3.11 Four-electrode method for resistivity measurement (*1*).

Tests have been performed to investigate the resistivity of concrete in various conditions. Moist concrete typically displays a resistivity of 3900 ohm-in, while oven-dried concrete exhibits a resistivity of 9400 Mohm-in (*1*). Data from a study in Great Britain indicate that corrosion is almost certain to occur when resistivity measurements are less than 2,000 ohm-in (*1*). When resistivity measurements are between 2,000 and 4,700 ohm-in, corrosion is probable (*1*). The research results suggest that corrosion is unlikely to occur when resistivity measurements are in excess of 4,700 ohm-in (*1*). Another study performed in Great Britain indicates that corrosion is unlikely to occur when the resistivity exceeds 7,900 ohm-in (*1*). In addition, the study states that resistivity values between 2,000 and 3,900 ohm-in are needed to induce corrosion (*1*). While low levels of resistivity may occur due to the presence of diverse types of ions in the pore water, these suggested threshold values presume that the increased electrical conductivity stems from the presence of sufficient chloride concentrations to induce corrosion of the reinforcing steel (*1*).

While resistivity testing shows promise as an effective non-destructive method, it lacks the efficiency to be used as a primary source of detecting bridge deck deterioration (*1*). One deficiency of the method is the absence of an established resistivity standard to which measurements can be compared. That is, although numerous suggestions have been reported, a consensus has not yet been reached regarding appropriate threshold resistivity values. Further research is needed to establish levels of resistivity that are reliably linked to corrosion potential and occurrence. Another deficiency is that the resistivity of concrete is most sensitive to near-surface conditions rather than to conditions in the vicinity of the reinforcement (*1*). Therefore, resistivity testing cannot be used as a primary testing method; it can, however, be used to provide information to supplement other testing methods (*1*).

3.9 IMPACT-ECHO TESTING

The impact-echo method is a non-destructive evaluation technique described in ASTM C 1383, Standard Test Method for Measuring the P-Wave Speed and the Thickness of Concrete Plates Using the Impact-Echo Method. Based on the propagation of stress waves that are reflected by internal flaws and external surfaces, the method has been used in a variety of ways to assess the condition and to measure the thickness of concrete decks (*26, 29, 50, 51*). The impact-echo method is capable of determining the location and the extent of delaminations, voids, honeycombing, and cracks in reinforced and post-tensioned concrete decks (*50, 51*).

The impact-echo method is based on a seismic analysis of materials. A low-frequency stress wave, typically less than 80 kHz, is generated by mechanically impacting the surface of the concrete (*26, 50, 51*). The stress wave propagates through the concrete at a velocity that is characteristic of the material (*26, 50*). Stress waves are reflected by discontinuities in the concrete and travel back toward the source, where they are detected by a transducer (*26, 50, 51*). The transducer digitizes the reflected information, after which the data are usually recorded on a computer (*50, 51*).

Equation 3 is used in the impact-echo method to estimate the thickness and depth of concrete deterioration (*26, 50, 51*):

$$d = \frac{C}{2 \cdot f} \tag{3}$$

where d = the depth of a flaw or the thickness of a solid structure, in

C = the wave velocity within the concrete, in/s

f = frequency of the transmitted or reflected waves, Hz

Thus, for calculating the depth of a flaw, the velocity and frequency of the wave must be known. Wave velocity is determined by measuring the travel time of a stress wave between two transducers separated by a known distance, while the wave frequency is obtained using accelerometers. The resulting frequencies constitute a response spectrum as depicted in Figure 3.12 (*50*). The peaks in the reflection spectrum designate dominant frequencies, which are associated with reflections of stress waves or with flexural vibrations in thin or delaminated layers (*29*).

The structural integrity of the concrete affects the frequency of the reflection waves by causing a shift in the response spectrum (*29*). Good quality concrete creates a peak in the response spectrum at low frequencies. However, for delaminated bridge decks, for example, the reflection waves return from depths much less than the deck thickness, causing higher frequencies that are marked by a peak farther to the right. When a delamination is just beginning, the high frequency peak may be accompanied by a second peak of lower frequency, corresponding to reflections from the bottom of the deck slab. As separation of the delaminated concrete increases, however, wave transmission across the delamination is prohibited so that only the higher frequency peak appears.

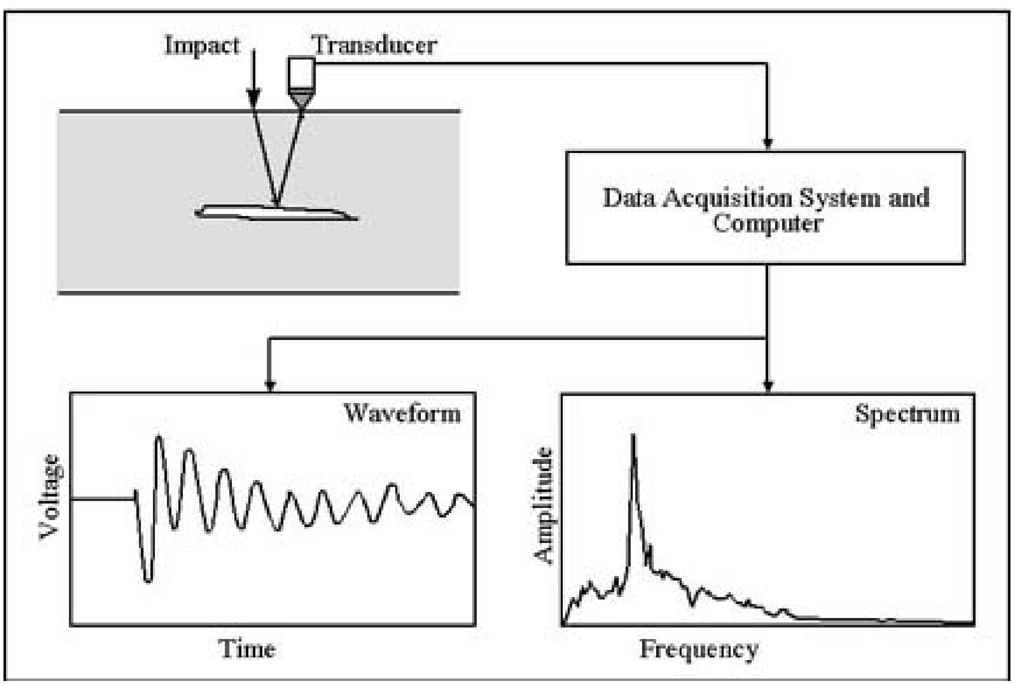

FIGURE 3.12 Diagram of the impact-echo method (*50*).

The impact-echo method can be successfully implemented in long-term condition monitoring of concrete bridge decks. Assessing concrete quality by the impact-echo method is ideal since it is non-destructive and can be used to measure the same locations repeatedly (*29*). However, impact-echo testing is extremely time consuming, and the results are not always conclusive. Also, the impact-echo method is not useful for investigating asphalt-overlaid bridge decks as a result of the potentially variable interface between asphalt concrete and Portland cement concrete. Therefore, a considerable number of bridge decks within the jurisdiction of a typical agency would have to be investigated using other techniques.

3.10 ULTRASONIC TESTING

Ultrasonic pulse-velocity tests were first developed in the 1940s using an instrument commonly known in North America as a soniscope (*1*). By the 1960s, the instrument was designed to be much smaller and more suitable for field use. While previous soniscopes were heavy and required expert training to operate, modern soniscopes or ultrasonic testers are smaller, battery-operated, and portable and boast a digital readout (*1*). In North America, the most commonly known units are the British-made PUNDIT (Portable Ultrasonic Non-destructive Digital Indicating Tester) and the American-made V-Meter (*1*).

An ultrasonic pulse-velocity test uses vibration frequencies in the range of 20 to 150 Hz to detect voids in concrete, although frequencies of 150 Hz have only been used in laboratory studies (*1, 52, 53*). The vibration frequencies are generated by electronic pulses and then converted into mechanical energy by a transducer. Although higher frequencies are more sensitive to smaller voids and can be used with much thinner specimens, they are also subject to greater attenuation (*1*). For these reasons, equipment with an operating frequency of 50 Hz, for example, should not be used on sections thinner than 6 inches, while equipment operating at a frequency of 20 Hz should be limited to use on sections greater than 12 inches in thickness (*1, 52, 53*).

Two different types of transducers are used during an ultrasonic test. One is a transmitting transducer, and the other is a receiving transducer (*1, 53*). Both are attached to the concrete deck at a specified distance apart from each other (*1*). The electronic pulses are generated by the transmitting transducer and collected at the receiving transducer, and the travel time between the two transducers is measured electronically (*1*). The recorded data include the frequencies at which the pulses resonate, which directly correlate with the concrete thickness and compressional wave velocity (*54*). The transducers are attached to the concrete with a suitable acoustic coupling medium such as petroleum jelly, liquid soap, or a kaolin-glycerol paste, which should be kept as thin as possible (*1*). If the concrete surface does not allow for a smooth contact, then the surface may be filed down, or a thin layer of epoxy mortar, plaster of Paris, or other similar material may be applied (*1*).

Contact points for transducers can be configured in three different ways as illustrated in Figure 3.13 (*1*). In the direct-transmission method, the transmitter is attached to one side of the concrete section, and the receiver is attached to the opposite side of the deck section (*1*). The placement of the transducers in this pattern results in maximum sensitivity and a well-defined path. A second possible configuration uses the semi-direct transmission, in which the transmitter is attached to one face of the concrete deck while the receiver is attached to an adjacent face so that they are 90 degrees to each other (*1*). Semi-direct transmission is occasionally used when the arrangement of a deck section does not permit the direct-transmission method to be utilized. An example of such condition would be a bridge abutment or a foundation slab. The third way of arranging the contact points is the surface-transmission method (*1*). Surface transmission is the least ideal of the three methods because it measures the quality of concrete near the surface only. In addition, reinforcement placed parallel to the surface may magnify or distort the pulse readings. For these reasons, the surface-transmission method should only be used when all but one surface of a concrete section are inaccessible.

Pulse-velocity measurements are reliable for assessing concrete quality and uniformity. As the pulse passes through concrete, its velocity decreases due to the presence of voids associated with porosity and internal cracking. Several researchers have developed scales correlating pulse-velocity measurements to concrete quality to be used as guides in interpreting pulse-velocity readings in concrete (*1*). Examples of two such scales are presented in Table 3.1 (*1*).

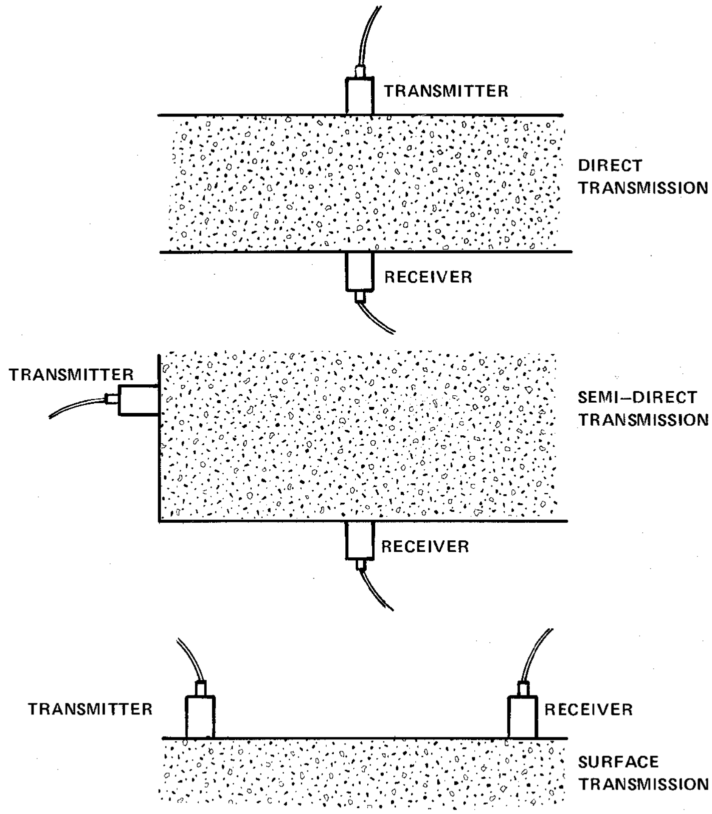

FIGURE 3.13 Methods of measuring pulse velocity through concrete (*1*).

TABLE 3.1 Pulse-Velocity Ratings for Concrete (*1*)

	Pulse Velocity (ft/sec)	
Quality of Concrete	**Malhotra Scale**	**Leslie and Cheesman Scale**
Excellent	> 15,000	--
Good	12,000 to 15,000	> 16,000
Fair	10,000 to 12,000	13,000 to 16,000
Poor	7,000 to 10,000	10,000 to 12,000
Very Poor	< 7,000	--

Several factors affect the accuracy of pulse-velocity measurements. One is the presence and orientation of reinforcing steel, in which the pulse velocity is 1.2 to 1.9 times greater than that of the surrounding concrete (*1*). Therefore, technicians should choose a location on the concrete section that will have little influence from the reinforcing steel. If reinforcing steel is present throughout the bridge deck and cannot be avoided, a correction can be applied to the readings. In order to use a correction factor, the size and location of the reinforcement must be known.

When the wave propagation is perpendicular to the longitudinal axis of the reinforcement, the effect of the reinforcing steel on the pulse-velocity measurements is small. However, when wave propagation is parallel to the longitudinal axis of the reinforcing steel, the effect of the reinforcement on the measurements is significant. Most equipment manuals provide correction factors for both of these situations when they occur independently of each other; however, no correction is readily available when the reinforcement runs both parallel and perpendicular to the propagation of the wave.

Other factors that affect pulse-velocity measurements through concrete are the smoothness of the concrete surface, concrete temperature, moisture content, mixture proportions, and concrete age (*1*). As discussed earlier, smoothness is important for ensuring adequate contact between the concrete and transducers. Concrete temperature does not significantly affect pulse-velocity measurements as long as the temperature is between 40 and 85°F; temperatures outside of this range may require corrections, however (*1*). The presence of moisture increases pulse-velocity measurements by no more than 2 percent (*1*). The quality of the original concrete mixture and the development of distress with increasing age most directly impact pulse-velocity measurements and are the basis for condition assessments using this technique.

Pulse-velocity measurements are primarily used to detect voids and cracks in concrete. Ideally, the cracks should be nearly perpendicular to the direction of wave propagation and large enough to disrupt the normal transmission path (*1*). Such cracks are detected because they cause an unusually long transit time or a decrease in the amplitude of received waves. Figure 3.14 illustrates some of the factors that affect the transmission of ultrasonic waves in concrete (*1*).

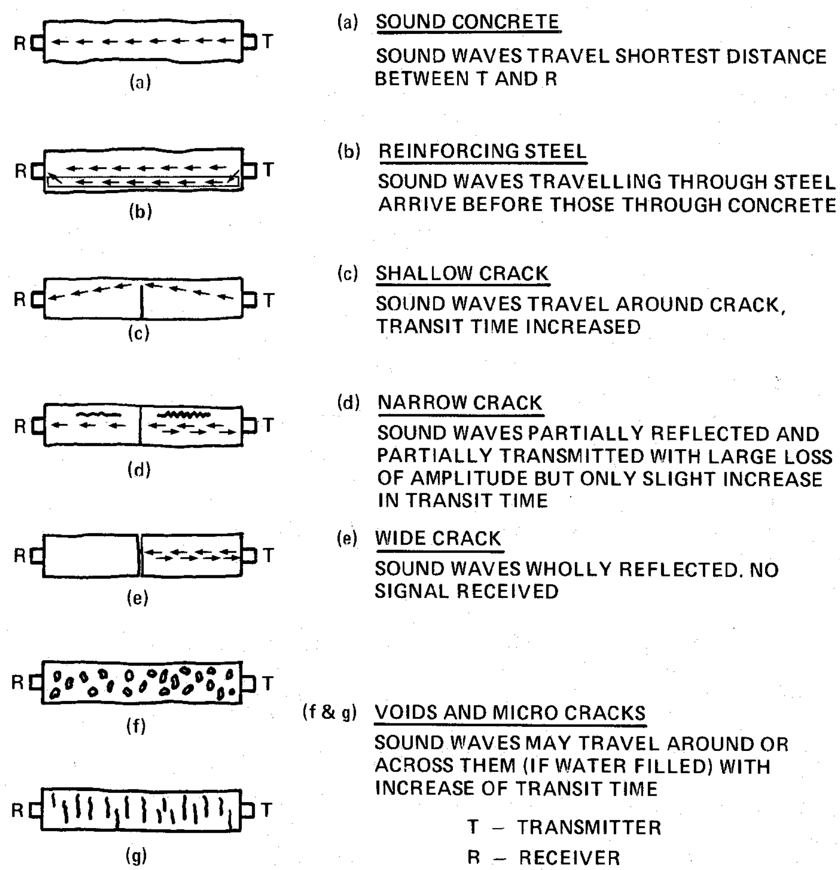

FIGURE 3.14 Factors affecting the transmission of sound waves through concrete (*1*).

Few studies exist on the use of ultrasonic tests on highway structures. In one study, researchers performed ultrasonic tests on several old bridges over the span of a couple of years (*1*). The results of the tests were inconsistent due to moisture conditions and changes in crack widths on the decks. Data from another study reporting ultrasonic measurements on a bridge deck were not well accepted because the underside of the bridge deck was relatively inaccessible due to the presence of girders and diaphragms (*1*). Such a condition is probably the main shortcoming of using ultrasonic methods on highway structures. Essentially these structures are so heavily reinforced that the direct-transmission method must be used to produce useful data. However, the opposite side of the concrete section is not always accessible, nor is precise alignment of the transducers always possible (*1*). The limitations of this method should not discourage its use but should be taken into consideration when determining the appropriate testing method for a particular structure (*1*).

3.11 CHLORIDE CONCENTRATION TESTING

Chloride concentration testing is used to identify areas of a bridge deck where chloride concentrations may be high enough to initiate corrosion of the reinforcing steel. Measuring the chloride concentrations in concrete is performed by extracting a core from the sample section of concrete (*1*). The cores are then transported to the laboratory for vertical sectioning, pulverization, and individual analyses (*1*). The sectioning is particularly desirable for obtaining chloride concentration profiles within the concrete, from which estimations of the rate at which chlorides are penetrating the concrete and predictions of the condition of the reinforcing steel may be possible.

Another way of extracting concrete samples is to use a rotary hammer, as shown in Figure 3.15 (*1, 55*). A rotary hammer is a drill that pulverizes the sample during the extraction process (*1, 18*). Drilling to specific depths and removing samples in succession is a particularly efficient method for generating chloride concentration profiles with the rotary hammer (*18*). Samples are collected in turn, and the hole is subsequently cleaned using a vacuum or compressed air. The process is repeated for each depth increment of interest. To directly determine the chloride concentration near the reinforcement, a cover meter is needed to identify the depth of the reinforcement. Once the depth is known, a hole is drilled in the vicinity of the reinforcement and vacuumed clean, and the rotary hammer is used to extract a sample of concrete from the same depth as the reinforcement (*1*).

The rotary-hammer method follows the same procedure as the core-sampling method, except that the drill bit performs the pulverization in the field. Sometimes, however, additional pulverization is required in the laboratory before tests can commence. All samples must pass a No. 50 (0.0118-inch) screen before laboratory tests can be conducted (*1*). Combination core bits with carbide-tipped starter bits enhance pulverization of the samples in the field and are recommended when the rotary hammer is used (*1*).

FIGURE 3.15 Extraction of concrete samples with a rotary hammer (*55*).

While the use of rotary hammers is an economical and rapid method for extracting samples from concrete for chloride determinations, care must be taken to avoid contamination of the samples. For example, contamination can occur when the drill bit scrapes the side of the hole, especially near the surface of the concrete where chloride concentrations are higher. The residue falls to the bottom of the hole, where it is blended together with the pulverized sample at that depth. One way of avoiding this problem is to use an increasingly smaller diameter bit with increasing hole depth (*1*).

The laboratory method of determining the chloride concentration in concrete is the wet chemical process (*1*). Figure 3.16 shows a commercially developed kit that may be used in the field to measure the chloride ion content of concrete (*56*). There are two ways of performing chloride concentration tests, based on the method used to extract the chloride ions. In the total-chloride-ion method, a powdered sample of the concrete is dissolved in dilute nitric acid (*1*). A silver nitrate solution is then used to conduct a potentiometric titration of the chloride ions. The total-chloride-ion method is a measurement of the free and chemically bound chloride ions in concrete. Consequently, the results contain measurements of chloride ions that do not contribute to the corrosion of the reinforcement.

FIGURE 3.16 Field kit for measuring chloride concentrations (*56*).

The second method of determining the chloride concentration in concrete is the soluble-chloride-ion method (*1*). This method uses water to extract the chloride ions. The test duration and the temperature of the water to which the concrete sample is exposed determine the amount of chloride ions extracted. Generally, the samples are boiled for 5 minutes and then cooled for a period of 24 hours. The water-soluble test measures both the free chloride ions and a portion of the chemically bound ions. Therefore, no clear advantage exists between the total chloride and the water-soluble methods.

Researchers at the Kansas DOT developed an in-situ method of evaluating chloride concentrations in concrete bridge decks (*1*). The method was simple to conduct and caused negligible damage. However, the method was discontinued for several reasons, including the failure of the electrodes during field usage, the need for extensive personnel training, and the need to routinely calibrate the electrodes to known chloride concentrations.

Researchers working for the Strategic Highway Research Program (SHRP) also developed a field method for chloride analysis. The method is described in detail in the American Association of State Highway and Transportation Officials (AASHTO) T 260, Sampling and Testing for Chloride Ion in Concrete and Concrete Raw Materials, as an alternative procedure for determining the total chloride ion content in concrete. The method requires the use of a specific-ion electrode, high-impedance digital voltmeter, calibration solution, digestion solution, and stabilizing solution (*57*). The specific-ion electrode is first calibrated in the calibration solution. A sample of the concrete powder is then placed in a bottle, where it is mixed with the digestion solution. After several minutes, the stabilizing solution is added to the bottle. The calibrated ion electrode is then inserted into the bottle to measure the voltage of the solution. Chloride contents can be calculated from the voltage reading using equations found in AASHTO T 260.

3.12 PETROGRAPHIC ANALYSIS

Petrography is the evaluation and assessment of the microstructure and composition of a material in order to judge its performance (*4*). A petrographic analysis includes both visual and microscopical techniques to identify the constituents of concrete, detect performance problems, or assess the integrity of concrete (*1*). The method is extremely useful in identifying mechanisms of deterioration such as freeze-thaw cycling and chloride infiltration. Figure 3.17 is a diagram of the petrographic process as applied to pottery (*58*).

Specimens can be prepared in different ways for evaluation in a petrographic analysis (*59*). Specimens can have polished or etched surfaces, or they can be thinly sliced (*1*). The thinly sliced sample, also known as a thin-section, uses a cross-sectional slice of a concrete core (*4*). The analysis begins with a visual examination of the concrete sample in order to gather information concerning construction practices, unique characteristics of the specimen, or noticeable deterioration (*4*). Occasionally, the information gathered during a visual inspection is sufficient to meet the needs of the investigation (*4*). However, when a more thorough examination of the specimen is necessary, any of the following techniques may be used: stereo microscopy, transmitted-light microscopy, reflected-light microscopy, or scanning-electron microscopy (*4*). These tests are highly specialized and are usually performed by an expert petrographer (*1*).

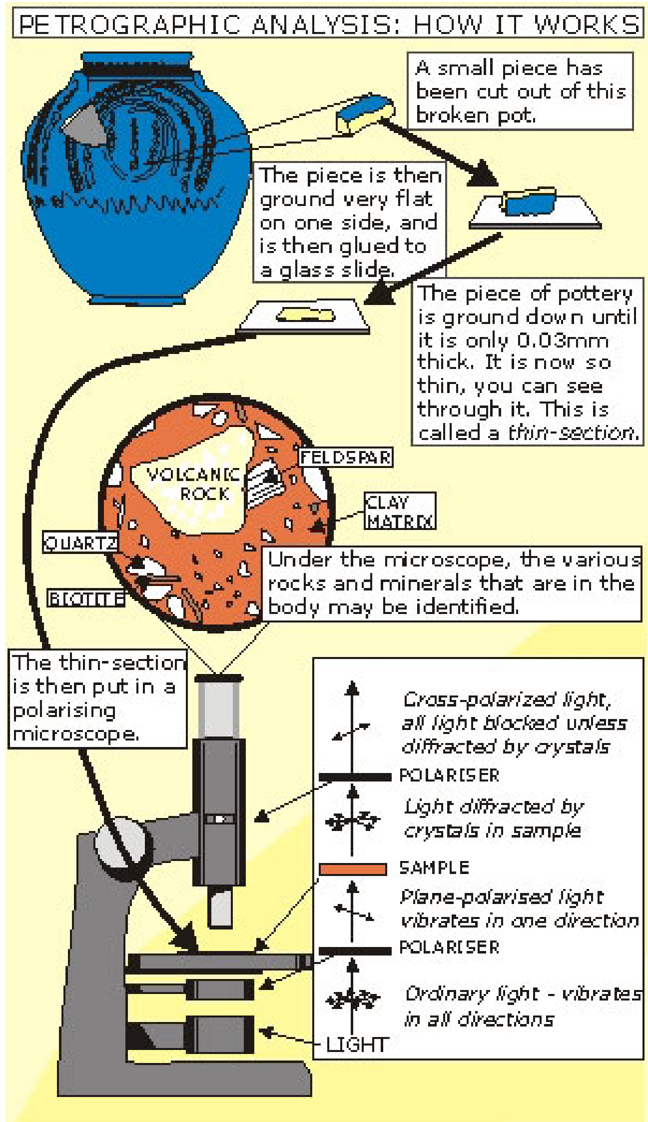

FIGURE 3.17 Petrographic analysis (*58*).

Petrographic image analysis (PIA) requires the use of advanced equipment (*60*). The equipment is completely automated and uses digital image acquisition to obtain quantitative information on sizes, shapes, and numbers of pores in a given thin-section. PIA yields a high-resolution scan of thin-sections, cuttings, and core samples in true color (*61*). PIA also provides details concerning texture and composition of concrete constituents, including specific textural parameters such as pore size and pore geometry. The time needed to conduct a PIA is considerably reduced since the equipment can detect characteristics and make classifications at a faster rate than a traditional petrographic analysis (*60*).

3.13 PENETRATION DYES

Inspection methods using penetration dyes are primarily utilized for detecting surface defects that are not detected during a visual inspection (*62*). Penetration dyes help to establish an observable contrast between discontinuities and the surrounding intact concrete. The dyes are applied as shown in Figure 3.18 to clean concrete surfaces and seep into surface discontinuities and open voids (*63*). Any excess dye is wiped away, and a white developer, which dries to a powder, is applied to the concrete surface. The developer acts as a blotter, drawing the dye out from the discontinuities. Upon contact, the dye stains the developer, marking the location of surface defects. A white or blank surface indicates an absence of cracks or other surface defects.

Penetration inspection can be performed using visible or fluorescent light (*62*). Visible penetrant inspection uses visible white light, while fluorescent penetrant inspection uses a black, ultraviolet (UV) light to create a color contrast between the discontinuity and the stain. Both methods require a concrete surface free of any contaminants that may impede the migration of penetrants into discontinuities. Following the inspection process, penetrant materials are removed from the concrete surface using specified cleaning procedures.

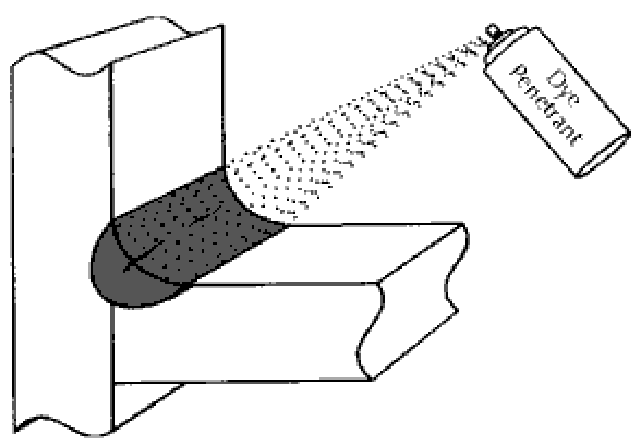

FIGURE 3.18 Inspection of a cracked surface using a dye penetrant (*63*).

The advantages of using visible dye penetrants are rapidity and low cost (*62*). Surface roughness and porosity, however, can both limit the use of liquid penetrants (*62*). Additionally, the visible-dye method only identifies surface defects. Detection of subsurface cracks and defects requires other techniques.

Unlike the visible-dye method, the fluorescent-dye method can be used on rough surfaces (*62*). One major disadvantage of this method is that field inspections must be performed at night when a black light can be used effectively (*62*). Like the visible-dye method, the fluorescent-dye method can only detect surface cracking. Therefore, investigations of subsurface cracks must be conducted with other techniques.

3.14 SCHMIDT REBOUND HAMMER

The Schmidt Rebound Hammer, also known as the Swiss Hammer, is useful in determining the uniformity of concrete with a focus on identifying areas that require further investigation (*1, 3*). The apparatus measures the rebound number, which has been empirically correlated to concrete strength (*7, 31*). The rebound number is determined by measuring the rebound of a spring-loaded plunger as a percentage of the initial length of the spring (*7, 31*). Figure 3.19 is a diagram of the testing apparatus, which provides a cross-sectional view of the internal components (*64*). Several advantages of the rebound hammer are low cost, durability, and ease of use. Tests should be performed in areas of concern, but direct contact with coarse aggregate particles should be avoided to improve accuracy (*1, 3*).

Results of a rebound hammer measurement are affected by several factors, including the angle of test, surface smoothness, concrete mixture proportions, type of coarse aggregate, moisture content, and carbonation of the surface (*1*). Calibration techniques should be used to ensure that the readings are accurate, especially regarding the angle of test. Placement of the instrument in vertically up, vertically down, or horizontal positions should be noted during testing (*1, 3*).

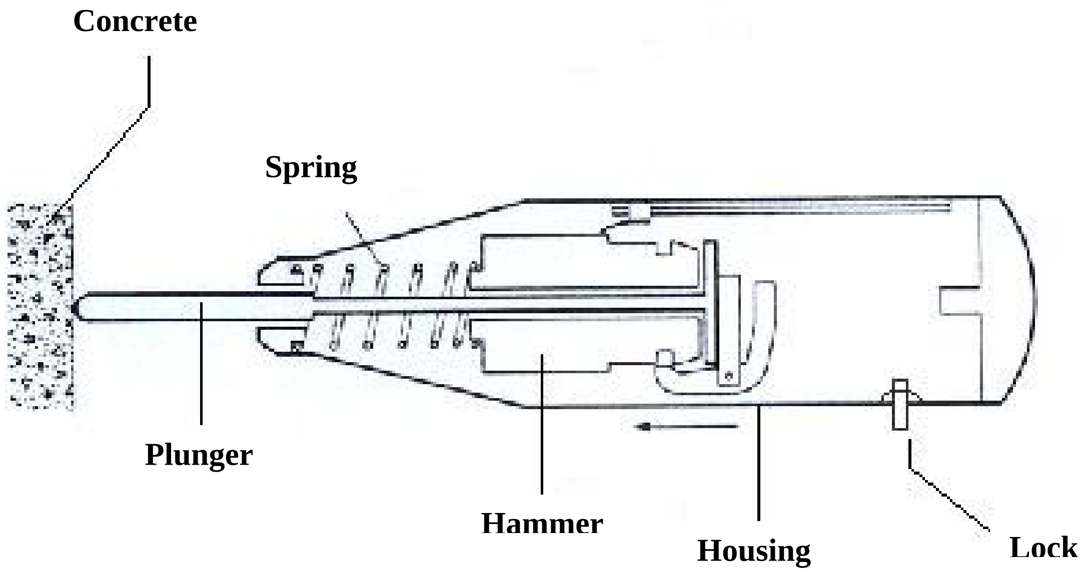

FIGURE 3.19 Cross-sectional view of a Schmidt rebound hammer (*64*).

While the accuracy of strength prediction for a concrete section is no greater than 25 percent when using the rebound hammer alone, data from a study performed in Romania suggests that using the rebound hammer in conjunction with ultrasonic tests markedly increases the accuracy of the measurement (*1*). Concrete having a compressive strength of less than 1000 psi should not be evaluated by this method, as the hammer may cause damage to the concrete (*1, 3*).

3.15 HALF-CELL POTENTIAL TESTING

The severity of steel corrosion in concrete can be determined by measuring the electrical half-cell potential of uncoated reinforcing steel (*18, 65*). The procedure for measuring half-cell potentials is fairly simple. A half-cell, normally a copper-copper sulfate (Cu-$CuSO_4$) reference electrode (CSE), is placed on the surface of the concrete where the steel reinforcement is located (*66*). The reference electrode is then connected to the positive end of a high-input impedance voltmeter that is connected to a data-logging device. The negative end of the voltmeter is connected to the reinforcing steel being investigated. To enable the latter connection, a hole should be drilled into the concrete to expose the steel, and a screw should be tapped into the reinforcing steel so that a good electrical connection is made (*67, 68*). In addition, a moist sponge should be placed between the half-cell and the concrete to improve the electrical coupling between the deck and the instrument during the survey (*68*).

Once the electrode and the reinforcing steel are adequately connected to the voltmeter, the corrosion potential of the steel reinforcement near the point of contact can be measured. A diagram of the testing equipment is shown in Figure 3.20 (*65*). After a large number of corrosion potential readings have been made, a contour map can be generated to delineate areas of corrosion. Measurements should be taken in a grid pattern to facilitate the drawing of equipotential lines on a two-dimensional contour map, as shown in Figure 3.21 (*68*).

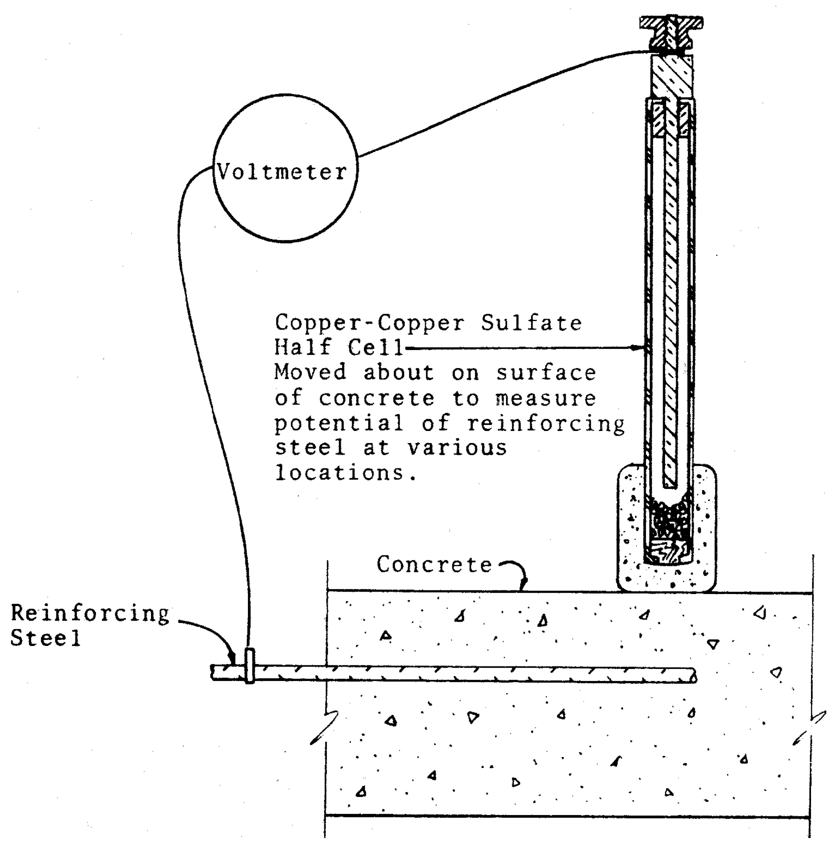

FIGURE 3.20 Diagram of a half-cell potentiometer (*65*).

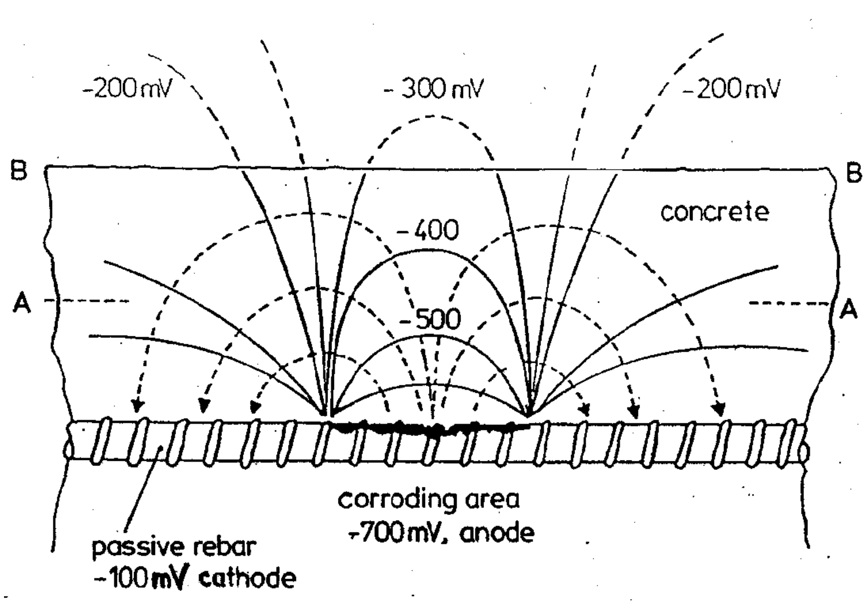

FIGURE 3.21 Two-dimensional contour map from half-cell potential testing (*68*).

Surface potential measurements are affected by several factors, including concrete cover thickness, concrete resistivity, and polarization. As concrete cover thickness increases, the difference between the potential values of passive steel and actively corroding steel decreases. In fact, for a thick cover, the potential values become nearly identical. Therefore, locating small corroding areas becomes extremely difficult with increasing cover depth, as illustrated in Figure 3.22 (*68*).

As previously discussed, the resistivity of concrete is affected by concrete humidity and ion concentrations in the pore solution. Using empirical equations, researchers have shown that reduced electrical resistance of the concrete increases the current flow in the reference electrode used in half-cell potential surveys (*68*). When surface potential measurements are performed on highly resistive concrete, however, corrosion areas can be masked. Macrocell currents are inclined to avoid the highly resistive layer, which affects potential readings in a way similar to cover depth.

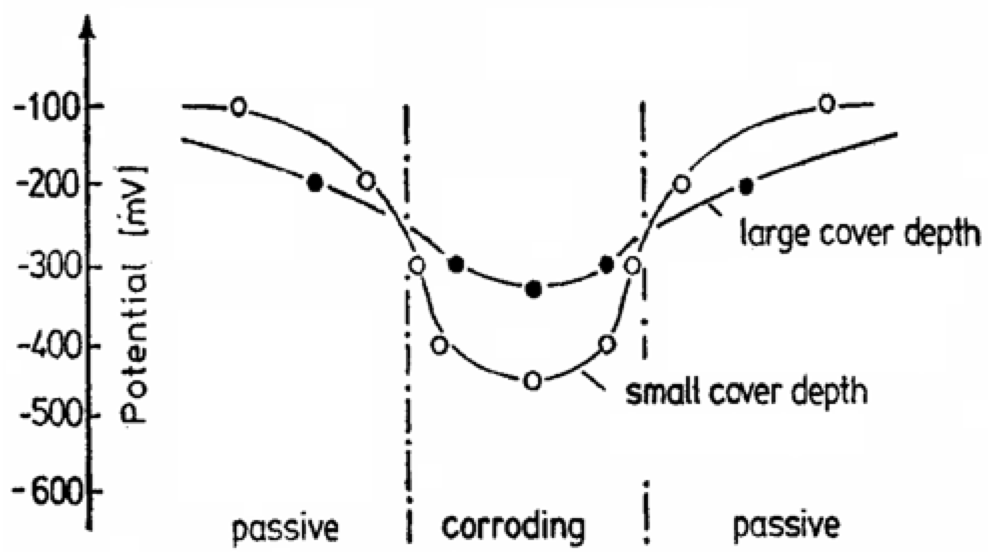

FIGURE 3.22 Plot of potential values with increasing cover depth (*68*).

Polarization effects may also cause distortions in surface potential measurements. When concrete is submerged in water or in earth, oxygen access to the reinforcing steel is restricted, causing a very negative potential measurement (*68*). Some structures have transition zones where part of the concrete member is submerged, but another part is protruding from the earth or water. Another type of transition area may be a splash zone on a bridge, where negative potential values are usually measured as a result of galvanic coupling on the immersed reinforcing steel (*68*). Therefore, negative values in transition areas must not be mistaken for corrosion.

ASTM C 876, Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete, specifies that potential measurements more negative than −0.35 V measured with a CSE indicate a probability greater than 90 percent that corrosion is occurring. Potential measurements more positive than −0.20 V indicate a probability greater than 90 percent that corrosion is not occurring in that area. Potential measurements between −0.20 and −0.35 V indicate that corrosion in that area is uncertain. However, studies have been conducted that conflict with these threshold values designated in ASTM C 876 (*66, 68*). The studies indicate that different conditions, such as concrete moisture content, chloride content, temperature, carbonation, and cover thickness, alter the potential values that suggest active corrosion of the reinforcing steel (*66, 67, 68*). In other words, corrosion may occur on one bridge deck at values more positive than −0.20 V, while for another it may occur at values far more negative than −0.35 V. Thus, published threshold values in ASTM C 876 should only be used as guidelines since a precise delineation of steel from a passive to an active state cannot be made to encompass all types of bridges. Engineers and technical specialists familiar with concrete materials and corrosion testing should interpret potential measurements using supplementary data and other factors necessary to accurately formulate conclusions about corrosion of reinforcing steel in concrete (*65*). An understanding of how various factors influence potential measurements is key to a meaningful data interpretation.

Surface potential measurements are a reliable indicator of corrosion activity on reinforcing steel. Although the rate of corrosion cannot be quantified using surface potential measurements, the amount of corrosion can be inferred (*17, 18*). In general, an extensive area of potentials more negative than −0.35 V suggests a high probability that corrosion is occurring (*17, 18*).

3.16 RAPID CHLORIDE PERMEABILITY

Low-permeability concrete generally possesses high strengths and is resistant to the infiltration of water and chlorides (*69*). Conversely, extremely porous concrete allows water, salts, and oxygen to more easily reach the reinforcing steel, which accelerates corrosion of the reinforcement. By measuring the chloride permeability of concrete, durability problems can be detected early so that timely and cost-effective protective measures can be implemented before the occurrence of any significant corrosion or deterioration of the concrete (*69*). The Rapid Chloride Permeability Test (RCPT), given as ASTM C 1202, Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration, is a method that measures concrete permeability (*69, 70*). Before the development of the RCPT, DOTs relied on ponding tests, such as that prescribed by AASHTO T 259, Standard Method of Test for Resistance of Concrete to Chloride Ion Penetration (Salt Ponding Test) (*70*). This test requires the ponding of a 3 percent sodium chloride (NaCl) solution on the surface of a concrete sample for 90 days. The chloride concentration of the concrete specimen is then determined at incremental depths. From beginning to end, the ponding test takes approximately four months. The RCPT is as accurate as the ponding test but requires a much shorter time frame to complete.

The RCPT method is performed on concrete specimens 4 inches in diameter and 2 inches in thickness (*69, 70*). One side of the specimen is immersed in an NaCl solution, while the other side is immersed in a sodium hydroxide solution (NaOH), as shown in Figure 3.23 (*70*). An electrical voltage of 60 V DC is then applied to the specimen to force the chloride ions to migrate into the concrete. Current readings are taken every 30 minutes during the 6-hour test and then plotted as a function of time. The area under the curve indicates the total charge passed, which is a measure of the resistance of the concrete to the diffusion of chloride ions. A high charge indicates a low resistance to chloride ions, or poor quality concrete. Table 3.2 provides values that relate the charge passed to chloride-ion penetrability (*69*).

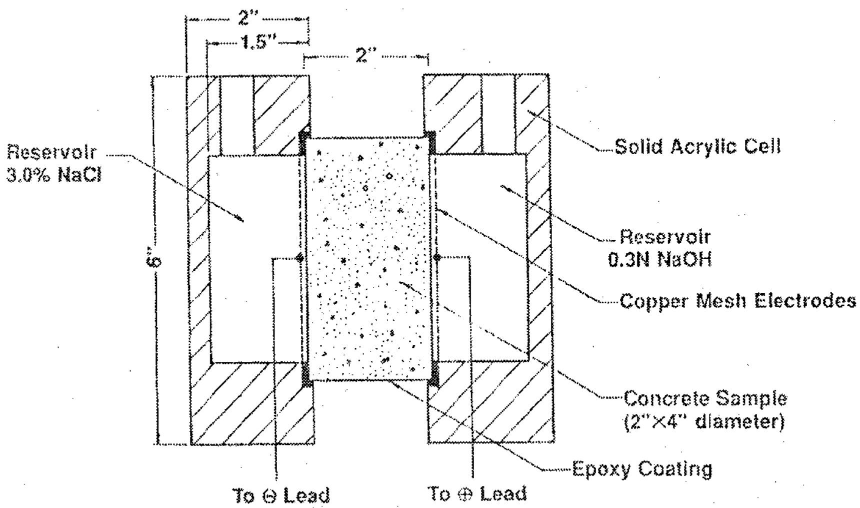

FIGURE 3.23 Schematic of the rapid chloride permeability test (*70*).

TABLE 3.2 Chloride-Ion Penetrability Ratings (*69*)

Charge Passed (Coulombs)	Chloride Ion Penetrability
> 4,000	High
2,000 - 4,000	Moderate
1,000 - 2,000	Low
100 - 1,000	Very Low
< 100	Negligible

Several researchers have criticized the accuracy of the RCPT even though the method has been adopted in both ASTM and AASHTO standards (*69*). One of the main criticisms is that the current passed through the specimen is a measure of all ions in the pore solution and not solely the chloride ions. Critics also suggest that measurements are made prematurely, before steady-state migration is attained. In addition, the high voltage passed through the specimen causes an increase in the temperature of the concrete, especially in poor quality concrete. The temperature increase intensifies the charge passed compared to the current flow that would occur if the specimen were to remain at a constant temperature. Therefore, results indicate that poor quality concrete appears to be worse than it actually is. Additionally, the accuracy and precision of the RCPT is poor. ASTM C 1202 requires that the average value of three samples cannot differ by more than 29 percent between two separate laboratories, which is viewed by many researchers as excessive. Furthermore, the method depends on a relationship between the conductivity of concrete and the chloride-ion permeability. Consequently, if conductive materials, such as reinforcing steel, carbon fiber, or corrosion-inhibiting admixtures, are present within the specimen, measurements will be higher than they would normally be.

Despite the limitations of the RCPT, the test provides a quick and reliable way of estimating the permeation of chloride ions into concrete. In fact, many researchers have concluded that the relationships between RCPT and ponding test results are valid even for concrete containing conductive materials. Further research, however, must be conducted to address the various opinions about the accuracy of the test.

3.17 SKID RESISTANCE TESTING

The British Pendulum Tester was designed by researchers at the Transport and Road Research Laboratory of Great Britain to study design and maintenance problems on highways (*71*). The testing apparatus, or skid tester, measures the skid resistance value (SRV) of roadway surfaces. The procedure is described in ASTM E 303, Standard Test Method for Measuring Surface Frictional Properties Using the British Pendulum Tester, and a diagram of the testing apparatus is given in Figure 3.24 (*72*).

Driver safety is the most important factor in roadway design and maintenance. Once a roadway surface has lost adequate friction, vehicles may require greater distances to stop, regardless of the condition of the vehicle tires. Such occurrences jeopardize the safety of everyone on the road. Routine skid resistance tests provide a way for engineers to determine the resistance to slipping and skidding of a particular road surface.

The testing apparatus is comprised of a tubular, pendulum arm that is attached to a vertical support (*71*). On the bottom end of the pendulum is a weighted head with a rubber slider. The opposite end of the pendulum is allowed to rotate freely around a spindle. A skid resistance test is conducted by placing the apparatus on the roadway surface to be tested with the pendulum arm brought up to a horizontal position, from which it is released. The pendulum is then allowed to freely swing to the roadway surface. As the rubber slider comes in contact with the surface, the friction between the rubber slider and the road surface cause the pendulum to come to rest. The distance that the slider travels across the road surface indicates the resistance to skidding of the surface. A frictionally constrained pointer that is affixed to the pendulum arm measures the surface resistance on a scale from 0 to 150. Corrections for temperature are needed since vehicle tires are typically at a higher temperature than that of the rubber slider.

The British Pendulum Tester provides a quick and reliable way to evaluate the skid resistance of roadway surfaces (*71*). The testing apparatus is relatively cheap, easy to transport, and possible to use in any location. Also, interpretation of test results can be performed on-site with remarkable accuracy.

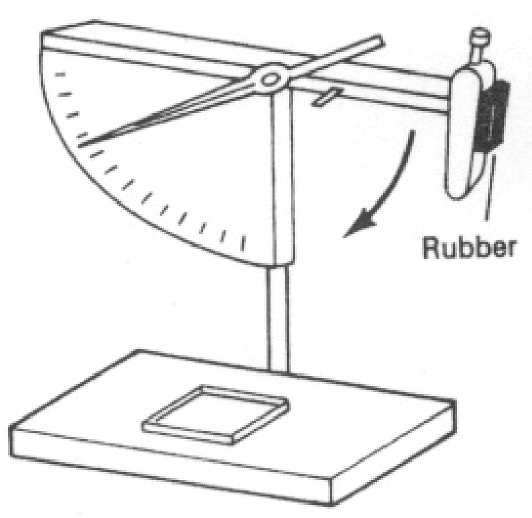

FIGURE 3.24 Diagram of a British pendulum tester (*72*).

The locked-wheel trailer test is also used to measure the skid resistance of wet roadway surfaces and should be performed according to ASTM E 274, Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire (*73*). Essentially, the testing apparatus is a trailer that is towed across the roadway surface of interest. The vehicle is maintained at a constant speed while water is applied to the road surface directly in front of the test wheel. Brakes are then applied to the test wheel so that the wheel stops spinning and begins to skid. At this point, the horizontal and vertical forces are recorded and correlated to a measurement called the friction number or skid number (*73*).

The friction value of dry pavements is relatively high, which aids in preventing traffic accidents. However, moisture on pavements causes a loss of friction, which can be potentially dangerous to motor vehicles, especially those with tires in poor condition. By using the results of the locked-wheel trailer test, engineers are able to identify pavements that are hazardous in the presence of moisture (*73*).

3.18 CORROSION SENSORS

Monitoring corrosion processes on bridge decks by traditional methods requires routine measurements so that the rate of chloride infiltration through time can be determined (*74*). Such methods usually necessitate lane closures, as well as expensive maintenance crews to perform the testing. Laboratory tests often require days, if not weeks, to complete. New technologies are emerging, however, that may reduce costs and more readily enable the collection of bridge condition data and the design of maintenance schedules (*75*). The technologies are in the form of tiny sensors, such as the one shown in Figure 3.25, which monitor parameters associated with in-situ corrosion of concrete bridge decks (*76*). Currently, three kinds of sensors are under development, including the Smart Pebble, Smart Aggregate, and Embedded Corrosion Instrument (ECI-1).

FIGURE 3.25 Placement of a corrosion sensor during new deck construction (*76*).

3.18.1 Smart Pebble

The Smart Pebble is a wireless sensor that monitors the condition of a bridge from within the deck (*74, 75*). Sensors are inserted into the bridge deck during initial construction or through a back-filled core hole (*74*). The sensors are approximately the size and weight of a typical coarse aggregate particle used in a concrete mixture (*74*). Batteries are not necessary, as each sensor is powered remotely through the use of a radio frequency identification (RFID) chip (*74, 75*).

Data are obtained by a data acquisition system that can be handheld or vehicle-mounted (*74, 75*). As the reader device passes over the Smart Pebble sensors, they are activated and begin relaying information about the condition of the bridge (*74, 75*). The RFID chip identifies the location of the sensor and indicates chloride levels in that part of the bridge (*74, 75*). Over time, individual readings can be plotted and the rate of chloride ingress mapped for the entire bridge, without the use of destructive, time-consuming methods. For immediate assessment of the bridge, the reader display can identify areas of the bridge under the chloride threshold value with a green light and areas over the chloride threshold value with a red light (*74*).

The Smart Pebble is currently in the early stages of development (*75*). According to SRI International, the maker of the Smart Pebble, individual sensors are expected to cost less than $100. The reader system that activates the sensors will cost approximately $1000.

3.18.2 Smart Aggregate

The Smart Aggregate sensor is a wireless device that measures the conductivity and temperature of the concrete from within the bridge deck (*77*). The sensor is powered by near-field induction coupling, and information is relayed out of the device by a radio frequency (RF) signal (*77*). Sensors are placed along the top mat of the reinforcing steel during initial construction of the bridge deck to allow for the earliest possible warning against harmful conditions that may initiate corrosion (*77*). The sensor is roughly the size of a quarter and has an estimated design life of 50 years (*75*). Smart Aggregate sensors are in the early stages of development and are expected to cost less than $20 (*77*).

Like the Smart Pebble sensor, the Smart Aggregate sensor is a passive device, which means that it remains idle until activated by a passing interrogation unit (*77*). The interrogation unit contains a power field generator and a data acquisition system. The power field generator provides energy to the Smart Aggregate sensor through an induction field. When the sensor is activated, environmental information is transmitted out of the sensor and stored in the data acquisition system.

3.18.3 Embedded Corrosion Instrument

The ECI-1 contains five individual sensors to monitor the condition of a bridge deck (*75*). The five sensors include a chloride threshold indicator, a temperature sensor, a resistivity sensor, a polarization resistance sensor, and an open-circuit potential sensor. The ECI-1 is contained in a box-like structure that measures just a few inches on its side. The sensors are wired to a data acquisition system that can be powered by a solar battery. Access to the information may be obtained from an on-site laptop computer or remotely. Individual devices are currently being developed and are expected to cost approximately $1000 each.

CHAPTER 4
REHABILITATION METHODS

4.1 REHABILITATION AND REPAIR

The technology of bridge rehabilitation is constantly improving as researchers develop new rehabilitation methods that are more efficient and specific to particular bridge elements. Nonetheless, the decision to rehabilitate a bridge should come only after considering whether replacement of the bridge deck is more economical. For example, deck replacement may be appropriate whenever deck repairs exceed 35 percent of the total deck area (*78*). In making such a decision, DOTs need to consider total costs that will be incurred throughout the service life of a bridge, in addition to factors such as traffic, maintenance, structure longevity, convenience to the public, practicality of either option, and rehabilitation longevity. This chapter presents an overview of common types of rehabilitation and repair methods, including electrochemical rehabilitation, methods of concrete removal and patching, surface treatments, corrosion inhibitors, and epoxy injections.

4.2 ELECTROCHEMICAL REHABILITATION

Concrete is a permeable material that is susceptible to the ingress of chloride ions, with the two primary sources being deicing salts and seawater (*79*). Although the greatest concentrations of chloride ions typically occur near the upper surfaces of bridge decks, chloride ions can migrate deeper into the deck through convection and diffusion processes. Continuing migration of chloride ions into the deck ultimately leads to sufficiently high concentrations at the reinforcing steel to induce corrosion (*79*). As mentioned earlier, a standard threshold chloride concentration applicable to all bridge decks does not exist, since corrosion caused by chloride ions is a combination of several factors including moisture and oxygen (*15*). Without the presence of both of these factors, corrosion will not occur regardless of the concentration of chloride ions (*17*). Because concrete is heterogeneous in nature, it allows uneven distributions of chloride ions, moisture, and oxygen throughout the structure, which creates electrochemical cells with different potentials on adjacent reinforcing bars (*79*). The electrochemical cells drive the corrosion process and the eventual deterioration of the reinforcing steel and concrete (*79*). The following sections discuss several types of electrochemical methods that may be used to rehabilitate concrete bridge decks, including cathodic protection (CP), electrochemical chloride extraction (ECE), and electrochemical realkalization (ER).

4.2.1 Cathodic Protection

CP has been shown to increase the service lives of concrete structures by preventing or reducing corrosion of the reinforcing steel within the concrete (*79*). The idea of CP is to artificially shift the potential of the steel so that it becomes immune or passive (*80*).

One form of CP is the sacrificial-anode method, which is based on the principle of dissimilar metal corrosion and the relative positions of different metals in the galvanic series (*80, 81*). The method uses a metal that is more reactive than steel, such as zinc, to create a dominant flow of electrons to the reinforcing steel, forcing the steel to behave as the cathode in the electrochemical cell (*80, 81, 82*). The electrical current is generated simply by the potential difference between the anode (zinc) and the cathode (reinforcing steel) (*80, 81*). As long as the reinforcing steel acts as the cathode, only the anode will be consumed by the corrosion process, thus leaving the steel unharmed (*80, 81, 82*). Typically, the sacrificial anode is sprayed onto the surface of exposed reinforcing steel (*80, 81*). Sacrificial-anode CP systems do not require an auxiliary power supply and are particularly useful for protecting pre-stressed or post-tensioned concrete without the risk of elevated potential levels that can lead to hydrogen embrittlement of the steel (*80, 81*). The life of the sacrificial anode, however, may be relatively short compared to titanium-based anodes that are used with impressed-current systems (*79*).

Impressed-current CP systems use a supplemental anode that corrodes in place of the steel (*80, 83*). The anode is typically a non-reactive conductor such as carbon or titanium that is capable of sustaining considerable oxidation reactions while sustaining little physical damage (*80*). The supplemental anode is installed within or on the concrete surface and attached to the positive terminal of an electrical power source as shown in Figure 4.1 (*80, 83, 84*). The entire network of reinforcing steel in the structure is then connected to the negative terminal of the power supply, which forces it to become cathodic (*80, 83*). As long as the power supply is active, corrosion takes place on the supplemental anode and not on the reinforcing steel (*80, 82,* *83*). In order for the process to be effective, electrical continuity of the reinforcing bars is mandatory (*83*). If one of the bars is left isolated, it will not be protected by the CP system (*83*).

The service life of a CP system is based on the components of the entire system. For the impressed-current system, the components consist of a power source, a wiring system, and an anode (*79*). For the galvanic (sacrificial-anode) system, the components consist of only a wiring system and an anode (*79*). The power source is not required since the potential difference between the anode and the cathode provides the necessary protection current. In both systems, the wiring system should be maintenance-free if it is properly installed and well protected from moisture. In CP systems where an external power source is required, the power source incurs minimal cost and requires little maintenance as long as regular replacement of parts is performed and sufficient protection from power surges is provided. Therefore, the life of the anode is the most significant factor in determining the service life of a CP system.

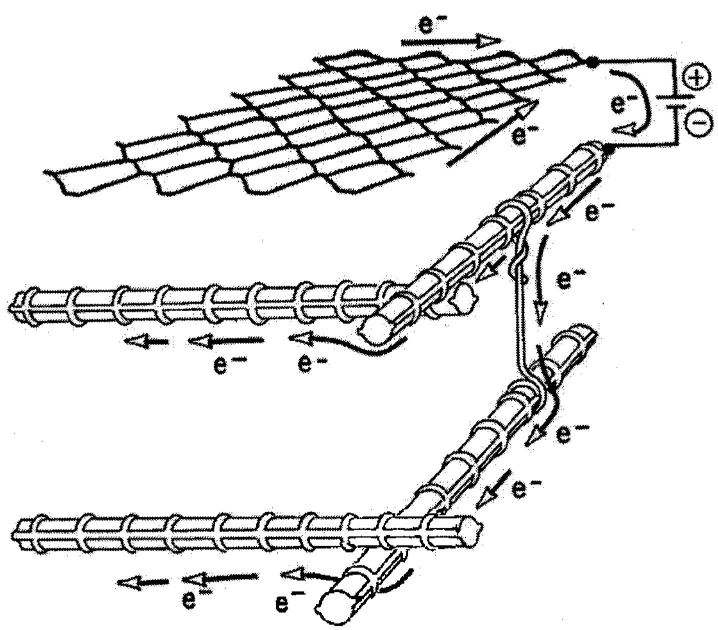

FIGURE 4.1 Supplemental anode for impressed-current cathodic protection (*84*).

Research has been conducted to predict the service lives of several anodes used in impressed-current and galvanic CP systems (*79*). The predictions are based on data extrapolation and engineering judgment. Titanium-mesh anodes used in impressed-current systems have a projected design life of 60 to 90 years, while thermally sprayed zinc coatings used in galvanic systems have projected life spans of less than 10 years (*79*). Although titanium-mesh anodes have an impressive design life, their higher costs should be considered before they are selected for use.

CP systems have proven to be an effective deterrent to corrosion problems. However, the use of CP has been limited to only major bridges and to concrete piers (*79*). One reason for the limited use of the impressed-current CP system is the maintenance of the electrical system (*79*). Although maintenance is relatively simple and inexpensive, it is new to many bridge engineers (*79*). The single most important factor limiting the use of CP systems has been the cost of installation, which can be significantly reduced if the system is installed simultaneously with the construction of a new bridge (*79, 84*).

Simultaneous installation offers three distinct advantages (*84*). First, it reduces cost by eliminating the need to remove large amounts of concrete in order to install the system, as is the case for treatment of existing bridge decks. Second, the amount of direct electrical current required to prevent the initiation of corrosion is only a fraction of that required to stop existing corrosion. Therefore, in new bridge decks, a smaller percentage of anodes would be required to keep the bridge corrosion-free. Third and last, the system would prevent chloride ions that infiltrate the concrete from coming in contact with the reinforcing bars. This would significantly maintain the integrity of the concrete for a longer duration of time.

4.2.2 Electrochemical Chloride Extraction

ECE is based on the fact that opposite electrical charges are attracted and like electrical charges are repelled (*79*). By applying an electrical current to the reinforcing steel, the ECE process actually reverses the course of corrosion, in addition to repassivating the steel (*79*). However, ECE should only be used where corrosion of the reinforcing bars has just started and not where a severe loss of cross-section has already occurred (*66*). ECE is similar to CP but requires an electrical current that is 50 to 500 times stronger. Unlike CP, however, ECE requires no anode materials or electrical components that must be regularly maintained during treatment (*79*).

Chloride extraction is performed by first constructing wooden dams around the area to be treated as shown in Figure 4.2 (*79*). The undersides of the wooden dams are caulked to ensure that no leaks occur. Titanium-mesh anodes are then sandwiched between two layers of synthetic felt and placed over the deck area enclosed by the dams. A liquid electrolyte (sodium borate or lithium borate) is then used to sufficiently inundate the sheets of felt and titanium anodes (*79*).

To minimize evaporation of the electrolyte, sheets of plastic are placed over the treatment area. A positive electrical charge is then applied to the anode, and a negative charge is applied to the network of reinforcing bars. Since chloride ions are negatively charged, they are repelled by the negatively charged reinforcing bars and attracted by the positively charged anode. Therefore, the chloride ions are driven from the reinforcing bars toward the surface of the concrete. The system is usually left in place for a couple of weeks (*79*).

During treatment of the bridge deck, the electrolyte may become too acidic due to the presence of chlorine that is formed by the electrolysis of water molecules and the oxidation of migrating chloride ions (*79*). The acidity of the electrolyte can lead to etching or softening of the cement paste that holds the aggregate together. In order to avoid this problem, an adequate liquid pumping system must be installed into each dam area in order to flush out electrolyte that is too acidic and replenish it with fresh electrolyte. Another way of addressing this problem is to remove the damaged concrete surface after treatment and then apply an overlay.

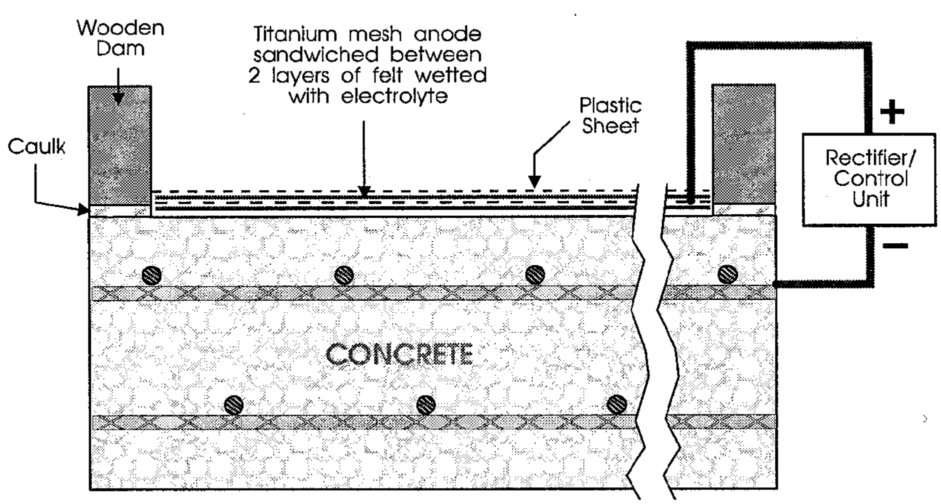

FIGURE 4.2 Schematic of the electrochemical chloride extraction process (*79*).

ECE is a relatively new rehabilitation method, which makes obtaining information on the length of protection very difficult. Similar to CP, the projected life of this method is only a prediction since long-term studies on ECE have yet to be completed. Estimations of the duration of ECE protection are based on the amount of time required for chloride levels to reach concentrations that will induce corrosion and the duration of the polarizing effects imparted to the reinforcing steel during the ECE treatment (*79*). Based on these considerations, the projected life of an ECE treatment is between 10 to 15 years (*79*). The application of a low-permeability concrete overlay after treatment will considerably reduce the recurrence of corrosion (*79*).

4.2.3 Electrochemical Realkalization

The reinforcing steel in concrete is protected by the highly alkaline environment (usually a pH of 13.5) provided by the surrounding concrete. The alkalinity of the concrete is predominantly due to the presence of calcium hydroxide ($Ca(OH)_2$) and alkali elements in the cement paste (*85*). When carbon dioxide from the atmosphere infiltrates the concrete surface, it reacts with the calcium hydroxide to form calcium carbonate ($CaCO_3$), which causes the pH of the concrete to decrease below 10, as discussed earlier (*85*). Because the passive oxide film on the steel becomes chemically unstable at a pH of approximately 11.5, this process of concrete carbonation can be a principal cause of corrosion of reinforced concrete structures in hot and dry environments (*85*). Corrosion caused by concrete carbonation can only be stopped by restoring the alkalinity of the concrete surrounding the reinforcing steel by the ER process.

The ER treatment process uses a direct electrical current to restore the alkalinity of the concrete (*85*). An anode is placed on the surface of the concrete and covered with a sodium carbonate (Na_2CO_3) electrolyte (*85*). As is the case with CP and ECE, the anode is connected to the positive terminal of an external power source and the cathode, or reinforcing steel, is connected to the negative terminal. When the power source is activated, the voltage potential forces the negatively charged ions (hydroxides) in the concrete to migrate toward the cathode, and the cathodic reaction at the reinforcing steel results in the formation of new hydroxyl ions (*85*). The accumulation of hydroxyl ions in the vicinity of the reinforcement elevates the pH of the concrete to approximately 13.5 and thereby repassivates the steel (*85*). A schematic representation of the process is shown in Figure 4.3 (*86*).

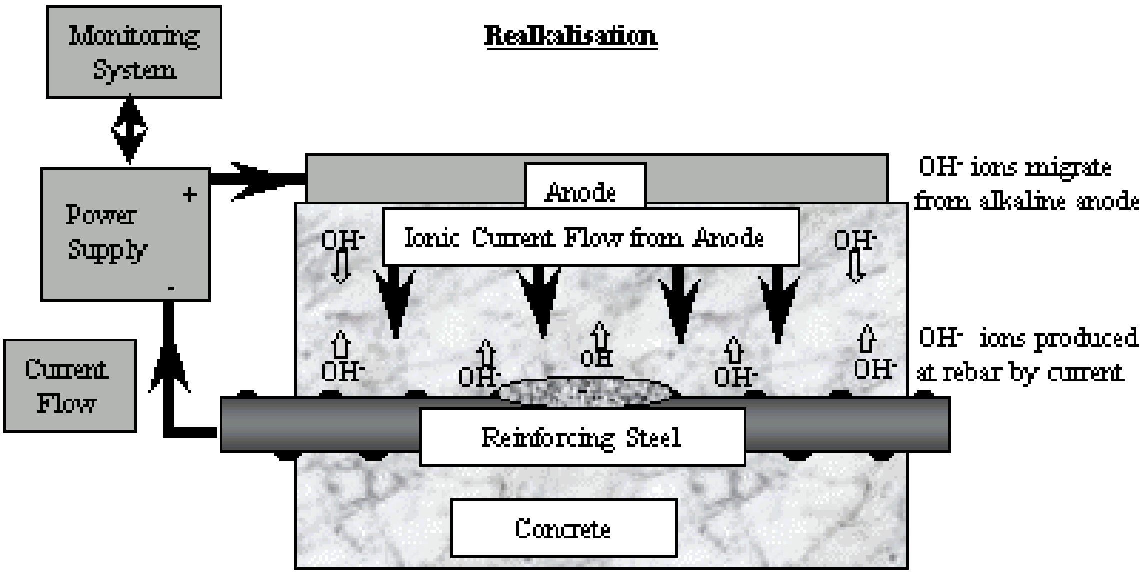

FIGURE 4.3 Schematic of the electrochemical realkalization process (*86*).

The ER process is very similar to CP and ECE but has several advantages. First, ER treatment is much easier and faster than chloride removal, only requiring a few days to perform (*80*). Second, the process changes the chemistry of the concrete, making it fundamentally different than untreated concrete (*85*). Studies have demonstrated that realkalized concrete is no longer susceptible to carbonation, for example (*80, 85*). Last, realkalized concrete does not easily leach alkali elements (*85*). Some leaching may occur in concrete with inadequate cover thickness that is exposed to severe weather conditions, but, for the most part, leaching does not occur. Such a characteristic of realkalized concrete is a major factor in retaining the passivity of the reinforcing steel.

One disadvantage of the ER treatment is that the elevated pH may accelerate ASR in concrete or may cause ASR to manifest itself in concrete that would not usually be affected (*80, 85*). As previously discussed, a Na_2CO_3 electrolyte is used during the ER process to provide lasting protection against carbonation by increasing the pH of the concrete (*80, 85*). However, the pH of the concrete may be increased to levels at which ASR can occur (*80, 85*). Additionally, the electrolyte may affect the adhesion of protective coatings on the concrete surface (*80*). Therefore, only low doses of Na_2CO_3 should be used (*80*). Any potential negative side effects of the realkalization processes can be controlled by careful engineering during design and installation of the system.

4.3 METHODS OF CONCRETE REMOVAL AND PATCHING

Until recently, concrete rehabilitation typically consisted of the removal of deteriorated concrete by saw-cutting or jack-hammering and then patching the damaged area with fresh concrete (*87*). However, these methods are often time-consuming and require careful treatment so as not to damage intact sections of concrete (*88*). In the past decade, new methods have been developed to accelerate the process of concrete removal and improve the performance of patch treatments. Thus, a variety of possible techniques is now available for concrete removal, including saw-cutting, hydrodemolition, jack-hammering, milling, shot-blasting, and sand-blasting.

Despite the widespread use of these methods, the removal of chloride-damaged concrete, in particular, and patching with fresh concrete can form strong electrochemical cells near the interface between the old chloride-contaminated concrete and the new chloride-free concrete (*79*). These cells cause strong potential gradients that may accelerate future corrosion. In many cases, patch repair requires rehabilitation within 1 to 2 years (*79*). Such undesirable patch performance has caused many transportation agencies to employ different rehabilitation strategies.

4.3.1 Saw-Cutting

Saw-cutting makes precise cuts at the perimeter of a deteriorated area of concrete and creates a clean separation between the existing concrete and the concrete to be removed (*89*). Removal of deteriorated sections is generally performed with jack-hammers. Before the area is patched, the smooth surface created by the saw must be roughened to ensure appropriate bonding between the patch material and the original concrete.

4.3.2 Hydrodemolition

Hydrodemolition removes deteriorated concrete through the use of a high-pressure water jet (*88, 90, 91, 92*). The jet removes weakened, fractured concrete while leaving sound concrete intact. The jet also cleans any exposed reinforcing steel by removing corrosion scale. However, hydrodemolition will not affect sound reinforcing steel, conduits, or electrical wires (*88*). Key advantages of hydrodemolition include the production of very little dust, relatively quiet operation, absence of significant vibrations that cause cracking in the remaining concrete, effective washing of aggregate, creation of a superior bonding surface, and retention of the bond quality between the reinforcing steel and sound concrete (*88*). The only disadvantage is that hydrodemolition uses significant amounts of water, which might cause drainage problems during rehabilitation.

4.3.3 Jack-Hammering

Jack-hammering requires the use of jack-hammers to break apart deteriorated concrete (*88*). A major disadvantage of this method is the amount and magnitude of vibrations caused when the jack-hammer strikes the embedded reinforcing steel. The vibrations can propagate through the bridge deck to areas of sound concrete, where they may cause microcracking and debonding of the concrete and rebar. Jack-hammering is also very slow, covering only 5 to 10 square feet an hour as opposed to 90 to 100 square feet an hour covered by hydrodemolition (*88*). Furthermore, jack-hammers produce considerable dust and debris while creating high levels of noise. Another disadvantage of the method is that the reinforcing steel must be sand-blasted afterwards since jack-hammers cannot remove rust and other corrosion products from steel.

4.3.4 Milling

Milling, also known as scarification, uses special grinding equipment to remove the upper surface layer of concrete down to a specified depth (*90, 93, 94*). A machine rotates a steel drum at high revolutions, and carbide or steel cutters mounted to the drum chip away the concrete as the drum rotates. The depth of penetration is typically only 0.5 inch, but deeper cuts can be obtained through multiple passes. The milling equipment can also reshape an existing deck to achieve a new surface profile, as well as to remove rutting, humps, and other roadway imperfections (*94, 95*). If the milling depth is sufficiently shallow, the resulting textured surface can be used by traffic even before the section is repaved (*95*). Milling is relatively simple, but the work is very tedious, labor-intensive, and costly (*90*). In addition, it generates significant noise and dust during removal of the concrete surface (*90*).

4.3.5 Shot-Blasting

Shot-blasting consists of projecting a large amount of small, steel balls downward onto the concrete, thereby chipping away the concrete surface (*89, 94*). The balls are recycled within the equipment and are used constantly throughout the entire process. Debris produced from the process is recovered using a special vacuum system. The method is fast and clean, producing very little dust, but is not ideal for thick repairs.

4.3.6 Sand-Blasting

Sand-blasting consists of projecting silica sand at high velocities to remove rust from steel (*89, 96*). The depth of removal depends on the size of the sand particles, quantity used, and projection speed. The method is quick and effective but produces significant amounts of dust. Cleaning is tedious, and the sand may cause the concrete surface to become smooth, thereby reducing the ability of the patch concrete to adhere to the rehabilitated concrete surface. The method is generally used for the removal of rust and corrosion from the surface of reinforcing steel but not for the removal of deteriorated concrete.

4.4 SURFACE TREATMENTS

For many years, the most common method of protecting concrete bridge decks has been the application of surface treatments. Surface treatments control corrosion of the reinforcing steel by minimizing the infiltration of moisture and chloride ions into the concrete. The general classifications of surface treatments are overlays, sealants, surface sealers, and membranes (*97*).

4.4.1 Overlays

Overlays are the simplest and most traditional rehabilitation option (*93*). Properly designed and constructed overlays will add structural strength, correct surface defects, and provide a smooth riding surface (*87, 93*). They also reduce oxygen, moisture, and chloride infiltration into the underlying bridge deck, thereby giving added protection to the embedded reinforcing steel. Overlays are not considered true rehabilitation methods, but rather repair methods, since the chloride-contaminated concrete is not removed (*87*). Although overlays provide added protection from harmful elements, the service life of an overlaid bridge deck is governed by the chloride contamination within the underlying concrete deck (*87*). That is, bridge deck overlays by themselves do not rehabilitate concrete decks but simply conceal manifested distress (*93*). In as little as 3 years, for example, underlying longitudinal and transverse cracking can be reflected into the overlay surface (*93*). In order to enhance the performance of overlays and maximize the service life of deteriorated bridge decks, concrete removal techniques should be considered in conjunction with overlays. In fact, many transportation agencies require concrete removal with overlay applications in order to maximize their investment.

Overlay materials should both impede the ingress of harmful chemicals and provide a wearing surface for traffic. However, other distinguishing features warrant the use of particular overlays in specific conditions. Overlay types include latex-modified concrete (LMC); low-slump, high-density concrete (LSHDC); microsilica-modified concrete (MSMC); and asphalt concrete (*78, 87, 92*).

LMC is made by introducing styrene butadiene latex into a concrete mixture (*92*). The use of LMC on bridge decks is highly desirable due to its superior bonding characteristics, impermeability to chlorides and moisture, and ability to be placed in thin layers (2.25 inches minimum) without excessive shrinkage cracking. The expected service life of an LMC overlay is 10 to 15 years with some patching required during that period (*78*).

LSHDC is based on the principle that moisture penetration into concrete can be reduced by decreasing the water content of the original mixture (*92*). Consequently, low water content results in a low-slump concrete, while proper consolidation results in a dense concrete matrix, thus rendering the concrete more resistant to the ingress of harmful chemicals and moisture. The expected service life of an LSHDC overlay is also 10 to 15 years (*78*). One disadvantage of LSHDC overlays is their construction difficulty. Although LSHDC overlays do exhibit adequate bonding and low shrinkage, difficulties in placing, compacting, and finishing have made them a less attractive alternative than LMC overlays.

MSMC is a concrete mixture that is modified by the addition of an admixture containing microscopic silica particles (*92*). The major economic advantage of MSMC is that it can be batched at any concrete plant with facilities capable of handling the microsilica admixture, whereas LMC requires the use of high-cost mobile mixers. The ability to batch at a concrete plant allows microsilica to be purchased in bulk quantities, which reduces the cost of MSMC to about half that of LMC. The expected service life of an MSMC overlay is also 10 to 15 years

(*78*). MSMC displays high early strength and very low permeability, which are ideal for preventing the intrusion of chloride ions and moisture. Unfortunately, MSMC does not bond as well as LMC and exhibits more shrinkage and less resiliency.

Asphalt concrete, or bituminous, overlays are generally used as temporary repairs on bridge decks scheduled to be replaced within a few years (*92*). The bituminous overlay is used simply to provide additional service life until replacement of the bridge deck can be performed. Typically, when a bituminous overlay is applied to a bridge deck, a waterproofing membrane is first installed to prevent further migration of chlorides into the underlying deck. The expected service life of a bituminous overlay is 5 to 10 years (*78*).

4.4.2 Sealants

The application of sealers is an effective method of inhibiting corrosion of reinforcing steel in concrete (*98, 99*). Bridge deck sealers provide a film that prevents the intrusion of harmful chemicals and moisture. While most sealers allow for little or no water vapor transmission into the atmosphere, some sealers are designed to allow for water vapor transmission out of the deck, and a few provide excellent perspiration. In unsealed concrete, the typical moisture level is approximately 50 to 80 percent of the saturation level (*98*). In contrast, the moisture levels in concrete protected by a breathable sealer are reduced to about 30 to 40 percent of the saturation level (*98*). When applied properly, concrete sealers offer protection for up to 5 years (*99*). Two types of concrete sealers are available: surface sealers and penetrating sealers.

Surface sealers, or coatings, are products that adhere to the surface of the concrete and form a waterproofing film (*98*). Surface sealers are generally used when the appearance of a bridge deck is of concern (*99*). When rehabilitation of a bridge deck involves only partial patching of the deck, for example, a coating may be applied to hide the repair work (*99*). The drawbacks of surface sealers are that they do not effectively bridge moving cracks (only non-moving cracks) and they are negatively affected by UV exposure and surface abrasion from traffic loads (*97*). Additionally, surface sealers do not allow for water vapor transmission, since the coating seals the pore openings (*100*). Therefore, if appearance of the bridge deck is not a concern, penetrating sealers should be used since they provide better protection against water and chlorides (*99*).

Penetrating sealers are products that are absorbed into concrete surfaces, where they chemically react to form a hydrophobic, water-repelling surface (*97, 98, 99*). Because the sealer is applied directly to the concrete surface, proper surface preparation is essential to achieve maximum penetration (*99*). Contaminants must be completely removed, and the surface of the concrete must also be dry (*99*).

Two types of penetrating sealers are available: water repellants and pore blockers (*100*). Water repellants provide moisture protection by coating pore walls with a hydrophobic film. Since the coating does not block pore openings, it allows the concrete to release excess moisture while simultaneously preventing moisture from infiltrating the concrete. The molecular size of pore blockers is larger than that of water repellants. Pore blockers provide moisture and chemical protection to reinforced concrete but do not allow for vapor transmission of excess moisture.

Penetrating sealers are very durable since they are not subject to abrasion and do not degrade under UV exposure (*97, 99*). Furthermore, penetrating sealers do not substantially alter the appearance of treated concrete (*97, 99*). However, unlike surface sealers, penetrating sealers neither conceal concrete repairs nor prevent water intrusion through open cracks (*97*).

Concrete sealers offer an efficient and cost-effective method of protecting concrete bridge decks. The cost of a concrete sealant is relatively low when compared to the cost of rehabilitating an entire deck. Many DOTs are now stressing prevention and protection by requiring the application of concrete sealers to all newly constructed and overlaid bridges (*99*).

4.4.3 Membranes

Membranes are typically applied to bridge decks in conjunction with protective overlays as barriers against water infiltration and as deterrents to reflective cracking (*101, 102*). Surface preparation of the bridge deck prior to applying the membrane is the most important part of the process (*102*). The deck must be completely repaired to ensure that air voids will not be trapped under the membrane (*102*). Afterwards, the deck must be cleaned of all dirt, grease, oil, and other materials that may damage the membrane or inhibit proper bonding (*102*). The bridge deck surface should then be allowed to dry before the membrane is applied (*102*).

Paving membranes are made of fiber-mesh fabric impregnated with polymer-modified or rubberized asphalt (*103*). Membranes have a sticky surface on one side that allows them to adhere to bridge deck surfaces (*103*). The application of a hot asphalt overlay causes the membrane to melt, consequently filling surface voids and tightly bonding the asphalt overlay to the bridge deck surface. The bond is often improved through the use of a concrete sealer placed prior to the installation of the membrane (*103*). Upon completion of the process, a watertight seal is created that limits the ingress of water and other harmful chemicals into the bridge deck.

In the United States, transportation agencies have moved away from the use of membranes and overlays on new bridge decks. Instead, they are requiring epoxy-coated reinforcing steel, higher concrete quality, and thicker concrete cover over the reinforcing steel (*101*). Nonetheless, membranes and overlays remain viable options for extending the service life of aged bridges; however, in these cases, research has shown that their use only retards the corrosion rate of reinforcing steel rather than stopping the corrosion process completely (*104*).

4.5 CORROSION INHIBITORS

A corrosion inhibitor is a substance that when added to an environment effectively reduces the corrosion rate of a metal exposed to that environment (*105*). In the case of bridge decks, the environment is the concrete, and corrosion inhibitors are added to the concrete mixture during the production process. Although corrosion inhibitors may be most effectively utilized in conjunction with new deck construction, their use in concrete patches and overlays is an appropriate application for rehabilitation and repair.

Inhibitors retard corrosion by increasing anodic and cathodic polarization, reducing the diffusion of ions, or increasing the electrical resistance of the metallic surface (*106*). As discussed earlier, an increase in electrical resistance restricts anodic and cathodic reactions from occurring on the reinforcement. Generally, inhibiting admixtures are classified as anodic, cathodic, or mixed inhibitors (*107*). The admixture classifications reflect the functionality of each inhibitor and the location of inhibitor action within the electrochemical cell (*106, 107*). Cost, toxicity, availability, and environmental friendliness are all important factors that should be considered when selecting an inhibitor (*106*).

Anodic inhibitors, such as calcium nitrite, block chloride-induced corrosion by chemically reinforcing and stabilizing the passive protective film on the reinforcement (*20, 108*). Research has shown that nitrites are required in order to prevent pitting corrosion (*20*). The efficacy of anodic inhibitors relies heavily on maintaining critical concentration levels within concrete (*20*). If the concentration level of calcium nitrite falls below a threshold level, then passivity will be lost, and corrosion may occur at greater rates than would occur with no inhibitor at all.

Cathodic inhibitors slow the corrosion rate by precipitating at cathodic areas or by limiting the availability of oxygen needed to drive cathodic reactions (*107*). Inhibitors containing calcium, zinc, or magnesium ions may be precipitated as oxides to recreate the protective layer on the reinforcement originally generated by the highly alkaline environment of the concrete (*106*). Other cathodic inhibitors, known as oxygen scavengers, starve the cathodic reaction of the oxygen necessary for the reaction to occur. The most commonly used oxygen scavenger is sodium sulfite (Na_2SO_3) (*106*).

Mixed corrosion inhibitors display characteristics of both anodic and cathodic inhibitors. These inhibitors influence the potential of the reinforcing steel by causing it to shift in either a positive or negative direction (*108*).

4.6 EPOXY INJECTIONS

Epoxy injections are an effective means of restoring the integrity of a concrete structure by completely sealing any open cracks that allow the ingress of harmful substances (*109, 110*). Injections effectively weld the concrete back together, thereby creating a structural bond that is stronger than the concrete itself and preventing cracks from propagating any further (*109, 110*).

The typical method of epoxy injection involves drilling a series of holes along the crack that are no farther apart than the depth of the crack (*110*). This ensures that the crack will be completely filled (*110*). Small tubes, or ports, are then installed into the holes, and epoxy is injected using a high-pressure, low-velocity flow machine (*109, 110*). The injection must be a low-viscosity epoxy in order to fully penetrate the crack, which must be clean of debris that may weaken the bond between the epoxy and the concrete (*110*). If necessary, the crack may be flushed out with water or compressed air before treatment (*110*). While moist cracks can be injected with epoxy, moisture may decrease the efficacy of the repair (*109*). After the crack is filled with epoxy resin, the ports are removed, and the epoxy is allowed to cure (*110*).

The use of epoxy injections offers several advantages that make it ideal as a repair method (*110*). Other methods may simply bridge the cracks or may deteriorate due to environmental conditions (*111*). Epoxy injections, on the other hand, completely fill crack voids, thereby starving the corrosion process of air, water, and chlorides (*110*). Additionally, epoxy injections exhibit high bond and tensile strengths, which prevent deterioration of the epoxy resin due to atmospheric and mechanical wear (*111*). Continued propagation of repaired cracks is also prevented as a result of the epoxy resin strength (*111*). However, epoxy injections do not guarantee that cracks will not form adjacent to the repaired crack if movement continues to occur (*109*).

Several disadvantages exist that may affect the use of this technique. Typically, epoxy injections must be performed by an experienced and qualified technician (*109*). Also, epoxy injections require significant amounts of time in preparation work and in performing the injections, which results in lane closures for extended periods of time. The damage would have to be severe to justify the closure of a bridge for the amount of time that would be required to perform epoxy injections.

CHAPTER 5
BRIDGE MANAGEMENT SYSTEMS

5.1 PURPOSE OF BRIDGE MANAGEMENT SYSTEMS

Over the past several decades, infrastructure problems in the United States have grown increasingly worse for several reasons, including lack of investing in public works programs, lack of an effective infrastructure management system, failure to recognize the value of a reliable infrastructure to the future economy, decreases in public works budgets, inability to replace the infrastructure as fast as it deteriorates, inability to recognize that a poor physical infrastructure limits the level and types of services that the government can provide to its citizens, tendency by government officials to postpone the maintenance of the infrastructure, and increases in costs to taxpayers to repair and rebuild the infrastructure (*112*).

Condition surveys of bridges in the United States continue to show that more than 200,000 bridges have deficient capacity or lack functional performance (*113*). More than 100,000 of these bridges have load capacities less than the legal limit. Another 5,000 bridges are completely closed. Approximately 150 to 200 bridges either partially or completely collapse every year. The cause of many bridge deficiencies can be attributed to poor maintenance. Deficiencies are increasingly pronounced as a result of deicing salt buildup, blocked drainage systems, leaky joints, and failed paint systems (*113*). As bridge deficiencies remain unrepaired, problems compound and ultimately limit the operational capabilities of the structure.

The easiest and most cost-effective way of minimizing bridge MR&R costs is to develop a BMS. A BMS is an organized approach to bridge MR&R that provides a planned procedure for performing these activities on all bridges within a given jurisdiction. The overall objective of a BMS is to maximize the service life of bridges through scheduled maintenance and repairs, where service life is the time between construction and replacement of a bridge. A BMS allows decision-makers at all bridge management levels to select optimum solutions from a variety of cost-effective alternatives, which should deliver the desired level of service while minimizing the overall life-cycle costs of a bridge.

The steps and objectives of a BMS are to predict bridge needs, define bridge conditions, allocate funds for both construction and MR&R actions, identify and prioritize bridges for

MR&R actions, identify bridges that require a load posting, find cost-effective alternatives for each bridge, recommend MR&R actions, account for MR&R actions, schedule and perform minor maintenance, monitor and rate bridges, and maintain an appropriate database of information (*114*). Figure 5.1 shows the fundamental organization of a BMS, which is divided into three distinct and interrelated groups: administrative planning, programming, and implementation (*114*).

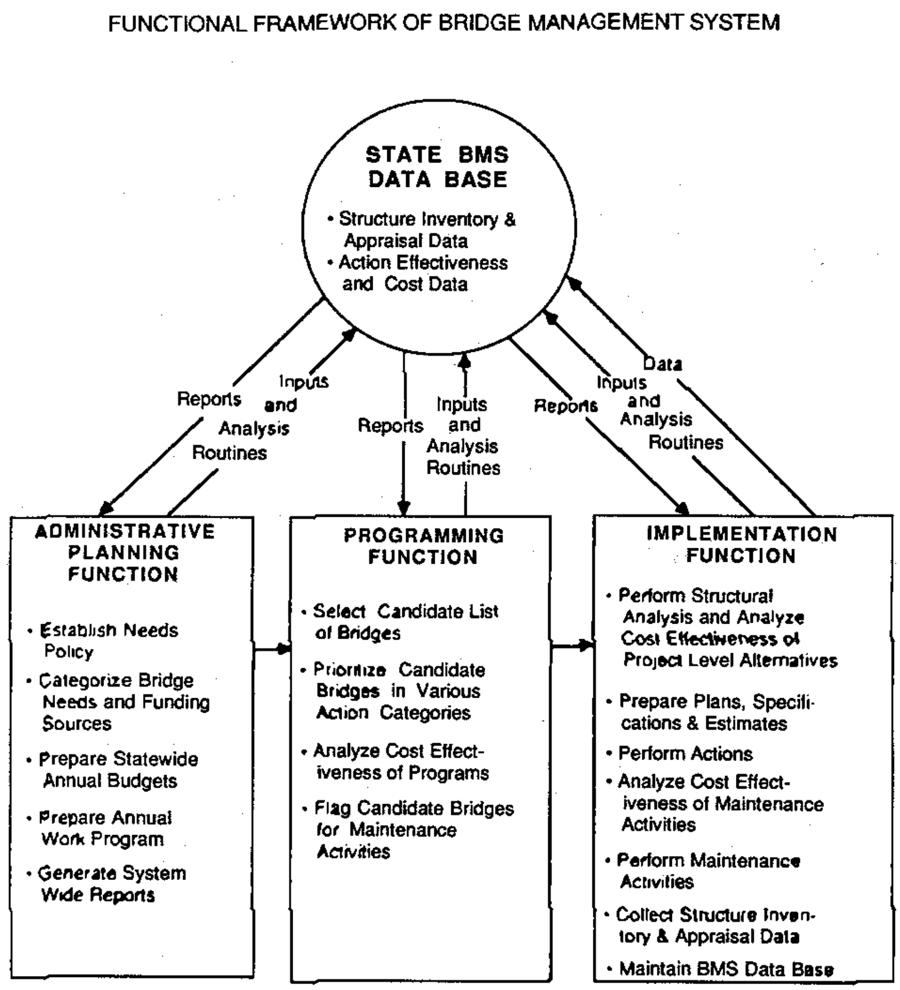

FIGURE 5.1 Conceptual framework of a BMS (*114*).

The administrative planning group is accountable for establishing needs policies, categorizing bridge needs and funding sources, preparing annual budgets for the organization, preparing annual work programs, and generating system-wide reports. The programming group is responsible for selecting candidate bridges, prioritizing candidate bridges into various action categories, analyzing the cost effectiveness of the programs, and flagging candidate bridges for maintenance activities. The responsibilities of the implementation group consist of performing structural analyses; analyzing the cost effectiveness of project-level alternatives; preparing plans, specifications, and estimates; performing the actions; analyzing the cost effectiveness of maintenance activities; performing maintenance activities; and maintaining the BMS database (*112, 114*).

The fundamental organization and distribution of responsibilities of a BMS can be adjusted to meet the needs and abilities of individual transportation agencies. For example, some responsibilities may be moved into other areas of the organization so that each group carries an equal responsibility and workload throughout the process. The following sections discuss in further detail the basic organization of a BMS, including network and project levels, data collection, performance indices, costs, and institutional issues.

5.2 NETWORK AND PROJECT LEVELS

In the early 1990s, an analysis was conducted on the spending trends of the United States government for the period 1945 to 1985 (*112*). The results showed that under-investment in infrastructure began in about 1968, and signs of deterioration became evident 5 years later (*112*). Due to this under-investment and the lack of effective infrastructure management, organizational changes were necessary to better utilize the workforce responsible for the bridge inventory nationwide. These personnel changes allowed for better management of the funds allocated for bridge MR&R. The reorganization of the management occurred at two distinct but closely integrated levels: network-level and project-level management. Figure 5.2 provides a diagram delineating the major operational activities of each level of management as applied to pavements (*112*).

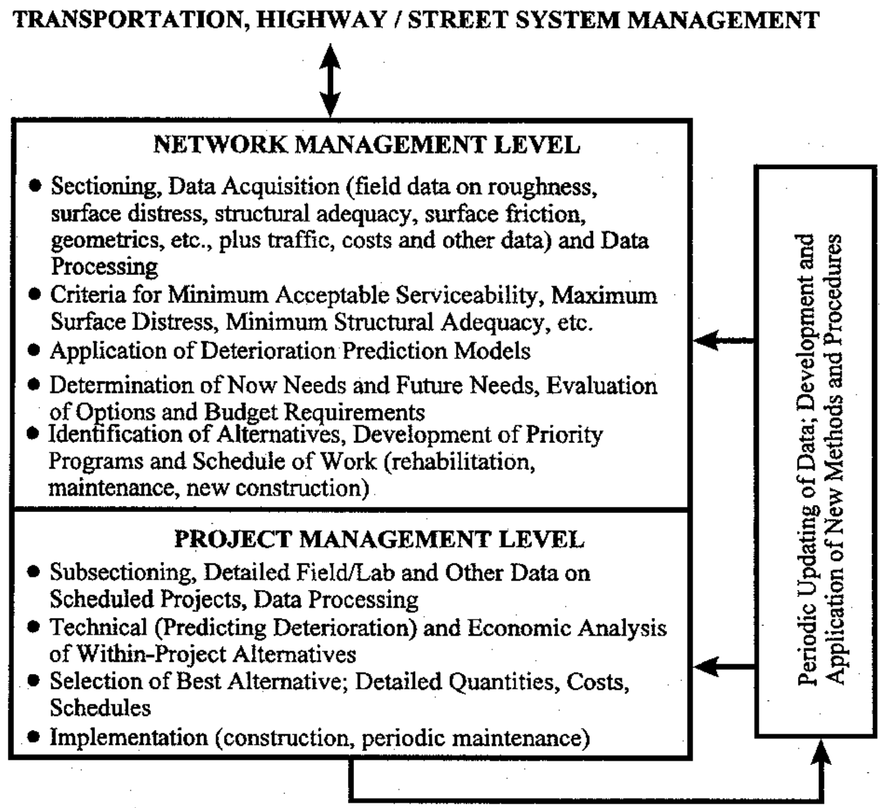

FIGURE 5.2 Basic operating levels of pavement management (*112*).

5.2.1 Network Level

Network-level management uses summary information related to the entire bridge network in order to maintain the overall condition of the system. Decisions made at the network level must consider the condition of the entire bridge network under a specific jurisdiction as opposed to the condition of an individual bridge (*115*). The main concern at the network level is the acquisition of appropriate funds to maintain the performance of the bridge network at a desirable level. Once funds have been secured, network-level managers allocate those funds to individual bridges, regions, or districts within the organization and ensure that the funds are used effectively (*114*). The important tasks at the network level include identification of bridge maintenance and rehabilitation needs, selection of candidate bridges, and evaluation of impacts that particular funding alternatives may have on the overall condition of the bridge network (*116*).

Features of a network-level BMS include an inventory database, an assessment of condition and needs, a program for capital and maintenance projects, an evaluation of budget options, and a reporting mechanism (*117*). Based on these features, the principal results of a network-level analysis include maintenance and rehabilitation needs, funding requirements, prioritization of candidate projects requiring MR&R, and forecasting of future conditions for different funding alternatives.

5.2.2 Project Level

Project-level management deals with bridges on an individual basis when considering various MR&R alternatives (*114*). Priority and funding decisions handed down from network-level managers are followed up with a detailed, project-level analysis of each selected bridge (*114*). The main purpose of the analysis is to assess the factors causing deterioration of each bridge and identify the most efficient and cost-effective MR&R strategies. Selection of the MR&R strategy is typically based on bridge condition assessment tests, economic analyses, and optimization techniques (*117*).

5.3 DATA COLLECTION

Routine analysis of bridge condition information is an essential operational component of a BMS (*114*). Therefore, collection and storage of bridge inventory, condition, and MR&R data are important tasks for bridge inspectors and managers. These data form the basis by which bridges are analyzed and selected for rehabilitation or replacement. Only data that contribute to an accurate life-cycle cost analysis (LCCA) should be collected, where the life cycle is defined as the actions, events, and outcomes that occur during the service life of a bridge (*118*). Excess data can make the system less manageable, more expensive, and less accurate. All data included in the BMS should contribute directly to the objectives of the system. In fact, excessive data collection is a principal reason for the abandonment of BMSs (*119*). Therefore, in the development of a BMS, every piece of data that is collected should be useful for at least one of the following reasons: identifying bridges or decks with poor performance, establishing priority, selecting maintenance or rehabilitation actions, calculating the cost of maintenance or rehabilitation actions, or estimating life-cycle costs for each maintenance and rehabilitation action (*120*). Any information that is not used in these tasks should be discarded. Additional information may be collected, but the criteria for selection should consider how the information is used in the BMS and the purpose that the information serves the agency.

The condition of bridge decks can be assessed based on several performance indicators, including pavement thickness; structural capacity index based on deflection; distress index based on distress type, severity, and extent; and surface smoothness (*112*). Additional information that may be of interest includes the average and standard deviation of surface chloride concentrations (from 0.25- to 0.75-inch depths), the average depth and standard deviation of bar cover, the percentage of delaminated area (not including spalls), the percentage of spalled area, the percentage of concrete samples with chloride contents higher than 3.5 percent by weight of cement at the reinforcement, the concrete water-cement ratio, the area of the deck slab, the bridge identification number, the year constructed (or reconstructed), the year of survey, and the snowfall range within the jurisdiction (*121*).

The frequency of data collection will depend on the available workforce, the size of the bridge network, and the available funds to support inspection efforts (*121*). Visual inspection data may be collected every year since no equipment is involved; however, data requiring expensive equipment may be collected less frequently until deterioration of the bridge deck has become severe. The timing of data collection will also vary according to the frequency of LCCAs conducted.

5.4 PERFORMANCE INDICES

The use of performance indices is a simple way for ranking projects and screening the network for general MR&R needs (*114*). The aggregate score for a project is based on a formula that considers bridge condition, function, use, and importance. The score represents the ability of a bridge to serve its intended functions with respect to other bridges in the network. MR&R options may be suggested based on threshold values of the particular index. However, determining MR&R strategies from this simple ranking will only provide a rough estimate of needs and should not be considered as an effective bridge management strategy by itself. The overall cost of a bridge project can be estimated based on the size of the bridge structure and associated costs for the type of work to be performed.

The Federal Highway Administration (FHWA) has established a sufficiency index called the Federal Sufficiency Rating System (FSRS). The FSRS is a screening index to determine if bridges qualify for federal funding through the Federal Highway Bridge Replacement and Rehabilitation Program (HBRRP). The FSRS rating is a statistic that allows for a non-biased distribution of federal funds to state transportation organizations (*114*).

In addition to the FSRS, most transportation agencies have developed their own sufficiency index ratings that are tailored to conditions specific to their own jurisdictions (*114*). The individual rating system is designed to include those factors that have the most significant impact on bridges within the stewardship of a given agency. Those factors may be adjusted over time to reflect policy updates and regulation changes within the organization. The agency may also experiment with different combinations of variables and weighting factors with the purpose of optimizing the efficacy of the sufficiency index (*114*). Such a customized rating system allows agencies the flexibility to manage their bridge networks based on the factors that have the most impact on performance.

Sufficiency index ratings for transportation agencies generally rate the conditions of individual bridge components separately. Such an approach is desirable because some areas of a bridge experience more environmental distress and vehicle wear than other parts. For example, bridge deck condition often rates significantly lower than the bridge substructure or superstructure since bridge decks are exposed to the elements and harmful chemicals while the substructure and superstructure are relatively protected (*112*). The individual ratings of each bridge element are then combined into a composite condition index (CI) score. The CI score serves as a summary evaluation to administrators at the network level by condensing data into a more manageable format that promotes better organization and decision-making for the entire bridge network (*112*). The rankings are typically based on a scale from 0 to 100, where 0 represents the worst possible, or failed, condition and 100 represents the best possible condition (*112*).

Prediction models are often developed to assist transportation agencies in forecasting bridge deck conditions in terms of either individual condition assessment ratings or CI scores. Usually an S-shaped curve is used to define performance. Figure 5.3 displays a typical model (*112*). The deterioration process and performance curve can be categorized into three separate phases. Phase I commences immediately following construction or major rehabilitation. The slope of the curve and the length of the time period in each phase are indicative of the performance of the bridge deck. Good performance is characterized by a small slope and a long time period. Ideally, the length of the Phase I period would be equal to the design life of the bridge deck. However, the intrusion of deicing chemicals and harmful substances into the bridge deck causes accelerated deterioration that is reflected in a low CI score, which causes the slope of the curve to become steeper.

Without appropriate maintenance and repair, the condition increasingly deteriorates along the S-shaped curve toward the minimum acceptable level, as shown in Phase II. At some point in Phase III, major rehabilitation is required to raise the CI score of the bridge deck to an acceptable value, at which time the deterioration process recommences. With reference to Curve 2 shown in Figure 5.3, however, major rehabilitation is not sufficient to restore the CI score to 100, which can only be accomplished by a full-deck replacement. Consideration should also be given to the fact that deterioration occurs more rapidly after rehabilitation than after original construction.

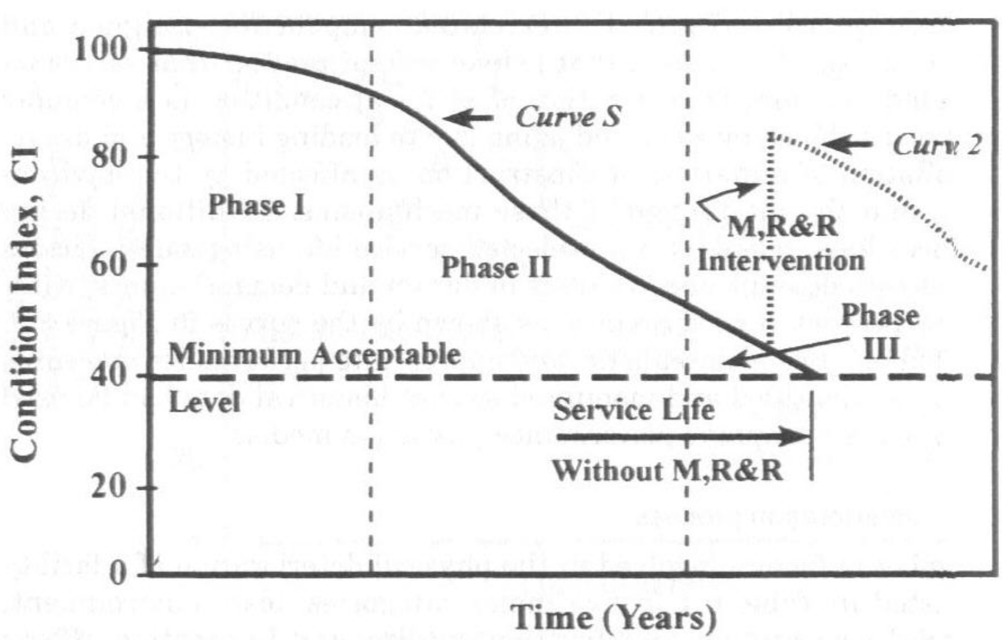

FIGURE 5.3 Effect of MR&R action on performance (*112*).

5.5 COSTS

A major challenge to transportation agencies is scheduling economically efficient bridge MR&R actions to achieve the 50- to 100-year service life that most agencies designate as an acceptable target. Recently, improvements have been made to BMSs that have allowed engineers to better schedule MR&R actions so that the service life of a given bridge is maximized, while the costs of maintaining the bridge are minimized. These improvements are based on an LCCA, which assists decision-makers in comparing and selecting alternative strategies for managing a bridge (*118*).

The cost of a bridge is not a one-time expenditure, but rather a long-term investment that is prolonged over decades. Over the life span of a bridge, periodic maintenance and rehabilitation are required to ensure that the bridge is safe and functions effectively. The life span of a bridge is terminated when the bridge fails functionally or structurally and must be replaced. In order to properly maintain a bridge, management strategies must be implemented to specify the type of material and design to use, as well as the type and timing of repairs needed. These decisions are based on the agencies' expectations of acceptable service life and associated costs (*118*).

LCCAs can be conducted based on performance curves generated for specific bridge elements, such as a bridge deck, to estimate the performance and service life of different repair strategies and to compare the long-term costs of competing maintenance scenarios (*118*). Essentially, an LCCA is a method for considering the efficiency of expenditures and for identifying the actions that provide the most life extension for the least cost, therefore allowing transportation agencies to capitalize on the full potential of scarce resources. The economic evaluation of a bridge structure should consider agency costs and user costs associated with possible MR&R strategies. These costs can be combined to determine total life-cycle costs that are considerate of both implicit and explicit factors.

5.5.1 Agency Costs

Agency costs include those directly related to MR&R actions. The routine maintenance of a bridge structure is typically performed by the agency's workforce. Costs for such maintenance should be developed as a function of the material type, condition, location, average daily traffic (ADT), highway classification, and other important factors relating to bridge elements. While most rehabilitation activities are designed by in-house engineers and performed by internal maintenance crews, some major rehabilitation options may require the assistance of an outside consulting engineer or contractor, depending on the difficulty of the project and the level of experience of in-house personnel (*118*). Costs for element rehabilitation activities should consider individual bridge element types and the different rehabilitation options applicable to each element.

Replacement costs are incurred from the replacement of bridge decks and entire bridge structures. In general, bridge deck replacements are considered major rehabilitation options, and their costs should be estimated in the same manner as element rehabilitation costs. However, they should be separated into individual categories because the funding sources may be different for each one. The replacement of an entire bridge structure is classified in the replacement category. New bridge construction caused by the construction of new roads is also included in the replacement category. Costs associated with new bridge construction are a function of the length, width, and height of a bridge; the number and length of individual spans; the types of materials used for the substructure and superstructure; structural type; bridge location; and the feature being bridged (*118*). Division of the project into different elements and the use of cost data from previous replacement projects for particular elements may assist in reducing the complexity of bridge MR&R cost estimates.

Included in the subcategories of agency costs are expenses associated with materials, personnel, and equipment (*118*). In order to estimate costs in these subcategories, maintenance of a good cost-accounting system is essential. Relevant data such as the type of action performed on each bridge element, the costs incurred, the condition of the bridge element preceding and following the activity, and other related information should be registered in a database to be used for future estimates.

5.5.2 User Costs

User costs primarily stem from the functional deficiencies of a bridge (*118*). Deficiencies such as load postings, clearance restrictions, and closures may result in higher vehicle operating costs due to detours, higher accident rates, and congestion (*118*).

During bridge MR&R, vehicles must often take detour routes to bypass bridge closures. The costs incurred by detoured traffic consist of additional vehicle operating costs and the value of lost time. Total user costs for a particular detour may thus be estimated by multiplying the number of detoured vehicles with a monetary value that accounts for these factors. Future user costs may be similarly estimated based on projected future traffic volumes.

Vehicle accidents attributed to bridge deficiencies also contribute to user costs. The frequency of certain types of accidents can often be correlated with specific functional bridge deficiencies. Costs associated with these accidents can be projected by rating the severity of each crash with respect to the bridge deficiency. The overall crash costs can then be determined by multiplying the rate of accidents for a specific deficiency to the projected costs of that particular accident.

Bridge MR&R typically causes increased traffic congestion on the bridge, as well as on arterial roadways adjacent to the bridge. The costs of congestion are thus felt by both the users of the bridge and the users of the surrounding roads. The designation of alternative routes to alleviate congestion in the immediate vicinity of the bridge may be appropriate, but this strategy may accelerate deterioration along those routes.

5.5.3 Software

Many transportation agencies have developed BMS software to assist in funding distribution and bridge MR&R prioritization. One such computer-based system is PONTIS, which was developed under an FHWA project and is available through AASHTO (*118*). Another computer-based BMS similar to PONTIS is called BRIDGIT (*118*). BRIDGIT was developed under the National Cooperative Highway Research Program (NCHRP) and is available from National Engineering Technology.

PONTIS and BRIDGIT support a string of activities including information gathering and interpretation, prediction of bridge conditions, cost accounting, decision-making, budgeting, and planning. The software systematically addresses each of these factors in order to predict deterioration, provide costs, and compare possible actions. Like most computer-based management systems, PONTIS relies on mathematical assumptions to generate life-cycle predictions. While it is not necessary for transportation agencies to understand the mathematical equations driving the model, they should clearly understand the strengths and weaknesses of the projections.

5.6 INSTITUTIONAL ISSUES

Considerable time and effort have been invested in the development and implementation of BMSs. However, the greatest problems many transportation agencies face regarding the successful implementation of a BMS are institutional issues. Institutional issues stem primarily from the reluctance of individuals within a given agency to adopt new policies and practices, even though implementation of new systems will allow them to be more effective in their jobs. In order for change to be effective, top administrators must therefore champion the cause themselves by communicating the vision to others, offering training programs to staff members, and ensuring the availability of resources needed to complete the tasks.

Communication is essential in maintaining an effective BMS. The organization of the system should be such that administrators at the top level of the agency are in relatively close contact with the design and maintenance crews at the project level. The separate entities are actually interrelated groups that depend on each other to make proper decisions for the entire network. For example, the administrative planning team requires prediction models from the programming group. Design teams require the administrators to allocate sufficient funding to support project-level bridge maintenance. The programming group requires the maintenance crews to accurately provide bridge condition information so that correct future funding needs are anticipated. Thus, communication at all levels is vital for the success of a BMS. Indeed, a breakdown in communication could result in a breakdown of the entire BMS.

A well-trained staff is also important to the successful management of a BMS (*112*). Several areas of the BMS organization require constant training programs that keep personnel current on technological advancements and new equipment. In particular, extensive changes in computer technology have necessitated the need for routine training for all computer-based jobs (*113*). Changes within the computer industry have revolutionized the organization of bridge network databases. The changes have allowed for better management of collected data and improved decision-making processes. As the computer industry continues to develop, transportation agencies should take advantage of emerging technologies that can improve their current BMSs. Through training programs, agencies will be able to improve the overall efficacy of their systems by improving the skills of their employees.

Bridge maintenance crews should also be trained on emerging technologies within their field (*113*). Bridge deck condition assessment proves to be more accurate and reliable every year due to continuing developments in testing equipment. Effective use of the equipment requires training of personnel, which also ensures that bridge condition assessments across the network are as repeatable and reliable as possible. Personnel who complete training programs satisfactorily should be recognized for their accomplishments. Where it is appropriate, completing proper training courses should be a condition for advancement.

In addition, equipment, staff, garage, and office space is needed to develop and implement in-service monitoring, evaluation plans, and a BMS database (*112*). The amount of needed resources and the complexity of the database system depend on the size of the bridge network and the short-term and long-term goals for the BMS. If the resources are strictly for administrative needs, then the resource demand should not be significant. However, as the need for resources trickles down into the design and maintenance teams, the demand on resources increases. Thus, effective utilization of all resources becomes essential for managing a BMS, as funding is always limited.

CHAPTER 6
SURVEY RESULTS

6.1 SURVEY PURPOSE

As the culminating element of this research, a questionnaire survey was conducted to investigate current concrete bridge deck MR&R techniques practiced by DOTs throughout the United States. UDOT personnel e-mailed the surveys to 43 DOTs, and responses were received from the following 28 states: Arizona, California, Connecticut, Illinois, Iowa, Kansas, Maryland, Minnesota, Missouri, Montana, Nebraska, Nevada, New Hampshire, New Jersey, New Mexico, New York, North Carolina, Oklahoma, Pennsylvania, Rhode Island, South Carolina, Tennessee, Texas, Utah, Vermont, Virginia, Washington, and Wyoming. The survey included 36 questions organized into the following sections: participant, climate and traffic, deck construction, winter deck maintenance, deck deterioration, deck condition assessment, and deck improvement. Survey responses are summarized in this chapter.

6.2 PARTICIPANT

The purpose of the participant section was to obtain contact information to facilitate follow-up questioning as needed. Therefore, participant information is not included in this report. The following questions 1 to 5 identify the information that was acquired.

1. What is your name?
2. What is your job title?
3. For which state department of transportation do you work?
4. What is your phone number?
5. What is your e-mail address?

6.3 CLIMATE AND TRAFFIC

The intent of collecting climate and traffic data was to identify the impact of these factors on concrete bridge deck performance. In addition, this information enables UDOT to make direct correlations between bridges found in Utah and those found in other states with similar climate and traffic conditions. The climate and traffic section deals with information concerning freeze-thaw cycles, relative humidity, and ADT demands. The respondents also indicated the name of the nearest major city within their jurisdiction. Information were obtained using questions 6 to 9, shown below. Summaries of the information are tabulated in Table 6.1 and Figures 6.1 and 6.2 and discussed in the commentary.

6. What is the name of the nearest major city whose climate is representative of the weather in your jurisdiction?
7. How many freeze-thaw cycles do concrete bridge decks typically experience within your jurisdiction?
8. What is the typical relative humidity within your jurisdiction?
9. What is the average daily traffic (ADT) for a typical concrete bridge deck on a major highway within your jurisdiction?

The following cities were represented in the surveys: Phoenix, Arizona; Sacramento, California; Hartford, Connecticut; Springfield, Illinois; Des Moines, Iowa; Kansas City, Kansas; Baltimore, Maryland; Minneapolis, Minnesota; St. Louis, Missouri; Helena, Montana; Omaha, Nebraska; Reno, Nevada; Concord, New Hampshire; Trenton, New Jersey; Albuquerque, New Mexico; Raleigh, North Carolina; Oklahoma City, Oklahoma; Pittsburgh, Pennsylvania; Providence, Rhode Island; Columbia, South Carolina; Nashville, Tennessee; Dallas, Texas; Salt Lake City, Utah; Montpelier, Vermont; Washington D.C. (Virginia); and Cheyenne, Wyoming. Respondents from the states of New York and Washington did not cite a specific city, indicating instead that the climatic variation was too great across those states to be represented by a single location. The purpose of collecting the data was to enable analysis of weather trends and to allow special consideration of practices used in states with climatic conditions most similar to Utah. Table 6.1 provides temperature data for each major city identified in the survey.

TABLE 6.1 Climatic Data for Participating DOTs

State	City	Weather Station	Air Temp.	Mean	Std. Dev.	Min.	Max.	Yrs.
Arizona	Phoenix	Phoenix City	High 7 day (°C)	43.3	1.2	41.7	46.1	27
			Low 1 day (°C)	14.6	3.3	10.0	26.1	29
California	Sacramento	Sacramento Executive Airport	High 7 day (°C)	38.3	1.5	34.2	41.7	40
			Low 1 day (°C)	13.1	2.2	8.9	17.8	39
Connecticut	Hartford	Hartford Bradley Airport	High 7 day (°C)	32.9	1.5	30.1	36.3	39
			Low 1 day (°C)	16.0	3.9	10.0	25.0	40
Illinois	Springfield	Springfield Capital Airport	High 7 day (°C)	34.2	1.8	30.2	39.1	47
			Low 1 day (°C)	15.6	5.6	5.0	27.8	47
Iowa	Des Moines	Des Moines International Airport	High 7 day (°C)	34.1	1.9	30.0	38.7	47
			Low 1 day (°C)	13.8	6.2	5.0	32.2	48
Kansas	Kansas City	Kansas City Municipal Airport	High 7 day (°C)	36.1	2.0	32.9	42.2	32
			Low 1 day (°C)	16.3	7.2	5.0	35.0	33
Maryland	Baltimore	Baltimore	High 7 day (°C)	34.1	1.3	31.7	36.9	46
			Low 1 day (°C)	12.9	4.0	5.0	24.4	48
Minnesota	Minneapolis	Minneapolis International Airport	High 7 day (°C)	32.5	2.2	27.7	40.0	92
			Low 1 day (°C)	14.4	5.9	4.4	29.4	95
Missouri	St. Louis	St. Louis Lambert Airport	High 7 day (°C)	35.3	1.8	32.2	39.7	47
			Low 1 day (°C)	15.2	5.5	4.4	30.0	47
Montana	Helena	Helena Airport	High 7 day (°C)	32.6	1.7	28.1	36.8	99
			Low 1 day (°C)	13.4	5.3	5.6	29.4	98
Nebraska	Omaha	Omaha Eppley Airfield	High 7 day (°C)	35.0	1.9	30.1	39.5	45
			Low 1 day (°C)	15.5	5.6	6.1	32.8	44
Nevada	Reno	Reno Cannon International Airport	High 7 day (°C)	36.3	1.2	33.0	39.2	58
			Low 1 day (°C)	17.4	2.9	11.1	22.8	51
New Hampshire	Concord	Concord Municipal Airport	High 7 day (°C)	31.9	1.6	29.1	35.9	67
			Low 1 day (°C)	19.3	4.0	11.1	28.9	63
New Jersey	Trenton	Trenton WSO City	High 7 day (°C)	33.0	1.5	30.7	36.2	29
			Low 1 day (°C)	12.0	4.1	3.3	19.4	30
New Mexico	Albuquerque	Albuquerque International Airport	High 7 day (°C)	36.4	1.4	33.6	39.8	62
			Low 1 day (°C)	15.1	4.1	6.1	28.3	55
New York	New York	New York John F. Kennedy Airport	High 7 day (°C)	31.8	1.7	28.2	34.5	32
			Low 1 day (°C)	12.3	5.1	5.0	21.1	33
North Carolina	Raleigh	Raleigh Durham Airport	High 7 day (°C)	34.4	1.5	31.5	37.8	48
			Low 1 day (°C)	14.5	4.7	7.2	27.8	48
Oklahoma	Oklahoma City	Oklahoma City Rogers	High 7 day (°C)	37.5	1.9	33.7	41.1	43
			Low 1 day (°C)	12.9	6.4	5	32.2	44
Pennsylvania	Pittsburgh	Pittsburgh GR P'Burg	High 7 day (°C)	31.9	1.7	29.2	36.2	43
			Low 1 day (°C)	14.6	5.7	5.6	31.7	44
Rhode Island	Providence	Providence Green Street	High 7 day (°C)	31.5	1.8	27.6	35.8	46
			Low 1 day (°C)	13.8	5.7	5.6	27.2	44
South Carolina	Columbia	Columbia Metropolitan Airport	High 7 day (°C)	36.6	1.7	32.3	39.8	48
			Low 1 day (°C)	16.1	4.7	9.4	29.4	46
Tennessee	Nashville	Nashville Metropolitan Airport	High 7 day (°C)	35.1	1.6	32.8	39.3	48
			Low 1 day (°C)	14.5	6.0	6.1	35.0	46
Texas	Dallas	Dallas/Fort Worth Airport	High 7 day (°C)	38.5	1.5	35.7	43.5	23
			Low 1 day (°C)	12.0	6.9	3.3	29.4	21
Utah	Salt Lake City	Salt Lake City International Airport	High 7 day (°C)	36.5	1.3	33.3	39.8	47
			Low 1 day (°C)	15.6	3.2	7.2	22.8	46
Vermont	Montpelier	Montpelier Airport	High 7 day (°C)	29.3	1.5	26.2	32.1	39
			Low 1 day (°C)	17.7	6.1	8.3	33.9	39
Virginia	Washington D.C.	Washington D.C. Dulles Airport	High 7 day (°C)	33.9	1.6	30.2	36.6	34
			Low 1 day (°C)	16.2	4.4	10.0	27.8	34
Washington	Seattle	Seattle-Tacoma Airport	High 7 day (°C)	29.1	2.2	23.3	33.6	46
			Low 1 day (°C)	8.7	2.2	5.0	16.7	44
Wyoming	Cheyenne	Cheyenne Municipal Airport	High 7 day (°C)	31.3	1.5	27.7	35.3	80
			Low 1 day (°C)	17.8	7.1	5.6	36.1	76

The data were obtained from the DataPave software developed through the SHRP Long-Term Pavement Performance (LTPP) program (*8*). An analysis of the climatic data identified several states with temperature extremes similar to Salt Lake City, Utah. In order of most similar to least similar, these are Albuquerque, New Mexico; Reno, Nevada; Columbia, South Carolina; Nashville, Tennessee; and Omaha, Nebraska. Among these, Tennessee and South Carolina may be categorized as wet with freeze-thaw cycling, while Nevada and New Mexico may be classified as dry with freeze-thaw cycling (*14*). Only Nebraska has a dry, hard freeze and spring thaw that are characteristic of much of Utah. Therefore, to the extent that regional weather trends influence the design, maintenance, and rehabilitation of concrete bridge decks, UDOT may benefit most by comparing practices with the Nebraska DOT.

Figure 6.1 shows that most states experience more than 20 freeze-thaw cycles in a year. According to the survey received from UDOT, bridge decks within Utah also experience more than 20 freeze-thaw cycles. Additionally, UDOT indicated that the typical relative humidity in the state is between 0 and 10 percent. As shown in Figure 6.2, all respondents specified a relative humidity greater than 10 percent. Therefore, UDOT has a climate that is significantly drier than that of most other states. A graph of ADT values is not included because of the difficulty encountered in tabulating that data. However, respondents generally indicated an ADT range of 50,000 to 120,000 vehicles. Eight other respondents answered the question with the term "varies" or with a variable range of numbers. The ADT for UDOT was specified as 10,000 vehicles. Traffic data collected by UDOT show that the ADT is 7280 at mile post 395 on the I-15 freeway; 12,007 at mile post 5 on US-40; and only 1364 at mile post 190 on SR-89 (*122*).

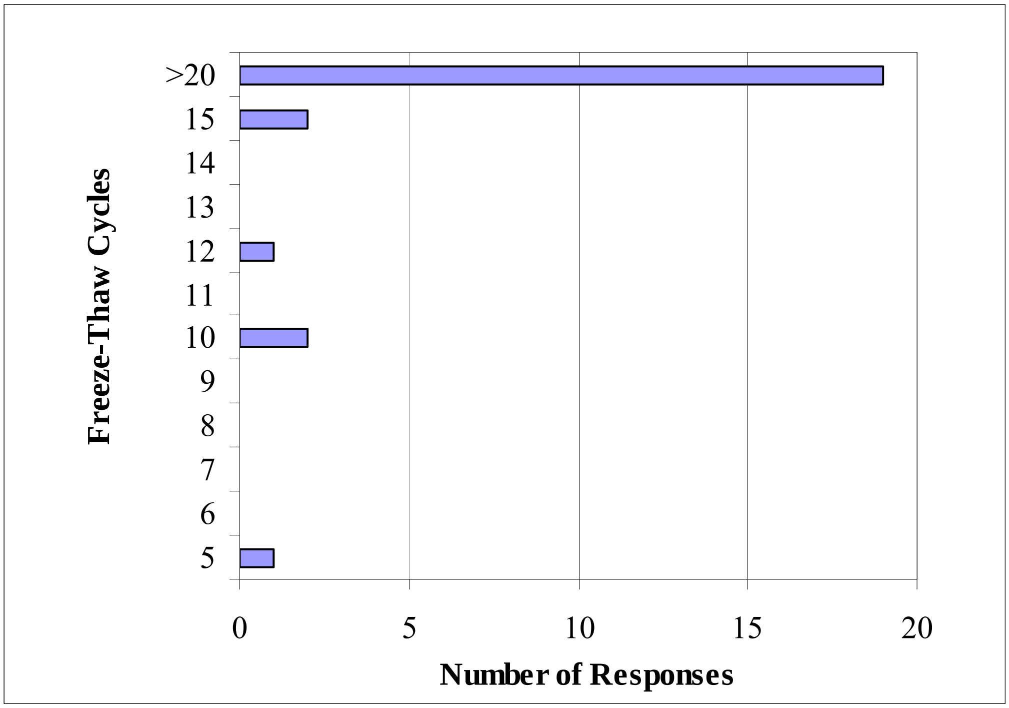

FIGURE 6.1 Number of freeze-thaw cycles.

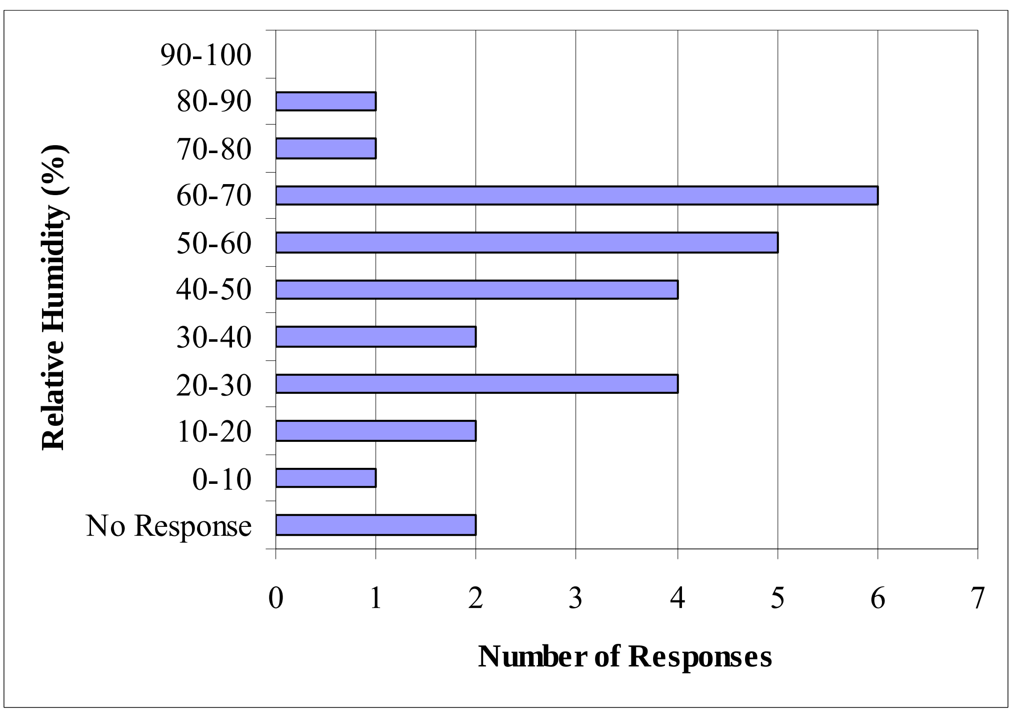

FIGURE 6.2 Relative humidity.

6.4 DECK CONSTRUCTION

The purpose of the deck construction section was to obtain data about current construction specifications and practices for new concrete bridge decks. The section provides information about deck thickness, minimum concrete cover, types of reinforcing steel, concrete compressive strength, water-cement ratios, slump specifications, admixtures, and curing techniques. Information was obtained using questions 10 to 19. Summaries of the information are shown in Figures 6.3 to 6.14 and further discussed in the commentary.

10. What is the historical thickness specification for concrete bridge decks in your jurisdiction?
11. What is the current thickness specification for concrete bridge decks in your jurisdiction?
12. What is the minimum concrete cover thickness presently required over the reinforcement?
13. What types of reinforcement are typically utilized and at what locations in the deck?
14. What is the average 7-day compressive strength of concrete required in new deck construction? If a 28-day compressive strength is specified instead, what is the typical value?
15. What is the typical water-cement ratio for concrete mixtures required in new deck construction?
16. What is the typical slump for concrete mixtures required in new deck construction?
17. What admixtures are usually added to the concrete?
18. What other additives are usually added to the concrete?
19. What curing techniques are typically utilized in concrete bridge deck construction?

Figures 6.3 to 6.5 show the historical thickness, current thickness, and change in deck thickness, respectively. Approximately 56 percent of the transportation agencies surveyed have increased the design thickness of their bridge decks compared to historical values. All agencies use a bridge thickness of 7.5 inches or greater. One agency actually indicated a 0.5-inch decrease in thickness, from an 8.5-inch historical thickness to an 8.0-inch current thickness. UDOT specified historical and current deck thicknesses of 8.0 inches. The general increase in bridge deck thickness is a possible result of increased traffic volumes and loads that mandate greater structural capacity.

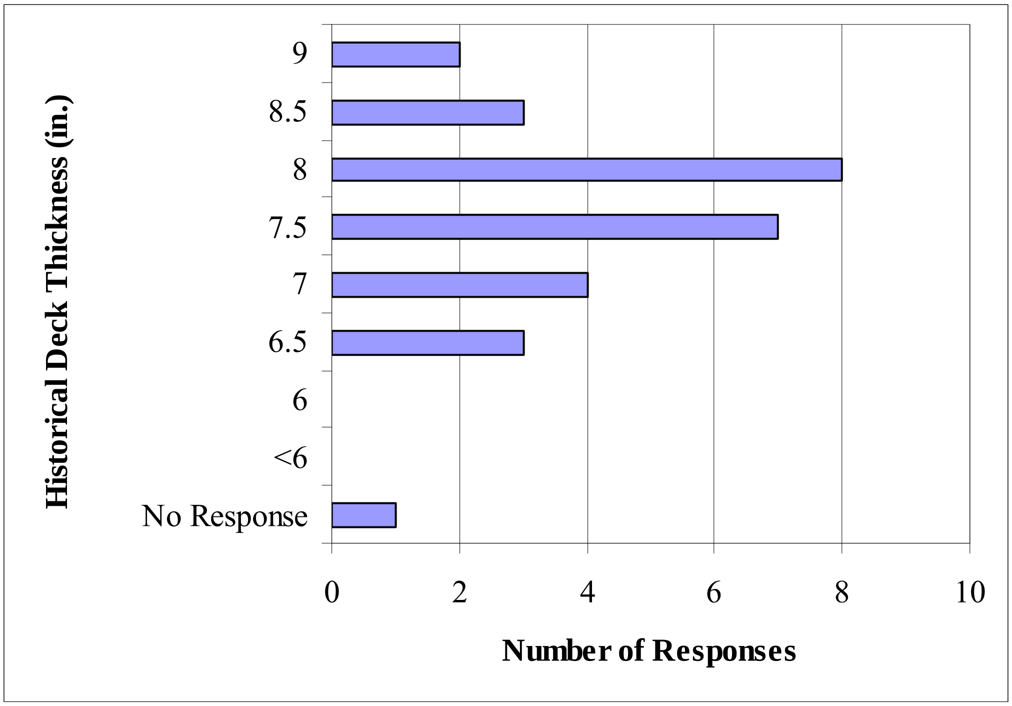

FIGURE 6.3 Historical deck thickness.

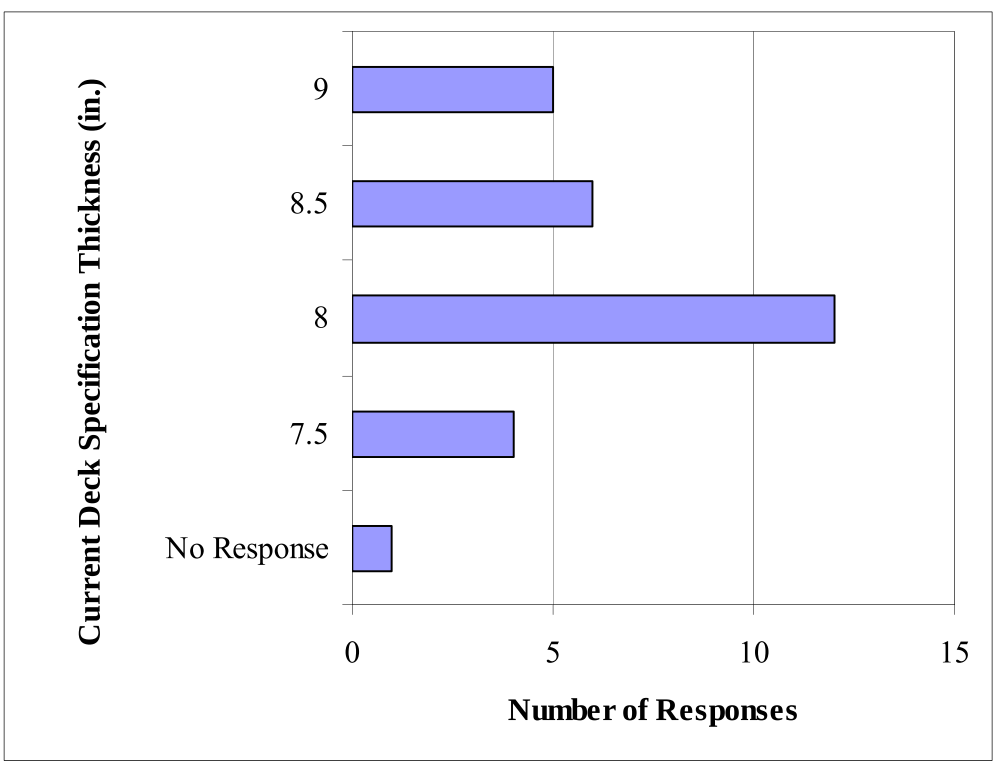

FIGURE 6.4 Current deck thickness.

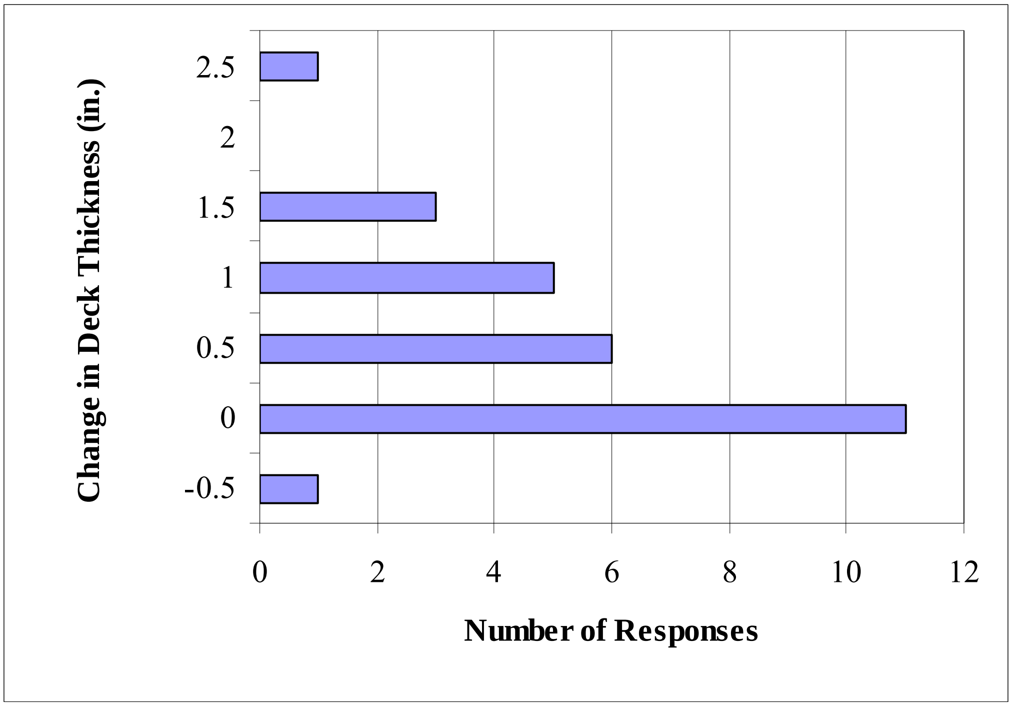

FIGURE 6.5 Change in deck thickness.

Figure 6.6 summarizes the minimum concrete cover over the top mat of reinforcement. Approximately 72 percent of the respondents require a minimum concrete cover of 2.5 inches. Currently, UDOT requires a minimum cover of 2.0 inches. Increased concrete cover provides added protection against chloride-induced corrosion for the top mat of reinforcing steel. The added cover lengthens the time for chloride concentrations to reach threshold values that might induce corrosion of the reinforcing steel.

Figure 6.7 summarizes the types of reinforcement used in bridge decks. Typically, decks are constructed with only three types of reinforcing steel: galvanized, epoxy-coated, and black steel. Other reinforcing materials such as fiberglass, stainless-clad, and stainless have obvious advantages in protecting against chloride-induced corrosion but are not routinely used by any of the agencies surveyed. Nineteen of the 28 agencies indicated that epoxy-coated steel is used throughout the entire bridge deck. Four others indicated that epoxy-coated steel is only used in the top reinforcement mat. UDOT stated that epoxy-coated steel is used throughout the entire bridge deck. The responses in Figure 6.7 are not mutually exclusive; the respondents could select more than one option.

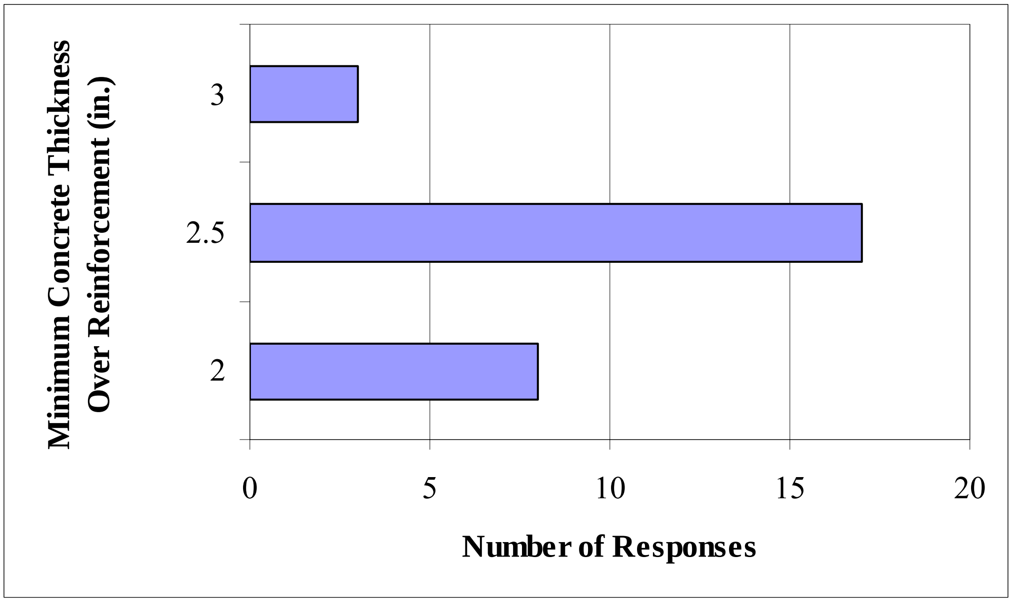

FIGURE 6.6 Minimum concrete cover over the top mat of reinforcement.

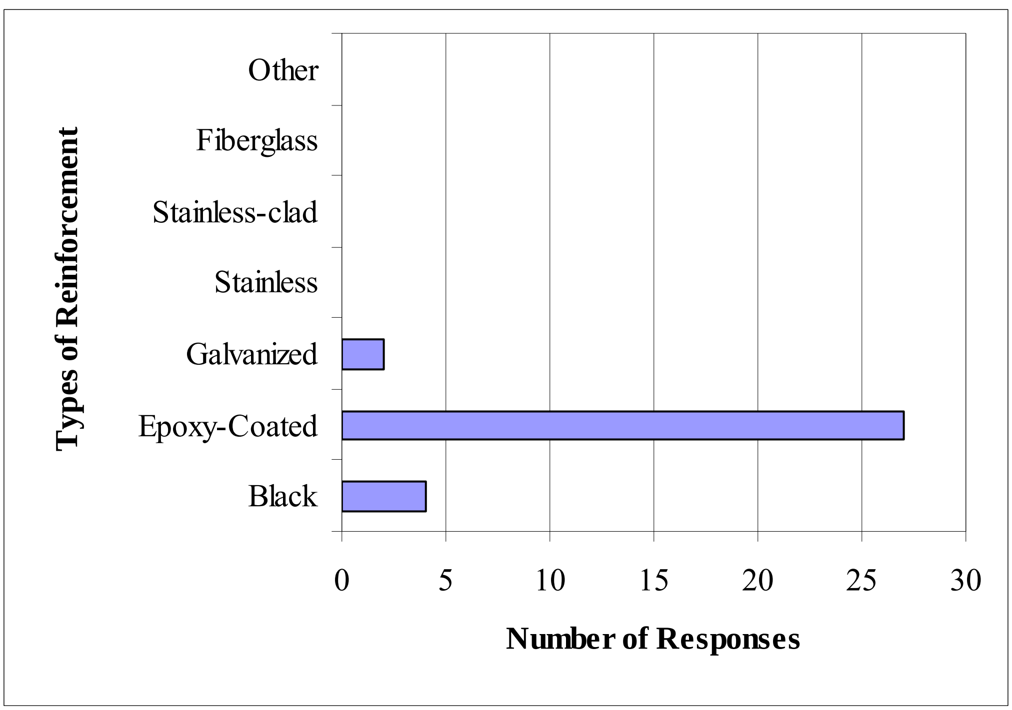

FIGURE 6.7 Types of reinforcing steel used in bridge decks.

Figures 6.8 and 6.9 summarize the 7-day and 28-day concrete compressive strength requirements, respectively. The average 7-day and 28-day concrete compressive strength requirements are approximately 3500 psi and 4000 psi, respectively. The majority of the participants did not provide an answer for the 7-day compression test, suggesting that only a 28-day compression test is required by those agencies. One respondent indicated a 14-day compressive strength requirement of 4000 psi. UDOT reported a value of 3500 psi for both 7-day and 28-day compressive strength specifications.

Figures 6.10 and 6.11 summarize the typical water-cement ratios and slumps for concrete mixtures used in new deck construction. UDOT specifies a water-cement ratio of 0.44 and a slump of 2.0 inches.

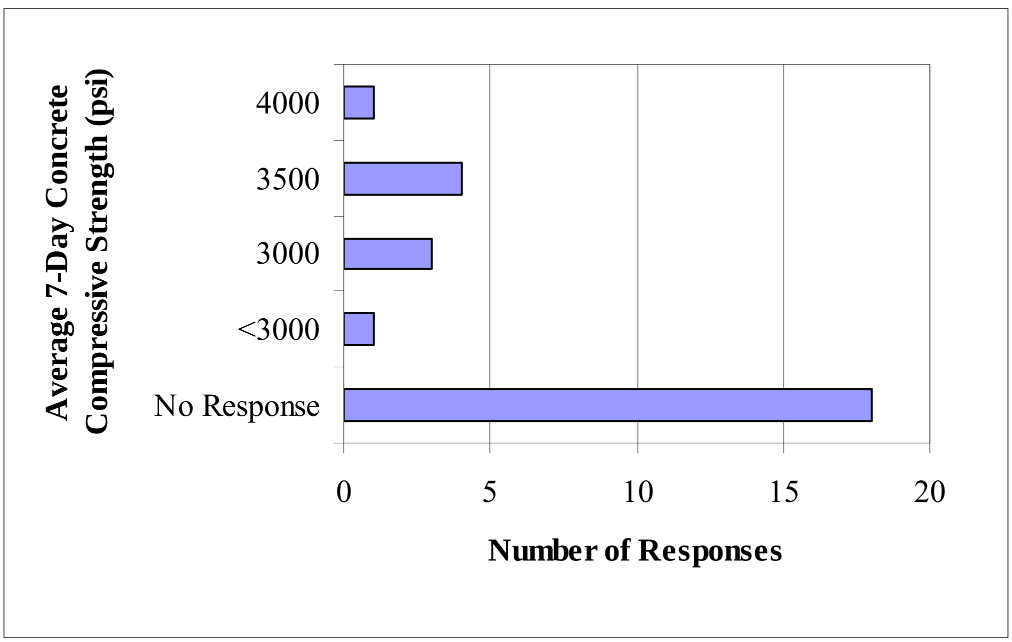

FIGURE 6.8 Required 7-day concrete compressive strength.

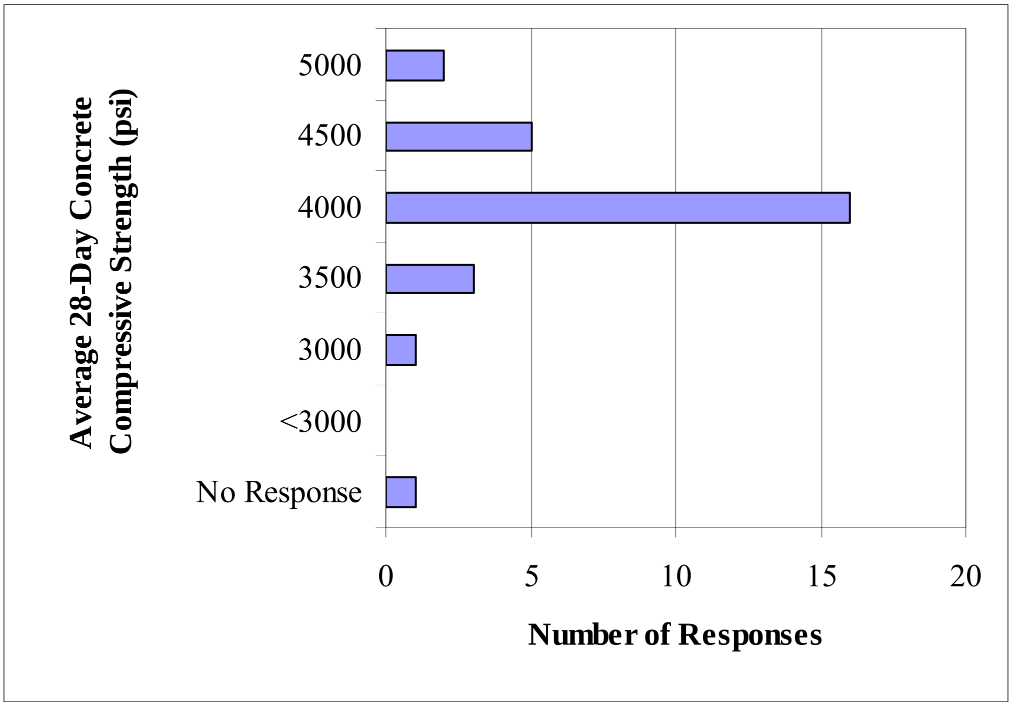

FIGURE 6.9 Required 28-day concrete compressive strength.

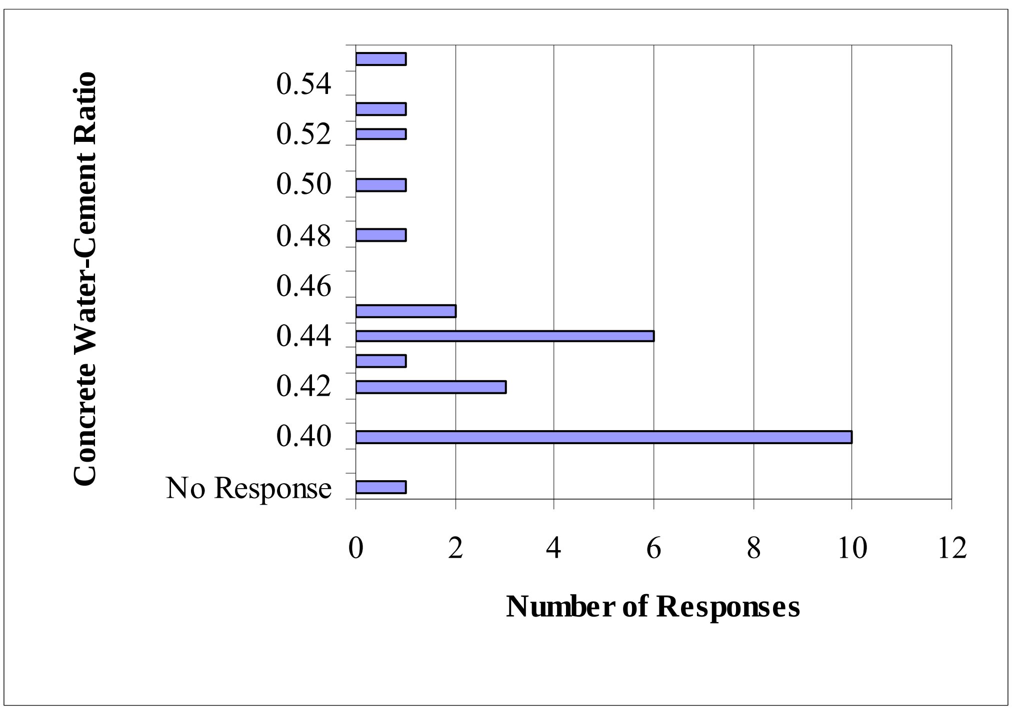

FIGURE 6.10 Typical concrete water-cement ratios.

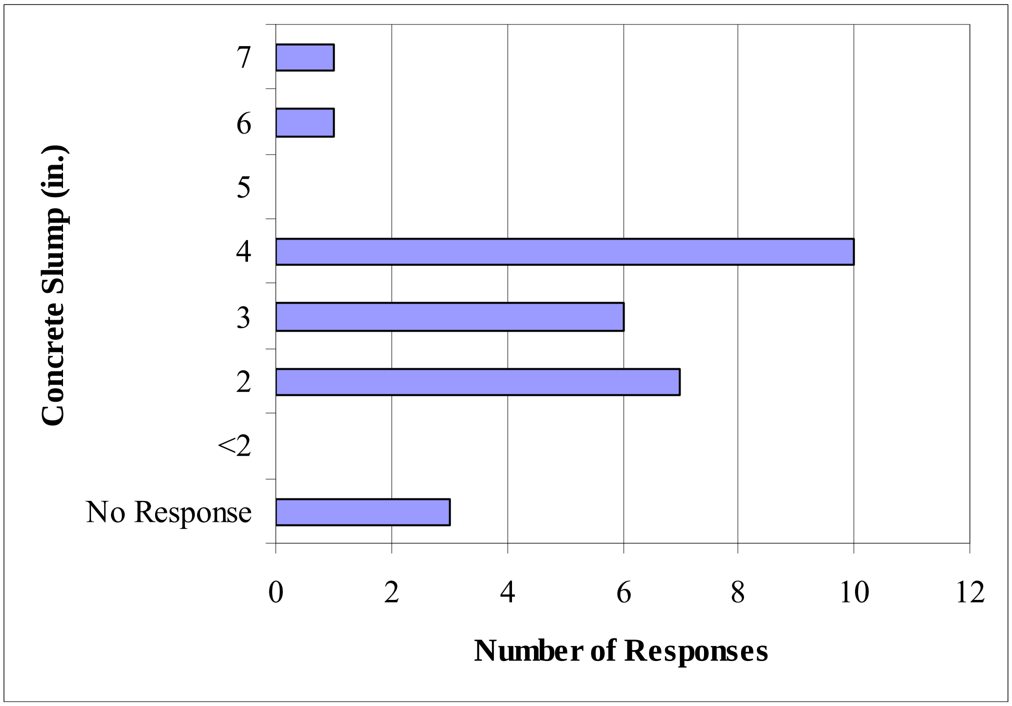

FIGURE 6.11 Typical concrete slumps.

Figure 6.12 summarizes the typical admixtures used in new deck construction. Air entrainers and water reducers are the most common admixtures used in new deck construction. Two agencies reported using corrosion inhibitors, one citing calcium nitrite and the other citing calcium nitrate. The seven respondents that selected "Other" indicated the use of retarders (5 responses), accelerators (1 response), and shrinkage reducers (1 response). UDOT reported using air entrainers and water reducers for new deck construction. The responses in Figure 6.12 are not mutually exclusive.

Figure 6.13 summarizes the use of supplementary admixtures in new deck construction. Most respondents use fly ash or a combination of fly ash, silica fume, and slag as supplementary admixtures in new deck concrete mixtures. UDOT reported the use of only fly ash in its concrete mixtures.

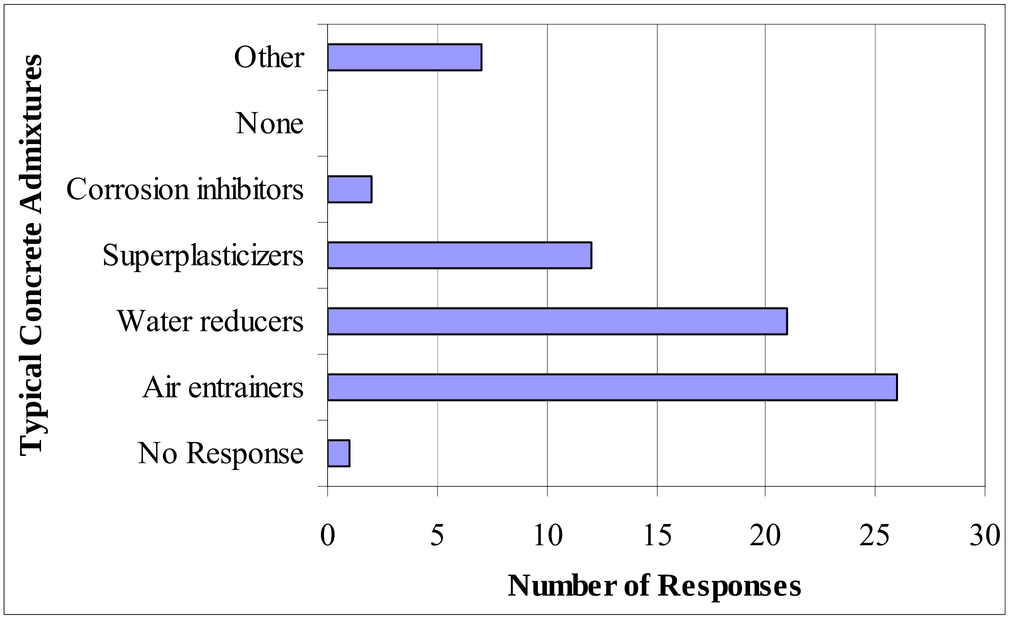

FIGURE 6.12 Typical concrete admixtures.

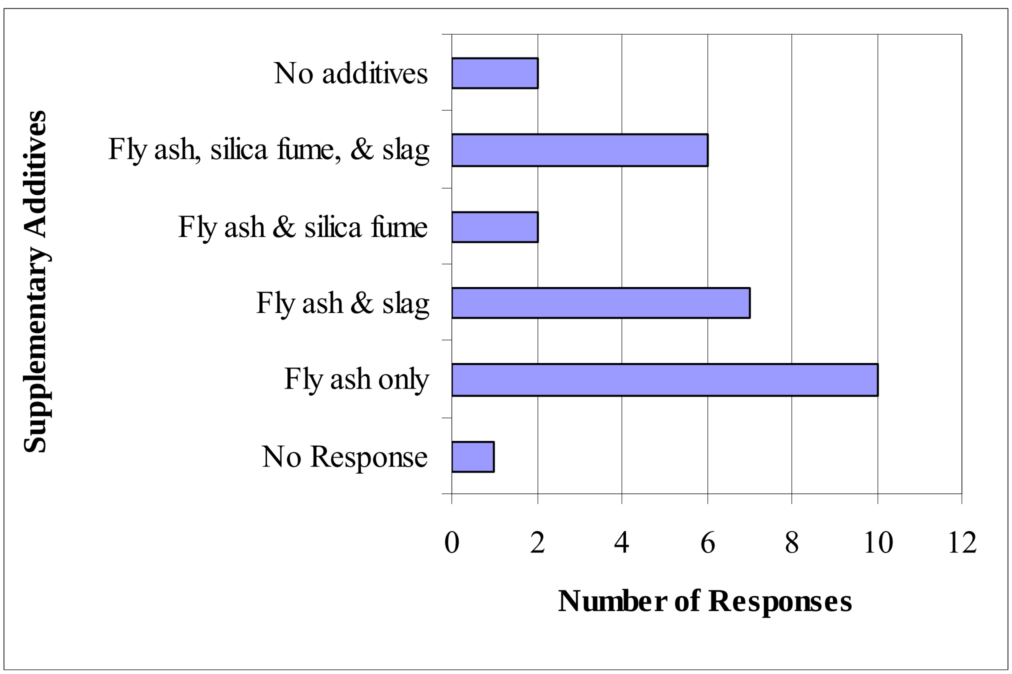

FIGURE 6.13 Supplementary concrete additives.

Figure 6.14 summarizes the types of concrete curing techniques for new deck construction. Almost all respondents indicated the use of moist burlap covers in addition to some other form of curing. Since the question is not mutually exclusive, respondents could choose more than one option. The three participants that selected "Other" indicated the use of cotton mats. UDOT reported the use of only moist burlap covers.

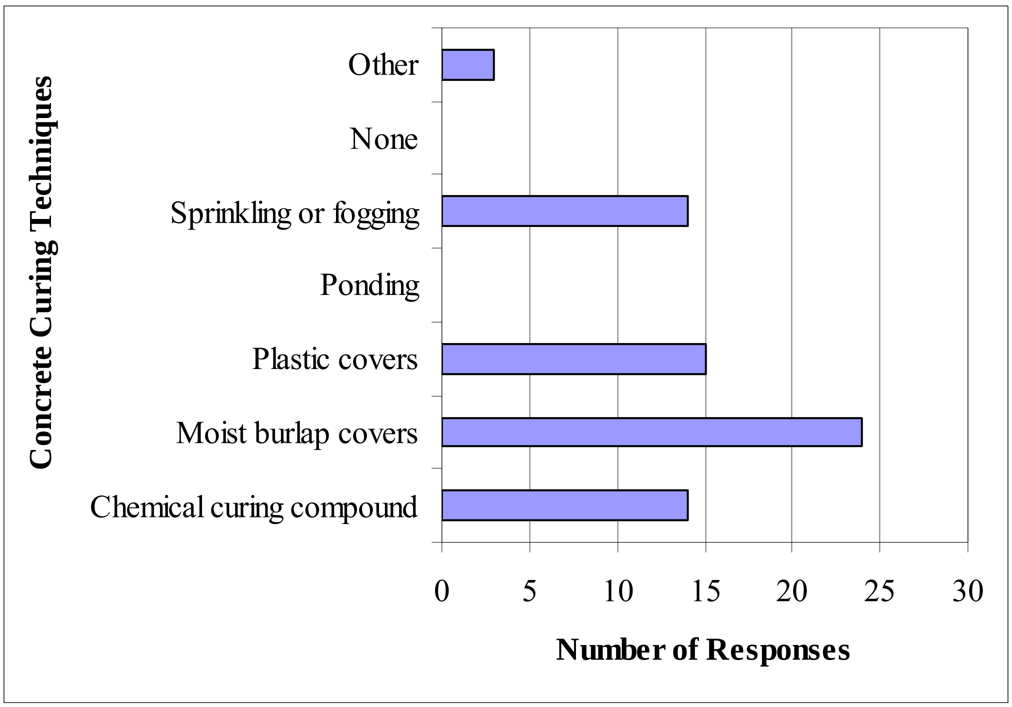

FIGURE 6.14 Concrete curing techniques.

6.5 WINTER DECK MAINTENANCE

The objective of collecting winter deck maintenance data was to identify the amounts and types of deicing salts used on concrete bridge decks. Information was obtained using questions 20 and 21. A summary of the information is presented in Figure 6.15 and further discussed in the commentary.

20. What types of deicing salts are used, and in what forms are the salts usually distributed?
21. What is the typical application rate of salt in pounds or gallons per lane mile?

Figure 6.15 shows that sodium chloride (NaCl) is the most commonly used deicing salt. Of the 28 participants in the survey, 36 percent use NaCl in dry crystal form, 7 percent in liquid form, and 43 percent in both dry crystal and liquid form. The remaining 14 percent indicated that they use a different deicing agent. Of the participants who reported using calcium chloride ($CaCl_2$), 50 percent indicated the use of dry crystals, while the other 50 percent specified the use of a liquid solution. All respondents who reported using magnesium chloride ($MgCl_2$), with the exception of one, indicated the use of $MgCl_2$ in a liquid solution. The seven respondents who reported using potassium chloride (KCl) and calcium magnesium acetate (CMA) employ these deicers in liquid form. CMA is one of the most widely known non-chloride deicing agents. However, its relatively high cost has been a deterrent to more widespread use of the product. Those who selected "Other" identified the deicing agent as brine and sand, ice ban with NaCl, and sand only. UDOT reported the use of $MgCl_2$ in liquid form, as well as NaCl in dry crystal and liquid form. Since the question is not mutually exclusive, participants were able to select more than one of the available options.

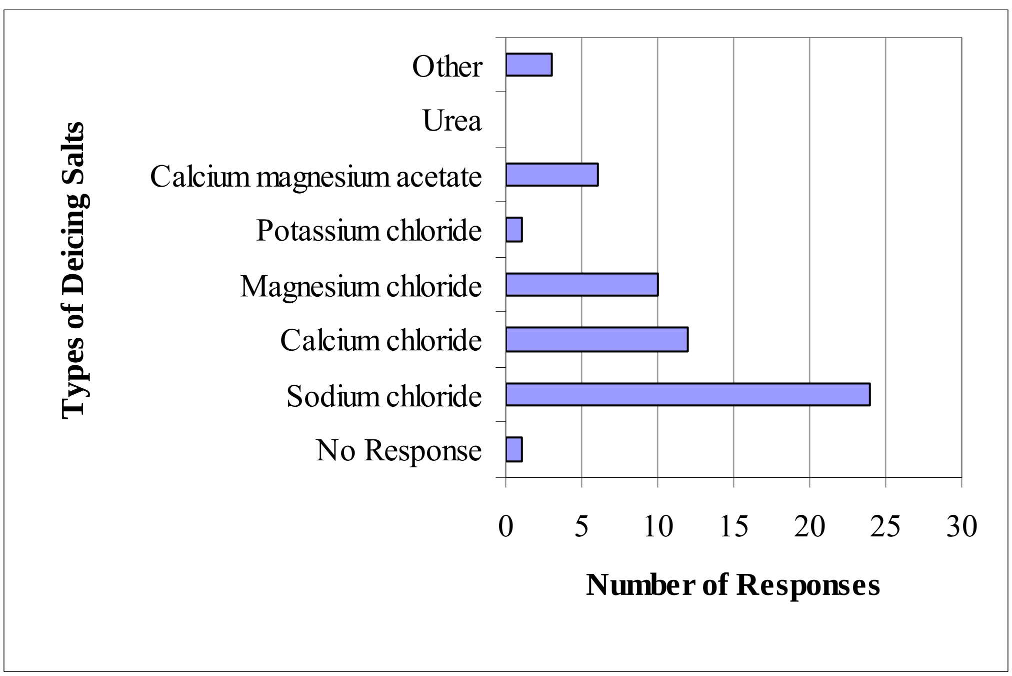

FIGURE 6.15 Types of deicing salts used on bridge decks.

Question 21 asked respondents to state the application rate of salt in pounds or gallons per lane mile. A graph of the data is not provided due to large variations in the responses. Approximately 36 percent of the respondents did not respond or did not know the answer to the question. Twenty-eight percent stated that the amount varied but gave no further indication of possible values. The remaining 36 percent reported using 100 to 400 pounds of salt crystals or 15 to 50 gallons of anti-icing liquid per lane mile. UDOT reported that the amount of deicing agents used per lane mile in Utah was not known but did indicate that it was "high."

6.6 DECK DETERIORATION

The goal of the deck deterioration section was to identify common distresses manifested on concrete bridge decks and the likely causes of those distresses. Information was obtained using questions 22 and 23. Summaries of the information are given in Figures 6.16 and 6.17 and further discussed in the commentary. Since the questions in this section are not mutually exclusive, the respondents were able to select more than one of the available options.

22. What are the most common distresses observed in concrete bridge decks in your jurisdiction, and when do they typically appear?
23. What are the likely causes of the observed distresses?

Figure 6.16 summarizes common distresses observed on concrete bridge decks. A large amount of bridge deck deterioration is manifested as transverse cracking and delamination. Seventy-eight percent of the participants who selected transverse cracking reported that it typically develops less than a year after construction. Seventy-one percent of the respondents who selected delamination reported that it is manifested 10 years after construction or longer. Four of the seven respondents who selected diagonal cracking specified occurrences within a year after construction. Ninety-three percent of the respondents who selected joint spalling indicated occurrences within 5 to 10 years following construction. The respondent who selected "Other" identified spalling of the concrete as the main distress. UDOT reported that the most common distresses are transverse cracking, diagonal cracking, and joint spalling, which are manifested less than 1 year, 3 years, and more than 10 years after construction, respectively.

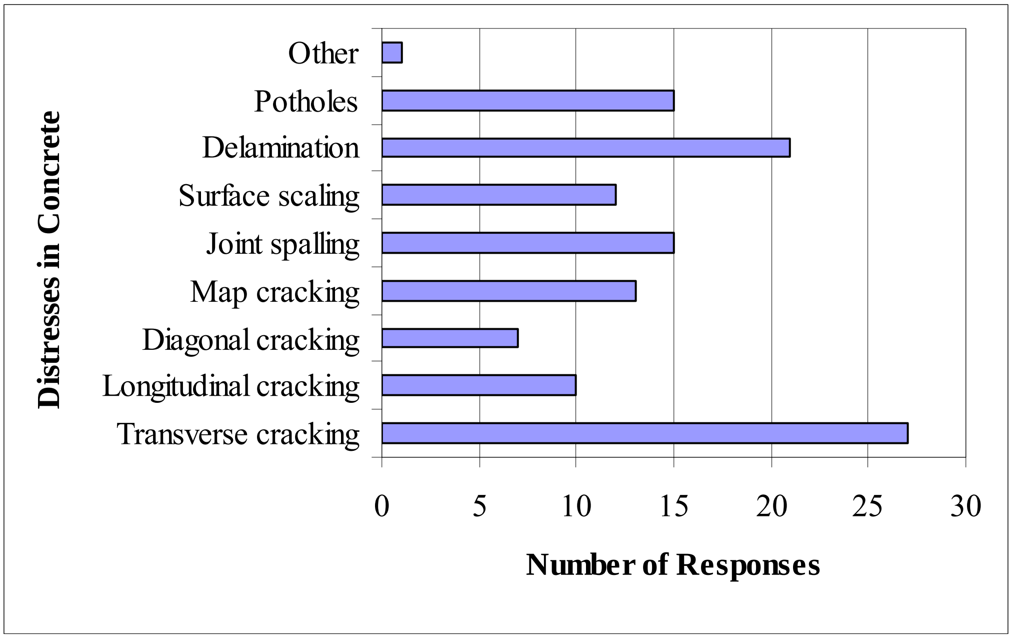

FIGURE 6.16 Common distresses observed on concrete bridge decks.

Figure 6.17 summarizes the likely causes of the aforementioned bridge deck distresses. A large majority of the respondents indicated that chloride-induced corrosion of the reinforcing steel is a significant contributor to deck deterioration, yet, as seen in Figure 6.12, most do not use a corrosion inhibitor to mitigate the effects of chloride attack. In the "Other" category, responses were generally related to construction problems or structural deficiencies. Shrinkage cracking, drying shrinkage, inadequate curing, failed membranes, and finishing operations were all listed as causes of distress originating from poor construction practices, which eventually lead to durability problems. Traffic loading and inadequate structural stiffness were causes of distress associated with structural deficiencies. UDOT reported the likely causes of bridge deck distress to be salt-induced corrosion, freeze-thaw cycling, and construction practices.

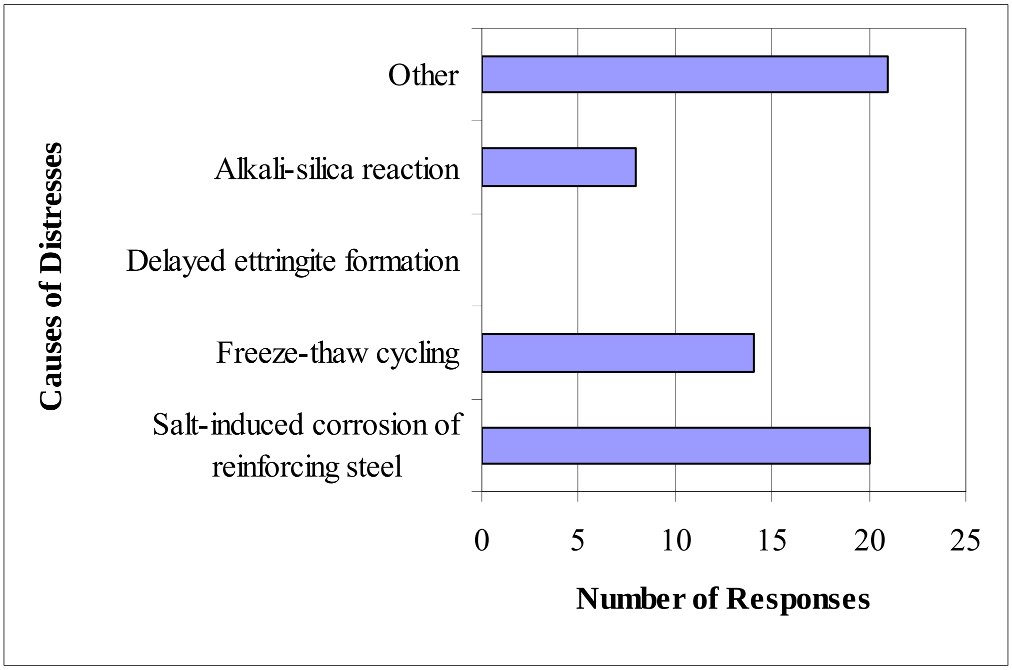

FIGURE 6.17 Common causes of bridge deck distress.

6.7 DECK CONDITION ASSESSMENT

The purpose of collecting deck condition assessment data was to determine the types of methods being employed to assess and rate the condition of concrete bridge decks. The condition assessment section incorporates information dealing with condition assessment methods and equipment, rating parameters, and prediction parameters. Information was obtained using questions 24 to 26. Summaries of the information are given in Figures 6.18 to 6.20 and further discussed in the commentary. The questions in this section are not mutually exclusive; therefore, the respondents were able to select more than one of the available options.

24. What methods do you typically employ to assess the condition of concrete bridge decks, and are they used at the network or project levels?
25. What parameters are typically used in quantifying or rating bridge deck condition, and what are the critical values of each parameter that would indicate a need for deck improvement?
26. What parameters are typically used in predicting future bridge deck condition for anticipating and scheduling future deck improvement projects?

Figure 6.18 summarizes the state of the practice for condition assessment methods. Visual inspection, chaining, chloride concentration testing, coring, and half-cell potential testing are the most common deck condition assessment methods. Although the first three methods are effective in detecting cracks and delaminations, the results suggest that DOTs place an equal emphasis on methods that measure corrosion potential (chloride concentration and half-cell potential). All five of the methods mentioned above are principally used at the project level only, with the exception of visual inspections, which are used on both the project and network levels. A significant number of the respondents reported the use of coring as an assessment technique, yet only three use petrographic testing. Coring is effective in detecting delaminations; however, a large number of cores are required in order for the sampling to be representative of the entire bridge deck. The use of a more efficient and rapid method for detecting delaminations would be ideal. UDOT reported the use of visual inspections, coring, chloride concentration testing, and skid resistance testing.

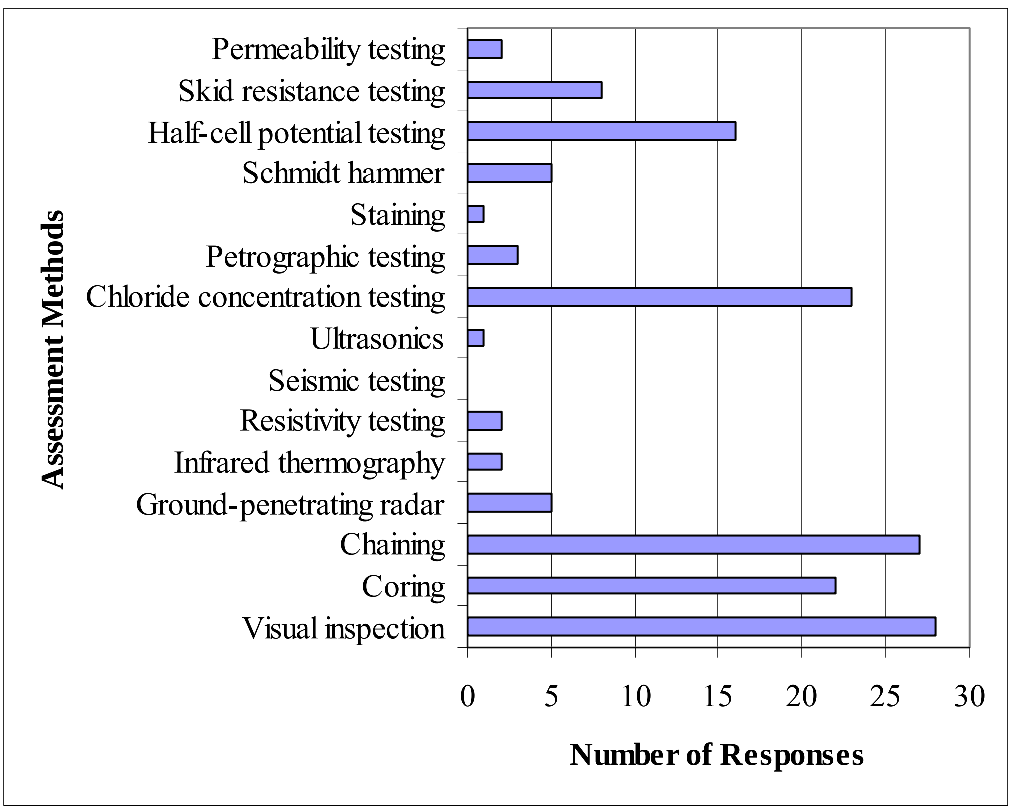

FIGURE 6.18 Condition assessment methods for concrete bridge decks.

Figure 6.19 summarizes parameters used to rate the condition of bridge decks. Delamination, cracking, chloride concentration, joint spalling, and surface scaling are the most common parameters used to rate the condition of concrete bridge decks. An analysis of the data indicates that 89 percent of the agencies use both delamination and cracking in their bridge deck ratings. These two parameters may be sufficient to provide an accurate bridge deck rating; however, accuracy of the rating will improve as the number of rating parameters that affect the functionality and structural integrity of the deck increases. The two parameters indicated by "Other" refer to potholes and funding. UDOT reported the use of cracking, joint spalling, delaminations, and chloride concentrations as the parameters used to rate their decks, which are consistent with those currently used by other state transportation agencies.

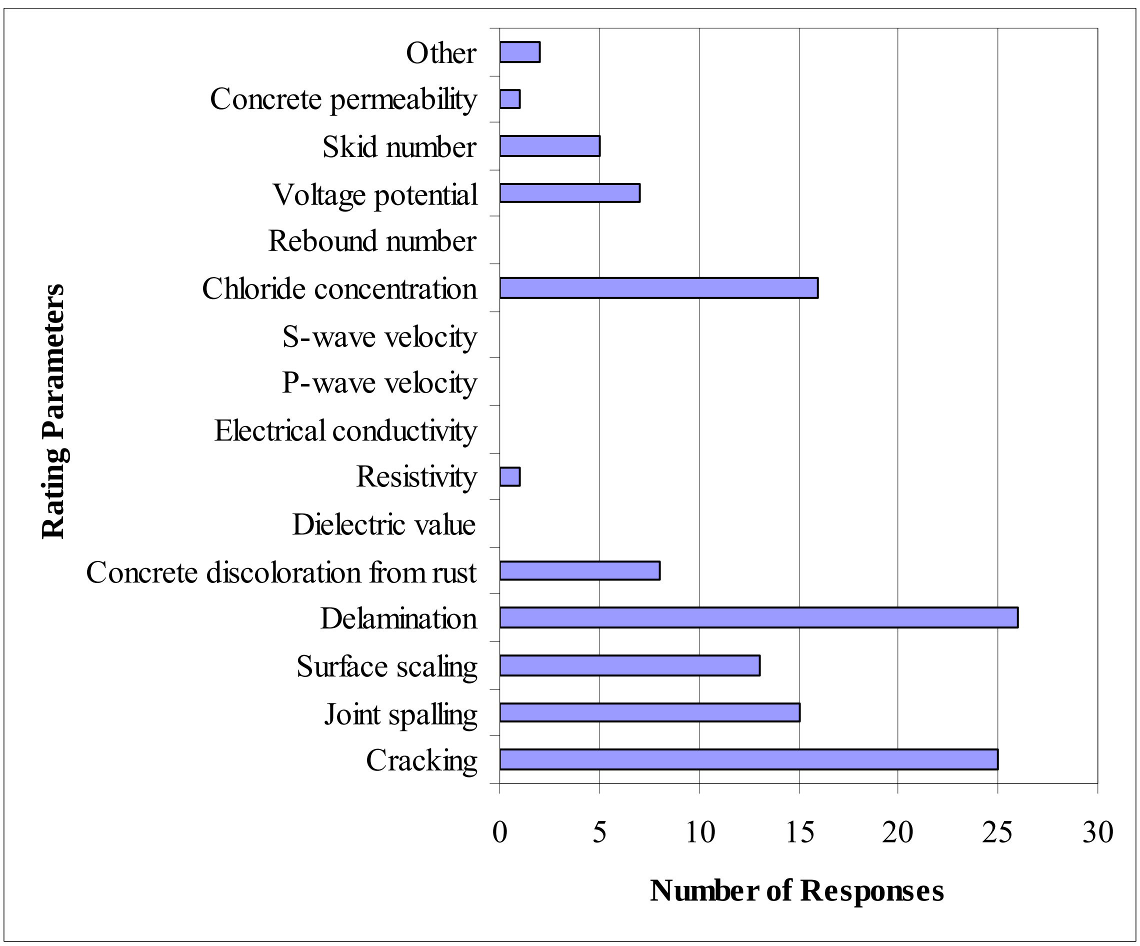

FIGURE 6.19 Parameters used to rate concrete bridge deck condition.

The second part of question 25 requested threshold values established for each parameter that would indicate a need for deck improvement. Several of the respondents reported taking action against cracking when efflorescence is present or when crack widths exceed 0.0625 inch. For surface scaling, two respondents stated that action was taken when 25 percent of the deck was affected with depths of 0.25 inch or greater. No definitive thresholds were identified for delaminations or joint spalling. Many respondents reported taking action when chloride concentrations reached 2.0 pounds per cubic yard of concrete.

Figure 6.20 summarizes parameters used to predict future deck condition. Close correlation exists between the parameters used to rate the condition of a bridge deck and those used to predict the future condition of a bridge deck. By using the same parameters, agencies are able to reduce the size of the BMS database. The eleven responses in the "Other" category refer to age, concrete strength testing, engineering judgment, full-depth patching, PONTIS, curb and rail condition, potholing, patching, and availability of funds. UDOT reported using cracking, spalling, delaminations, and age to predict the future condition of bridge decks.

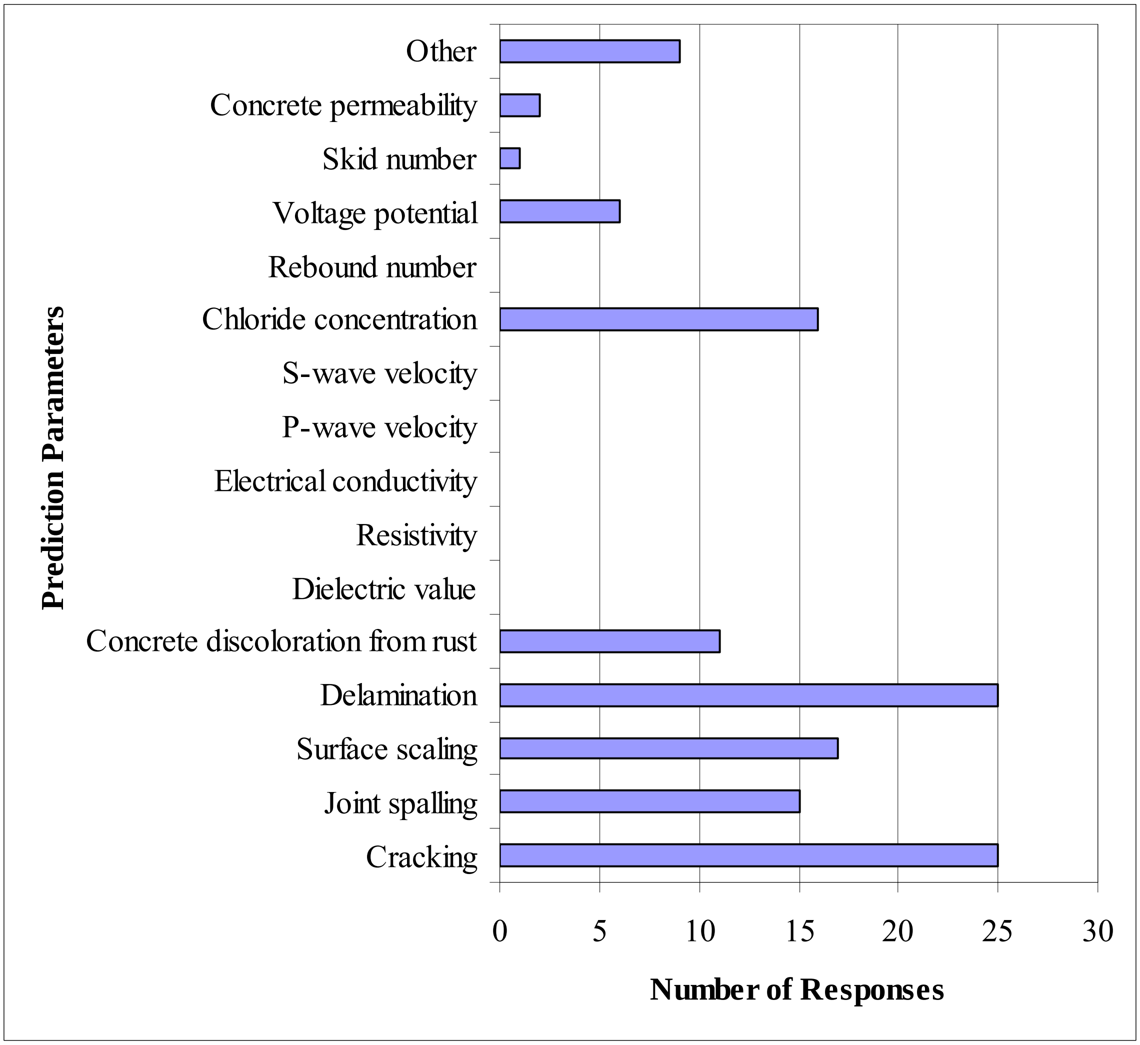

FIGURE 6.20 Parameters used to predict future bridge deck condition.

6.8 DECK IMPROVEMENT

The objective of the deck improvement section was to identify the methods and practices used in improving the condition of concrete bridge decks. Questions required information dealing with parameters for selecting a full-deck replacement or rehabilitation, estimating and mapping the extent of concrete damage, techniques and specifications for the removal of damaged concrete, patching materials and procedures, specifications for joint sealants and deck sealers, and service-life extension. Information was obtained using questions 27 to 36. Summaries of the information are shown in Figures 6.21 to 6.27 and further discussed in the commentary.

27. When a selected parameter indicates that the bridge deck should be improved, how do you determine whether the deck should be rehabilitated or fully replaced? You may address costs, geographic bridge location, traffic access issues, remaining life of bridge super-structure and sub-structure, type of deck reinforcement, and concrete mixture design details in addition to the physical condition of the bridge. Please be as specific as possible.
28. If deck rehabilitation is chosen, what methods are typically used to estimate or map the extent of the concrete damage, and are they usually used for evaluating bare concrete decks or decks with asphalt overlays?
29. After the extent of the damage is determined, what methods are typically employed for removing or treating the damaged concrete?
30. When damaged concrete must be removed, to what depth beyond the limits of the damage is removal recommended? To what lateral extent beyond the limits of the damaged concrete is removal recommended?
31. For repair of damage caused by corrosion of steel deck reinforcement, to what depth below the top mat of reinforcement is the concrete removed?
32. What specifications do you use for patching materials and procedures?
33. What specifications do you use for joint sealants?
34. What specifications do you use for deck sealers and coatings?
35. Under what conditions do you consider electrical treatment of a damaged bridge deck to be a viable rehabilitation strategy? Please be as specific as possible.
36. What is the typical extension in service life of concrete bridge decks due to rehabilitation strategies employed in your jurisdiction?

Question 27 asks the respondent to identify the parameters that determine whether a bridge deck should be rehabilitated or replaced. A chart of the data is not included since the information could not be readily graphed. Participants of the survey reported a wide variety of decision thresholds on whether to rehabilitate or replace an existing bridge deck. The most common decision-making factors were overall bridge condition, chloride concentration, type of bridge system, remaining service life, cost, available funds, and traffic loads.

For overall bridge condition, respondents indicated the need to replace bridge decks if deterioration exceeded a particular threshold value. Generally, the value for deterioration that constituted a full-deck replacement was approximately 30 percent of the deck area. However, some were as low as 20 percent and others as high as 50 percent. These values varied because they were all used in combination with other threshold values. Survey respondents suggested that delaminations in excess of 15 to 25 percent require a full-deck repair. One agency reported that localized deterioration is typically rehabilitated through patching. However, when deterioration has spread throughout a large area, the deck is replaced. The most specific response was from the New Jersey Department of Transportation, which stated that 30 percent deterioration requires patching, 20 to 60 percent deterioration requires patching with an overlay, and 50 to 100 percent deterioration requires a full-deck replacement.

Several agencies reported that chloride concentrations are monitored exclusively at the top mat of reinforcement. The North Carolina Department of Transportation stated that a full-deck replacement is required when concentrations exceed 2.0 pounds per cubic yard of concrete in more than 30 percent of the deck area. Other agencies simply stated that they replace a deck when concentrations are high; however, no definitive threshold values were given.

Some agencies reported that the type of bridge affected rehabilitation and replacement decisions. Integral bridge decks (box girders) cannot be replaced since the deck is part of the structural system. In this case, rehabilitation of the bridge deck is the only option until funds are available to replace the entire bridge. Because of the difficulty in replacing post-tensioned or pre-stressed decks, agencies reported that they usually rehabilitate these decks for longer durations. However, the Tennessee Department of Transportation reported that replacing a post-tensioned or pre-stressed deck is often quicker than rehabilitating it.

Remaining service life is probably one of the most important factors considered. Almost all agencies reported that the age of the bridge deck plays a significant role in decision-making. Bridges that are nearing the end of their service life are almost always replaced as opposed to being rehabilitated. Rehabilitation of a bridge component may significantly extend the service life of that particular element, but if the rest of the bridge structure requires replacement soon afterward, the benefit of the earlier rehabilitation is substantially reduced. Therefore, replacement of the entire bridge structure may be more economical than attempting to rehabilitate an aging bridge element. The Oklahoma Department of Transportation replaces bridge decks when rehabilitation efforts cost more than 70 to 75 percent of the replacement cost for an additional 25 years of service life.

Because of the ever-increasing traffic loads, some transportation agencies prefer replacement over rehabilitation. Many agencies identified roadway expansion projects as an example of when they would simply replace a bridge. Also, performing a complete replacement of a bridge is often easier than widening it. Usually, bridges are replaced when the structure can no longer support the weight of traffic.

Figure 6.21 summarizes the methods used to estimate the extent of deck damage. Visual inspection, chaining, coring, chloride concentration testing, and half-cell potential testing are the most common methods employed. These five methods are consistent with the common condition assessment methods identified in Figure 6.18. As identified in the deck condition and assessment section, the occurrence of delaminations was the most common factor used to rate the condition of a bridge deck, which seems to suggest that most transportation agencies are using some type of equipment or method that would detect delaminations. However, Figure 6.21 shows that only a small percentage of transportation agencies are using equipment or methods that are capable of this action.

Among the more commonly used non-destructive methods, chaining is the only technique that specifically detects delaminations. Visual inspections, chloride concentration testing, and half-cell potential tests cannot detect delaminations. These methods are, however, useful in mapping and estimating the extent of surface damage, the durability of the concrete, and the condition of the reinforcing steel. UDOT reported the use of visual inspections and chloride concentration testing on bare and overlaid decks. Question 28 is not mutually exclusive; therefore, participants could select more than one option.

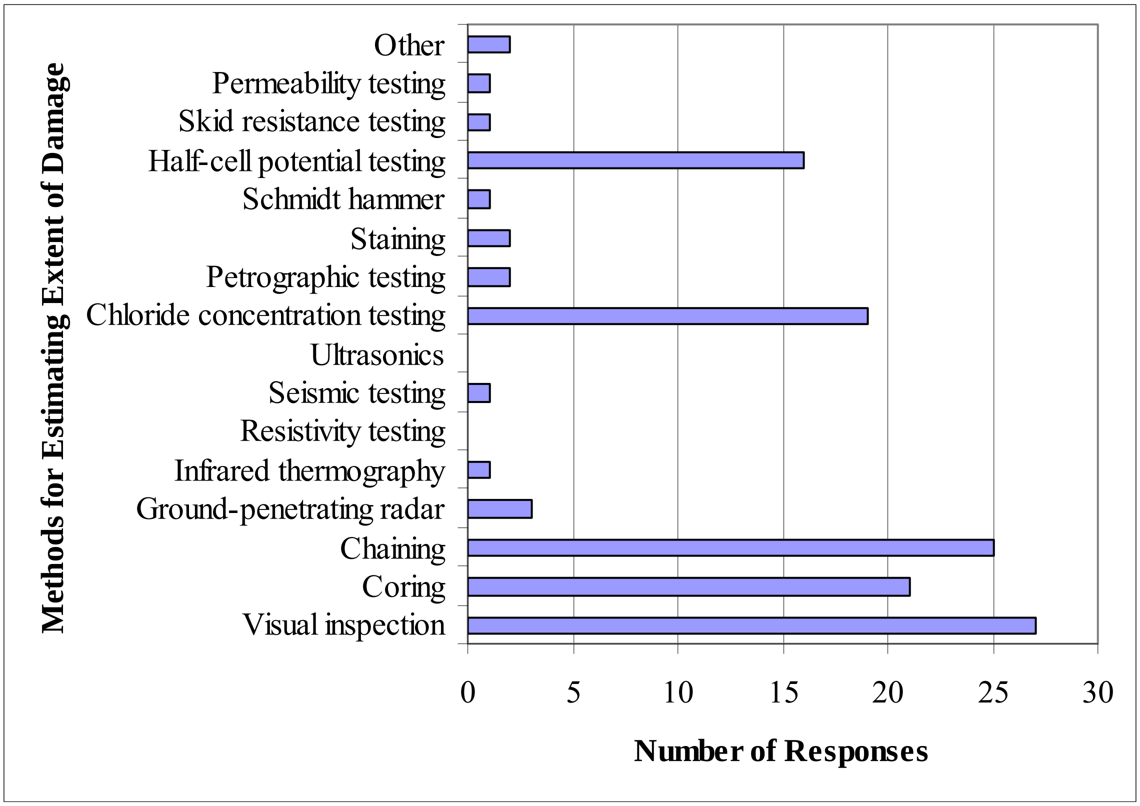

FIGURE 6.21 Methods used to estimate extent of concrete bridge deck damage.

Figure 6.22 summarizes the removal and treatment methods for damaged concrete. Jack-hammering, saw-cutting, and hydrodemolition are the most common methods of removing damaged concrete. Approximately 79 percent of the respondents who specified the use of jack-hammers employ a hammer weighing between 15 to 30 pounds, while 15 percent of the respondents employ a hammer weighing 45 pounds. Only 6 percent of the respondents employ a hammer that weighs 60 pounds. UDOT reported the use of saw-cutting and jack-hammering, with hammers weighing 15 pounds. Question 29 is not mutually exclusive; therefore, participants could select more than one option.

Figures 6.23 and 6.24 summarize the vertical and lateral extents beyond the limits of damage that concrete is removed. "Vertical" refers to a depth measurement, while "lateral" refers to a horizontal measurement beyond the limits of the damaged area. Approximately 88 percent of the participants indicated that only 1 inch of concrete or less is removed beyond the limits of the damage in the vertical direction. Vertical removal is governed by the thickness of the bridge deck, where excessive removal could puncture the bottom of the deck.

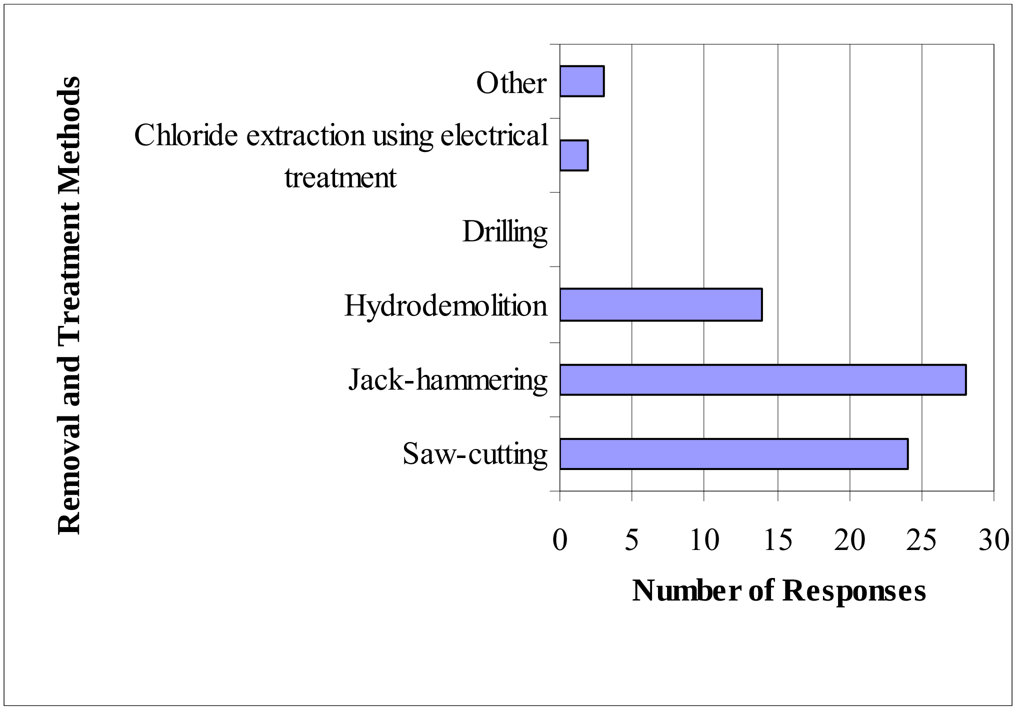

FIGURE 6.22 Removal and treatment methods for damaged concrete.

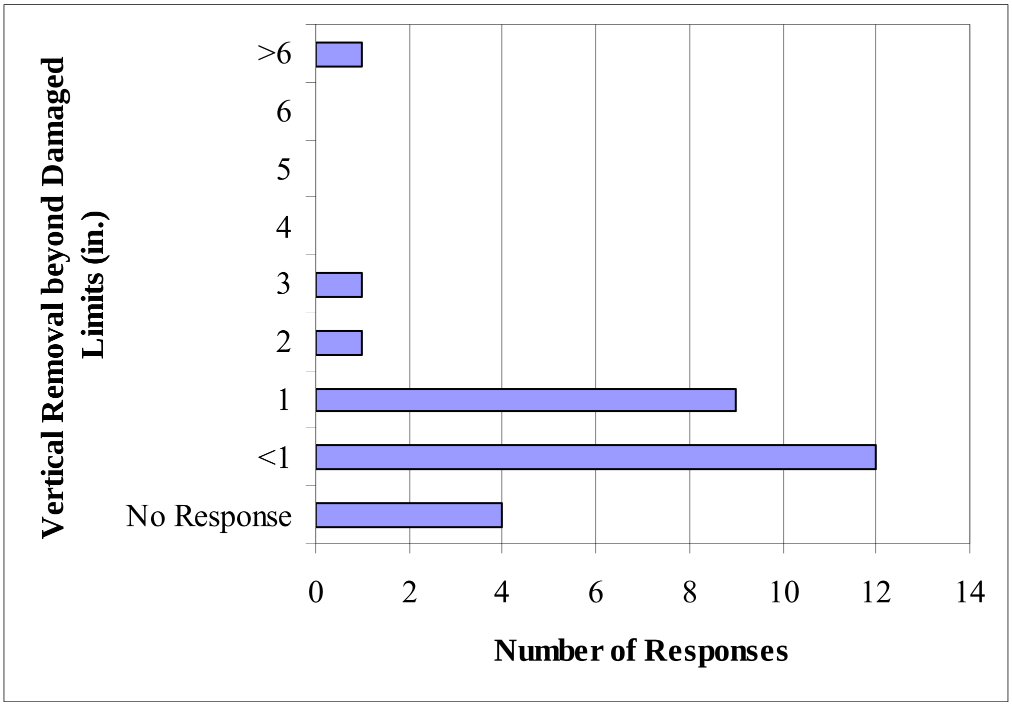

FIGURE 6.23 Vertical removal of concrete beyond the limits of damaged concrete.

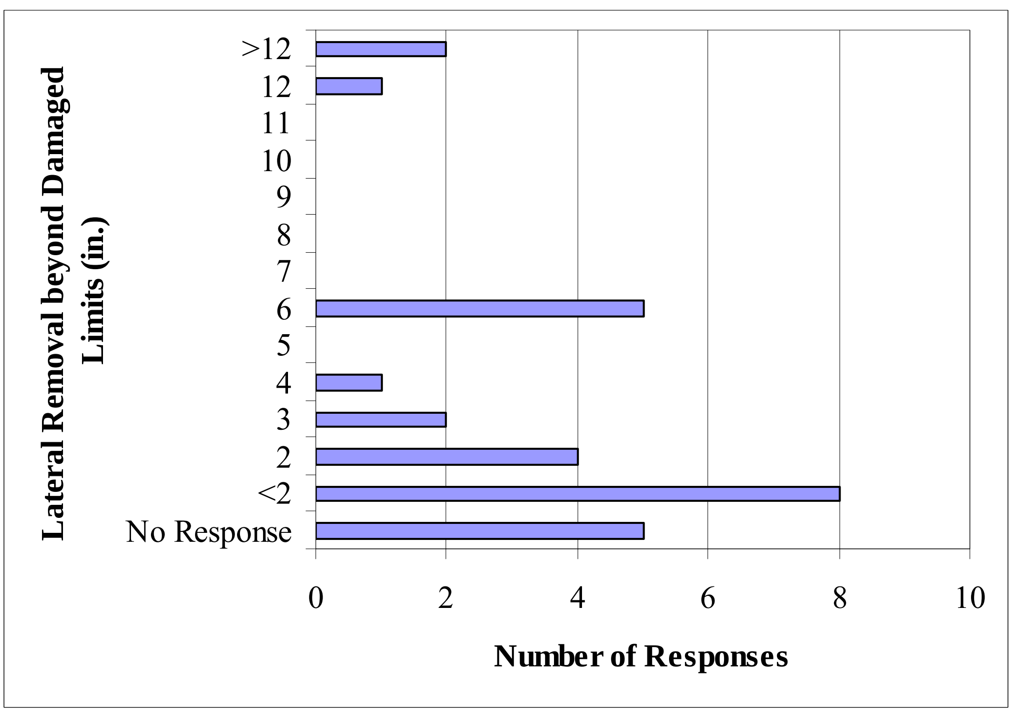

FIGURE 6.24 Lateral removal of concrete beyond the limits of damaged concrete.

The DOT that specified greater than 6 inches of vertical concrete removal beyond the limits of damage provided a copy of their bridge deck repair guidelines. The guidelines describe three classifications (Type A, B, and C) of repair that may be performed. Type A repairs consist of removing concrete down to the top layer of reinforcement. Type B repairs require the removal of concrete down to 1 inch below the bottom layer of reinforcement. Type C repairs consist of removing all delaminated and deteriorated concrete through the full depth of the deck. The Type C repair specification reportedly influenced their response in the survey. For removal of concrete in the lateral direction, 52 percent of the participants reported removal of 2 inches or less beyond the limits of the damage. UDOT reported a vertical removal of less than 1 inch and a lateral removal of 2 inches.

Figure 6.25 summarizes the depth of concrete removal beneath the top mat due to corrosion of the reinforcing steel. Ninety-two percent of the respondents indicated a removal of 1 inch or less. UDOT reported that less than 1 inch of concrete below the top mat is removed in this situation.

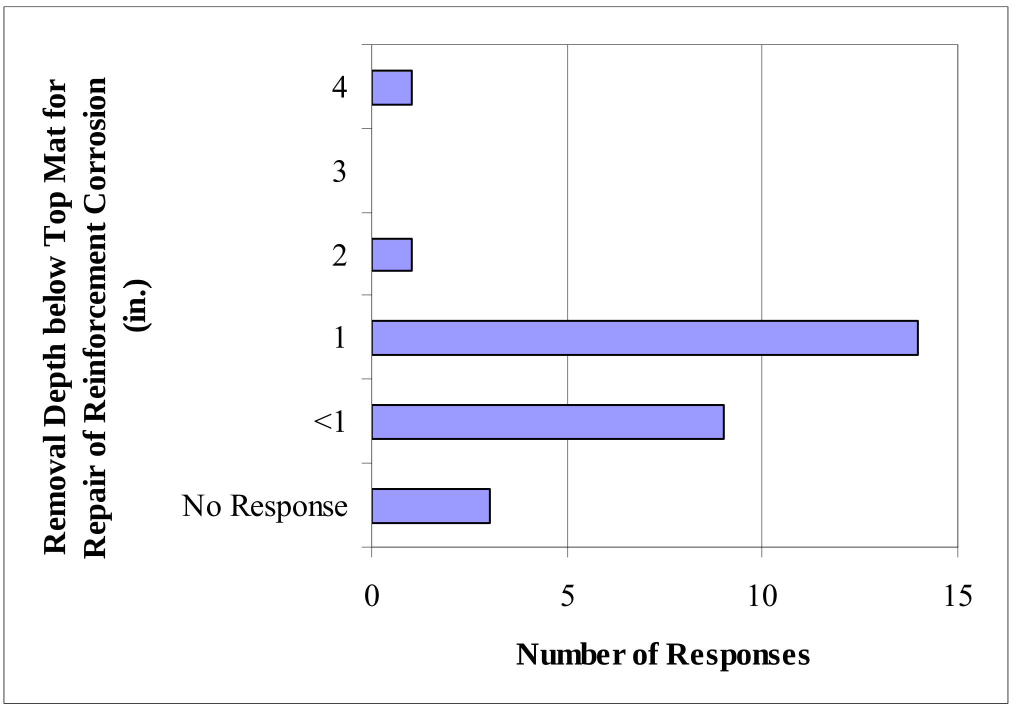

FIGURE 6.25 Removal depth of concrete below the top mat due to steel corrosion.

Figure 6.26 summarizes specification parameters for patching materials and procedures. Participants generally reported that vertical cut planes should be performed with a squared off, 1-inch perimeter around areas in need of patching and that blasting and flushing should then be used to clean the repair site. One agency indicated the use of bonding epoxy on the vertical planes prior to patching. The range of 7-day compressive strengths required for concrete patch materials was specified as 3000 to 4000 psi, with the exception of one agency that reported 6000 psi. The concrete shrinkage limit for patch materials was reported as 0.05 to 0.15 percent. The water-cement ratio was specified between 0.40 and 0.48, and concrete slump was reported to be 2 to 4 inches. Concrete bond strength was identified as 2000 psi. Supplemental quantities of reinforcing steel are added when the section has lost approximately 20 to 25 percent of its cross-section. The five responses in the "Other" category refer to ASTM C 928, Standard Specification for Packaged, Dry, Rapid-Hardening Cementitious Materials for Concrete Repairs; a requirement for a 10-minute set time; the use of pre-approved materials; a procedure dependent on the size and type of repair; and a requirement to use the same concrete mix for patching as used in the overlay. UDOT also specified that vertical cut planes should be performed with a squared off, 1-inch perimeter around the damaged area. UDOT further indicated that the patch material should have a minimum 7-day compressive strength of 3500 psi, a concrete water-cement ratio of 0.44, and a concrete slump of 2 inches. As the question is not mutually exclusive, participants were able to select more than one of the available options.

Figure 6.27 summarizes significant increases in the service lives of rehabilitated bridge decks. All respondents, including UDOT, indicated an extension in the service life of 10 years or more.

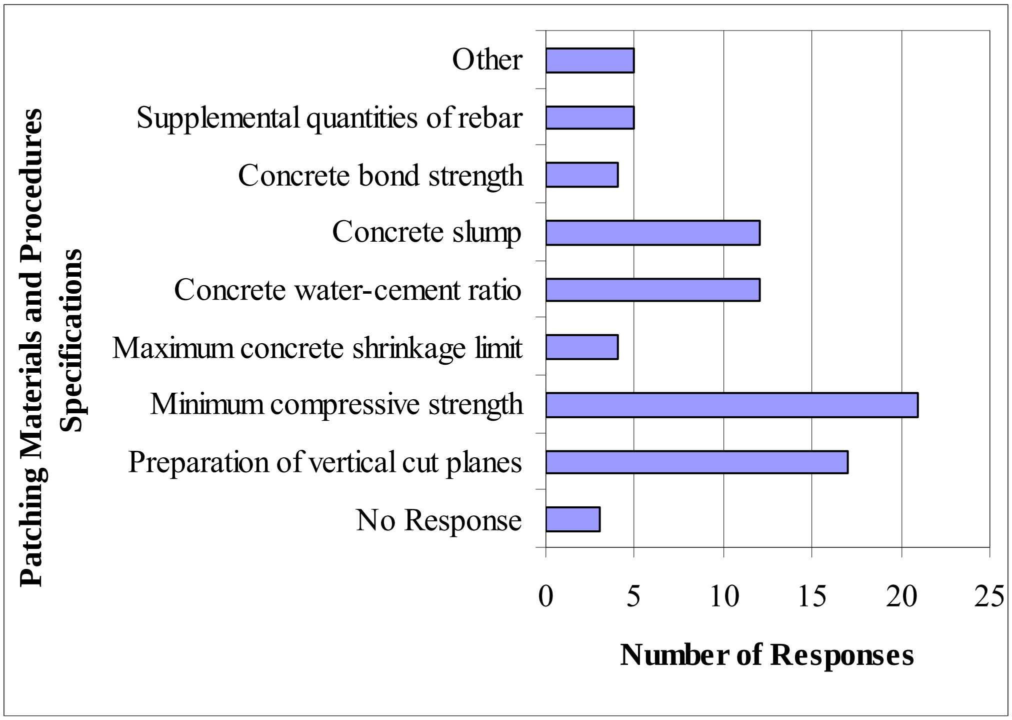

FIGURE 6.26 Specification parameters for patching materials and procedures.

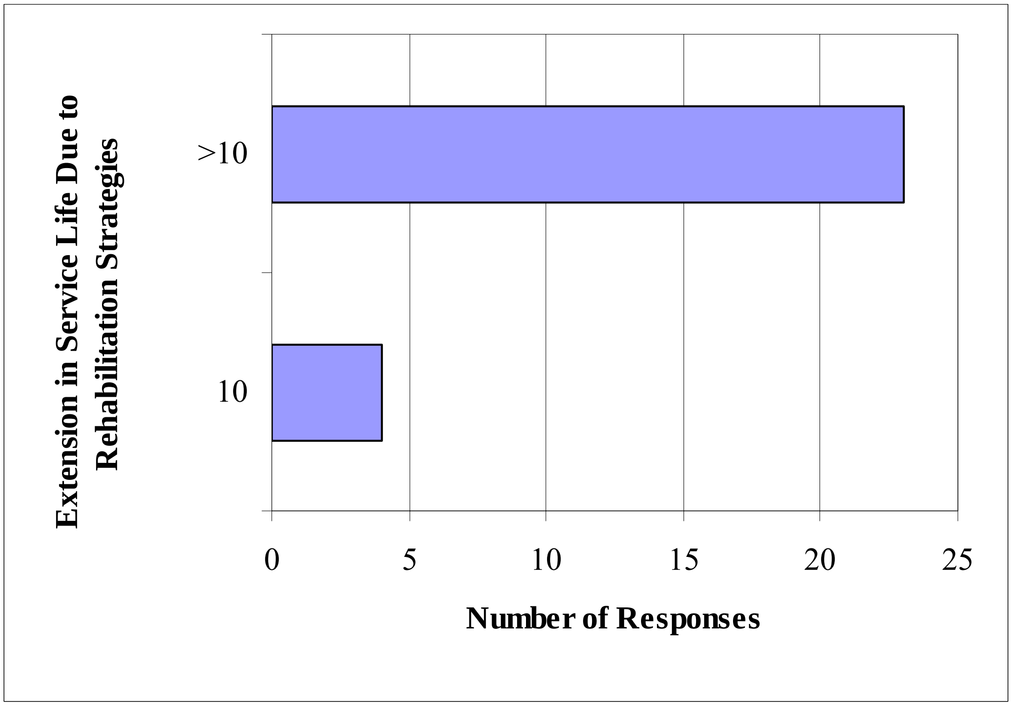

FIGURE 6.27 Service life extensions due to rehabilitation strategies.

CHAPTER 7
CONCLUSION

7.1 SUMMARY

The aging and deterioration of bridges in Utah mandates increasingly cost-effective strategies for bridge MR&R. Although the substructures and superstructures of bridges in Utah are in relatively good structural condition, the bridge decks are deteriorating more rapidly due to the routine application of deicing salts, repeated freeze-thaw cycles, and other damaging effects. Therefore, to assist UDOT with developing a protocol offering guidance about when and how a bridge deck should be rehabilitated or when it should be replaced, this research investigated concrete bridge deck performance issues, condition assessment techniques, rehabilitation methods, and BMS concepts. Development of a decision-making protocol that utilizes bridge deck condition assessment information in combination with life-cycle costs is especially important, since the costs associated with replacing every bridge deck in Utah are extremely high.

An extensive literature review was conducted on these topics. Research on concrete performance addressed concrete composition and durability, corrosion of reinforcing steel, and types of deterioration commonly exhibited by concrete bridge decks, including cracking, scaling, popouts, honeycombing and air pockets, AAR, carbonation, sulfate attack, and corrosion of reinforcing steel.

Research on bridge deck condition assessment methods focused on visual inspection, coring, chain dragging, hammer sounding, GPR, infrared thermography, resistivity testing, impact-echo testing, ultrasonic testing, chloride concentration testing, petrographic analysis, penetration dyes, the Schmidt Rebound Hammer, half-cell potential testing, rapid chloride permeability, skid resistance, and in-situ corrosion monitoring using embedded sensors.

Rehabilitation methods identified in the research include electrochemical rehabilitation, concrete removal and patching, surface treatments, corrosion inhibitors, and epoxy injections. Among electrochemical methods, cathodic protection, chloride extraction, and realkalization were specifically discussed. Concrete removal methods such as saw-cutting, hydrodemolition, jack-hammering, milling, shot-blasting, and sand-blasting were also reviewed, and a variety of overlays, sealants, and membranes were identified.

Information about utilization of BMSs in evaluating available rehabilitation methods mainly emphasized the importance of using accurate costs associated with specific rehabilitation techniques and obtaining reliable estimations of the extension in service life expected from each method. The value of BMSs at both the network and project levels was described, and details regarding data collection, development of performance indices, estimation of both agency and user costs, and the possible impacts of institutional issues on BMS implementation were reported.

To determine the state of the practice for bridge deck management, a questionnaire survey was conducted to identify common concrete bridge deck deterioration mechanisms, condition assessment procedures, rehabilitation methods, and decision-making protocols within individual state transportation agencies. The survey was sent to 43 state DOTs, and 28 responded. Analyses of the survey results identified MR&R strategies that could be implemented by UDOT to improve management of its bridge network.

7.2 FINDINGS

The literature review and survey results suggest that many DOTs attribute the performance of concrete bridge decks within their jurisdictions to specific issues regarding concrete mixture design, deck design, and construction practices. In addition, the data show that certain types of condition assessment and rehabilitation techniques have proven effective in the process of identifying and repairing damaged sections of concrete.

For new deck construction, the results of the survey indicate that many DOTs have increased the thickness of their bridge decks compared to historical values. The majority of DOTs presently use a bridge deck thickness of 8 inches or more with a minimum concrete cover of 2.5 inches. Increasing the concrete cover over the reinforcement delays chloride-induced corrosion by lengthening the duration of time before chloride concentrations reach threshold values in the vicinity of the reinforcing steel. Nearly 85 percent of the DOTs responding to the survey indicated the use of epoxy-coated reinforcement in the top mat or throughout the entire bridge deck. Epoxy coating on the reinforcement is intended to provide an extra protective layer against the elements required for steel corrosion, such as moisture, air, and chlorides.

The most commonly used admixtures in new deck construction are air entrainers, water reducers, and superplasticizers. These admixtures improve the durability of concrete by decreasing the effects of freeze-thaw cycling and by facilitating the use of lower water-cement ratios while still maintaining adequate concrete workability. Only two DOTs specified the use of corrosion inhibitors in new deck construction. The limited use of this type of admixture is most likely due to its relatively high cost. Supplementary admixtures, such as fly ash, silica fume, and slag, are used in new deck construction to decrease the permeability of the concrete. These supplementary admixtures are employed by nearly 90 percent of the DOTs surveyed. For concrete curing, DOTs typically utilize a combination of moist burlap covers, plastic covers, chemical curing compounds, and sprinkling or fogging. These techniques are used to prevent rapid drying of the concrete that could otherwise exacerbate shrinkage cracking.

The survey respondents ranked chloride-induced corrosion of reinforcing steel and freeze-thaw cycling as the most common sources of distress in concrete bridge decks and reported that such distress is most frequently manifested as cracking and delaminations. The occurrence of distresses can particularly accelerate the corrosion of reinforcing steel by providing new avenues for the ingress of chlorides. Every DOT surveyed reported the use of a chloride-based deicing agent to melt snow and ice from bridge decks as part of routine winter maintenance operations.

Survey responses on the topic of condition assessment identified visual inspection, chaining, chloride concentration testing, coring, and half-cell potential testing to be the methods most frequently used for detecting and quantifying bridge deck deterioration. New technologies that facilitate in-situ corrosion monitoring through the use of embedded corrosion sensors are being developed, but the technologies are not yet widely available. Delaminations, cracking, and chloride concentration, followed closely by joint spalling and surface scaling, are the parameters most frequently used for rating bridge deck deterioration. These same parameters are also used to predict the future condition of a bridge deck.

For deck improvement and rehabilitation, the results of the survey show that most DOTs use combinations of visual inspection, chaining, chloride concentration testing, coring, and half-cell potential testing to map and estimate the extent of concrete deterioration. In repair operations, the majority of the respondents employ jack-hammering, saw-cutting, and hydrodemolition to remove damaged concrete. Survey results show variations in the amount of concrete that is removed vertically and laterally. Approximately 80 percent of the DOTs specified that only 1 inch of concrete or less is removed vertically beyond the limits of damaged concrete. Specifications for lateral removal of damaged concrete varied greatly, however, with many of the respondents removing 4 inches of concrete or less, some removing 6 inches, and others removing 12 inches. When corrosion of the reinforcing steel is the cause of the damage, DOTs reported that only 1 inch of concrete or less is removed from below the top reinforcement mat. DOT responses to the survey questions about patching and material specifications consistently indicated that high strength, low slump, and low water-cement ratio are required for patch materials.

The most common factors influencing the decision to rehabilitate or replace a concrete bridge deck are overall bridge condition, chloride concentration, type of bridge system, remaining service life, cost, available funds, and traffic loads. Generally, deterioration exceeding 30 to 50 percent of the deck area warranted consideration of deck replacement, where deterioration may be defined as the presence of cracking or delaminations, excessive chloride concentration levels, or other distresses. The most specific response on this issue was received from the New Jersey Department of Transportation, which stated that 30 percent deterioration requires patching, 20 to 60 percent deterioration requires patching with an overlay, and 50 to 100 percent deterioration requires full-deck replacement. The threshold value for chloride concentration reported by most state DOTs is approximately 2.0 pounds per cubic yard of concrete.

7.3 RECOMMENDATIONS

UDOT should develop and implement a formal BMS with a searchable database containing information about the types of distress manifested on individual bridges, causes for the distress, values of measured test parameters, types of rehabilitation methods performed on the bridge deck, costs for rehabilitation methods, and service life extensions as a result of particular rehabilitation methods. Supporting data should be regularly collected through inspection and monitoring programs to facilitate prioritization of MR&R strategies for individual bridges and to evaluate the impact of such strategies on the overall condition of the network. A variety of non-destructive testing technologies should be employed to assess the condition of bridge decks and map the extent of damage. In particular, half-cell potential testing, GPR, and corrosion sensors warrant further investigation.

Performance indices based on selected condition assessment parameters should be developed for use in BMS analyses, and mathematical deterioration models should be calibrated for forecasting network condition and predicting funding requirements for various possible MR&R strategies. UDOT should also consider revising concrete mixture designs, deck specifications, and construction practices to improve the performance of concrete bridge decks in Utah. For example, lower water-cement ratios and thicker concrete cover over the reinforcing steel may offer improved deck durability.

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