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Development of an Index for Concrete Bridge Deck Management in Utah

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DEVELOPMENT OF AN INDEX FOR CONCRETE BRIDGE DECK MANAGEMENT IN UTAH

by

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ABSTRACT

DEVELOPMENT OF AN INDEX FOR CONCRETE BRIDGE DECK

MANAGEMENT IN UTAH

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Master of Science

The purpose of this research was to develop a new index for concrete bridge deck management in Utah. Data were collected in the summer of 2005 from 15 concrete bridge decks in the vicinity of Salt Lake City. The decks ranged from 2 to 21 years in age and were all constructed using epoxy-coated rebar. Visual inspection, sounding, Schmidt hammer testing, half-cell potential testing, and chloride concentration testing were performed on six 6-ft by 6-ft test areas randomly distributed within the single lane closed to traffic on each deck, and testing protocols followed American Society for Testing and Materials standards to the extent possible.

Collected data were analyzed using statistics, and age, cover, and half-cell potential were ultimately selected for inclusion in a new Utah Bridge Deck Index (UBDI); these variables effectively reflect chloride-induced corrosion mechanisms active on Utah bridge decks, are highly correlated to delamination distresses, and are relatively easy to measure compared to chloride concentration. At the request of Utah Department of Transportation (UDOT) personnel, the UBDI equation was structured around a deduct system using a 100-point scale similar to the sufficiency rating system, in which a perfect

bridge deck receives a score of 100. Coefficients were selected based largely on the judgment of the researchers and the UDOT personnel involved in the research, and threshold values for maintenance, rehabilitation, and replacement (MR&R) options were specified to be the same as those associated with the standard sufficiency ratings. The UBDI and corresponding MR&R recommendation were then provided for each of the bridge decks tested in this research; nine of the decks are recommended for preventive treatment, and six are recommended for rehabilitation.

In addition, the possibility of treatment applications was considered, leading to required adjustments in the UBDI calculation; the treatment options that were considered include an epoxy seal, an HPC overlay, and an asphalt membrane overlay. Four case scenarios were developed to demonstrate the response of the revised UBDI equation to these treatments. Finally, as aids for UDOT personnel implementing this research, charts were created to facilitate rapid determination of the required number of half-cell potential and concrete cover measurements for different levels of reliability and tolerance.

The UBDI developed in this research is recommended for implementation by UDOT personnel as a tool for optimizing the timing of MR&R treatments on concrete bridge decks similar to those evaluated in this project. In measuring cover and half-cell potential values, UDOT personnel should utilize the sampling guidelines presented in this report to ensure adequate characterization of each deck. Furthermore, to facilitate the inclusion of treatment effects in the UBDI, UDOT personnel should establish a policy of recording the types and dates of all MR&R treatments applied to bridge decks. As performance data are collected for specific treatments over time, the treatment lives proposed in this research for epoxy seals, HPC overlays, and asphalt membrane overlays should be revised as needed, and information for other treatments may be added. In addition, to maximize the predictive capabilities of the UBDI, more accurate relationships between half-cell potential values and deck age should be developed for estimating future deck condition.

ACKNOWLEDGMENTS

I wish to acknowledge Daniel Hsiao of UDOT for directing this project and coordinating communication between UDOT and Brigham Young University (BYU). Appreciation is also given to Mike Ellis of UDOT for arranging traffic control during the bridge deck testing. UDOT bridge engineers David Eixenberger, Todd Jensen, Danny Page, and Boyd Wheeler served on the technical advisory committee for this project. BYU research assistants Aimee Birdsall, Brandon Blankenagel, Ashley Brown, Rebecca Crane, Steve Frost, Jon Hanson, Russell Lay, Benjamin Reese, Loren Ross, and Robert Tuttle assisted in the bridge deck testing required for this research. In addition, my husband and family have given me tremendous support in completing my education.

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CHAPTER 1
INTRODUCTION

1.1 PROBLEM STATEMENT

Of the approximately 3,000 bridges in Utah, 1,700 are state-funded bridges for which the Utah Department of Transportation (UDOT) is responsible. In 2004, the national bridge inventory (NBI) report stated that 8.6 percent of these bridges were structurally deficient and that an additional 8.5 percent were functionally obsolete. In addition, 86.7 percent of Utah bridges were in need of some form of maintenance. The cost of these repairs was estimated to be $1.4 billion (*1*). Estimates suggest that nearly 315 million vehicles nationwide cross structurally deficient bridges every day (*2*).

Although several reasons exist for which a bridge can be labeled structurally deficient, one of the most common is a structurally inadequate deck (*2*). To maximize bridge deck service life amid increasing financial constraints, bridge engineers and managers have developed and implemented bridge management systems (BMSs). Two key components of a BMS are assessing current deck condition and predicting future deck condition.

As an aid in determining current deck condition, the NBI rating system has been developed for use in BMSs. In this system, bridges are rated on a scale from 0 to 9 based on current deck condition, with 9 indicating an excellent deck and 0 indicating a failed deck, or one that is deteriorated beyond corrective action and is out of service (*3*). Although the NBI rating system is utilized nationwide, recent studies have shown that it is highly subjective and that inspectors consistently overestimate the quality of bridge decks, creating a false sense of structural reliability (*3*). In addition, because the existing NBI rating system is based only on visual assessment, deck deterioration mechanisms are not detected until damage becomes visible, at which point the deck has probably declined below a condition at which cost-effective preventive maintenance measures could be applied. Instead, more expensive rehabilitation treatments must be considered.

The subjectivity of NBI ratings, together with the need for determining appropriate timing for preventive deck treatments, mandates a new index that is better able to convey current deck condition and facilitate more accurate predictions of future deck condition. The specific purpose of this research was to develop a new index for concrete bridge deck management in Utah. As the index is implemented, UDOT engineers will be better able to optimize deck maintenance, rehabilitation, and replacement (MR&R) programs.

1.2 SCOPE

This research includes data collected from 15 concrete bridge decks that were tested in the summer of 2005 and builds upon data collected from 12 decks in the summer of 2004 (*1*). Data collected in 2004 were used to identify effective testing techniques, while data from 2005 were used to create the new deck management index. Decks tested in 2005, which are the primary focus of this report, ranged from 2 to 21 years in age and included 3 decks with steel girders and 12 decks with concrete girders. In addition, all 15 decks contained epoxy-coated reinforcement.

The deck management index developed in this study is based in part upon assumptions about the efficacy of specific treatment actions as a function of the age of the deck to which they are applied. Treatment options can be analyzed in conjunction with the model proposed in this research to maximize the benefit and minimize the cost of MR&R actions. Specific benefit-cost analyses are beyond the scope of this report, however. In addition, the effects of environmental factors, traffic loading, stay-in-place metal forms, and girder type were not explicitly considered in this research.

1.3 OUTLINE OF REPORT

This report consists of five chapters. Chapter 1 discusses the purpose and scope of the report. Chapter 2 describes background information related to the study, including deck deterioration, BMSs, NBI ratings, sufficiency ratings, deterioration models, and cost information. Data collection procedures are presented in Chapter 3, and Chapter 4 details the development of the new deck management index. Chapter 5 concludes the report, summarizes the findings, and offers recommendations.

CHAPTER 2
BRIDGE DECK CONDITION ASSESSMENT

2.1 OVERVIEW

This chapter provides a discussion of several issues associated with bridge deck condition assessment, including deck deterioration, BMSs, condition ratings, deterioration models, and cost information.

2.2 DECK DETERIORATION

Bridge deck deterioration is a continuous and gradual process that is affected by traffic loading, environmental factors, current deck condition, bridge design, and material properties (*4*). Of all bridge elements, the deck is the most susceptible to deterioration due to its flatness, direct and regular exposure to deicing chemicals in cold climates, weather, and abrasion from traffic (*5, 6*). Deck deterioration has been termed a "serious national problem" (*7,* p. 60), a "tremendous problem" (*8,* p. 13), and a "problem of unprecedented magnitude" (*9,* p. 245).

The main source of deck deterioration is the corrosion of steel reinforcement due to deicing salts, which can cause both severe damage and premature failure (*10, 11*). In spite of many efforts to mitigate this problem, the rate of structural deterioration of bridge decks throughout the United States appears to be increasing, most likely due to the rapid amplification in the use of deicing salts; nationwide salt usage has increased from less than one million tons per year in the 1950s to approximately 15 million tons per year in the 1990s (*12*). The corrosion epidemic yields two major objectives for bridge managers: 1) slowing the rate of corrosion that will eventually result in costly repairs and 2) prioritizing individual bridges so that they are repaired before costly rehabilitation or reconstruction is required (*5*). In order to address these and other deck management problems, BMSs have been created.

2.3 BRIDGE MANAGEMENT SYSTEMS

BMSs were developed for use by bridge engineers in optimizing MR&R programs under financial constraints (*6*). The main purpose for using a BMS is to maximize bridge service life while minimizing life-cycle cost (LCC). The objectives of a BMS include predicting bridge needs, defining bridge conditions, allocating funds for both construction and MR&R actions, identifying and prioritizing bridges for MR&R actions, identifying bridges that require a load posting, finding cost-effective alternatives for each bridge, recommending and accounting for MR&R actions, scheduling and performing minor maintenance, monitoring and rating bridges, and maintaining an appropriate database of information (*1*). Each of these objectives can be placed in one of four categories representing the primary functions of a BMS: 1) condition assessment, 2) deterioration prediction, 3) risk assessment, or 4) maintenance optimization (*13*).

MR&R decision-making is based on two conditions: 1) current bridge condition, the accuracy of which depends on measurement technology, and 2) predicted future condition, the accuracy of which depends on the accuracy of the deterioration model (*6*, *14*, *15*). Both components are essential for an effective BMS. For this reason, the American Association of State Highway and Transportation Officials (AASHTO) suggests that a bridge deterioration model is one of the minimum requirements of a BMS (*6*).

Effective MR&R decision-making requires optimum timing of bridge treatments. Preventive treatment is the most cost-effective, followed by rehabilitation and, lastly, replacement. Because the benefit-cost ratio associated with preventive treatment is often as high as 4 to 1, BMSs should be designed to provide recommendations on the timing of preventive actions (*5*).

2.4 CONDITION RATINGS

UDOT currently uses NBI ratings and sufficiency ratings to determine current bridge condition and to prioritize bridge maintenance activities. These rating systems are described in the following sections.

2.4.1 National Bridge Inventory Ratings

The government initiated the NBI system to standardize bridge condition assessment procedures nationwide and to facilitate national bridge condition evaluations. This protocol mandates that each state department of transportation conduct an inspection on each bridge every two years and record information about the wearing surface, structural condition, expansion joints, railing, fencing, sidewalks, curbs, and median. In addition, all decks should be examined for skid resistance to determine if a hazard exists. Drains and drain outputs should also be checked, and concrete decks should be inspected for cracking, leaching, scaling, potholing, spalling, and other evidence of deterioration (*16*). The bridge deck, superstructure, and substructure are then each assigned a rating between 0 and 9 as shown in Table 2.1 (*1*). These ratings are associated with specific repair actions listed in Table 2.2 (*17*).

In 2001, a study was performed to determine the amount of subjectivity that exists in the NBI rating scale. Forty-nine inspectors with a minimum experience of 10 years each were selected from 10 states for participation in the experimental program. The inspectors, who were not informed of the previous NBI ratings on the bridges before completing their individual evaluations, provided ratings for deck, superstructure, and substructure integrity for six bridges included in the research. Prior to the testing, a consensus rating was determined for each deck by a separate group of experienced bridge inspectors. Of the 18 total ratings performed by each inspector, 13 were larger than the

TABLE 2.1 Bridge Deck Condition Rating (*1*)

NBI Rating	Description
9	Excellent
8	Very Good
7	Good
6	Satisfactory
5	Fair
4	Poor
3	Serious
2	Critical
1	Imminent Failure
0	Failed

TABLE 2.2 Condition Ratings and Repair Actions (*16*)

NBI Rating	Suggested Repair Action
7	Minor Maintenance
6	Major Maintenance
5	Minor Repair
4	Major Repair
3	Rehabilitate
2	Replace

consensus rating, and the results of a statistical *t*-test indicated that only four of the 18 ratings were representative of the consensus. The study further concluded that the NBI rating definitions could not be sufficiently redefined to enable reliable routine inspection results to be obtained (*3*).

Similarly, a previous analysis determined that the varied backgrounds of inspectors led to different ratings and concluded that the NBI rating is subjective and does not properly assess intrinsic bridge mechanisms or behavior. The authors of that study recommended that some objective evaluation should be included in condition assessment (*18*). Subjective or inaccurate condition assessment has been identified as the most serious technical barrier to the effective management of highway bridges (*19*). Effective BMSs must incorporate more accurate methods of measuring and predicting bridge condition (*20*).

2.4.2 Sufficiency Ratings

Unlike NBI ratings, sufficiency ratings are assigned to an entire bridge structure, not just to a specific element such as a bridge deck. The rating is based on a numerical scale from 0 to 100, with 100 being a new bridge and 0 being one that is deteriorated beyond corrective action and is out of service (*21*). The rating is derived from the observed condition of the bridge and can be used to supplement NBI ratings in BMSs. Bridges with values less than 80 have priority for federal rehabilitation funds (*22*).

2.5 DETERIORATION MODELS

As shown in Figure 2.1, a deterioration model relates deterioration to the age of a structure and should be designed to reflect improvements in the condition index (CI) associated with the model as a result of MR&R action. Because such a model is the basis for predictions of future condition, it is an essential component of an effective BMS (*23*). Deterioration models usually follow an s-shaped curve like the one shown in Figure 2.1 (*1*).

Several attempts have been made to develop bridge deck deterioration models (*11, 17, 24*). For example, one deterioration model developed for the Wisconsin Department of Transportation uses a scale ranging from 0 to 100 and is based on the percentages of spalled and delaminated areas and the chloride content at the level of the reinforcement. A rating of 100 represents a fully deteriorated bridge deck, while 0 represents a deck in perfect condition (*24*). An index developed by the Strategic Highway Research Program also uses a scale between 0 and 100 and is also computed as a function of the percentages of spalled and delaminated areas and chloride content. For that CI, 100 represents a deck in perfect condition, and 0 represents one that is

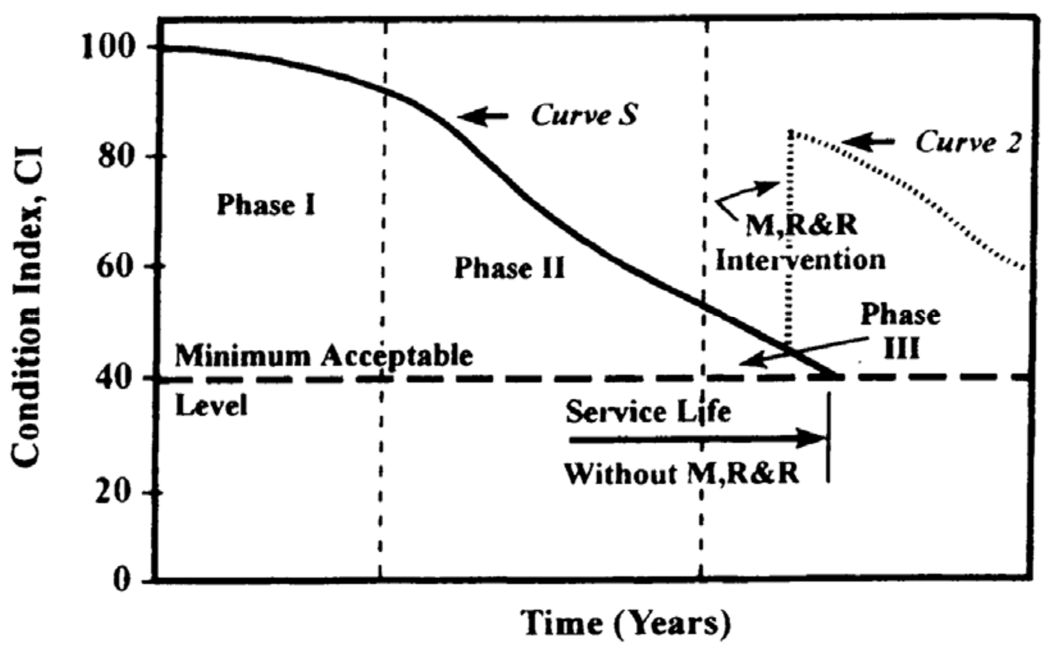

FIGURE 2.1 Deterioration curve (*1*).

completely deteriorated. When the value reaches 45, the bridge deck is considered to have reached the end of its service life (*11*). Although these two deterioration models utilize properties associated with deck distress, age and traffic flow data have also been incorporated into CIs for deck management (*17*).

None of the indices identified in the literature were determined to be suitable for concrete bridge deck management in Utah. The first two use areas of spalled and delaminated concrete to determine the CI. While such measures may be adequate for determining the timing for rehabilitation or reconstruction, they are not appropriate for optimizing the placement of preventive treatments, which are associated with the highest benefit-cost ratios as described previously. Indeed, by the time cracks or potholes are noticeably present on a deck, the reinforcement has already corroded past the point when preventive treatment would be effective. Because corrosion of reinforcement is the main cause of deck deterioration, this deficiency represents major limitations in the utility of these models, especially with regard to preventive maintenance programs.

The corrosion process involves two critical times: 1) time to corrosion initiation, which is affected by the concrete cover thickness and concrete porosity, and 2) time until development of visual distress, which is principally affected by age and chloride concentration. Because the efficacy of MR&R treatments depends on the state of the reinforcement corrosion and the deck distress, deterioration models must include factors associated with both of these critical times.

2.6 COST INFORMATION

A bridge deck represents the most expensive portion of a roadway (*16*). Estimates suggest that one-third to one-half of the projected bridge rehabilitation costs in North America will be used for the rehabilitation of deteriorated bridge decks (*13*). The cost of decks is usually measured in terms of LCC, which is the cost of the deck for the entire life of the bridge. The LCC should be minimized through the use of treatments having high benefit-cost ratios. A study completed in 2004 concluded that the use of asphalt-concrete overlays with membranes resulted in the lowest LCC as compared to patching, concrete overlays, asphalt-concrete overlays without membranes, and deck replacement with epoxy-coated bars (*24*). The study also determined that the highest

LCC resulted when patching was used as a means of treatment (*24*). These results reflect the fact that earlier intervention is usually associated with higher benefit-cost ratios (*5*).

Identifying the presence of damage early and accurately quantifying its effect on the structural integrity of the affected bridge are essential in achieving cost-effective bridge management. Given the comparatively high benefit-cost ratios associated with preventive maintenance, BMSs should facilitate optimization of preventive maintenance treatments, not just rehabilitation and reconstruction activities. Because previously developed CIs do not adequately fulfill this need, the purpose of this research was to develop a new Utah bridge deck index (UBDI) suitable for optimizing the timing of both preventive and rehabilitation treatments by UDOT.

2.7 SUMMARY

Deterioration of concrete bridge decks is an increasing problem in the United States. While BMSs have been developed to assist bridge engineers and managers with implementing MR&R programs, NBI and sufficiency ratings are too subjective and do not permit early identification of corrosion activity, which is the leading cause of deck distress. Although these measures may be adequate for determining the timing of rehabilitation or reconstruction activities, they are not appropriate for optimizing the placement of preventive treatments. This major deficiency is characteristic of all of the deck CIs identified in this research and warrants development of a new CI suitable for optimizing the timing of preventive maintenance treatments, which have higher benefit-cost ratios than both rehabilitation and reconstruction activities.

CHAPTER 3
EXPERIMENTAL METHODOLOGY

3.1 OVERVIEW

Several techniques are available for performing condition assessments of concrete bridge decks, including but not limited to visual inspection, sounding, measurement of dielectric values, ground-penetrating radar imaging, Schmidt hammer testing, resistivity testing, half-cell potential testing, and chloride concentration testing (*1*, *21*). In previous research at Brigham Young University (BYU), these assessment techniques were evaluated based on the variability inherent in the measurements, the correlation of the data to other measures of deck condition, and the existence and credibility of threshold values (*1*, *25*). The studies concluded that the most viable testing techniques were visual inspection, sounding, Schmidt hammer testing, half-cell potential testing, and chloride concentration testing. Therefore, these techniques were utilized in this research for testing 15 concrete bridge decks selected by UDOT engineers for evaluation during the summer of 2005. The decks ranged from 2 to 21 years in age and were all located in Salt Lake City. The testing included three bridges with steel girders, designated with a prefix F in the deck name, and 12 bridges with concrete girders, designated with a prefix C in the deck name, all of which were constructed using epoxy-coated rebar. The bridges and relevant characteristics are shown in Table 3.1. The locations of these bridges are shown in Figure 3.1.

Before each deck was tested, the deck length was measured, and six 6-ft by 6-ft test areas on the deck surface were randomly selected within the single lane closed to traffic. The number of test locations required per deck was determined using statistics from the spatial variation associated with test results obtained in previous work (*1*). The six test areas were marked with spray paint as shown in Figure 3.2 and swept prior to testing in order to remove any dirt or debris on the deck. The data collection procedures for each of the five testing techniques utilized in the research are discussed in the following sections.

TABLE 3.1 Bridge Data

Bridge ID	Direction of Travel	Direction Tested	Mile Post	Location	Facility	Featured Intersection	Polymer Overlay	Date Testing Performed
C-438	NB & SB	NB	102	North of Black Rock Int.	I-80	I-80 & Railroad	No	30-Jul
C-460	NB & SB	NB	21.4	850 S & 2000 W	I-215	Indiana Ave & Railroad	No	21-May
C-688	NB & SB	NB	21.9	500 S & 2000 W	I-215	I-215 & 500 S	No	14-May
C-698	NB	NB	21.8	500 S & 2000 W	Ramp from I-215 NB to I-80 EB	500 S & Railroad	No	21-May
C-699	NB	NB	21.8	N of 500 S at 2000 W	Ramp from I-215 NB to I-80	I-215 & Railroad	No	21-May
C-726	NB & SB	NB	9.5	6550 S & 900 E	SR-71 (900 E)	I-215 & 900 E	No	16-Jul
C-736	WB	WB	7.7	6600 S & 2000 E	On-ramp to I-215 WB	I-215 & SR-152	Yes	30-Jul
C-752	NB & SB	NB	20.6	W of Redwood Rd at California Ave	I-215	I-215 & California Ave	Yes	14-May
C-759	EB & WB	WB	6.5	0.2 mi SW of Knudson Cnr Int	I-215	I-215 & Holladay Blvd	Yes	4-Jun
C-760	WB	WB	6.5	0.2 mi SW of Knudson Cnr Int	On-ramp to I-215 WB	I-215 & Holladay Blvd	No	4-Jun
C-844	EB & WB	EB	17.4	SR-201 Int at 2400 S	SR-201 to I-15 NB	SR-201 & I-215 Ramp	No	16-Jul
C-919	EB & WB	WB	6.2	Near Copper Co. Magna Plant	SR-201	SR-201 & Copper Co. Haul Road	No	30-Jul
F-500	NB & SB	NB	23.3	700 N & 2000 W	I-215	I-215 & 700 N	No	16-Jul
F-504	NB & SB	SB	8.0	6650 S & 1300 E	1300 East	I-215 & 1300 E	No	4-Jun
F-506	NB & SB	NB	8.1	2300 E & 6450 S	2300 South	I-215 & 2300 S	No	16-Jul

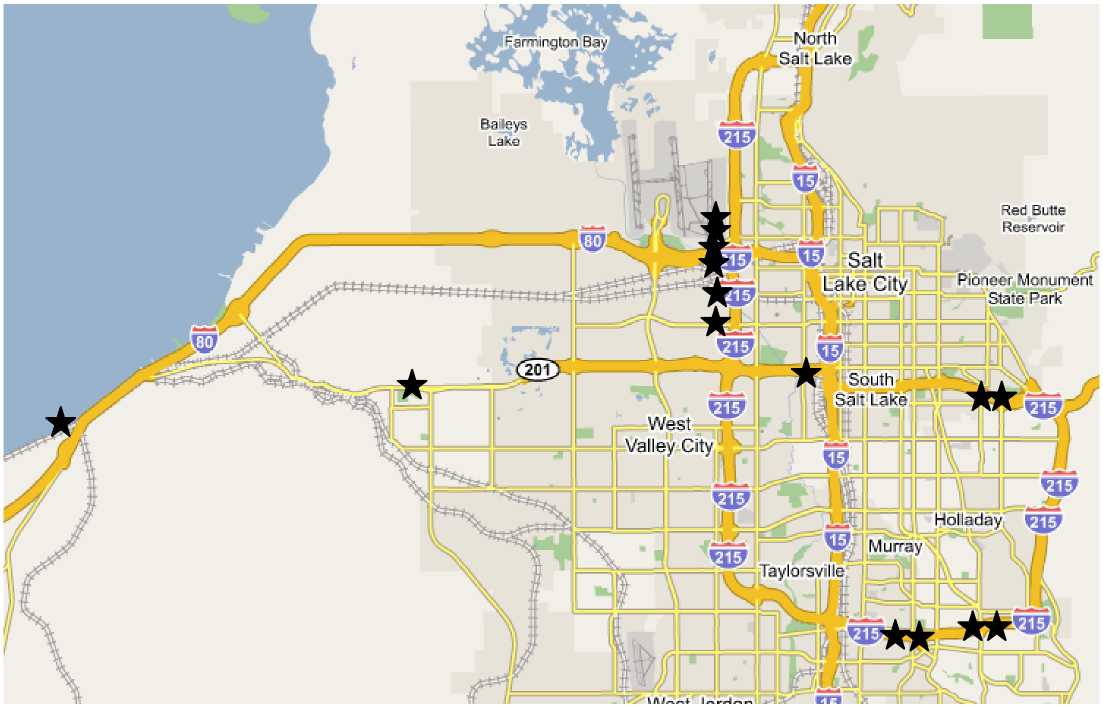

FIGURE 3.1 Bridge deck locations (*26*).

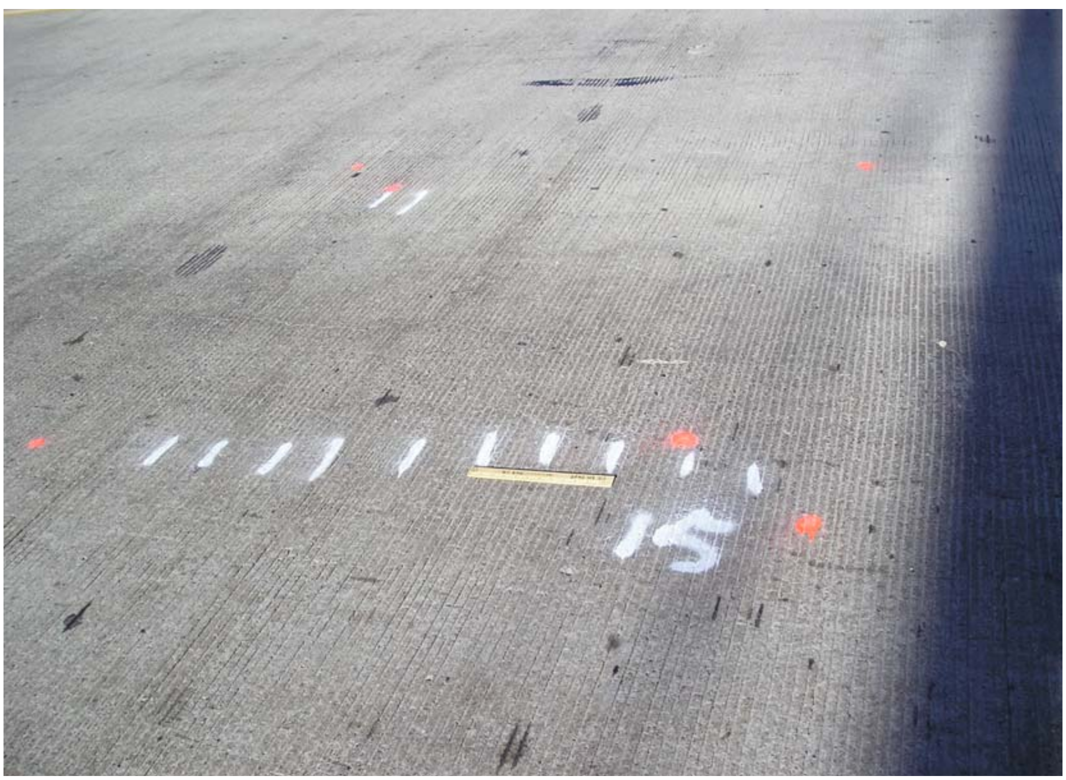

FIGURE 3.2 Typical test area.

3.2 VISUAL INSPECTION

Visual inspection is a systematic process that involves locating and recording any physical manifestations of deterioration (*27*). In this research, visual inspection was performed in each of the test areas on the 15 bridge decks. The testers inspected the deck for any noticeable cracking, spalling, scaling, potholes, efflorescence, or exposed reinforcement. The data collected within each of the six test areas were recorded on individual distress survey forms. If a protective overlay existed on the deck, crack data were not collected because the overlay masked the true condition of the underlying concrete. Pothole data, however, were collected in this situation since a pothole is likely to be seen even if an overlay is present. The lengths of all visible cracks were drawn to scale on the distress survey forms, and crack widths were measured using a crack comparator card as shown in Figure 3.3. The presence of scaling, efflorescence, or exposed reinforcement was also noted on the distress survey, although no calculations were performed using these data.

FIGURE 3.3 Crack comparator card.

3.3 SOUNDING

Sounding involved locating areas of the deck where delaminations existed, or where portions of the concrete had become detached from the subsurface. Delaminations typically result from volume-expanding corrosion of the steel reinforcement within the deck (*9*).

Sounding procedures followed those in the American Society for Testing and Materials (ASTM) D 4580, Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding. Two forms of sounding were performed in each of the test areas, including chain dragging and hammer sounding. Chain dragging was used to determine the probable locations of delaminations and involved dragging an ordinary steel chain over the surface of the concrete while listening for changes in the acoustical response (*9*). Normal concrete exhibits a clear ringing sound, while delaminated concrete produces a dull hollow sound (*28*). The chaining process is shown in Figure 3.4. After a delamination was detected using chaining, hammer sounding was performed as shown in Figure 3.5 to more precisely estimate the delamination size. In hammer sounding, the

FIGURE 3.4 Chain dragging.

FIGURE 3.5 Hammer sounding.

operator repeatedly struck the concrete surface with a hammer while listening to the acoustical response. The size of the delamination was then recorded on the distress survey. Although sounding can be effectively performed on decks with thin overlays, the operator cannot usually determine if detected delaminations are located in the underlying concrete or if they result from detachment of the overlay from the concrete substrate. Therefore, if an overlay existed, sounding was not performed in this study.

3.4 SCHMIDT HAMMER

Schmidt hammer testing measures the rebound of a hardened steel hammer impacted on the concrete by a spring (*29*). The test results reflect the rebound of the spring-loaded plunger as a percentage, or rebound number, of the initial extension of the spring (*30*). The Schmidt rebound number is a useful estimate of the compressive strength of the concrete; higher rebound numbers correspond to stiffer or higher quality concrete (*31*).

The procedures used to measure rebound numbers are described in ASTM C 805, Standard Test Method for Rebound Number of Hardened Concrete. The use of the Schmidt rebound hammer requires a smooth surface, which was provided by traffic wear in this study. Schmidt hammer readings were taken at nine locations in each test area as shown in Figure 3.6. In this testing, the Schmidt hammer was placed directly on the concrete surface between the tining grooves and operated in a vertical position. Measurements were not performed on decks with polymer overlays because the underlying concrete could not be easily exposed for testing. From the readings taken in the field, the strength of the concrete was estimated for each deck from a correlation between rebound number and compressive strength provided by the hammer manufacturer. Figure 3.7 shows the device being used.

Although Schmidt hammer testing is an effective way of estimating concrete strength, some shortcomings do exist. Test results are very dependent on the surface finish, moisture content, temperature, rigidity, and carbonation of the concrete. Moreover, values are also affected by the direction of impact, which is sometimes hard to control (*29*).

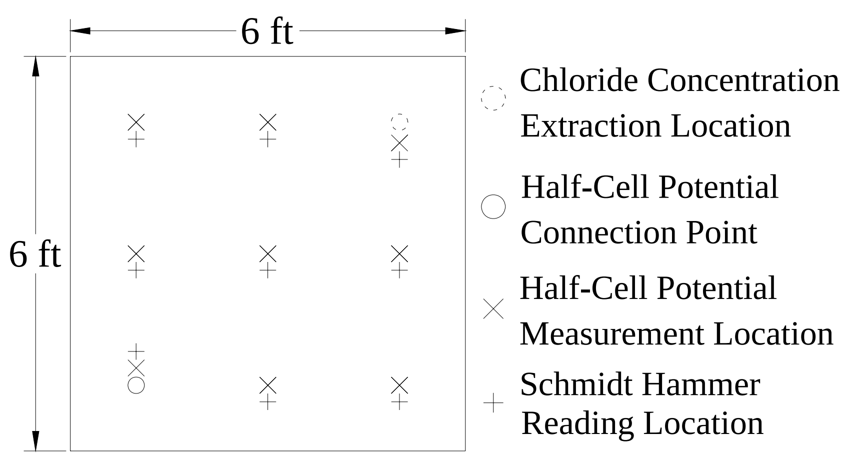

FIGURE 3.6 Test area layout.

FIGURE 3.7 Schmidt hammer rebound testing.

3.5 HALF-CELL POTENTIAL

Half-cell potential measurements can be used for identifying locations where steel is corroding, classifying the corrosion activity of steel, and determining the effectiveness of proposed repair methods (*32*). Half-cell potential values are generally obtained by measuring the electrical half-cell potential of reinforcing steel using a copper-copper sulfate ($Cu-CuSO_4$) reference electrode (CSE) (*1*).

In this research, testing followed the procedures given in ASTM C 876, Standard Method for Half-Cell Potential of Uncoated Reinforcing Steel in Concrete. However, because the rebar was epoxy-coated and electrical continuity between test locations was therefore not assured, separate connections to the rebar were established at each test location as indicated in Figure 3.6. Since half-cell potential measurements are only valid for the individual rebar to which the meter is attached when electrical continuity is not present, this protocol ensured that valid readings would be obtained at every test location; this process of establishing discrete connection points on the epoxy-coated rebar and then conducting the half-cell potential survey in the near vicinity of the connection point has also been utilized by other researchers (*33*). For each test area, a cover meter was used to locate the reinforcement, and a hole was drilled to the depth of reinforcement. The negative lead of the half-cell potential voltmeter was connected to the reinforcement, and the positive lead was connected to the CSE, which was coupled to the deck with a moistened sponge. The concrete was sprayed with water before the testing to ensure adequate electrical coupling between the deck and the sponge, and two half-cell potential measurements were then taken at each of the nine locations within each test area as indicated in Figure 3.6. Figure 3.8 shows this test being performed.

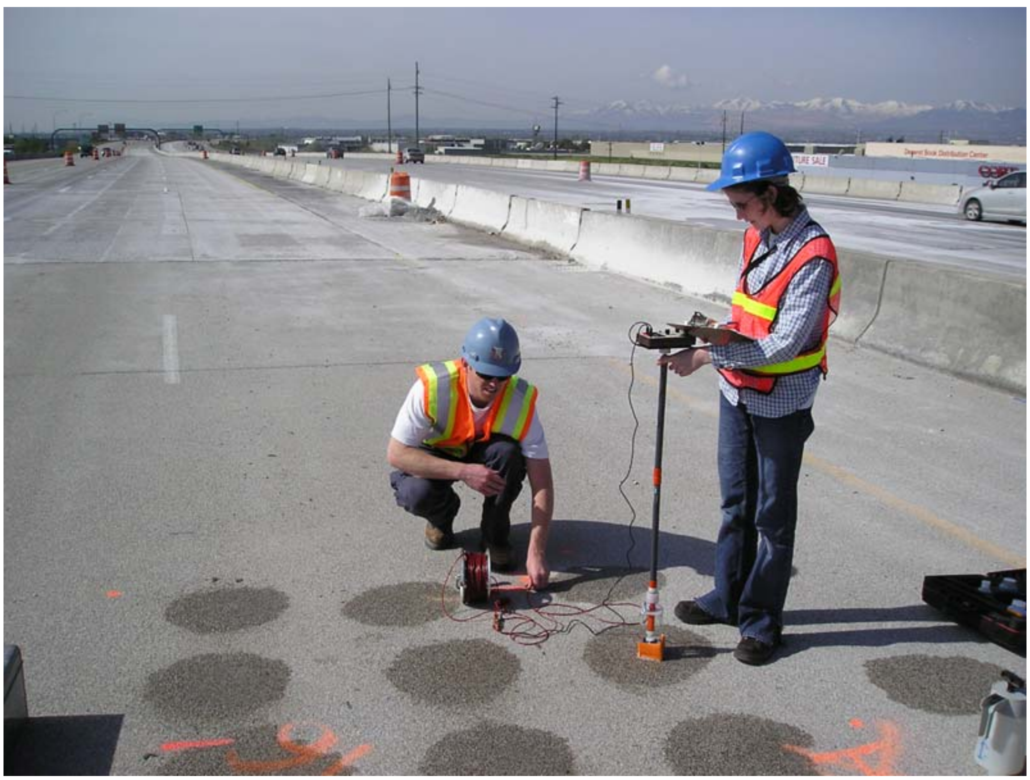

FIGURE 3.8 Half-cell potential testing.

3.6 CHLORIDE CONCENTRATION

Chloride concentration testing is a tedious process that involves both field sample collection and laboratory testing. The primary source of chlorides on bridge decks is deicing salt routinely used for winter deck maintenance (*12*). Corrosion can be initiated when chlorides within the concrete immediately surrounding the reinforcing steel reach a critical concentration, commonly assumed to be 2 pounds of chloride per cubic yard of concrete (*1*).

In this research, sampling for chloride concentration testing was performed to a depth of about 4 in. in order to obtain measurements below the reinforcement. This required drilling away from the reinforcement, which was located using a cover meter as previously discussed. The sample collection location in each test area is marked in Figure 3.6. A hammer drill was used to pulverize the concrete in approximately 1-in. depth intervals, and the bit size was decreased with increasing hole depth as shown in Figure 3.9; this practice minimized contamination of deeper samples by reducing the probability of inadvertently scraping near-surface concrete during the drilling process. After each lift was drilled, the pulverized concrete powder was manually removed from the test hole and placed into a plastic sample bag as shown in Figure 3.10. The hole and drill bit were then cleaned using compressed air, the depth of the hole was measured using a digital micrometer, and the next lift was drilled. The bags were individually labeled in succession and then transported to the BYU Highway Materials Laboratory for chemical analyses.

The chloride extraction procedures delineated in ASTM C 1218, Standard Test Method for Water-Soluble Chloride in Mortar and Concrete, were used to determine the chloride concentration of each sample. This method requires that sample particles pass through a No. 50 (0.0018 inch) screen, which was ensured through the use of the hammer drill in the field. Water was added to the dry samples, and the mixtures were then boiled for 5 minutes and subsequently cooled for 24 hours. Filtration was used to separate the

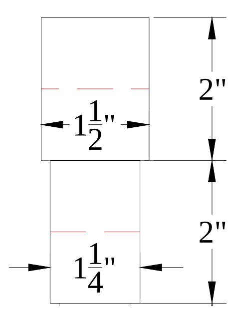

FIGURE 3.9 Drilling dimensions.

FIGURE 3.10 Concrete sampling for chloride concentration analysis.

soluble chloride ions from the pulverized concrete particles, and the solution was treated with 3 mL of nitric acid followed by 3 mL of hydrogen peroxide. The solution was again heated, but just until boiling, and then cooled for an additional 24 hours before a laboratory chloride-ion selective probe was used to determine the chloride content of the solution. The chloride concentration at the depth of the top of the rebar was then calculated by means of linear interpolation between laboratory-determined chloride concentrations above and below this point; the cover depth was measured in the field at the half-cell potential test locations.

3.7 SUMMARY

Visual inspection, sounding, Schmidt hammer testing, half-cell potential testing, and chloride concentration testing were utilized in this research for testing 15 concrete bridge decks selected by UDOT engineers for evaluation during the summer of 2005. The decks ranged from 2 to 21 years in age and were all located in Salt Lake City. Six 6-

ft by 6-ft test areas on the surface of each deck were randomly selected within the single lane closed to traffic, and testing protocols followed ASTM standards to the extent possible.

CHAPTER 4
RESULTS

4.1 OVERVIEW

The purpose of this chapter is to explain the process of developing a new index for concrete bridge deck management in Utah. All aspects of the index development are presented, including data calculations, statistical analyses, development of the new CI, consideration of treatment effects, and sampling guidelines.

4.2 DATA CALCULATIONS

The collected data were used to calculate values in the categories of visual inspection, sounding, Schmidt hammer, half-cell potential, and chloride concentration. Calculation procedures for each of these five types of data together with their relevant threshold values are discussed in the following sections.

4.2.1 Visual Inspection Data

The type, severity, and location of visual distress were recorded on distress survey forms, which are shown in Figures 4.1 through 4.12 for 12 of the 15 decks tested in this research; distress surveys for decks C-736, C-752, and C-759 are not shown since the presence of polymer overlays prevented visual inspection of the underlying concrete. In these figures, each large square represents one of the six randomly distributed 6-ft by 6-ft test areas evaluated on each deck, and circles labeled with "P" and "D" represent potholes and delaminations, respectively. Calculated visual data include crack width, crack severity, and crack density. Crack severity is a function of the average crack width in inches, and crack density is reported as the linear feet of cracking per square yard of deck surface. The cracks were classified as hairline, narrow, medium, or wide based on the widths given in Table 4.1 (*1*). Pothole density was calculated by dividing the surface area of potholes in square feet by the entire test area in square yards. Crack data are summarized in Table 4.2, and Table 4.3 summarizes the crack severity rankings for the 15 decks. Pothole data are summarized in Table 4.4. For parameters resulting from repeated measurements, the average and standard deviation were computed for each deck and are shown in the appropriate tables. In all cases, a hyphen indicates that a measurement could not be obtained or is not applicable.

Although crack severity, crack density, pothole size, and pothole density currently have no established, universally accepted threshold values, some engineers suggest that action should be taken if the average crack width is greater than 0.0625 in. and moderate crack density or efflorescence in the vicinity of the cracks exist (*1*). AASHTO suggests that the deck should be replaced if 10 to 50 percent of the deck is affected by potholes (*1*). Compared to these thresholds, the data collected in this research suggest that none of the 15 bridge decks need to be replaced.

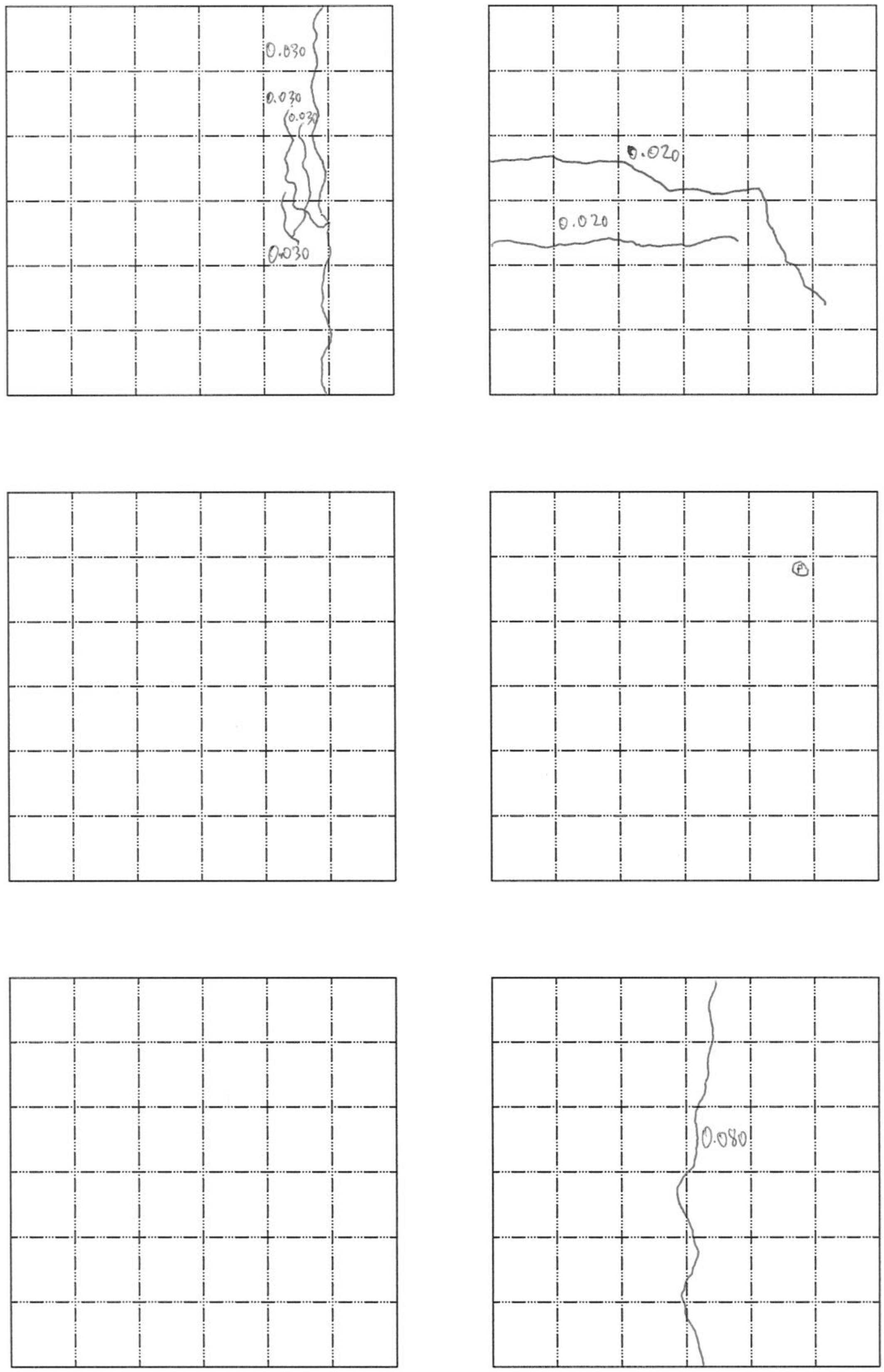

FIGURE 4.1 Distress survey for deck C-438.

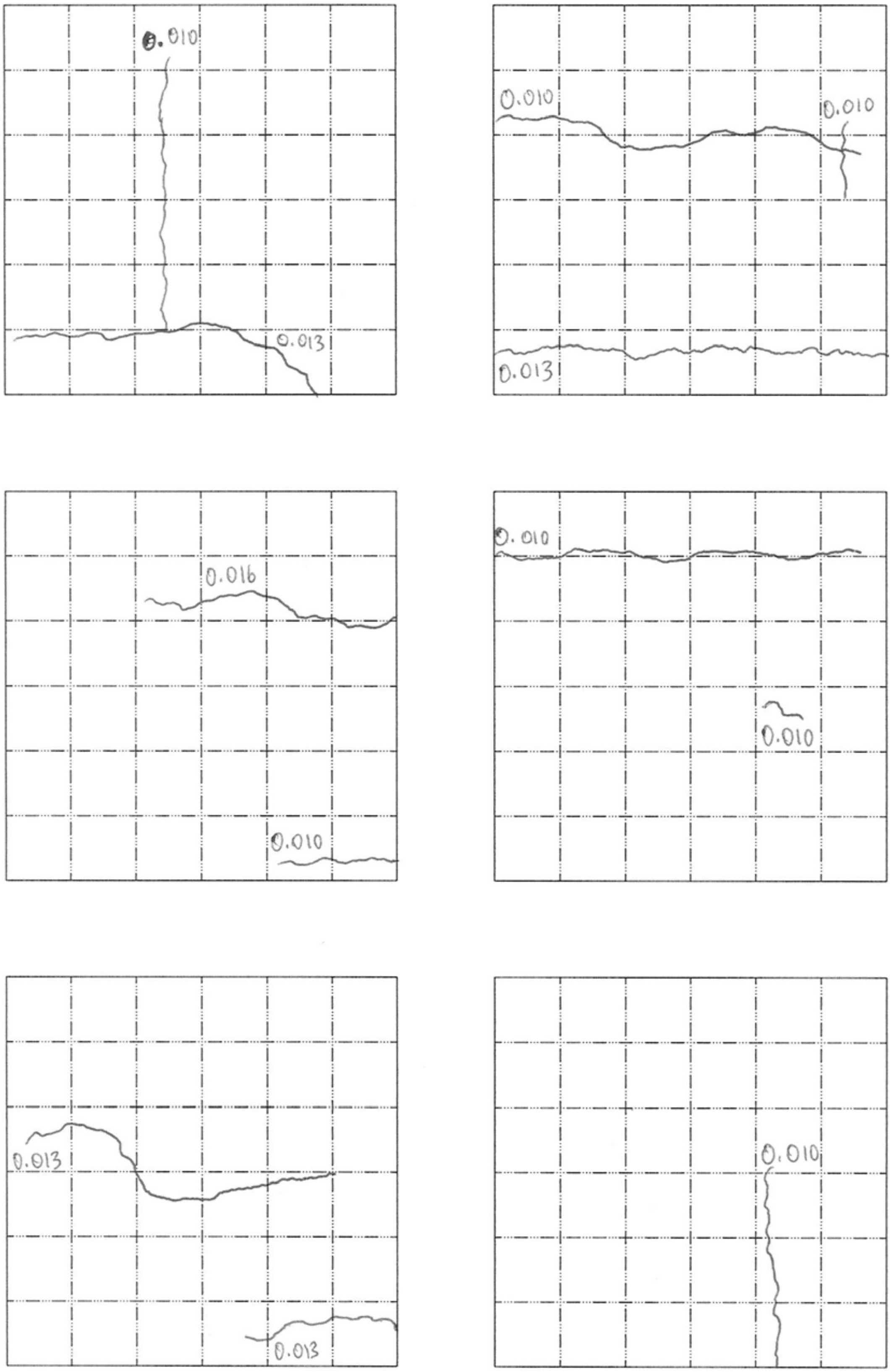

FIGURE 4.2 Distress survey for deck C-460.

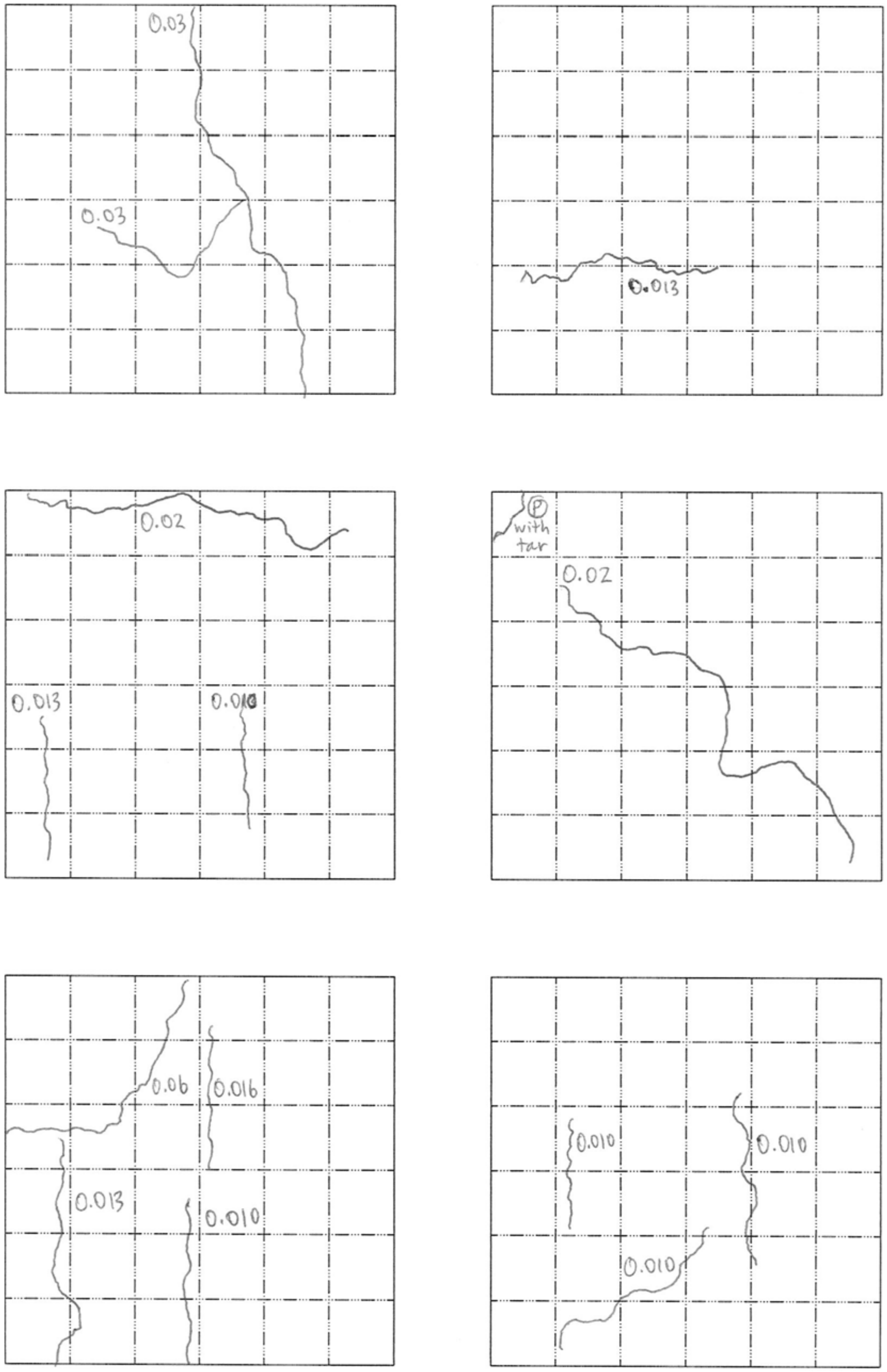

FIGURE 4.3 Distress survey for deck C-688.

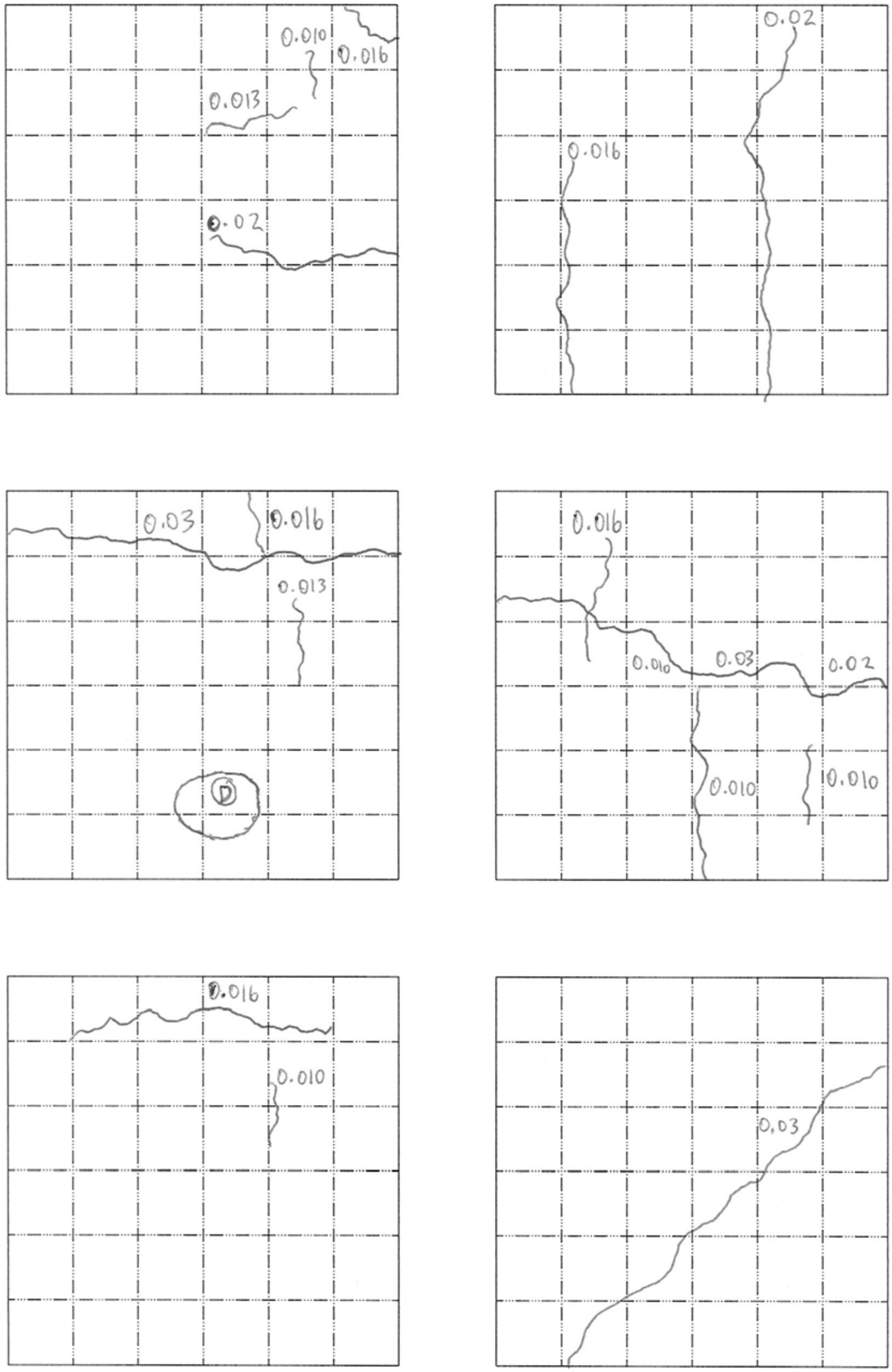

FIGURE 4.4 Distress survey for deck C-698.

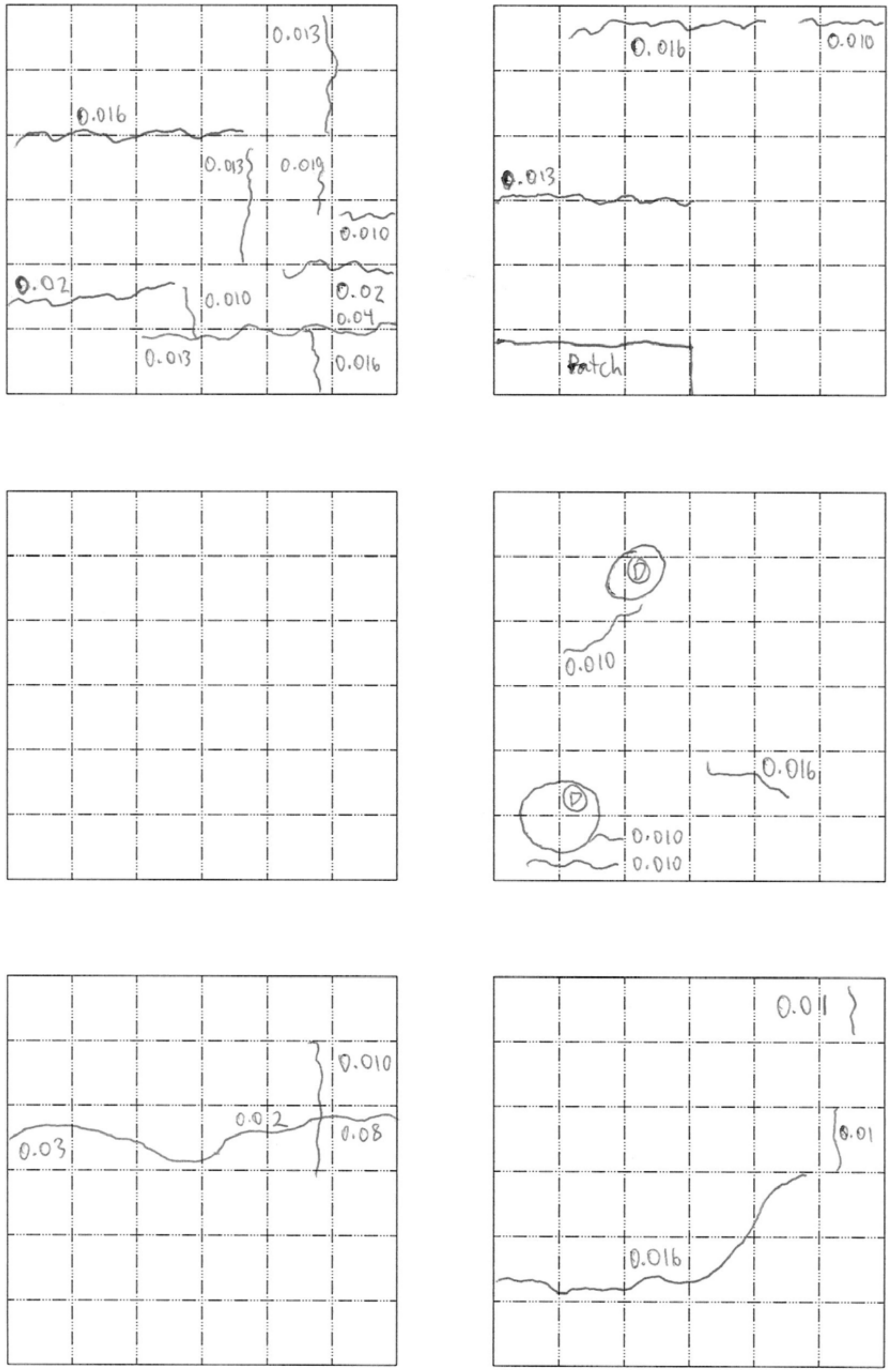

FIGURE 4.5 Distress survey for deck C-699.

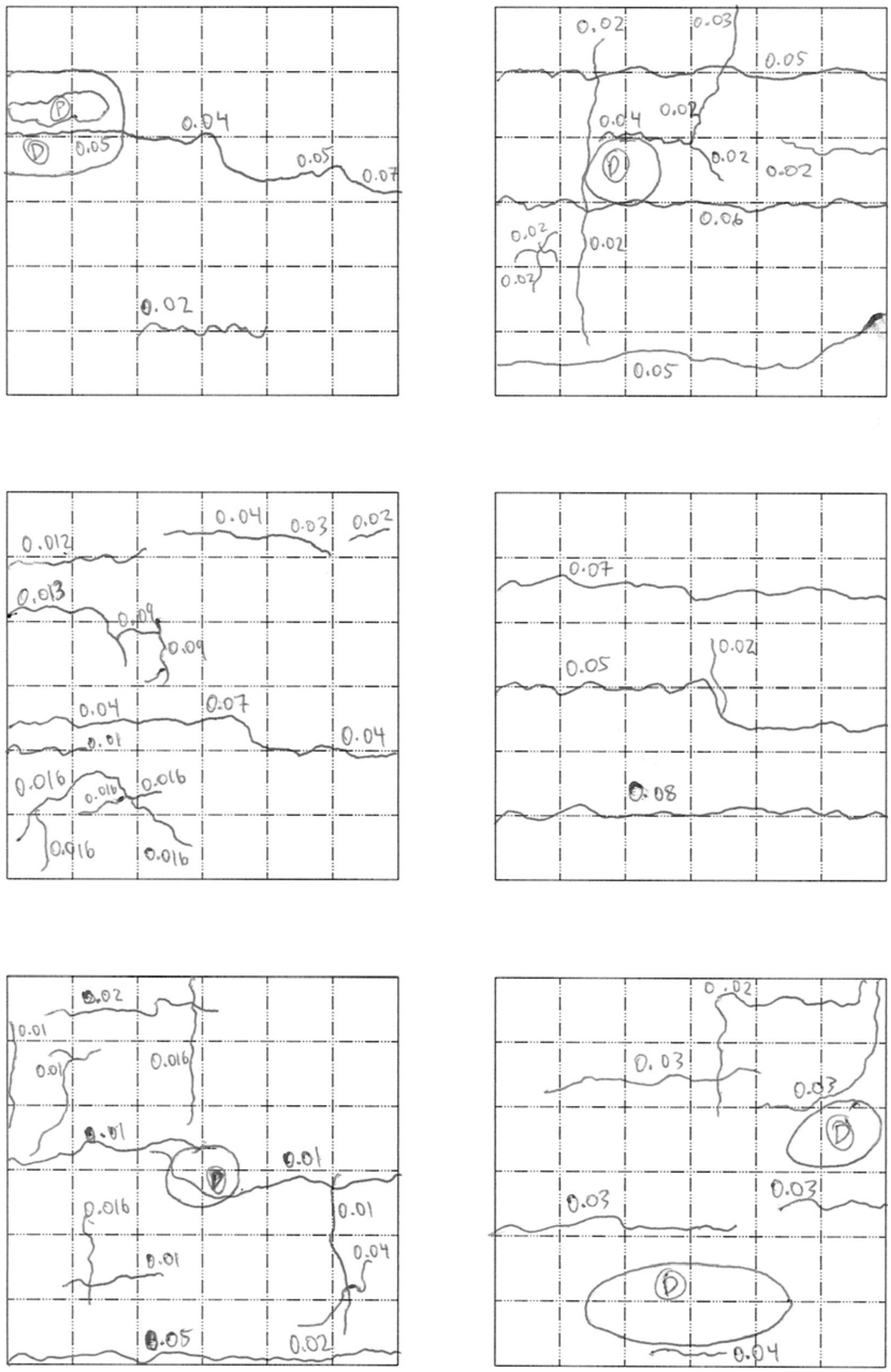

FIGURE 4.6 Distress survey for deck C-726.

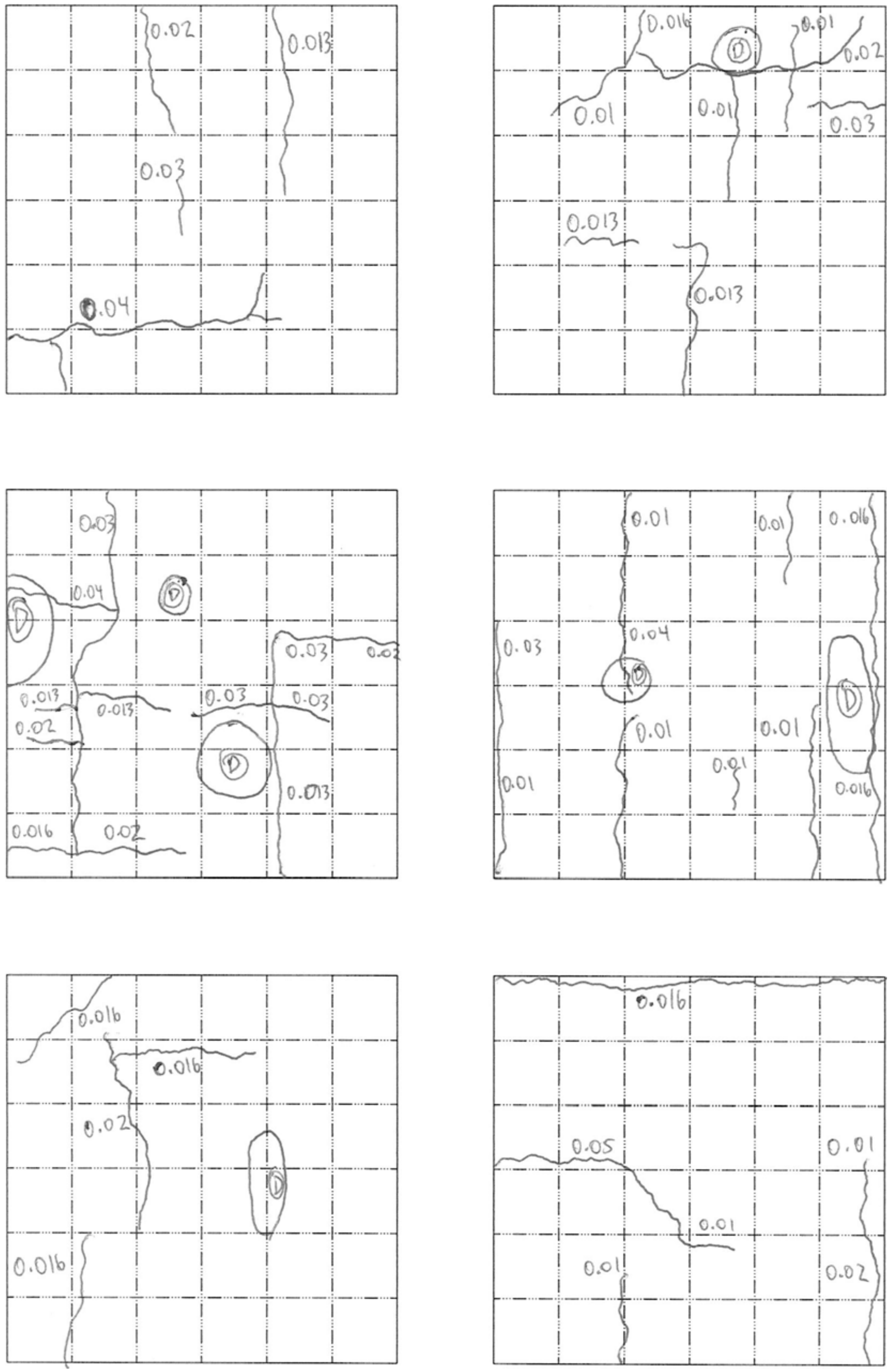

FIGURE 4.7 Distress survey for deck C-760.

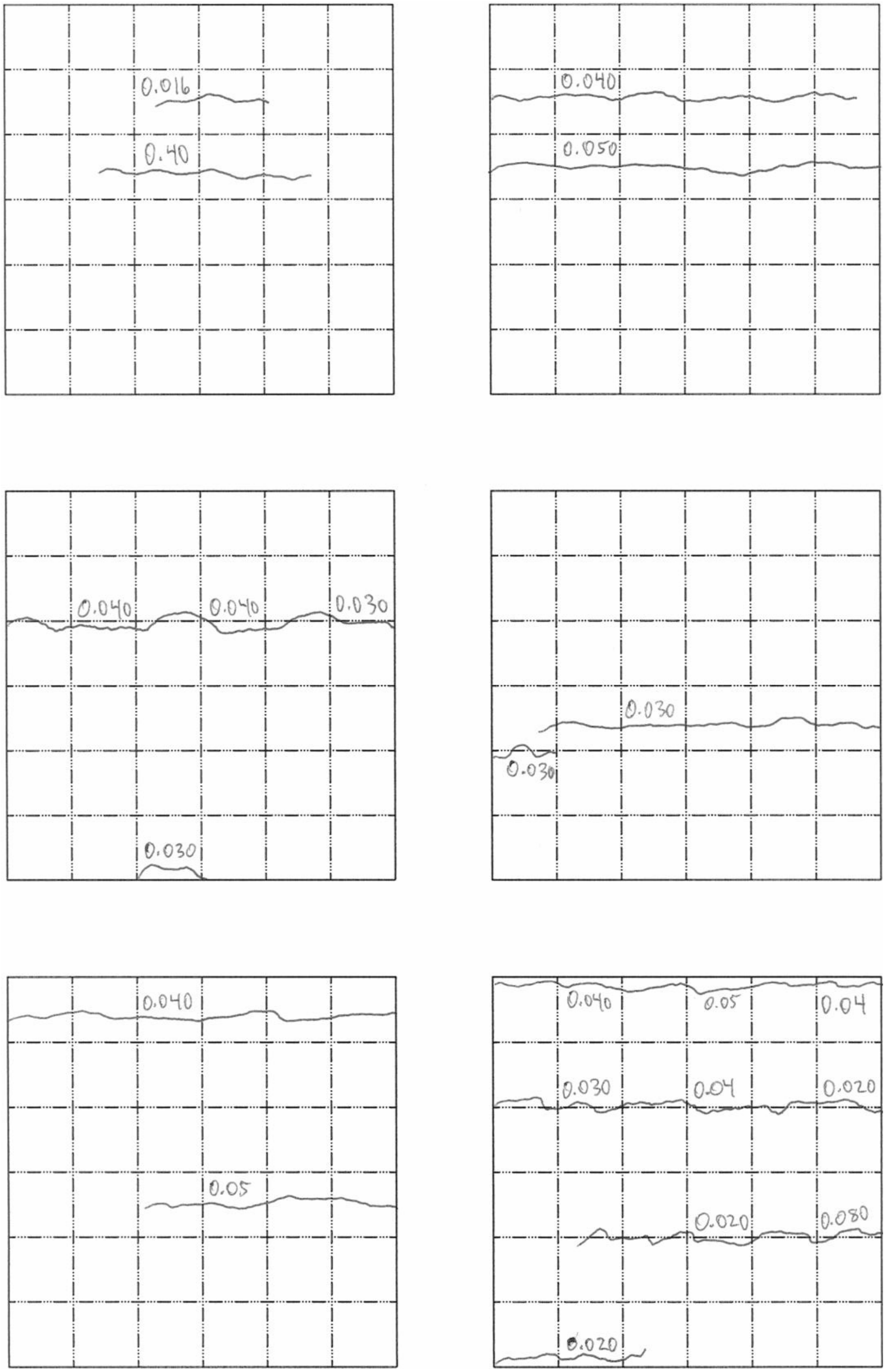

FIGURE 4.8 Distress survey for deck C-844.

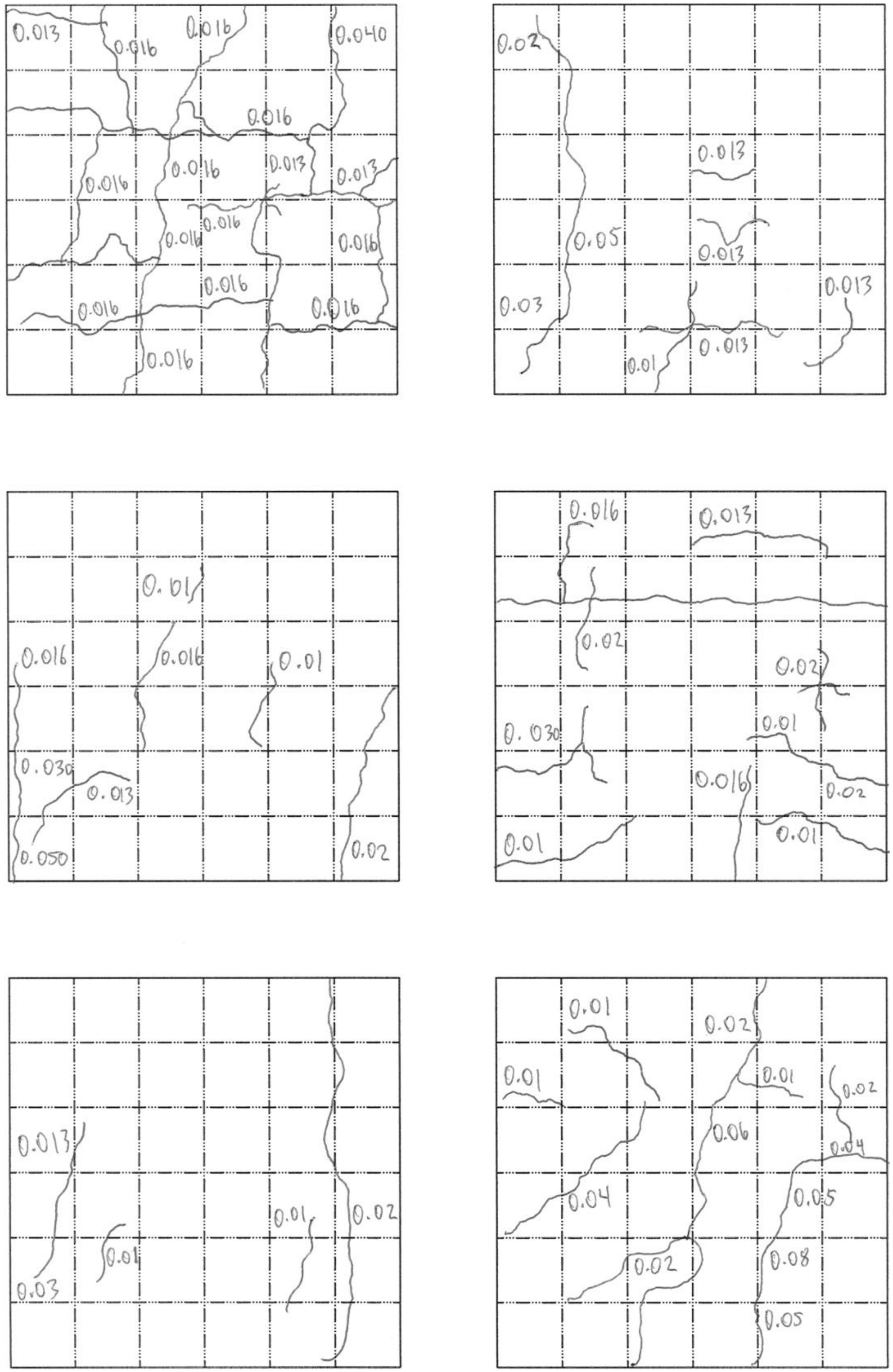

FIGURE 4.9 Distress survey for deck C-919.

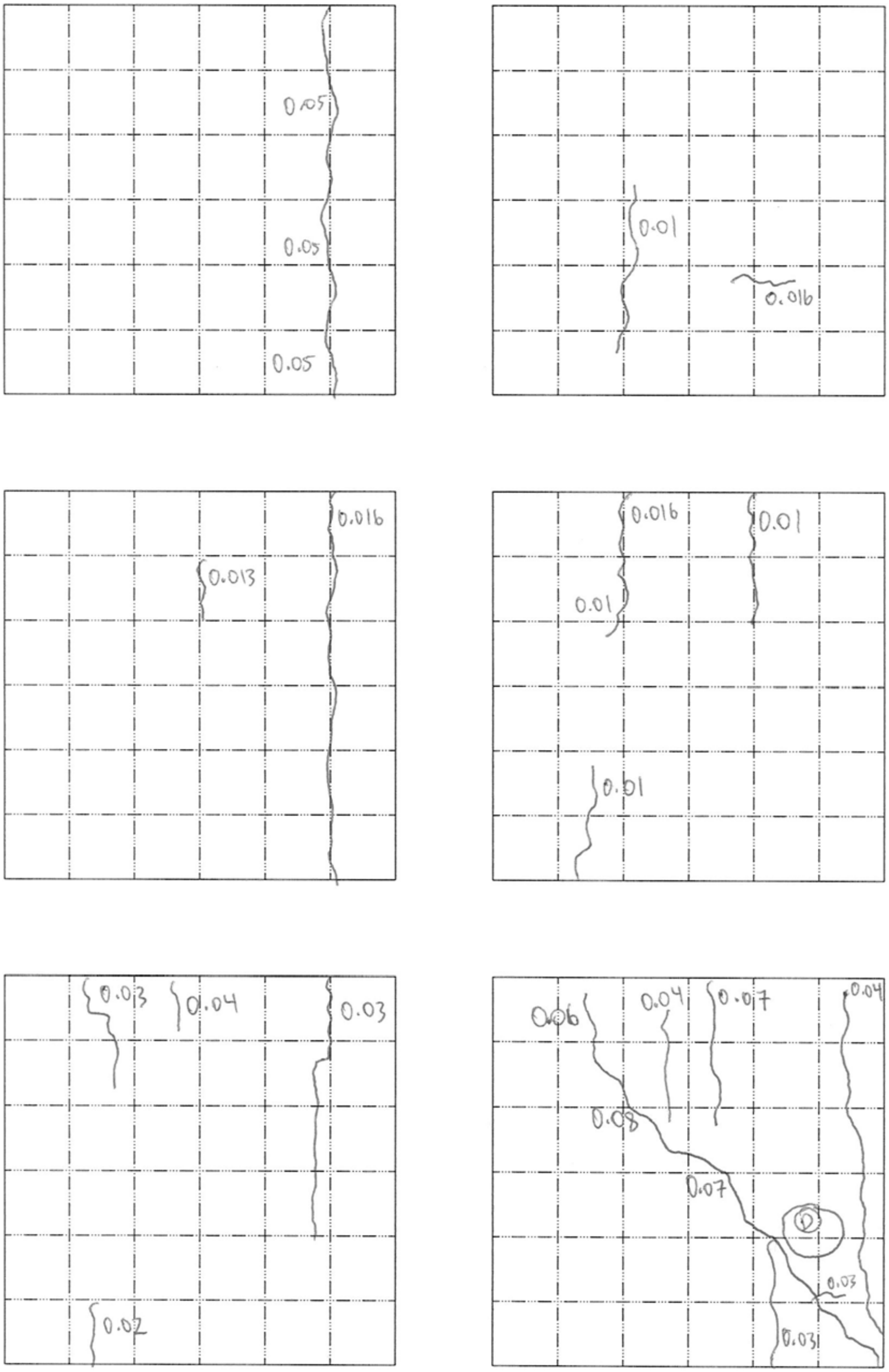

FIGURE 4.10 Distress survey for deck F-500.

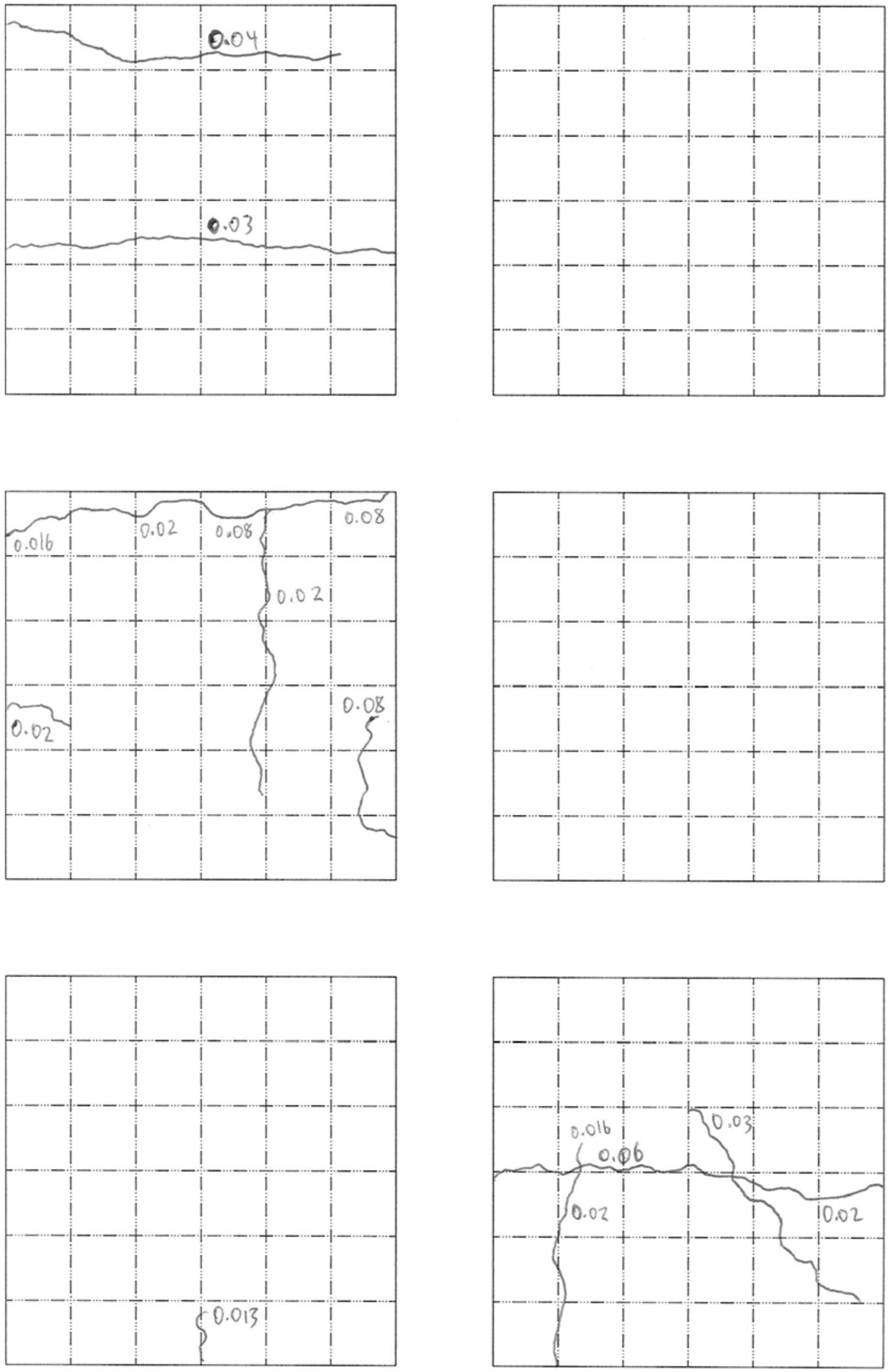

FIGURE 4.11 Distress survey for deck F-504.

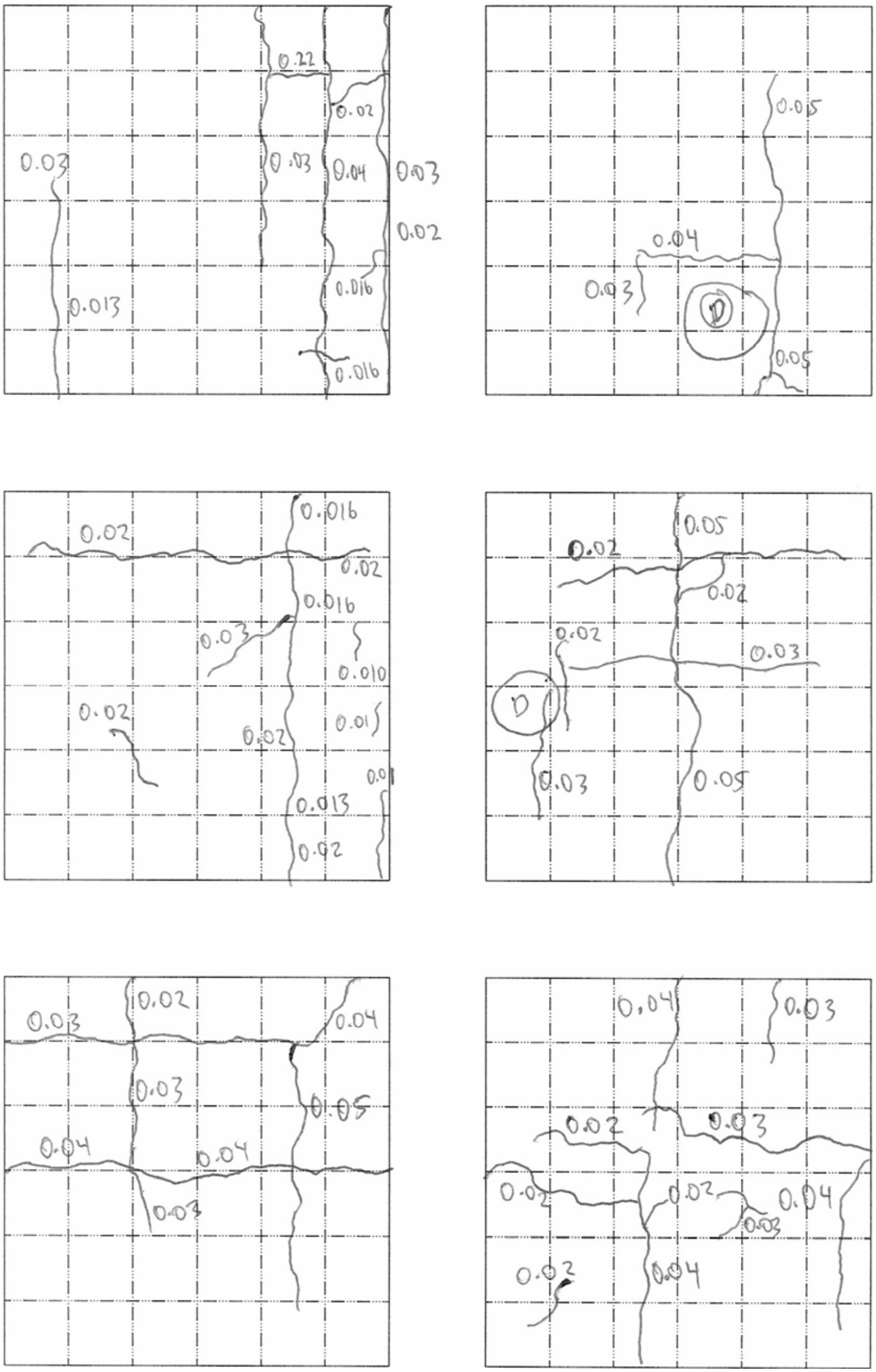

FIGURE 4.12 Distress survey for deck F-506.

TABLE 4.1 Crack Width Categories (*1*)

Category	Crack Width (in.)
Hairline	<0.004
Narrow	0.004 to 0.01
Medium	0.01 to 0.03
Wide	>0.03

TABLE 4.2 Crack Data

Deck ID	Crack Width (in.)		Crack Severity	Crack Density (ft/yd^2)
	Average	Std. Dev.		
C-438	0.034	0.021	Wide	1.07
C-460	0.013	0.006	Medium	1.85
C-688	0.017	0.009	Medium	2.01
C-698	0.017	0.006	Medium	1.99
C-699	0.016	0.009	Medium	1.83
C-726	0.029	0.016	Medium	5.25
C-736	-	-	-	-
C-752	-	-	-	-
C-759	-	-	-	-
C-760	0.019	0.009	Medium	3.69
C-844	0.036	0.011	Wide	2.47
C-919	0.019	0.012	Medium	5.37
F-500	0.029	0.019	Medium	1.92
F-504	0.033	0.021	Wide	1.66
F-506	0.029	0.012	Medium	4.72

TABLE 4.3 Crack Severity Categories

Crack Severity	Number of Decks	Percent of Decks (%)
Hairline	0	0.0
Narrow	0	0.0
Medium	9	60.0
Wide	3	20.0

TABLE 4.4 Pothole Data

Deck ID	No. of Potholes	Pothole Size (ft^2)		Pothole Density (ft^2/yd^2)
		Average	Std. Dev.	
C-438	0	-	-	0.000
C-460	0	-	-	0.000
C-688	0	-	-	0.000
C-698	0	-	-	0.000
C-699	0	-	-	0.000
C-726	1	0.6	-	0.025
C-736	0	-	-	0.000
C-752	0	-	-	0.000
C-759	0	-	-	0.000
C-760	0	-	-	0.000
C-844	0	-	-	0.000
C-919	0	-	-	0.000
F-500	0	-	-	0.000
F-504	0	-	-	0.000
F-506	0	-	-	0.000

4.2.2 Sounding Data

From the sounding data collected, the average delamination size was calculated in square feet, and delamination density in square feet per square yard of test area was calculated by dividing the total delamination area by the tested area. Sounding data are shown in Table 4.5. Again, hyphens indicate that measurements could not be obtained or were not applicable. As with visual distress, no universally accepted threshold values exist for delaminations. However, maintenance action is suggested if 5 to 10 percent of the deck is affected (*1, 21*). Compared to these thresholds, the data suggest that no maintenance is presently required for delaminations on any of the 15 decks.

TABLE 4.5 Sounding Data

Deck ID	No. of Delaminations	Delamination Size (ft^2)		Delamination Density (ft^2/yd^2)
		Average	Std. Dev.	
C-438	0	-	-	0.000
C-460	1	0.2	-	0.008
C-688	0	-	-	0.000
C-698	1	0.8	-	0.033
C-699	4	0.8	0.4	0.127
C-726	5	1.5	0.8	0.320
C-736	-	-	-	-
C-752	-	-	-	-
C-759	-	-	-	-
C-760	8	0.3	0.2	0.104
C-844	0	-	-	0.000
C-919	0	-	-	0.000
F-500	1	1.0	-	0.042
F-504	0	-	-	0.000
F-506	2	1.0	0.0	0.083

4.2.3 Schmidt Hammer Data

Relationships between Schmidt rebound number and concrete compressive strength published by the Schmidt hammer manufacturer were utilized to estimate concrete compressive strengths from the rebound numbers measured in the field; these data are given in Table 4.6. As stated previously, readings were not obtained for decks C-736, C-752, or C-759 because the polymer overlays on these decks prohibited evaluation of the bare concrete surfaces. Measurements were not obtained for decks C-438 or C-919 due to instrument failure. While these data reflect differences in concrete strengths, no general threshold values exist for determining the applicability of specific MR&R actions based on Schmidt hammer data. Nonetheless, the estimates all exceed the 28-day strength requirement of 3500 psi specified by UDOT for concrete bridge decks (*21*).

TABLE 4.6 Schmidt Hammer Data

Deck ID	Rebound Number		Compressive Strength (psi)
	Average	Std. Dev.	
C-438	-	-	-
C-460	38.8	0.9	4500
C-688	39.6	2.0	4900
C-698	41.2	0.9	5100
C-699	36.7	0.8	4300
C-726	36.6	4.3	4250
C-736	-	-	-
C-752	-	-	-
C-759	-	-	-
C-760	37.4	2.3	4375
C-844	33.8	2.2	4715
C-919	-	-	-
F-500	30.9	9.8	3550
F-504	32.8	5.0	3750
F-506	36.7	3.0	4300

4.2.4 Half-Cell Potential Data

Half-cell potential provides a reliable indicator of corrosion activity, although the rate of corrosion cannot be quantified (*21*). Based on the states of corrosion defined in Table 4.7 and the average half-cell potential measurements obtained in the research, each of the tested decks is rated in Table 4.8. Table 4.9 summarizes the corrosion classifications of the 15 decks.

TABLE 4.7 Half-Cell Potential Threshold Values for Corrosion

Half-Cell Potential (V)	State of Corrosion
<-0.35	Active
-0.35 to -0.20	Uncertain
>-0.20	Inactive

TABLE 4.8 Half-Cell Potential Data

Deck ID	Half-Cell Potential (V)		State of Corrosion
	Average	Std. Dev.	
C-438	-0.33	0.17	Uncertain
C-460	-0.39	0.10	Active
C-688	-0.34	0.07	Uncertain
C-698	-0.45	0.10	Active
C-699	-0.47	0.11	Active
C-726	-0.42	0.04	Active
C-736	-0.24	0.10	Uncertain
C-752	-0.31	0.09	Uncertain
C-759	-0.47	0.19	Active
C-760	-0.54	0.07	Active
C-844	-0.42	0.21	Active
C-919	-0.25	0.04	Uncertain
F-500	-0.27	0.19	Uncertain
F-504	-0.35	0.04	Uncertain
F-506	-0.33	0.09	Uncertain

TABLE 4.9 Distribution of Decks among Half-Cell Potential Ratings

State of Corrosion	Number of Decks	Percent of Decks (%)
Active	7	46.7
Uncertain	8	53.3
Inactive	0	0.0

4.2.5 Chloride Concentration Data

The chloride concentration in units of pounds of chloride per cubic yard of concrete was calculated in this research at the depth of the top of the reinforcement, which is the shallowest location at which corrosion may begin. The chloride concentration at this depth was calculated at each test area by linear interpolation between laboratory-determined chloride concentrations above and below this point as described previously. The accepted chloride concentration threshold for corrosion initiation of black bar is 2 lbs of chloride per cubic yard of concrete as specified by the Federal Highway Administration (*21*). Based on the assumption that this value is also a reasonable threshold for epoxy-coated rebar, the state of corrosion for each deck was classified based on the average chloride concentrations shown in Table 4.10. The validity of the assumption depends on the degree to which the epoxy-coated rebar utilized in construction of the decks evaluated in this research is deteriorated; previous research on similar decks in Utah indicates that the epoxy coatings are probably damaged in many areas, suggesting that the reinforcement is in fact exposed to chlorides (*1*). Table 4.11 summarizes the corrosion classifications of the 15 decks.

TABLE 4.10 Chloride Concentration Data

Deck ID	Cover (in.)		Chloride Concentration (lbs Cl^-/yd^3 Concrete)		State of Corrosion
	Average	Std. Dev.	Average	Std. Dev.	
C-438	2.3	0.49	0.8	0.4	Inactive
C-460	1.7	0.31	11.8	4.8	Active
C-688	2.9	0.43	6.3	4.1	Active
C-698	2.0	0.21	15.8	6.6	Active
C-699	2.3	0.52	13.0	8.1	Active
C-726	2.0	0.25	13.7	4.5	Active
C-736	3.0	0.18	1.2	1.8	Inactive
C-752	3.0	0.63	0.6	0.3	Inactive
C-759	3.0	0.42	6.0	1.4	Active
C-760	1.8	0.22	17.3	5.4	Active
C-844	2.9	0.88	0.4	0.6	Inactive
C-919	2.9	0.63	0.1	0.1	Inactive
F-500	2.5	0.13	0.4	0.4	Inactive
F-504	2.5	0.28	6.0	2.3	Active
F-506	2.7	0.36	2.3	1.4	Active

TABLE 4.11 Distribution of Decks among Chloride Concentration Ratings

State of Corrosion	Number of Decks	Percent of Decks (%)
Active	9	60.0
Inactive	6	40.0

4.3 STATISTICAL ANALYSES

Both correlation and regression analyses were performed on the data as described in the following sections.

4.3.1 Correlation Analysis

A full matrix correlation analysis was performed to investigate relationships between variables associated with deck damage and parameters potentially useful for predicting the occurrence of damage. As described previously, developing a model to predict damage before it is manifest on the deck would particularly facilitate preventive maintenance treatments. In Table 4.12, variables associated with deck damage are categorized as response variables, while variables representing deck properties are categorized as predictor variables. Every unique pair-wise combination of these parameters was analyzed to determine if correlations exist.

For each of the analyses, the null hypothesis was the postulation that no relationship exists between the variables, and the alternative hypothesis was the conjecture that a relationship does exist. A *p*-value representing the probability of observing a sample outcome more contradictory to the null hypothesis than the observed sample result, or, alternatively, the probability that the observed sample outcome occurred by chance, was computed for each comparison using a statistical software package. When the *p*-value was less than the value of 0.15 specified in this research as a tolerable level of error for the experimentation, the null hypothesis was rejected, leading to acceptance of the alternative hypothesis. However, when the *p*-value was greater than 0.15, the null hypothesis could not be rejected. Table 4.12 shows the *p*-values that resulted from the analyses. Values less than the threshold value of 0.15 are bolded to emphasize importance.

The results of the correlation analysis indicate that Schmidt rebound number is correlated to crack severity, age is correlated to delamination size, and half-cell potential is correlated to number of delaminations, while both cover and chloride concentration are correlated to number of delaminations, delamination size, and delamination density. The significant correlations between predictor variables and delamination distresses are consistent with physical mechanisms of deterioration explained previously. Decks with

TABLE 4.12 Correlation Matrix

Predictor Variables	Response Variables							
	Crack Severity	Crack Density	Number of Delaminations	Delamination Size	Delamination Density	Number of Potholes	Pothole Size	Pothole Density
	p -values							
Schmidt Rebound Number	**0.02**	0.88	0.77	0.70	0.70	0.96	0.96	0.96
Age	0.41	0.98	0.27	**0.06**	0.18	0.40	0.40	0.40
Cover	0.88	0.42	**0.01**	**0.09**	**0.08**	0.30	0.30	0.30
Half-Cell Potential	0.51	0.92	**0.01**	0.50	0.17	0.60	0.60	0.60
Chloride Concentration	0.85	0.44	**0.00**	**0.13**	**0.04**	0.24	0.24	0.24

high chloride concentrations, very negative half-cell potentials, inadequate cover, and advanced age usually exhibit damage resulting from corrosion of the reinforcing steel, and delaminations are the precursors in many cases to more severe distresses such as potholes.

Although chloride concentration is an ideal predictor variable from a theoretical perspective because it is the leading cause of bridge deck deterioration in cold climates such as Utah, measurement of chloride concentrations in practice is tedious and requires advanced skills. Therefore, the use of other variables that are more easily measured is desirable in development of a CI. In this research, the ability to utilize age, cover, and half-cell potential as surrogate variables in the place of chloride concentration was especially of interest. For that reason, regression analysis was performed to quantify the percentage of variation in chloride concentration that can be explained by variation in age, cover, and half-cell potential as described in the next section.

4.3.2 Regression Analysis

Linear regression was used to develop an equation relating chloride concentration to age, cover, and half-cell potential. The regression produced a coefficient of determination (R^2) of 0.874 associated with the following Equation 4.1:

$$C = 5.041 + 0.23A - 37.2H - 6.4D \tag{4.1}$$

where C = chloride concentration, lbs of chloride per yd^3 of concrete

A = deck age, yrs

H = half-cell potential, V

D = concrete cover thickness, in.

That is, 87.4 percent of the variation in chloride concentration can be explained by variation in age, cover, and half-cell potential. The measured and predicted chloride concentrations for each deck are shown together in Table 4.13.

TABLE 4.13 Predicted Chloride Concentrations

Deck ID	Age (yrs)	Cover (in.)	Half-Cell Potential (V)	Chloride Concentration (lb Cl^-/yd^3 Concrete)	
				Measured	Predicted
C-438	2	2.3	-0.33	0.8	2.9
C-460	17	1.7	-0.39	11.8	12.6
C-688	18	2.9	-0.34	6.3	3.4
C-698	18	2.0	-0.45	15.8	12.8
C-699	18	2.3	-0.47	13.0	12.1
C-726	21	2.0	-0.42	13.7	12.5
C-736	18	3.0	-0.24	1.2	0.0
C-752	17	3.0	-0.31	0.6	1.6
C-759	16	3.0	-0.47	6.0	6.9
C-760	16	1.8	-0.54	17.3	17.2
C-844	4	2.9	-0.42	0.4	2.8
C-919	2	2.9	-0.25	0.1	0.0
F-500	21	2.5	-0.27	0.4	4.1
F-504	21	2.5	-0.35	6.0	6.7
F-506	20	2.7	-0.33	2.3	4.5

In consideration of the satisfactory R^2 value associated with Equation 4.1 and the relative ease of measuring age, cover, and half-cell potential compared to chloride concentration, the latter three variables were selected for inclusion in the UBDI developed in this research. These variables effectively reflect chloride-induced corrosion mechanisms active on Utah bridge decks and are highly correlated to delamination distress.

4.4 DEVELOPMENT OF CONDITION INDEX

Determination of the UBDI equation was essentially a trial-and-error process. At the request of UDOT personnel, the equation was structured around a deduct system using a 100-point scale similar to the sufficiency rating system, in which a perfect bridge deck receives a score of 100. Coefficients were selected based largely on the judgment of the researchers and the UDOT personnel involved in the research, and threshold values for MR&R options were specified to be the same as those associated with the standard sufficiency ratings as shown in Table 4.14. The analysis ultimately produced Equation 4.2:

$$UBDI = 100 - 0.67A - 150(-0.35 - H) - 15(2.5 - D) \tag{4.2}$$

where $UBDI$ = Utah bridge deck index (between 0 and 100)

A = deck age, yrs

H = half-cell potential, V

If H > -0.35 V, then H = -0.35 V

D = concrete cover thickness, in.

If D > 2.5 in., then D = 2.5 in.

TABLE 4.14 UBDI Treatment Categories

UBDI	Recommendation
<50	Replacement
≥50 and <80	Rehabilitation
≥80	Preventive

According to UDOT personnel, a standard deck is said to have a service life of 35 years, or, in other words, a deck should transition into the replacement category within 35 years from the date of construction. Although some estimates suggest that the service life of a bridge deck ranges from 13 to 28 years (*34, 35*), 35 years was used in this research at the request of the UDOT engineers involved with this project. In Equation 4.2, the age coefficient of 0.67 was selected so that when a deck is 30 years old, 5 years less than the assumed average service life of 35 years, the deck will transition from a preventive category to a rehabilitation category even if no deductions are incurred from half-cell potential or cover. Regarding half-cell potential, the coefficient of 150 was selected in consultation with UDOT engineers to amplify the small half-cell potential values in relation to their relative importance in the UBDI. The threshold value of -0.35 V above which no deduct is required corresponds to the half-cell potential measurement indicating an active state of corrosion as specified in Table 4.7; with this threshold value, no deduct value is required for a deck having an average half-cell potential value more positive than -0.35 V. Regarding concrete cover thickness, the coefficient of 15 was selected in consultation with UDOT personnel, and a value of 2.5 in. was used as the threshold, as this is the specification established by UDOT for minimum concrete cover on bridge decks; only those decks with cover thicknesses less than 2.5 in. are penalized in the UBDI.

The UBDI and corresponding MR&R recommendation for each of the bridge decks tested in this research are given in Table 4.15 together with the values of each parameter utilized to compute the UBDI; although not necessary in the computations, the chloride concentrations, sufficiency ratings, and NBI ratings are also given for comparison. Table 4.16 summarizes the UBDI recommendations for the 15 decks included in the study.

Table 4.15 depicts interesting discrepancies between UDOT's current tools for deck assessment, which include the NBI and sufficiency ratings, and the new UBDI. Because an NBI rating reflects the condition of only the bridge deck and a sufficiency rating reflects the condition of the entire bridge, differences between these two parameters are expected. Notable differences between the relative classifications of decks using the NBI and the UBDI scales, however, are evident in the table. For

TABLE 4.15 Deck Treatment Recommendations

Deck ID	Age (yrs)	Cover (in.)	Half-Cell Potential (V)	Chloride Concentration (lb Cl^-/yd^3 Concrete)	Sufficiency Rating	NBI	UBDI	UBDI Recommendation
C-438	2	2.3	-0.33	0.9	75.9	7	96	Preventive
C-460	17	1.7	-0.39	12.0	89.0	6	71	Rehabilitation
C-688	18	2.9	-0.34	6.4	85.0	7	88	Preventive
C-698	18	2.0	-0.45	15.9	88.9	6	66	Rehabilitation
C-699	18	2.3	-0.47	13.1	87.8	6	67	Rehabilitation
C-726	21	2.0	-0.42	13.9	92.9	6	68	Rehabilitation
C-736	18	3.0	-0.24	1.3	95.4	7	88	Preventive
C-752	17	3.0	-0.31	0.7	96.0	6	89	Preventive
C-759	16	3.0	-0.47	6.2	96.0	7	71	Rehabilitation
C-760	16	1.8	-0.54	17.5	96.0	7	50	Rehabilitation
C-844	4	2.9	-0.42	0.5	81.9	7	87	Preventive
C-919	2	2.9	-0.25	0.2	99.6	8	99	Preventive
F-500	21	2.5	-0.27	0.5	85.6	7	86	Preventive
F-504	21	2.5	-0.35	6.1	86.4	7	86	Preventive
F-506	20	2.7	-0.33	2.4	84.4	7	87	Preventive

TABLE 4.16 Distribution of Decks among Treatment Recommendations

MR&R Category	Number of Decks	Percent of Decks (%)
Preventive	9	60.0
Rehabilitation	6	40.0
Replacement	0	0.0

example, the NBI ratings for decks C-438 and C-760 are both 7, which suggests a need for minor maintenance, but the UBDI ratings are 96 and 50, respectively, suggesting that the latter deck is actually approaching the replacement category. Indeed, according to Table 4.5, deck C-760 has more delaminations than any other deck tested in this study. In addition, while the NBI ratings for all of the decks are either 6, 7, or 8, the UBDI ratings range from 50 to 99, suggesting that the UBDI scale is more able to distinguish decks of differing conditions compared to the NBI rating system.

Because the UBDI decreases linearly with increasing age and because concrete cover thickness is assumed to remain essentially constant during the deck service life, the only variable that can potentially offer the UBDI an "s-shape" appearance like that illustrated in Figure 2.1 is half-cell potential. Therefore, for the sole purpose of demonstrating the model, the half-cell potential was assumed to be a cubic function based on age with the hypothetical values given in Table 4.17. The function produces an s-shaped curve and, as displayed in Figure 4.13, generates half-cell potential values similar to those measured in 2005. As UDOT personnel collect additional half-cell potential data over time, development of a more accurate trend will be possible; however, the proposed relationship between half-cell potential and deck age was satisfactory for the purposes of this research.

Based on the assumed half-cell potential data and a cover thickness of 2.0 in., the curve plotted in Figure 4.14 was produced from Equation 4.2. As shown in the figure, the UBDI falls from the preventive category into the rehabilitation category at a deck life of about 18 years in this example and further declines into the replacement category at a deck life of 35 years.

TABLE 4.17 Case 1 Data (No Applied Treatment)

Age (yrs)	Half-Cell Potential (V)	Cover (in.)	UBDI
5	-0.30	2.0	89
10	-0.31	2.0	86
15	-0.33	2.0	82
20	-0.36	2.0	77
25	-0.40	2.0	69
30	-0.44	2.0	59
35	-0.48	2.0	50
40	-0.52	2.0	41

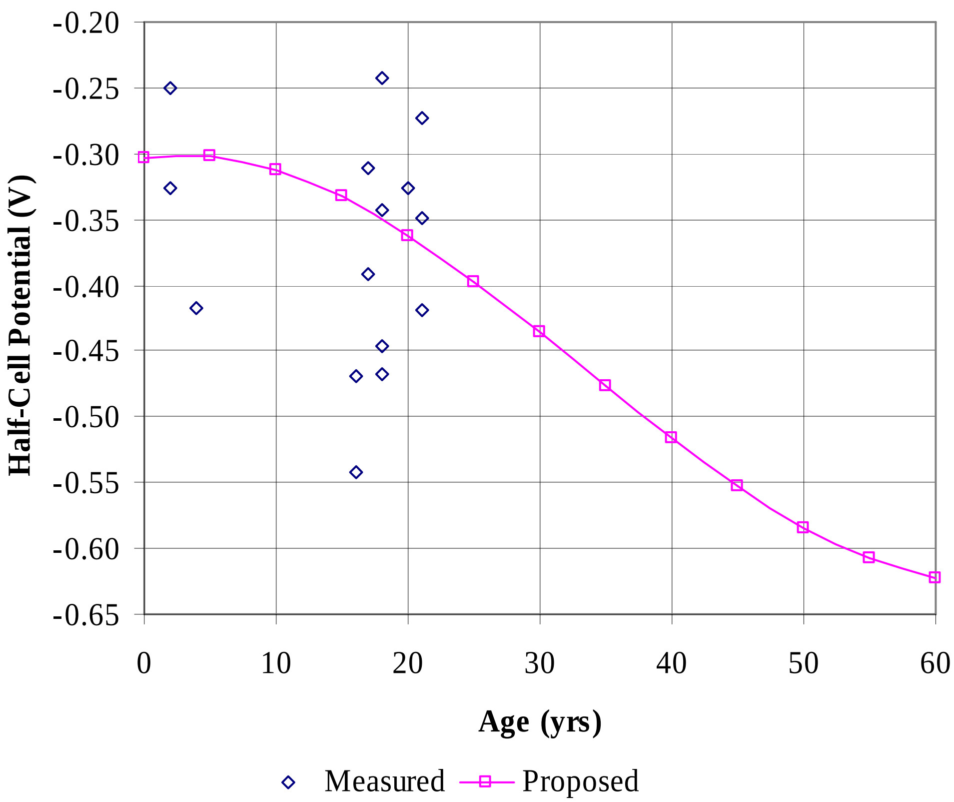

FIGURE 4.13 Relationships between half-cell potential and deck age.

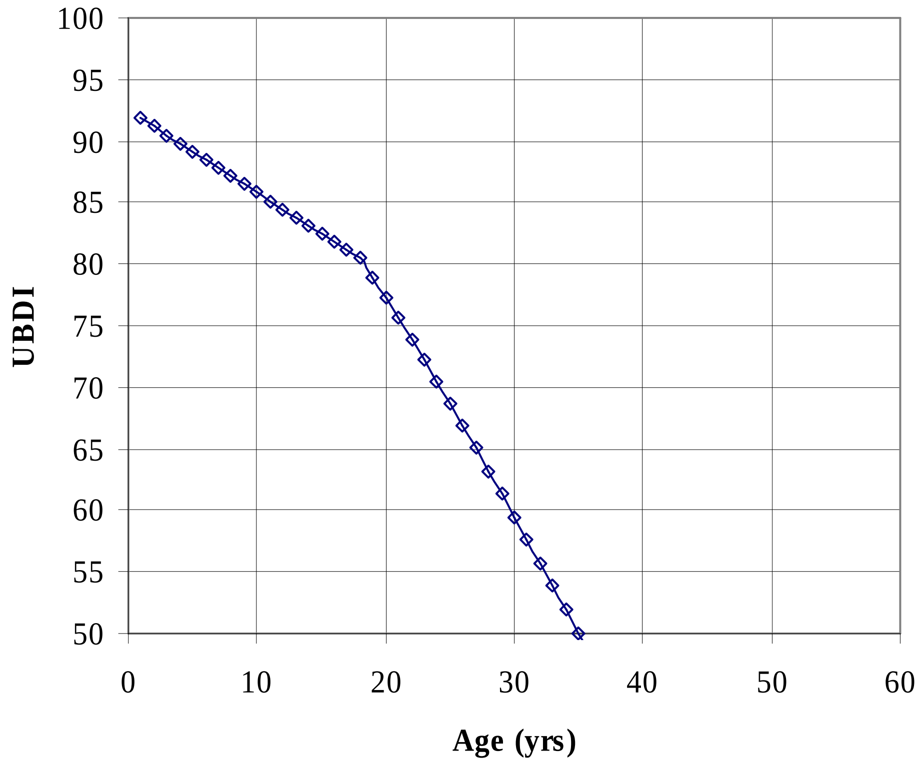

FIGURE 4.14 Deterioration curve for case 1.

4.5 TREATMENT EFFECTS

After Equation 4.2 was developed, the possibility of treatment applications was considered, leading to required adjustments in the UBDI calculation. The treatment options that were considered included an epoxy seal, a high-performance concrete (HPC) overlay, and an asphalt membrane overlay. The benefit associated with applying a particular treatment at a given deck age was determined in terms of deck life extension in consultation with UDOT bridge engineers and is listed for all three treatment types in Table 4.18. The adjusted UBDI formulation is given as Equation 4.3:

$$UBDI = 100 - 0.67(A - T) - 150(-0.35 - H) - 15(2.5 - D) \qquad (4.3)$$

where $UBDI$ = Utah bridge deck index (between 0 and 100)

A = deck age, yrs

T = cumulative treatment life as determined from Table 4.18, with A_T = deck age at time of treatment application, yrs

If $T > A$, then $T = A$

H = half-cell potential, V

If $H > -0.35$ V, then $H = -0.35$ V

D = concrete cover thickness, in.

If $D > 2.5$ in., then $D = 2.5$ in.

Because UDOT does not presently maintain a record of the timing or effects of treatments applied to bridge decks, fabrication of case-scenario data like that shown for case 1 in Table 4.17 was necessary to demonstrate the revised UBDI equation. Similar data were prepared for three additional cases. Case 2 involves an epoxy surface treatment applied at 5 years; case 3 involves an epoxy surface treatment applied at 5 years and an HPC overlay applied at 25 years; and case 4 involves an epoxy surface treatment applied at 5 years, an HPC overlay applied at 25 years, and an asphalt membrane overlay applied at 40 years.

TABLE 4.18 Proposed Treatment Life

Treatment Type	A_T (yrs)	T (yrs)
Epoxy Seal	0	10
	5	8
	10	6
	15	4
	20	2
HPC Overlay	15	20
	20	16
	25	12
	30	8
	35	4
Asphalt Membrane Overlay	15	7
	20	6
	25	5
	30	4
	35	3

The data corresponding to cases 2, 3, and 4 are shown in Tables 4.19, 4.20, and 4.21, respectively, and are graphically represented in Figure 4.15. In all cases, the half-cell potential values were held constant during the service life of the applied treatment. In theory, even though the deck surface is effectively closed to further chloride ingress due to the presence of an intact surface treatment, increases in the actual half-cell potential values could occur during the treatment life due to the gradual

TABLE 4.19 Case 2 Data

Age (yrs)	Half-Cell Potential (V)	Cover (in.)	T (yrs)	UBDI
5	-0.30	2.0	0	89
10	-0.30	2.0	8	91
15	-0.31	2.0	8	88
20	-0.32	2.0	8	84
25	-0.35	2.0	8	81
30	-0.38	2.0	8	73
35	-0.42	2.0	8	64
40	-0.46	2.0	8	55
45	-0.50	2.0	8	45
50	-0.54	2.0	8	36

TABLE 4.20 Case 3 Data

Age (yrs)	Half-Cell Potential (V)	Cover (in.)	T (yrs)	UBDI
5	-0.30	2.0	0	89
10	-0.30	2.0	8	91
15	-0.31	2.0	8	88
20	-0.32	2.0	8	84
25	-0.35	2.0	8	81
30	-0.35	2.0	20	86
35	-0.35	2.0	20	82
40	-0.38	2.0	20	75
45	-0.41	2.0	20	66
50	-0.45	2.0	20	57
55	-0.49	2.0	20	48
60	-0.53	2.0	20	39

TABLE 4.21 Case 4 Data

Age (yrs)	Half-Cell Potential (V)	Cover (in.)	T (yrs)	UBDI
5	-0.30	2.0	0	89
10	-0.30	2.0	8	91
15	-0.31	2.0	8	88
20	-0.32	2.0	8	84
25	-0.35	2.0	8	81
30	-0.35	2.0	20	86
35	-0.35	2.0	20	82
40	-0.38	2.0	20	75
45	-0.40	2.0	23	71
50	-0.44	2.0	23	61
55	-0.48	2.0	23	52
60	-0.52	2.0	23	43
65	-0.55	2.0	23	34

diffusion of near-surface chlorides already in the concrete to locations nearer the reinforcing steel. However, for simplification purposes in these demonstrations, this effect was considered to be negligible. The application of the treatment in case 2 increased the service life of the deck from 35 to 43 years. In cases 3 and 4, the service life increased to 54 and 56 years, respectively.

Four distinct improvements in UBDI are evident in Figure 4.15. One improvement occurs on the case 2 curve, two occur on the case 3 curve, and three occur on the case 4 curve. These improvements correspond to the applications of specific treatments. The two improvements that occur at 8 and 25 years correspond to the applications of an epoxy surface treatment and an HPC overlay, respectively, while the improvement located at 40 years corresponds to the application of the asphalt membrane overlay. The noticeable slope changes at 18, 26, 36, and 36 years for cases 1, 2, 3, and 4, respectively, occur when the half-cell potential values become more negative than the threshold value of -0.35 V and thus begin to generate non-zero deduct values with time.

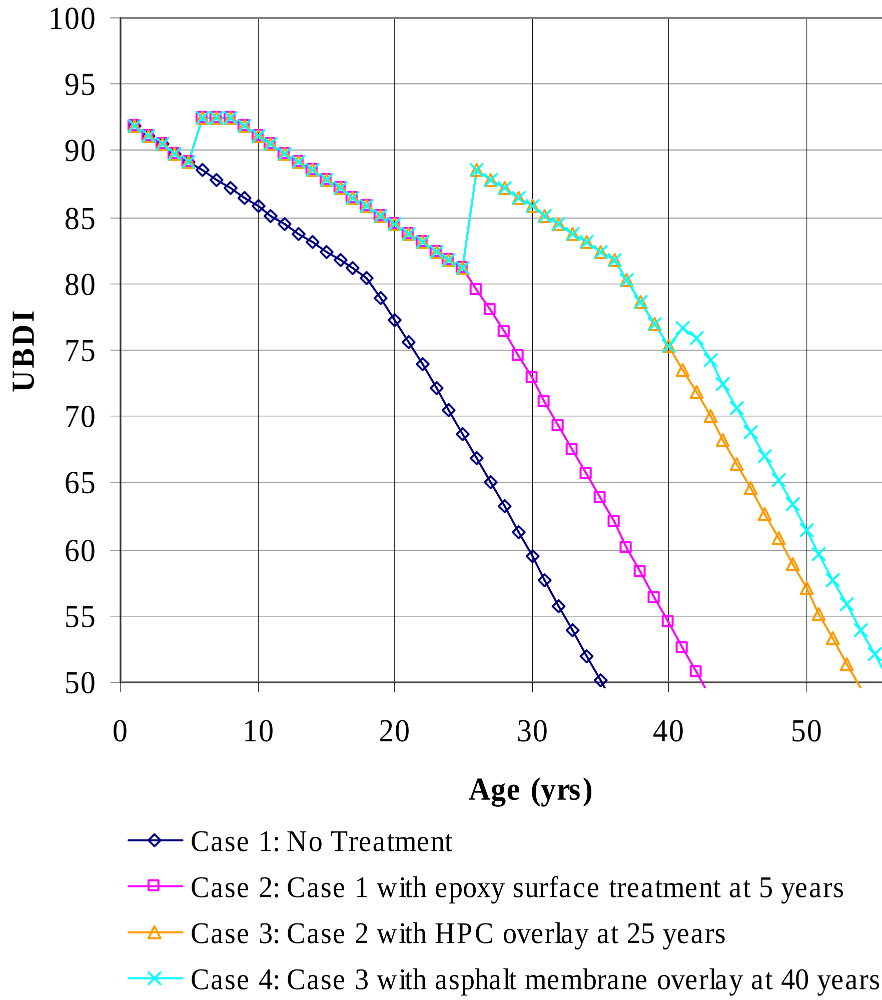

FIGURE 4.15 Deterioration curves for cases 1 to 4.

4.6 SAMPLING GUIDELINES

UBDI recommendations will be most meaningful when adequate testing has been conducted to properly characterize bridge decks of interest. Therefore, Equation 4.4 was utilized to determine the number of required measurements per deck for both half-cell potential and concrete cover (*36, 37*):

$$n = \left(\frac{Z \cdot s}{\Delta x} \right)^2 \tag{4.4}$$

where n = number of replicate measurements

Z = two-tailed probability statistic from the standard normal distribution

s = standard deviation

Δx = specified tolerance

Both the reliability and tolerance associated with Equation 4.4 may be specified by the user; the *Z*-values corresponding to specific reliability levels are given in Table 4.22. Typical standard deviations for half-cell potential and concrete cover measurements were calculated based on field data collected in this research. Standard deviations associated with the set of six average half-cell potential values and concrete cover thicknesses were computed for each deck. The average standard deviations for all 15 decks were then determined for both properties and yielded values of 0.11 V and 0.39 in. for half-cell potential and concrete cover, respectively.

As aids for UDOT personnel implementing this research, Figures 4.16 and 4.17 were created to facilitate rapid determination of the required number of half-cell potential and concrete cover measurements, respectively, that should be randomly collected from a given deck. In the figures, the values of both reliability and tolerance were varied within practical ranges. For example, if a reliability of 95 percent and tolerance values of 0.04 V and 0.20 in. for half-cell potential and concrete cover, respectively, were specified, 29 half-cell potential and 15 concrete cover measurements would be required. The engineer could then compute the means and standard deviations associated with the sample measurements and determine the actual tolerances associated with a given level of reliability by using Equation 4.4. At 95 percent reliability, the average of the sample readings in each case would be within the specified tolerance of the true mean 95 percent of the time.

TABLE 4.22 *Z*-Values (*36*, *37*)

Reliability (%)	*Z* -Value
99	2.58
95	1.96
90	1.65
85	1.44
80	1.28
75	1.15

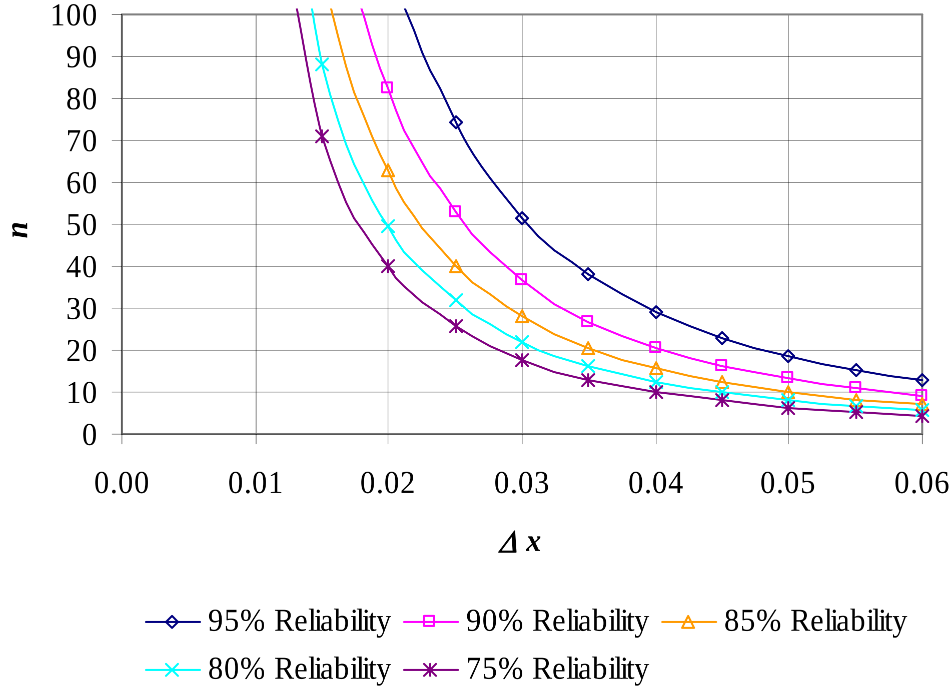

FIGURE 4.16 Sampling guidelines for half-cell potential measurements.

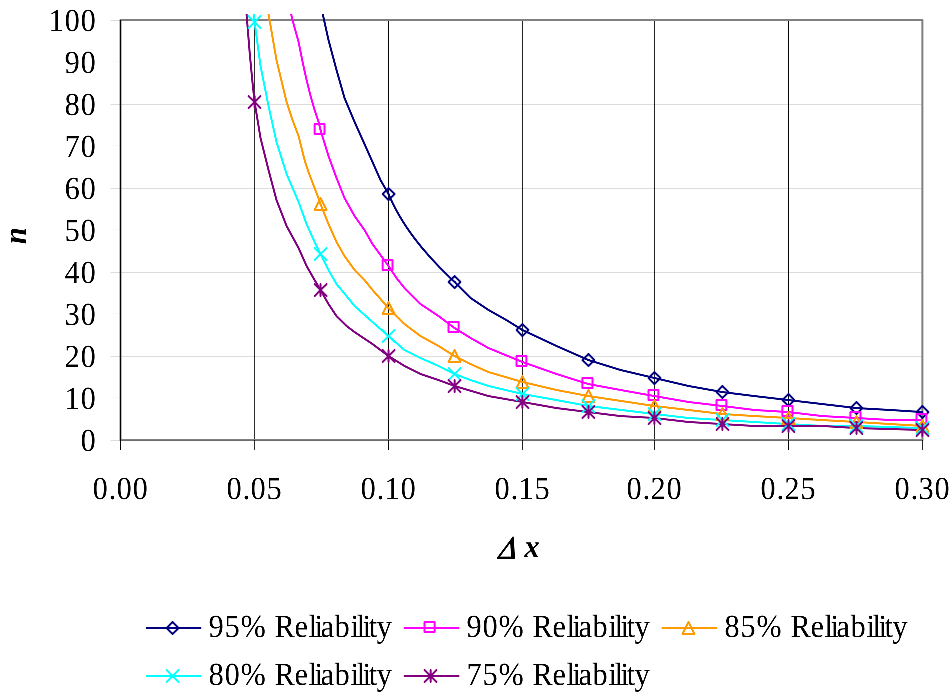

FIGURE 4.17 Sampling guidelines for concrete cover measurements.

4.7 SUMMARY

The process of developing the UBDI involved data calculations, statistical analyses, development of the new CI, consideration of treatment effects, and sampling guidelines. The statistical analyses were performed on field data collected from 15 decks in the vicinity of Salt Lake City, Utah. Based on the selected variables of age, cover, and half-cell potential, a trial-and-error process yielded a new bridge deck management index. Treatment effects were added to the model, and four case scenarios were developed to demonstrate the use of the revised equation. Finally, sampling guidelines for half-cell potential and concrete cover measurements were developed using further statistical techniques.

CHAPTER 5
CONCLUSION

5.1 SUMMARY

UDOT owns and operates 1,700 bridges across the state of Utah. The 2004 NBI report stated that 86.7 percent of these bridges were in need of some form of maintenance, the cost of which was estimated to be $1.4 billion (*1*). Although several reasons exist for which a bridge can be labeled structurally deficient, one of the most common is a structurally inadequate deck (*2*). Given the need to maximize bridge deck service life amid increasing financial constraints, as well as the subjectivity of existing NBI ratings, UDOT bridge engineers needed a new index for concrete bridge deck management that was useful, in particular, for determining the timing of preventive maintenance treatments.

To this end, data were collected in the summer of 2005 from 15 concrete bridge decks in the vicinity of Salt Lake City. The decks ranged in age from 2 to 21 years in age and were all constructed using epoxy-coated rebar. Visual inspection, sounding, Schmidt hammer testing, half-cell potential testing, and chloride concentration testing were performed on six 6-ft by 6-ft test areas randomly distributed on each deck within the single lane closed to traffic, and testing protocols followed ASTM standards to the extent possible.

Data collected from visual inspection, sounding, Schmidt hammer testing, half-cell potential testing, and chloride concentration testing were analyzed to develop the UBDI. Visual inspection yielded information on crack width, crack severity, crack density, number of potholes, average pothole size, and pothole density. Sounding produced data regarding number of delaminations, average delamination size, and delamination density. Schmidt hammer values were utilized to estimate the average compressive strength of each concrete deck. Half-cell potential values were used to determine the average state of corrosion of each deck, and chloride concentration data facilitated calculation of the average chloride concentration at the depth of the reinforcement on each deck.

5.2 FINDINGS

A full matrix correlation analysis was performed to investigate relationships between variables associated with deck damage and parameters potentially useful for predicting the occurrence of damage. The results of the correlation analysis indicate that Schmidt rebound number is correlated to crack severity, age is correlated to delamination size, and half-cell potential is correlated to number of delaminations, while both cover and chloride concentration are correlated to number of delaminations, delamination size, and delamination density.

In this research, the ability to utilize age, cover, and half-cell potential as surrogate variables in the place of chloride concentration was especially of interest. A regression analysis was therefore performed to quantify the percentage of variation in chloride concentration that can be explained by variation in age, cover, and half-cell potential. In consideration of the satisfactory R^2 value of 0.874 associated with the regression and the relative ease of measuring age, cover, and half-cell potential compared to chloride concentration, these three variables were selected for inclusion in the UBDI developed in this research. These variables effectively reflect chloride-induced corrosion mechanisms active on Utah bridge decks and are highly correlated to delamination distresses.

Determination of the UBDI equation was primarily a trial-and-error process. At the request of UDOT personnel, the equation was structured around a deduct system using a 100-point scale similar to the sufficiency rating system, in which a perfect bridge deck receives a score of 100. Coefficients were selected based largely on the judgment of the researchers and the UDOT personnel involved in the research, and threshold values for MR&R options were specified to be the same as those associated with the standard sufficiency ratings. The UBDI and corresponding MR&R recommendation were then provided for each of the bridge decks tested in this research; nine of the decks are recommended for preventive treatment, and six are recommended for rehabilitation.

In addition, the possibility of treatment applications was considered, leading to required adjustments in the UBDI calculation. The treatment options that were considered include an epoxy seal, an HPC overlay, and an asphalt membrane overlay. Because UDOT does not presently maintain a record of the timing or effects of treatments applied to bridge decks, case-scenario data were fabricated to demonstrate the response of the revised UBDI equation to MR&R treatments.

Finally, as aids for UDOT personnel implementing this research, charts were created to facilitate rapid determination of the required number of half-cell potential and concrete cover measurements for different levels of reliability and tolerance. UBDI recommendations will be most meaningful when adequate testing has been conducted to properly characterize bridge decks of interest.

5.3 RECOMMENDATIONS

The UBDI developed in this research is recommended for implementation by UDOT personnel as a tool for optimizing the timing of MR&R treatments on concrete bridge decks similar to those evaluated in this project. In measuring cover and half-cell potential values, UDOT personnel should utilize the sampling guidelines presented in this report to ensure adequate characterization of each deck. Furthermore, to facilitate the inclusion of treatment effects in the UBDI, UDOT personnel should establish a policy of recording the types and dates of all MR&R treatments applied to bridge decks. As performance data are collected for specific treatments over time, the treatment lives proposed in this research for epoxy seals, HPC overlays, and asphalt membrane overlays should be revised as needed, and information for other treatments may be added. In addition, to maximize the predictive capabilities of the UBDI, more accurate relationships between half-cell potential values and deck age should be developed for estimating future deck condition.

REFERENCES

1. Tuttle, R. S. *Condition Analysis of Concrete Bridge Decks in Utah*. M.S. thesis. Department of Civil and Environmental Engineering, Brigham Young University, Provo, UT, August 2005.

2. Case, S., and J. A. Laman. Dynamics and Field Testing of Bridges. Technical Committee A2C05 on Dynamics and Field Testing of Bridges, TRB, National Research Council, Washington, D.C. http://onlinepubs.trb.org/onlinepubs/millennium/00029.pdf. Accessed May 23, 2006.

3. Phares, B. M., B. A. Graybeal, D. D. Rolander, M. E. Moore, and G. A. Washer. Reliability and Accuracy of Routine Inspection of Highway Bridges. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1749*, TRB, National Research Council, Washington, D.C., 2001, pp. 82-92.

4. Mauch, M., and S. Madanat. Seimiparametric Hazard Rate Models of Reinforced Concrete Bridge Deck Deterioration. *Journal of Infrastructure Systems*, Vol. 7, No. 2, June 2001, pp. 49-57.

5. Carter, P. D. Preventive Maintenance of Concrete Bridge Decks. *Concrete International*, Vol. 11, No. 11, November 1989, pp. 33-36.

6. Morcous, G., H. Rivard, and A. M. Hanna. Modeling Bridge Deterioration Using Case-Based Reasoning. *Journal of Infrastructure Systems*, Vol. 8, No. 3, September 2002, pp. 86-95.

7. Cady, P. D. Bridge Deck Rehabilitation Decision Making. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1035*, TRB, National Research Council, Washington, D.C., 1985, pp. 13-19.

8. Fitch, M. G., R. E. Weyers, and S. T. Johnson. Determination of End of Functional Service Life for Concrete Bridge Decks. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1490*, TRB, National Research Council, Washington, D.C., 1995, pp. 60-66.

9. Scott, M., A. Rezaizadeh, A. Delahaza, C. G. Santos, M. Moore, B. Graybeal, and G. Washer. A Comparison of Nondestructive Evaluation Methods for Bridge

Deck Assessment. *NDT&E International*, Vol. 36, No. 4, June 2003, pp. 245-255.

10. Elsener, B. Half-Cell Potential Mapping to Assess Repair Work on RC Structures. *Construction and Building Materials,* Vol. 15, No. 2-3, March 2001, pp. 133-139.

11. Melhem, H. G., and Y. Cheng. Prediction of Remaining Service Life of Bridge Decks Using Machine Learning. *Journal of Computing in Civil Engineering*, Vol. 17, No. 1, January 2003, pp. 1-9.

12. Stewart, M. G., and D. V. Rosowsky. Time-Dependent Reliability of Deteriorating Reinforced Concrete Bridge Decks. *Structural Safety*, Vol. 20, No. 1, 1998, pp. 91-109.

13. Lounis, Z. Probabilistic Modeling of Chloride Contamination and Corrosion of Concrete Bridge Structures. In *Proceedings of the North American Fuzzy Information Processing Society Fourth International Symposium on Uncertainty Modeling and Analysis*, College Park, MD, September 21-24, 2003, pp. 447-451.

14. Mishalani, R. G., and S. M. Madanat. Computation of Infrastructure Transition Probabilities Using Stochastic Duration Models. *Journal of Infrastructure Systems*, Vol. 8, No. 4, December 2002, pp. 139-148.

15. Karlaftis, M. G., S. M. Madanat, and P. S. McCarthy. Development of Discrete Models of Infrastructure Deterioration with Panel Data. In *Infrastructure Condition Assessment: Art, Science, and Practice, Proceedings of the American Society of Civil Engineers Conference*, Boston, MA, August 25-27, 1997, pp. 21-29.

16. *AASHTO Maintenance Manual: The Maintenance and Management of Roadways and Bridges.* American Association of State Highway and Transportation Officials, Washington, D.C., 1999.

17. Hearn, G., D. M. Frangopol, T. Szanyi, and S. Marshall. Data and Data Interpretation in Bridge Management Systems. In *Proceedings of the American Society of Civil Engineers Structures Congress XIV: Building an International Community of Structural Engineers*, Chicago, IL, April 15-18, 1996, pp. 245-252.

18. Lenett, M. S., A. Griessmann, A. J. Helmicki, and A. E. Aktan. Subjective and Objective Evaluations of Bridge Damage. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1688*, TRB, National Research Council, Washington, D.C., 1999, pp.76-86.

19. Xia, P., and J. M. Brownjohn. Bridge Structural Condition Assessment Using Systematically Validated Finite-Element Model. *Journal of Bridge Engineering*, Vol. 9, No. 5, September/October 2004, pp. 418-423.

20. Enright, M. P., and D. M. Frangopol. Survey and Evaluation of Damaged Concrete Bridges. *Journal of Bridge Engineering*, Vol. 5, No. 1, February 2000, pp. 31-38.

21. Hema, J., W. S. Guthrie, and F. Fonseca. *Concrete Bridge Deck Condition Assessment and Improvement Strategies*. Report UT-04.16. Department of Civil and Environmental Engineering, Brigham Young University, Provo, UT, 2005.

22. Madanat, S., and D. Lin. Bridge Inspection Decision Making Based on Sequential Hypothesis Testing Methods. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1697*, TRB, National Research Council, Washington, D.C., 2000, pp. 14-18.

23. Mishalani, R. G., and S. M. Madanat. Infrastructure State Transition Probability Computation Using Duration Models. In *Proceedings of the American Society of Civil Engineers International Conference on Applications of Advanced Technologies in Transportation Engineering*, Newport Beach, CA, April 26-29, 1998, pp. 505-512.

24. Huang, Y., T. M. Adams, and J. A. Pincheira. Analysis of Life-Cycle Maintenance Strategies for Concrete Bridge Decks. *Journal of Bridge Engineering*, Vol. 9, No. 3, May/June 2004, pp. 250-258.

25. Frost, S. L. *Effect of Stay-in-Place Metal Forms on Performance of Concrete Bridge Decks*. M.S. thesis. Department of Civil and Environmental Engineering, Brigham Young University, Provo, UT, August 2006.

26. Mapquest. http://www.mapquest.com. Accessed April 2, 2006.

27. Fanous, F., H. Wu, and J. Pape. *Impact of Deck Cracking on Durability*. Report TR-405. Center for Transportation Research Education, Iowa State University, Ames, IA, 2000.

28. Henderson, M. E., G. N. Dion, and R. D. Costley. Acoustic Inspection of Concrete Bridge Decks. In *Proceedings of the International Society for Optical Engineering Conference on Nondestructive Evaluation of Bridges and Highways III*, Newport Beach, CA, March 1999, pp. 219-227.

29. Mindess, S., J. F. Young, and D. Darwin. *Concrete*, Second Edition. Pearson Education, Inc., Upper Saddle River, NJ, 2003.

30. Manning, D. G. *Detecting Defects and Deterioration in Highway Structures*. National Cooperative Highway Research Program Synthesis of Highway Practice 118. NCHRP, TRB, National Research Council, Washington, D.C., 1985.

31. Moore, W. Detection of Bridge Deck Deterioration. In *Highway Research Record 451*, HRB, National Research Council, Washington, D.C., 1975, pp. 53-61.

32. Stratfull, R. F., W. J. Jurkovich, and D. L. Spellman. Corrosion Testing of Bridge Decks. In *Transportation Research Record: Journal of the Transportation Research Board, No. 539*, TRB, National Research Council, Washington, D.C., 1975, pp. 50-59.

33. Peterman, R. J., J. A. Ramirez, and R. W. Poston. Durability Assessment of Bridges with Full-Span Prestressed Concrete Form Panels. In *ACI Materials Journal*, Vol. 96, No. 1, January/February 1999, pp. 11-19.

34. Tikalsky, P. J. Long-Term Durability of Concrete in Highway Bridges, and Practical Approaches to Durability-Based Design. In *Proceedings of the American Society of Civil Engineers Structures Congress 2004: Building on the Past, Securing the Future*. CD-ROM. Nashville, TN, May 2004.

35. Weyers, R. E., and I. L. Al-Qadi. Corrosion Model for Concrete Structures. In *Infrastructure Planning and Management: Proceedings of Two Parallel Conferences*, American Society of Civil Engineers, Denver, CO, June 21-23, 1993, pp. 92-96.

36. Guthrie, W. S., Ellis, P., and T. Scullion. Repeatability and Reliability of the Tube Suction Test. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1772*, TRB, National Research Council, Washington, D.C., 2001, pp. 151-157.

37. Ott, R. L., and M. Longnecker. *An Introduction to Statistical Methods and Data Analysis*, Fifth Edition. Duxbury, Pacific Grove, CA, 2001.