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16. Abstract

The practice of conducting a half-cell potential survey during the assessment of the condition of a concrete deck was reexamined with the objective of eliminating some of the doubts concerning its benefits. It was found that the survey grid size of 4.0 ft recommended in ASTM C 876 had partly contributed to the uncertainty because this spacing is too large to allow complete detection of all the rebar corrosion and the associated concrete delamination that may exist in a deck. It was shown that the effect of rebar corrosion on the potential of the concrete was relatively localized; thus, a grid size of no more than 2.0 ft should be used.

Another contributing factor was that the half-cell potential at any location on a concrete deck can fluctuate normally with temperature and other factors. Consequently, in contrast to the interpretation guidelines suggested in ASTM C 876, the numerical value of the potential measured at each location (by itself) would be an imprecise indicator of the condition of the rebar and/or concrete at the location. Instead, the actively corroding rebars are manifested on the concrete surface in large potential gradients that can be readily located in iso-potential contour maps made from the survey results.

It was found that the 3LP device provides a simple but time-consuming means for measuring the corrosion rates of rebars, which correlate reasonably well with the metal losses observed in the rebars. However, since corrosion rate also fluctuates with location on a concrete deck and time, the benefit of such a survey would not be fully realized until an appropriate method for determining the representative rebar corrosion rate for a deck and for relating such to remaining service life has been developed.

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FINAL REPORT

BENEFITS OF MEASURING HALF-CELL POTENTIALS AND REBAR CORROSION RATES IN CONDITION SURVEYS OF CONCRETE BRIDGE DECKS

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Principal Research Scientist

(The opinions, findings, and conclusions expressed in this report are those of the author and not necessarily those of the sponsoring agencies.)

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ABSTRACT

The benefit of using half-cell potential measurements in condition surveys of concrete bridge decks was examined using data collected from several decks by visual inspection, sounding with chain drags, and measurement of half-cell potentials. As suspected, half-cell potentials on a deck were observed to fluctuate from survey to survey, mostly in response to seasonal fluctuations of temperature and moisture in the concrete. Consequently, in contrast to the interpretation guidelines recommended in ASTM C 876, the numerical value of each measured half-cell potential by itself is a poor indicator of the probability of corrosion occurring on a rebar, and even less of an indicator of the condition of the surrounding concrete. Instead, the potential observed at each location should be considered in relative terms, i.e., in relation to potentials observed at the surrounding concrete. Therefore, when all the potentials observed were plotted on an iso-potential contour map, the locations of active rebar corrosion and delaminated concrete were found to be highly associated with high negative potential gradients.

In addition, due to the localized nature of rebar corrosion, the grid spacing of 4.0 ft recommended in ASTM C 876 for use in surveys of bridge decks (and the 5.0-ft spacing used by many state transportation agencies) was found to be too large for locating existing active corrosion and the associated damage to concrete. It was determined that a grid spacing of no more than 2.0 ft should be used.

When half-cell potential surveys are performed in a sufficiently small grid spacing and the collected half-cell potentials are plotted on contour maps of iso-potential lines, the locations of all existing active rebar corrosion and damaged concrete are indicated with a high degree of accuracy by relatively high potential gradients. When combined with the other inspection techniques, such a survey would, therefore, be extremely useful for estimation of repair quantity.

The benefit derived from being able to measure the corrosion rates of rebars in an existing concrete bridge deck, as part of condition surveys, was also examined. The 3LP device appeared to be a convenient tool for measuring corrosion rates. It yielded rebar corrosion rates that correlated reasonably well with metal losses observed in rebars specimens extracted from the decks. Further, it appeared that metal losses of 3 to 6 percent by weight of the rebars may be the threshold metal loss necessary to initiate delamination in reinforced concrete decks.

Rebar corrosion rates also vary with location in a deck and with temporal fluctuating conditions in the concrete. Therefore, until appropriate methods have been developed for determining the representative rebar corrosion rate for a deck (that accounts for these variations) and relating it to future concrete damage or remaining service life, the benefit of measuring rebar corrosion rates in condition surveys of concrete bridge decks will not be fully realized.

FINAL REPORT

BENEFITS OF MEASURING HALF-CELL POTENTIALS AND REBAR CORROSION RATES IN CONDITION SURVEYS OF CONCRETE BRIDGE DECKS

Gerardo G. Clemeña
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INTRODUCTION

Since concrete deterioration resulting from corrosion of embedded rebars is the primary cause of premature deterioration of concrete bridge decks in many states (including Virginia), relevant deck conditions (such as cracking, spalling, delamination, chloride content in the concrete, condition of the rebars, and thickness of the concrete cover over the top-mat rebars) are given considerable attention in condition surveys of bridge decks. Therefore, in addition to visual inspection, other inspection methods (such as sounding by chains for delamination, use of a pachometer, measurement of half-cell potentials, and chemical analysis of extracted concrete samples to determine chloride content) are routinely used in detailed condition surveys.

Since these inspection methods require many hours to perform, the adequacy and relevancy of each method must be reexamined whenever warranted. There are different concerns associated with the adequacy and relevancy of each of these inspection methods. Consequently, with the exception of half-cell potential measurements, various efforts have been made either to replace or to improve an existing method.

Visual inspection is not quantitative, is tedious, is time-consuming, and is disruptive to traffic. Research efforts that may lead to its partial replacement have included the use of a light-dependent resistor in measuring the width of cracks in concrete surfaces[1] and the development of image processing algorithms for analysis of the imagery of cracks in concrete surfaces.[2]

Users have similar concerns with sounding to detect concrete delaminations. To replace it, infrared thermography,[3,4] short-pulse radar,[5,6] and impact-echo[7] techniques have been studied.

The standard method for chloride analysis described in AASHTO T 260 is based on a potentiometric titration procedure developed by Berman[8] and improved by others.[9] The method is destructive and expensive because the procedures involved are lengthy and require powdered samples from a deck. To reduce its cost by simplifying the analysis, "short-cut" procedures based on the same potentiometric approach have been attempted.[10,11] Unfortunately, because of adverse solution ma-

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trix effects on the potentiometric readings of each sample solution, all of these procedures require reliance on a proper calibration curve. In addition, these procedures still require powderized samples from a deck.

Half-Cell Potential Measurements

Half-cell potential measurements are performed because the coexistence of corroding areas (or anodic half-cells) and noncorroding areas (or cathodic half-cells) on rebars is reflected in potential differences, or voltages, across the steel-concrete interfaces. These potentials can be measured relative to a constant reference potential provided by a suitable reference electrode. Accordingly, any change in the potential between the reference electrode and the steel-concrete interface can be attributed to, among other things, the corrosion activity at the surface of the steel.

The standard method for measuring half-cell potentials in concrete, ASTM C 876, is based on a procedure developed by Stratful.[12] In this method, the half-cell potential of the rebar in a concrete is measured by a high-impedance voltmeter connected between the rebar network and a $Cu/CuSO_4$ reference electrode that is in contact with the surface of the concrete. To survey an entire structure, this measurement is repeated by moving the electrode to other locations, or points, following a grid pattern laid on the surface of a concrete structure. The standard method noted that a 4.0-ft grid spacing had been found to be generally satisfactory for bridge decks and that larger spacings increase the probability that localized corrosion areas will not be detected.

Based on data by Stratful et al.[13] obtained from surveying several bridge decks and those from a laboratory study by Clear and Hay,[14] ASTM C 876 suggests using the following guidelines in the interpretation of data:

1. If potentials over an area are greater than –0.20 volt, there is a probability greater than 90 percent that no steel corrosion is occurring in the area at the time of measurement.

2. If potentials are in the range of –0.20 to –0.35 volt, corrosion activity of the steel in the area is uncertain.

3. If potentials are less than –0.35 volt, there is a probability greater than 90 percent that steel corrosion is occurring in the area at the time of measurement.

These guidelines indicate that there is significant uncertainty concerning the relationship between half-cell potentials and the condition of rebars. This uncertainty has created doubts among bridge engineers on the benefits of using half-cell potential measurements in condition surveys of bridge decks to prioritize deck repairs and prepare repair plans, where being able to delineate and quantify the amount of needed repair (instead of assessing the probability of rebar corrosion occurring in each deck) is desirable.

This investigator believes that the uncertainty arose from the constant fluctuation of potential at any location with time, which has been reported recently,[15] in response to the changing dynamics of the corrosion processes with fluctuations in the physical properties of concrete.

Rebar Corrosion Rate Measurements

When conducted by skilled inspectors, visual inspection combined with sounding reveals reasonably good information on the extent of the concrete deterioration already existing in a deck. The information provided by the other cited methods allows bridge maintenance engineers only to make guesses concerning the possible extent of future damage to the deck. To eliminate such guessing would require—to begin with—a method for measuring the existing rate of rebar corrosion, which governs the rate of concrete deterioration. (Such a method can also be useful in assessing the effectiveness of the various deck repair or treatment procedures in controlling rebar corrosion.)

Responding to this need, the Federal Highway Administration sponsored a study at the former National Bureau of Standards that led to the development of a prototype portable device for measuring the corrosion rate of rebars in concrete in the field.[16] Adapting the three-electrode linear polarization (3LP) resistance technique, this device measures potentiostatically the DC polarization resistance of a rebar, from which the rate of corrosion of the rebar is calculated by using the Stern-Geary[17] equation

$$I_{corr} = \frac{B_a B_c}{2.3(B_a + B_c)} \frac{dI}{dE} = k \frac{dI}{dE} \qquad [1]$$

where I_{corr} = corrosion current, in mA

(dI/dE) = slope of polarization plot, in mA/mV

B_a = anodic Tafel coefficient

B_c = cathodic Tafel coefficient

k = formula constant.

To facilitate its routine use in the field, this prototype 3LP device has subsequently been modified and made available commercially.[18]

The electrical resistance measured by this polarization technique includes the resistance of the concrete itself, which could contribute to considerable error in the calculated corrosion rate if not corrected for. The powerful AC impedance technique, which uses AC signals of varied frequencies to polarize the rebar, overcomes this problem by allowing separate measurement of the resistance of the concrete and, in addition, provides insight into the mechanism of the corrosion reactions.[19] However, the technique requires complex data interpretation, which makes transfer of this technique from the laboratory to the field less practical than the DC polarization technique. For this reason, only the 3LP device was used in this study to measure rebar corrosion rates.

To predict future damage, it is not enough to be able to measure rebar corrosion rates in bridge decks because (similar to half-cell potentials) they can be expected to exhibit a two-fold variation: with location and with time. Because of the lack of homogeneity in concrete, rebars at different locations are exposed to varied corrosive conditions and, therefore, undergo corrosion at different rates.

The corrosion rate of rebars at any location, in turn, changes with time—reflecting the seasonal fluctuations of variables such as moisture and oxygen content in the concrete and temperature. It is also possible for chloride concentration in the concrete to increase with time and thus influence the corrosion rate. Therefore, a measured corrosion rate is valid for only the time during which the measurement was made. It is therefore necessary to take into consideration these possible fluctuations of rebar corrosion rates in bridge decks when attempting to estimate future corrosion damage to a concrete deck in terms of projected metal loss in the rebars and its resulting damage to the surrounding concrete.

PURPOSE AND SCOPE

Since measuring half-cell potentials is so simple, inexpensive, and virtually nondestructive, the investigator believed it worthwhile to reexamine the relationship between half-cell potential and the condition of the rebar and concrete and to determine if the uncertainty could be resolved so that potential measurements could be used to locate and quantify deteriorated rebars and concrete.

Another aspect that warranted reexamination was the adequacy of the grid spacing of 4.0 ft suggested in ASTM C 876. It is quite likely that this spacing is so large that a significant percentage of corrosion-affected concrete areas are left undetected during a survey. This may have at least partially contributed to the reports of mixed success obtained from the results of using half-cell potential measurements.

To address these issues and facilitate the understanding of the manner in which the rebar corrosion rate may fluctuate with location in a deck, the investigator collected data from several concrete decks (using some of the various cited inspection methods and the 3LP method) and analyzed them. This report describes the collection and analysis of these data. It also discusses data obtained from a semicontinuous monitoring for more than a year of the corrosion rates of rebars in three fabricated concrete slabs that had a high concentration of chloride. These data were intended to shed light on the manner in which rebar corrosion rates may fluctuate seasonally.

METHODS

Deck Surveys

Five concrete bridge decks were surveyed in this study. During each survey, a preliminary sounding was conducted on the deck to allow selection of a survey area that included numerous delaminated areas. A square grid with either 1.0-ft or 2.5-ft spacing was then marked on the selected area for detailed inspection.

Visible concrete distresses, such as cracks (especially those above the top transverse rebars) and spalls, were recorded and carefully mapped to ±2 in. Sounding was used to detect concrete delaminations, which were similarly mapped to ±2 in.

The measurement of half-cell potentials was facilitated by the use of a multiple half-cell array, which consisted of an array of four $Cu/CuSO_4$ electrodes (see Figure 1). Depending on the grid spacing to be used in a survey, the electrodes were spaced at 1.0 to 2.5 ft apart on the graduated metal mounting bar. A battery-powered portable data logger (Polycorder 700), which was preprogrammed to serve as a digital voltmeter (with a resolution of 1 mV), was used in conjunction with the half-cell array to store automatically, throughout each survey, all potential readings from the four electrodes (see Figure 2). The stored data were then subsequently downloaded to a desktop computer for iso-potential contour mapping and statistical analysis, using an appropriate graphic software.

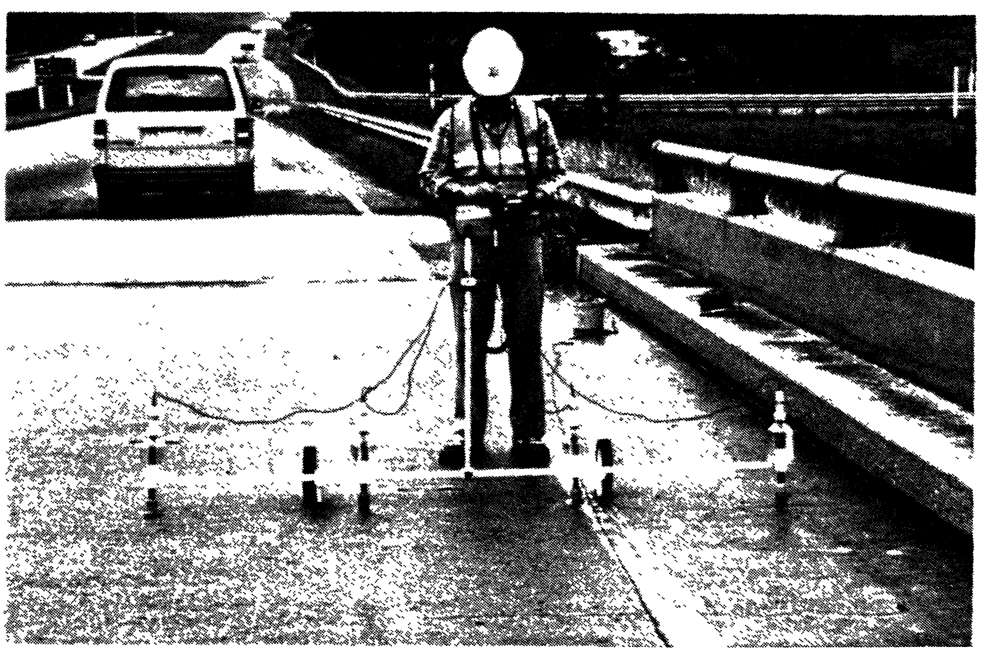

Figure 1. A half-cell array, consisting of four $Cu/CuSO_4$ reference cells, being used on a concrete bridge deck.

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Figure 2. A microprocessor-based data logger used for measurement and automatic logging of half-cell potentials.

The commercially modified 3LP device, as shown in Figure 3, was used to measure rebar corrosion rates in each deck. Due to the amount of time (approximately 3 min) required to perform the measurement at each location, or grid point, a grid spacing of 5.0 ft was used.

To measure the corrosion rate of a rebar at each grid point, the location and depth of the nearest rebar were determined with a pachometer. A probe assembly, which consisted of a pen-sized $Cu/CuSO_4$ reference electrode and a surrounding counter electrode that was made of a sponge-encased copper mesh, was then placed directly above the rebar (see Figure 4). The probe assembly was then connected to the rebar network in the deck and to the 3LP device, and procedures were conducted to measure the polarization resistance of the rebar. These procedures, which were described elsewhere,[18] essentially entailed incremental polarization of the rebar to achieve potential shifts of 4, 8, and 12 mV, in that ascending order, and noting of the respective amounts of direct current needed to be applied through the counter electrode to effect these potential shifts. Using these data, the linear polarization resistance (dE/dI), which is the slope of the "best-fitting" line, was calculated. Assuming a formula constant of 41, as suggested elsewhere,[18] the corrosion current was calculated from equation 1. To allow comparison, the calculated corrosion current (in mA) was expressed in terms of corrosion current density, I_d, i.e., current per unit area of rebar (that was under the influence of the counter electrode), by using the following formula:

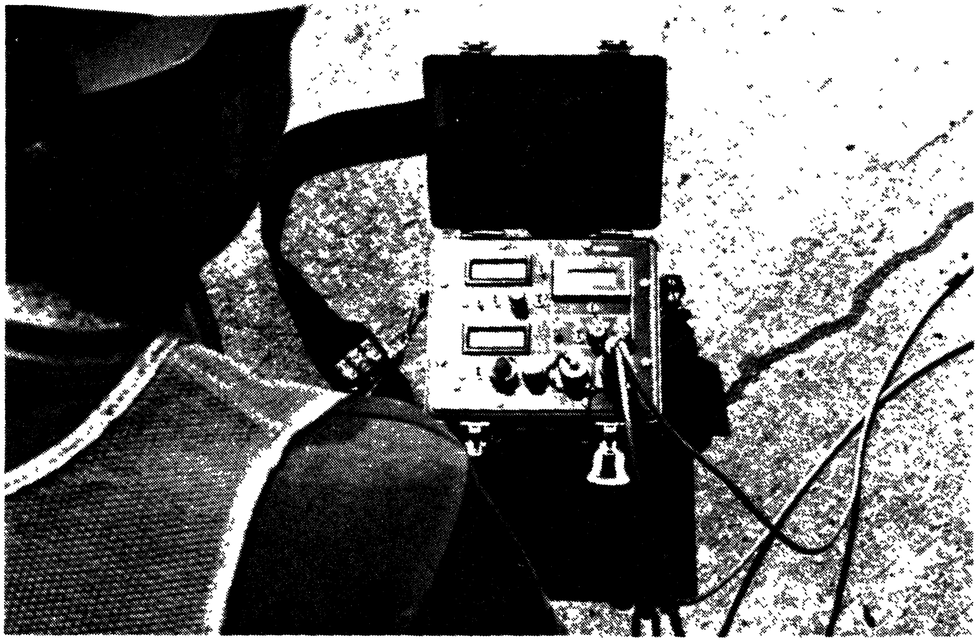

Figure 3. A 3LP device for measuring rebar corrosion rates in concrete bridge decks.

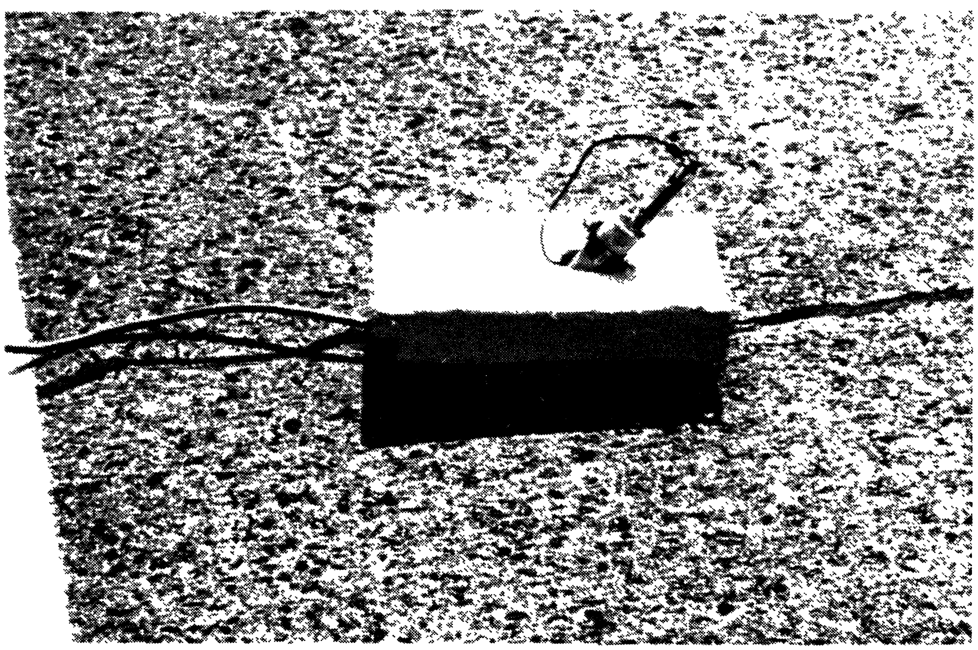

Figure 4. A probe assembly for use with the 3LP device.

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$$I_d = \frac{I_{corr}}{A} = \frac{367\ I_{corr}}{BK} \qquad [2]$$

where I_d = corrosion current density, in mA/sq ft

I_{corr} = corrosion current, in mA

A = surface area of rebar, in sq ft

B = equivalent bar size, in eighths of an inch

K = rebar length beneath the counter electrode, in in.

The calculated corrosion current densities were then plotted in iso-contour lines for each deck.

After completion of each survey, the rebars at several selected locations were extracted by partial concrete coring for examination and determination of the thicknesses of the concrete cover and accumulated metal losses, or weight losses, due to corrosion. The criterion used in the selection was that the measured corrosion rates of the rebars at these selected locations must closely represent the range of corrosion rates observed in the survey area.

To determine the metal loss on each piece of extracted rebar, the following procedures were used:

1. The rebar was thoroughly cleaned by careful chipping with chisels, which was followed by light sandblasting to remove corrosion products and portions weakened by pittings.

2. The curved ends of the rebar were cut off to provide straight ends that facilitate measurement of the length, L, of the rebar, which was then made with a caliper to the nearest 0.01 in.

3. Using a top-loading balance, the weight of the rebar, W, was then measured to the nearest 0.01 g. The weight was divided by 453.6 to convert to pounds.

4. The metal loss of the rebar, in percentage by weight, was then estimated by using the following relationship

$$ML = 100 \times [(W/L)_i - (W/L)_f]/(W/L)_i \qquad [3]$$

where ML = metal loss

$(W/L)_f$ = the final weight per unit length, in lb/in

$(W/L)_i$ = the estimated initial weight per unit length of the rebar, in lb/in.

The $(W/L)i$ was assumed to be the same for all extracted rebars in each survey area and was the average weight per unit length of two extracted rebars from each survey area that did not have any visible sign of corrosion. The major sources of errors in each estimate of metal loss included the cleaning of the rebar specimen—from either overcleaning or undercleaning the rebar—and the estimation of $(W/L)_i$ for each deck sampled. It was estimated that the combined error was at least ±1.2 percent by weight.

Monitoring Rebar Corrosion Rates in Concrete Slabs

Fabrication of Concrete Test Slabs

Three reinforced concrete slabs (3.0 ft × 3.0 ft), each with a different thickness of concrete cover (1, 2, or 3 in) over their top-mat rebars, were fabricated according to the plan shown in Figure 5. The concrete mixture used was a VDOT Class A4 concrete.[20] To accelerate corrosion of the top-mat rebars, enough NaCl was added to the concrete mixture for the top 3-in layer of each slab to yield a chloride ion concentration of 10.0 lb/cu yd of concrete.

To allow measurement of the temperature and moisture content of the concrete, thermocouples and calibrated soil moisture probes were installed in each slab at the two locations shown in Figure 5.

Monitoring of Slabs

The slabs were allowed to cure outdoors for at least 60 days before the corrosion rate of the rebars, concrete temperature, concrete moisture in each slab, and outdoor air temperature were monitored. Measurements of rebar corrosion rates in each slab were restricted to the two same rebars (see Figure 5) throughout the monitoring period, during which the slabs were exposed to the outdoor environment. The monitoring lasted for approximately 16 months—from December 1989 to April 1991.

RESULTS AND DISCUSSION

Half-Cell Potentials and Concrete Condition

The half-cell potentials obtained from all study areas were correlated with the condition of the concrete. To allow such correlation, it was necessary first to classify concrete as either sound or deteriorated, based on a criterion that the investigator believed reasonably conformed to the manner with which deteriorated concrete would be removed and repaired in a concrete deck. Accordingly, a concrete

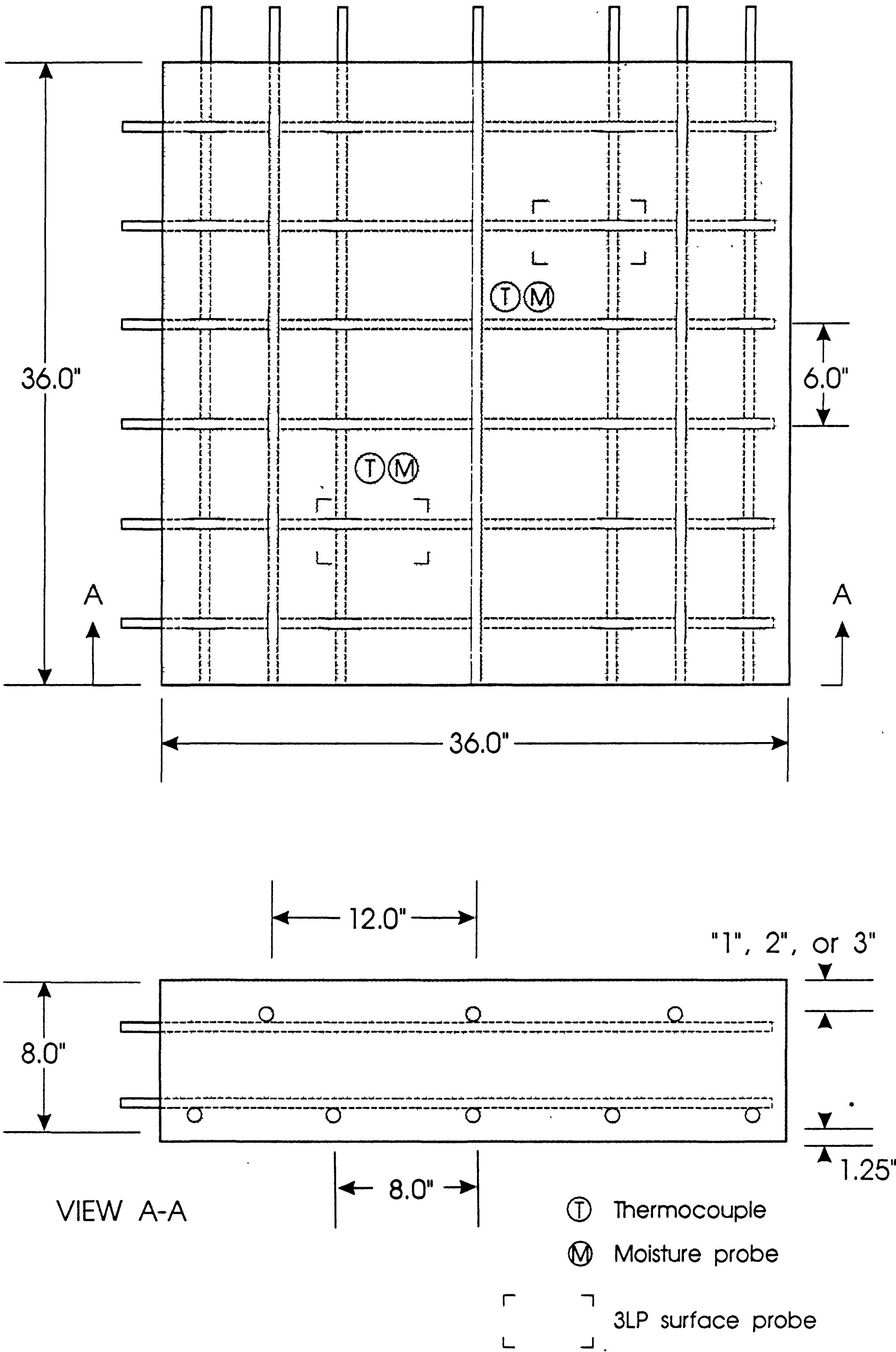

Figure 5. Outdoor exposed concrete slabs used to monitor the variation of rebar corrosion rates with the seasons.

was classified as sound if there was neither a transverse crack, a delamination, nor a spall within 6 in. of the point of measurement of the half-cell potential; otherwise, the concrete was classified as deteriorated. In addition, all the potential readings (approximately 2,000) were divided into categories in 50-mV increments, i.e., –151 to –200 mV, –201 to –250 mV, etc.

This correlation showed that virtually 100 percent of the concretes were in sound condition at potentials ≥ –150 mV and 100 percent of the concretes were deteriorated at ≤ –450 mV (see Figure 6). However, this correlation was characterized by deviations of varied degrees between these extremes. For example, for concrete with potentials between –301 to –350 mV, only 28% to 85% were actually deteriorated; for –351 to –400 mV, only 70% to 100% were deteriorated. (It is believed that this correlation would still be generally valid even if a criterion other than 6 in. was used in classifying the condition of the concrete.) This observation agrees, in general, with the ASTM C 876 guidelines for interpretation of half-cell potentials, even though the latter attempted to relate potential only with probability of rebar corrosion.

It is clear that there was not a sufficiently well-defined relationship between concrete condition and the numerical values of individual half-cell potentials, with the exception of potentials at the extreme ends, that engineers can use to prepare repair plans with a reasonable degree of confidence. This also raises concern on the appropriateness of replacing existing concrete simply because the potential is ≤ –350 mV, which is a common practice among many highway agencies.

As will be shown, the absence of such correlation is due to fluctuation of half-cell potential at any location with time—likely in response to fluctuating temperature and moisture in the concrete.

Potential Gradients and Concrete Condition

It was found that a convenient and effective way to locate active rebar corrosion and its associated damage to the concrete is to plot the results of a half-cell potential survey on an iso-potential contour map in which the variation in potential gradient is reflected by the spacing between the contour lines, such as that shown in Figure 7 for one of the areas surveyed.

Because of the lack of homogeneity of concrete and, therefore, the distribution of chloride ions across the concrete, the corrosion found on rebars and its associated damage to the surrounding concrete are often localized in nature. Consequently, as the reference electrode is moved from one location on the concrete where the rebar underneath is not corroding to another nearby location where the rebar is actively corroding, the half-cell potential should become more negative. Further, the probability that localized rebar corrosion is occurring at the second location increases as the potential shifts toward a more negative value at a relatively high potential gradient (i.e., the contour lines are closely spaced). Conversely, if the

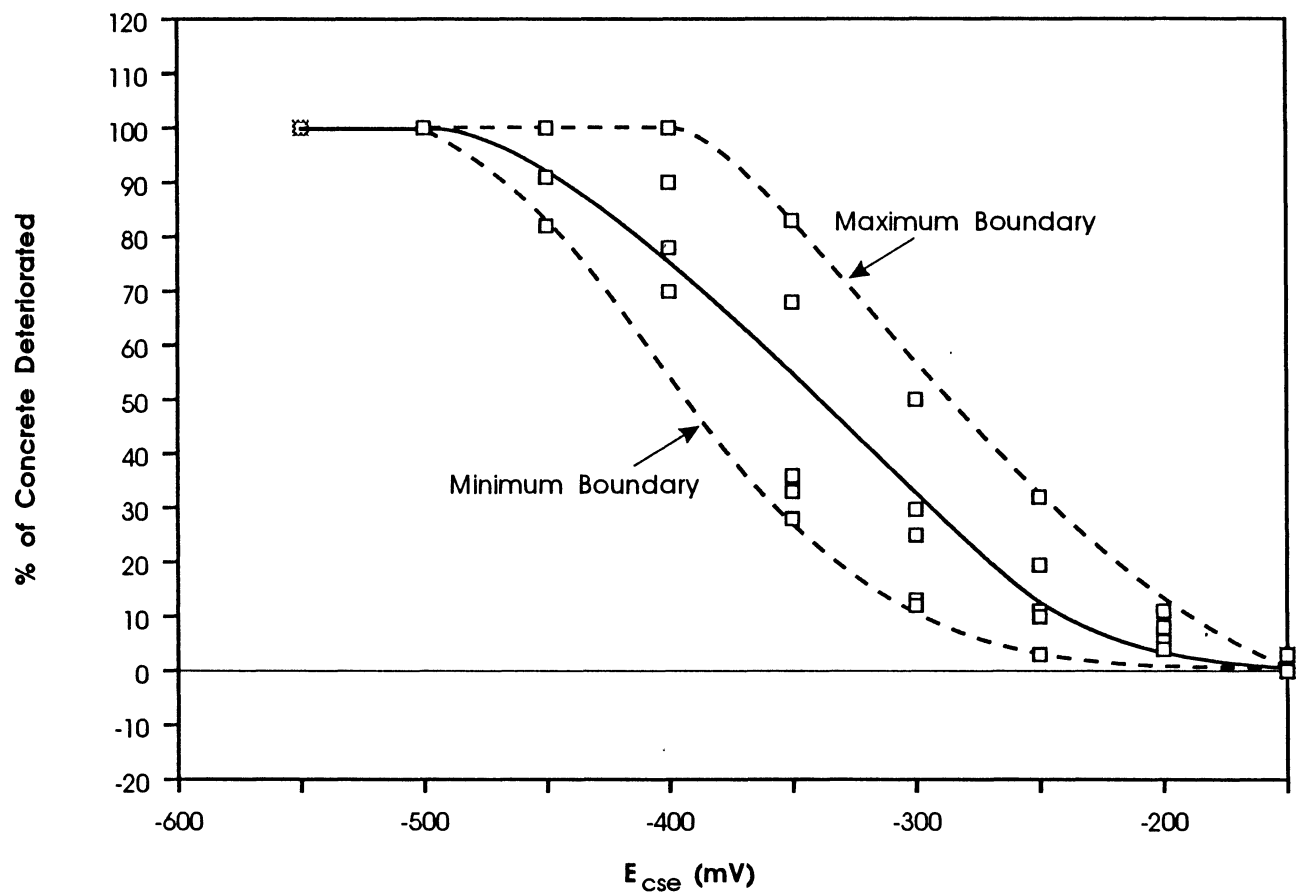

Figure 6. Correlation between half-cell potentials and frequency of damaged concrete observed in all survey areas.

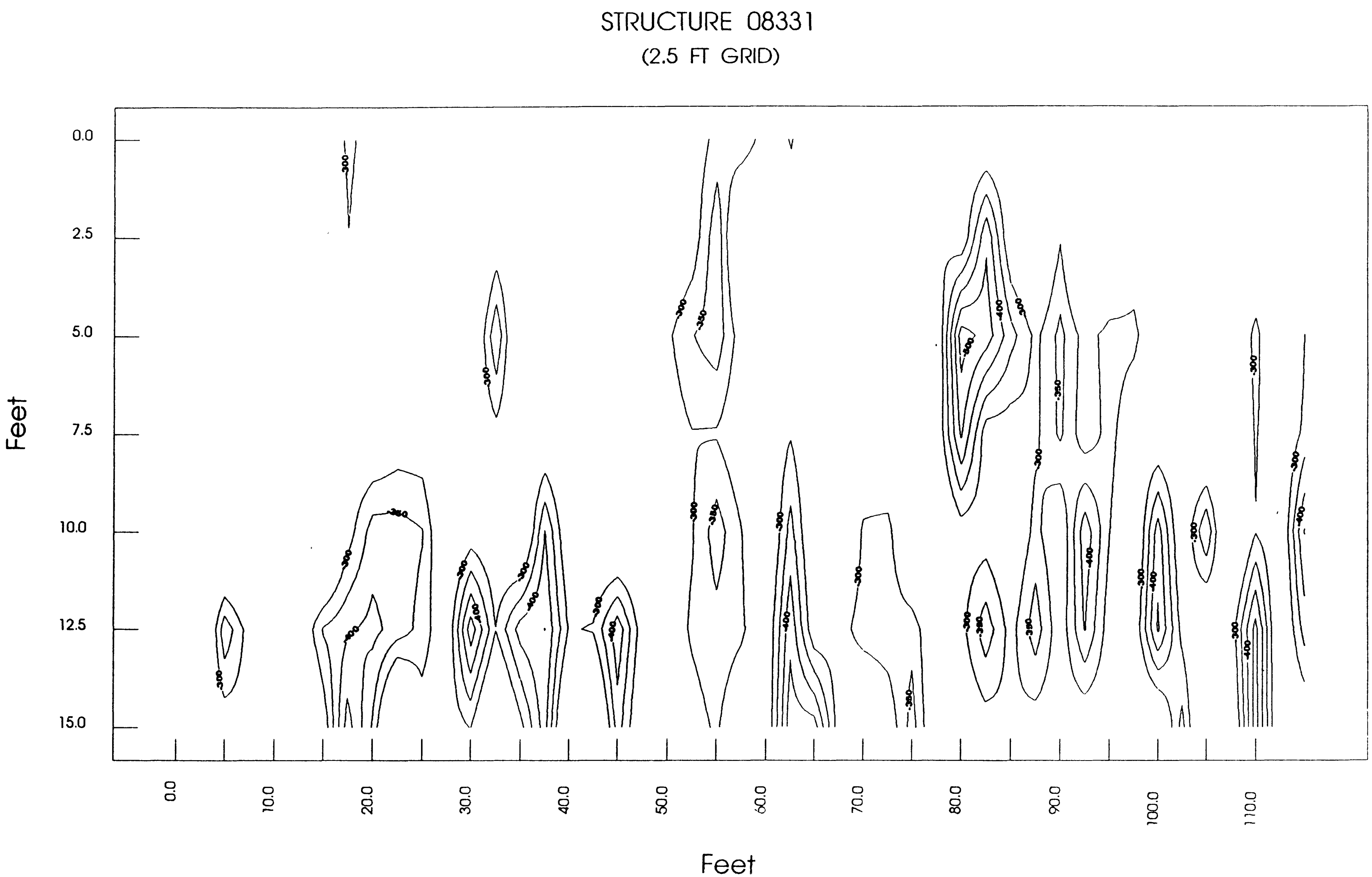

Figure 7. Iso-potential contour map for a survey area. (Only E ≤ −300 mV are plotted in 50-mV increments.)

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potential shifts toward a more positive value, the probability of localized rebar corrosion occurring at the next location decreases.

Accordingly, the existence of numerous areas where active rebar corrosion was likely to be occurring is indicated in Figure 7. Across the surface of the concrete, the potential gradients corresponding to these areas ranged from approximately –72 to –172 mV/ft. It is worthwhile to note that the orientation of these areas coincided with the general alignment of the top transverse rebars of the deck, which are the rebars that are likely to corrode first.

Given sufficient time to corrode, the rebars would eventually accumulate a sufficient amount of oxidation products to initiate delamination in the surrounding concrete. Therefore, it is not unreasonable to expect that the concrete at some of the active areas indicated in Figure 7 would already be damaged, especially when the deck has been in service for 20 years. In fact, this was the case. When the locations of all the transverse cracks and delaminations detected in the concrete were superimposed on the contour map in Figure 7 (as shown in Figure 8), it was evident that many of the deteriorated concrete areas matched the areas of high negative potential gradients.

A similar correlation between locations of high lateral potential gradients and locations of deteriorated concrete was found for the other survey areas involved in the study. In those areas, the potential gradients ranged from approximately –60 to –300 mV/ft.

It is estimated that at least 85 percent of the deteriorated concrete areas in the survey area depicted in Figure 8 were matched or accounted for by high potential gradients. The failure of this potential survey to account for or detect the remaining deteriorated concrete areas is attributed to the grid spacing of 2.5 ft used in the survey, which appeared to be too large. For example, one may consider the two narrow delaminated areas at around (61.0 ft, 2.5 ft) and (85.0 ft, 11.0 ft)—the edges of both areas were located considerably away (at least 0.7 ft) from their nearest grid points for the effect of the localized corrosion on the rebars to be manifested on the potentials measured at those grid points.

Optimum Grid Spacing for Half-Cell Potential Surveys

To determine the optimum grid spacing, it was necessary to determine the maximum lateral distance from a corroding rebar at which the effect of corrosion on the half-cell potential is still discernible. Figure 9 shows a contour map that resulted from detailed measurements of half-cell potentials, in 0.5-ft spacing, around a typical small delaminated concrete area that was approximately 1.2 ft wide. The transverse rebar on which the corrosion occurred that caused the delamination was situated directly beneath the crack at the center; other transverse rebars were spaced at 1 ft away. Again, the localized nature of rebar corrosion (and its resulting damage) was manifested in a large potential gradient, which was approximately –250 mV/ft in this case, as the potential shifted from –525 mV at the most active area (center top) to approximately –275 mV at 1 ft away on either side.

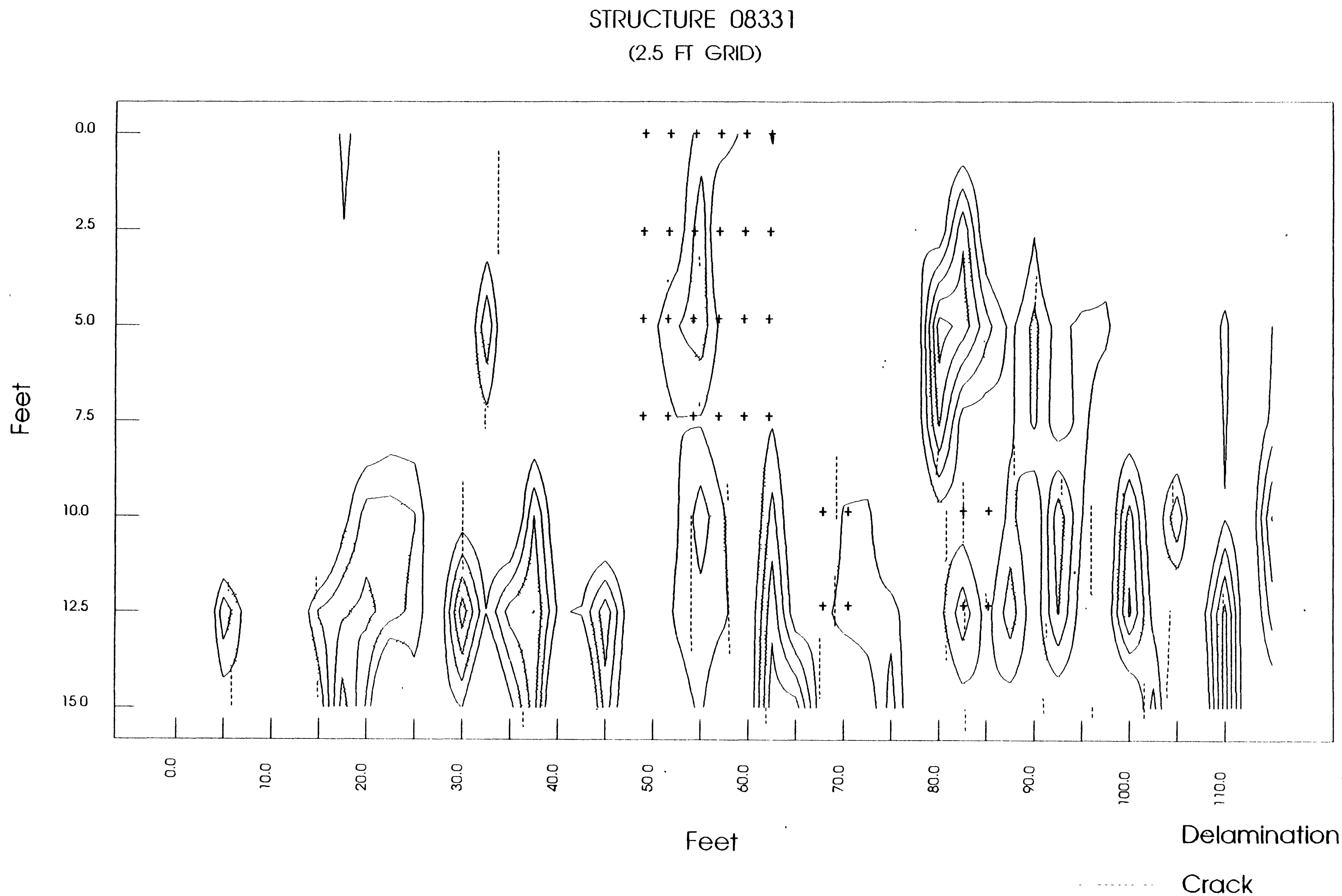

Figure 8. Correlation of transverse cracks and delaminations with potential gradients (2.5-ft grid spacing).

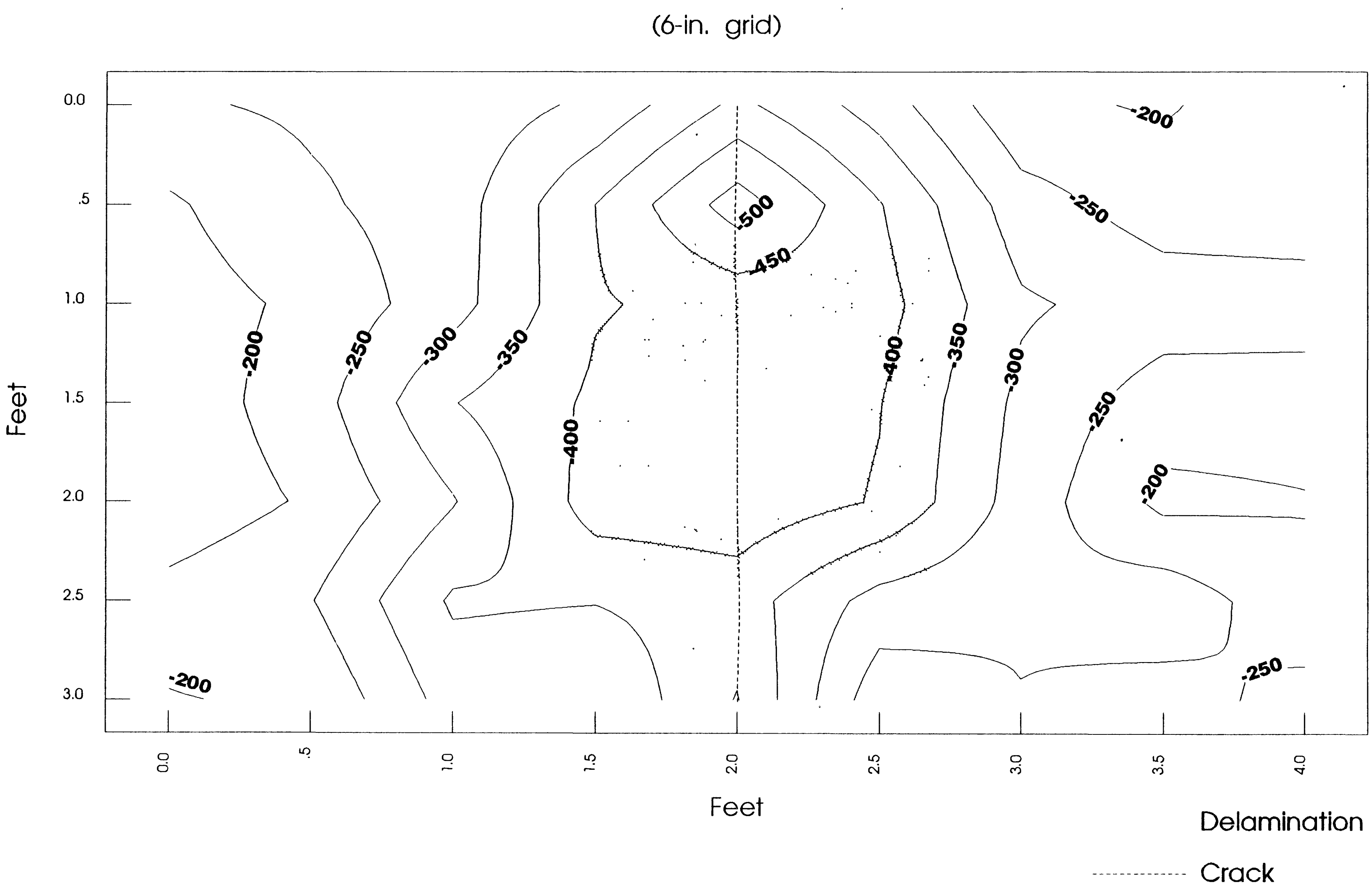

Figure 9. Iso-potential contour lines surrounding a deteriorated concrete area.

If we define "field of effect" as the maximum (lateral) distance from a crack or the edge of a delamination to a point on the surface of the concrete where the half-cell potential has shifted sufficiently toward positive to indicate that the underlying rebar corrosion is almost indiscernible and assume that this potential is equal to, say conservatively, –300 mV, Figure 9 shows that the field of effect in this case was less than 8 in. Therefore, if this delamination happened to fall in the middle of a 2.5-ft grid square, it would certainly not be detected in such a survey. Since similar observations in the other decks indicated that the field of effect was typically no more than 8 to 10 in., it is quite clear that a grid spacing of 2.5 ft—not to mention the 4.0-ft spacing recommended by ASTM C 876 or the 5.0-ft spacing used by many highway agencies—would still be too large to provide a complete "picture" of the condition of a concrete deck.

It appeared that the optimum grid spacing is approximately 1.0 ft. To ascertain this, a 1.0-ft grid spacing was used to conduct a half-cell potential survey on another concrete deck. Then, in addition to the usual analysis of the entire set of data, analyses to simulate the use of 2.0-ft and 4.0-ft spacings were also conducted by simply disregarding potentials measured at appropriate grid points during different simulations.

The iso-potential contour map that resulted from the use of 1.0-ft grid spacing (see Figure 10) clearly delineated the presence of numerous areas of high potential gradients, ranging from –100 to –300 mV/ft. These areas matched or accounted for all the delaminations and severe transverse cracks detected.

The contour map also showed several areas wherein the potential gradients were high, indicating the presence of relatively active rebar corrosion, but the concretes were still reasonably sound (with the exception of the presence of some fine cracks) by the sounding inspection. In fact, the rebar corrosion rates in a few of these areas (A, B, and C) were measured by the 3LP device and found to be 2.87, 2.94, and 3.09 mA/sq ft, respectively; these rates were approximately 3 times higher than those found at grid points (13, 3) and (21, 1), which were 0.99 and 1.06 mA/sq ft, respectively.

Based on a generalized correlation between rebar corrosion rate and metal loss that was observed in the other decks (to be discussed later), the level of corrosion rates observed in areas A, B, and C may translate to a metal loss of approximately 2% to 6% by weight. Such metal losses are just below the threshold of 3% to 6% by weight (estimated in this study) at which concrete could begin to fracture internally. This implies that, if repair is contemplated, a bridge engineer should include such small areas of A, B, C, and others (in Figure 10) in the estimation of repair quantity. This raises an issue that must be addressed in a future study—the determination of the extent of concrete in each area of high potential gradient that must be included in the estimation of repair quantity.

If a 2.0-ft grid spacing was used instead, as shown in Figure 11, the delamination at the vicinity of grid point (6, 5) became practically unaccounted for, as a consequence of the incomplete data provided by this slightly larger grid spacing.

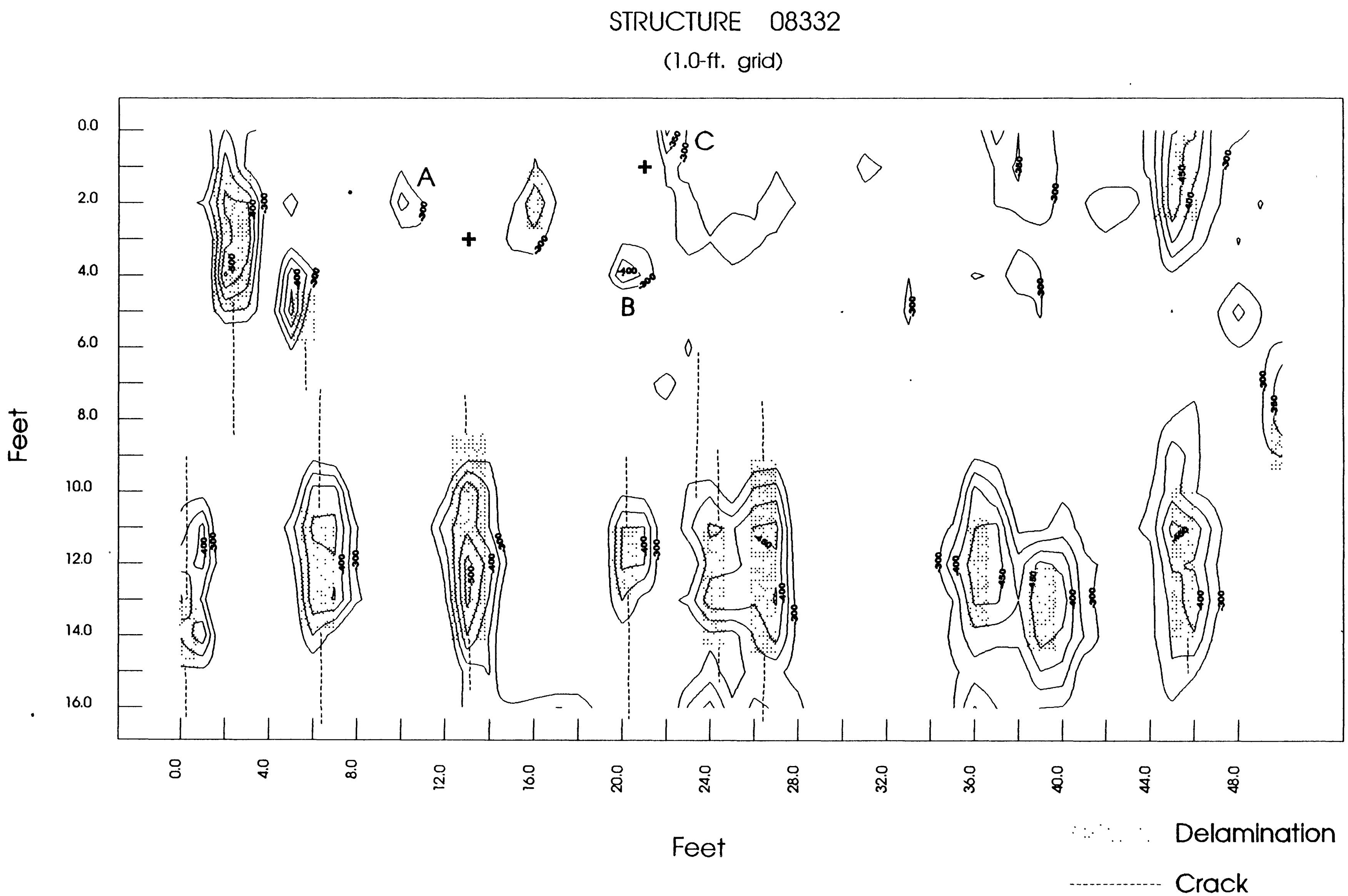

Figure 10. Iso-potential contour map obtained with 1.0-ft grid spacing. For a discussion of *A*, *B*, and *C*, see the text.

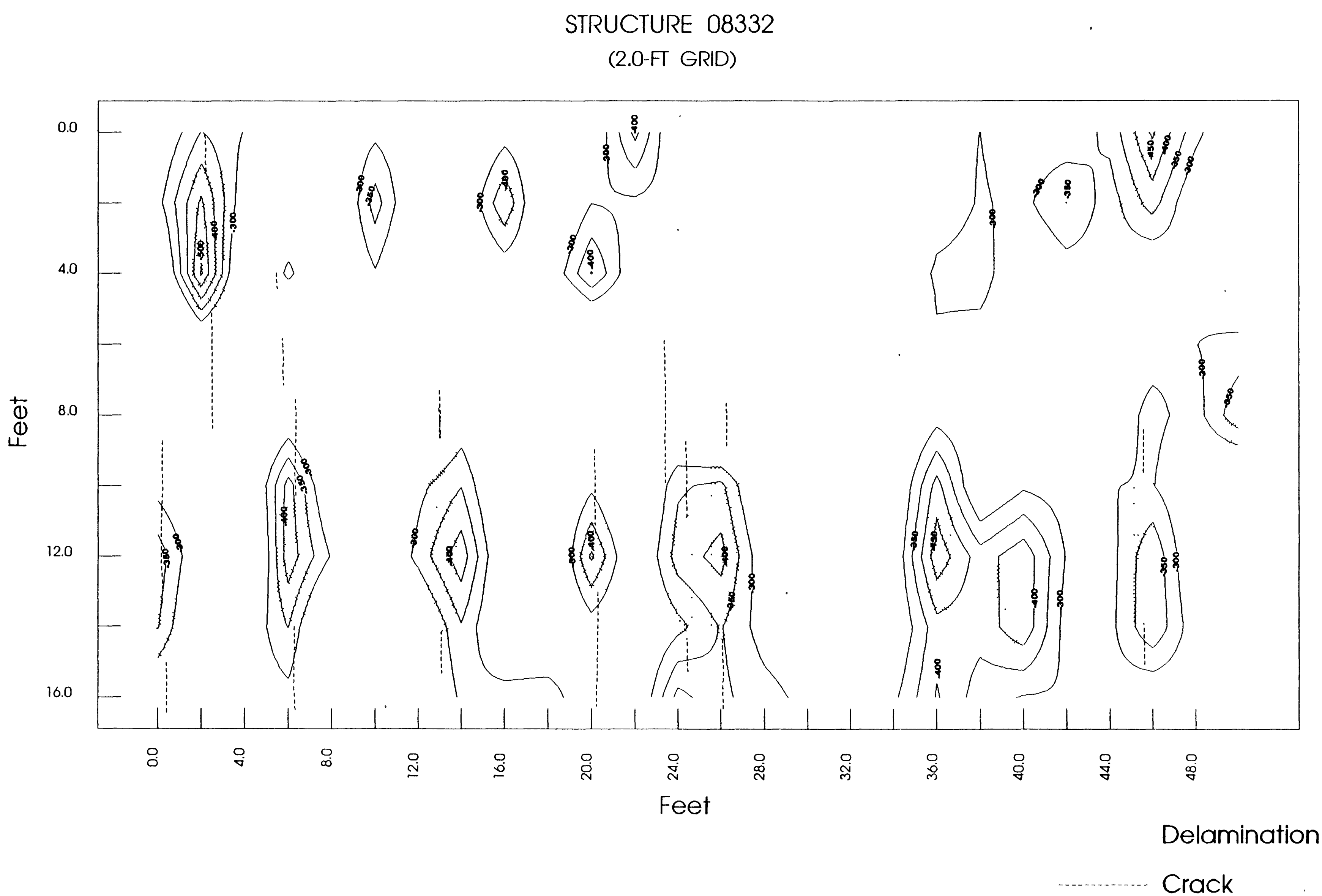

Figure 11. Iso-potential contour map obtained with 2.0-ft grid spacing.

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Further, there were some losses of details and distortion of the configurations of some areas of high potential gradients. However, these errors were relatively small and probably negligible.

If an even larger 4.0-ft grid spacing were used, Figure 12 shows that the corresponding sampling of the same area would be so insufficient that approximately 50 percent of the deteriorated areas would be left undetected and the areas of high potential gradients would be seriously distorted. The results would not represent the condition of the deck area surveyed with reasonable accuracy.

The failure rate in detecting areas of active rebar corrosion and concrete deterioration for a given grid spacing, with respect to the 1.0-ft grid spacing, was estimated based on two survey areas from which appropriate data were available (see Figure 13). Assuming that the failure rate for using 1.0-ft grid spacing was 0 percent, the rate increased to 10 percent for using a 2.0-ft grid spacing and then abruptly to approximately 62 percent for using a 4.0-ft grid spacing. Although these observed failure rates may represent only the two particular survey areas, they nevertheless serve to illustrate that the reliability of a half-cell potential survey to provide a complete picture of the condition of a deck is considerably jeopardized when a large grid spacing, especially those larger than 2.0 ft, is used.

Of course, as the grid spacing is reduced, the required amount of sampling (or number of potential measurements) in a survey increases exponentially, as illustrated in Figure 14. With a 1.0-ft spacing, 1,084 individual potential readings would have to be recorded for 1,000 sq ft of concrete deck area. The amount of sampling required decreases significantly by 73 percent or 87 percent when the spacing is increased to 2.0 ft or 3.0 ft, respectively. Beyond these grid spacings, the corresponding decrease in sample size required is comparatively less.

The selection of an optimum grid spacing should reflect a balance between reasonable accuracy (consistent with the purpose of the survey) and a reasonable sample size (with its associated survey time and cost). One may consider the influence of a desired accuracy on the sample size required per unit percentage of accuracy, as shown in Figure 15. For the 100 percent accuracy provided by 1.0-ft spacing, the sample size required would be 10.8 measurements of potential per 1,000 sq ft of concrete per percentage of accuracy. In contrast, for the 90 percent accuracy provided by using a 2.0-ft grid spacing, the required sample size would be only 3.26 measurements per 1,000 sq ft of concrete per percentage of accuracy.

Even though the use of small grid spacing would increase sample size exponentially, the concomitant survey and analysis time would increase only minimally by the use of an array of $Cu/CuSO_4$ electrodes and a portable microprocessor-based data recorder similar to those used in this study. In fact, the most time-consuming portion of a survey, which is the marking of a survey grid on the deck being surveyed, can be almost eliminated by attaching an adequate distance-measuring wheel on the half-cell array (see the Appendix). However, if a compromise between accuracy and required sample size is necessary, the 2.0-ft spacing is undoubtedly a reasonable choice—especially considering that the field of effect on the potential from a corrosion site may be 8 to 10 in.

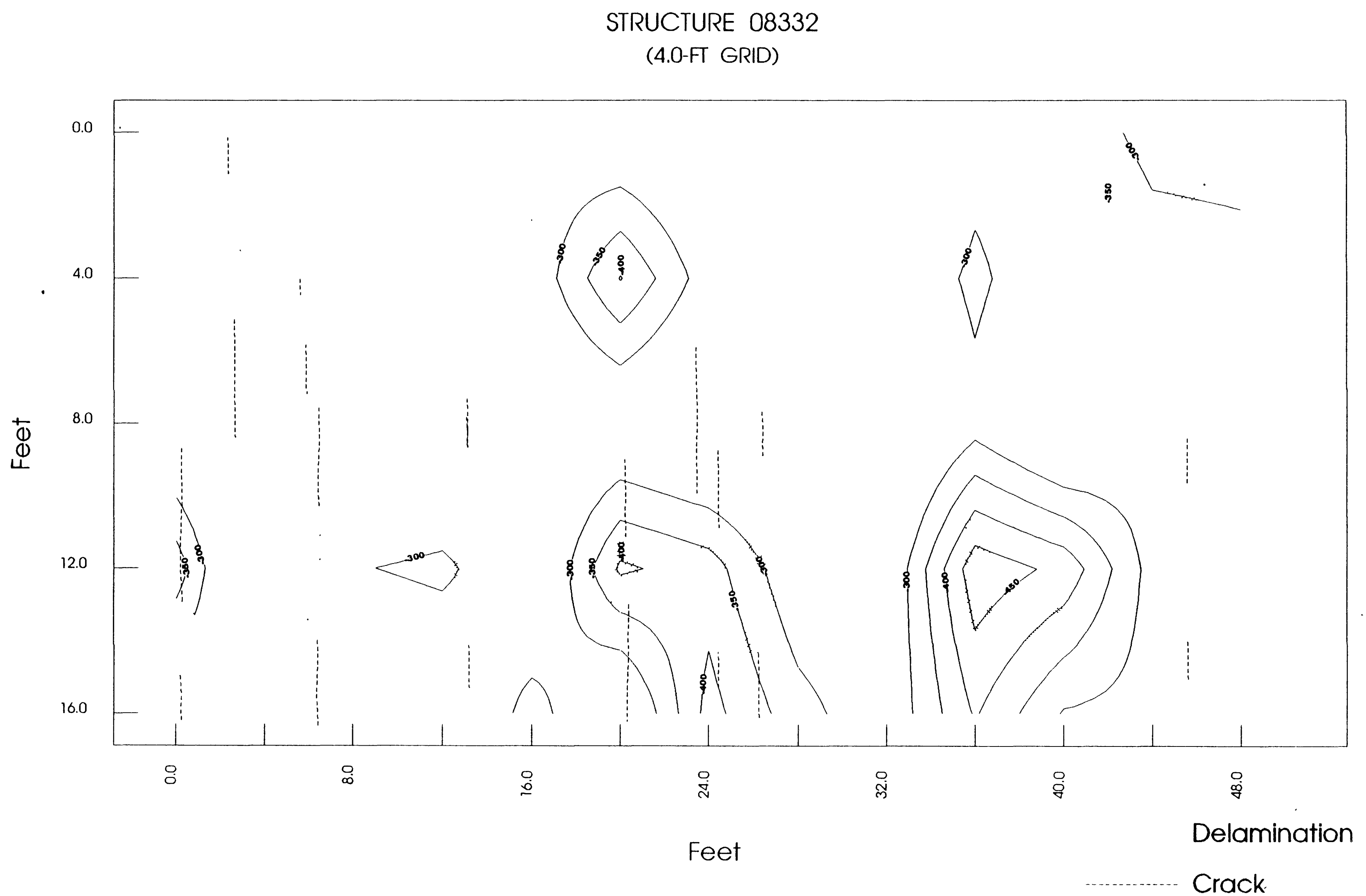

Figure 12. Iso-potential contour map obtained with 4.0-ft grid spacing.

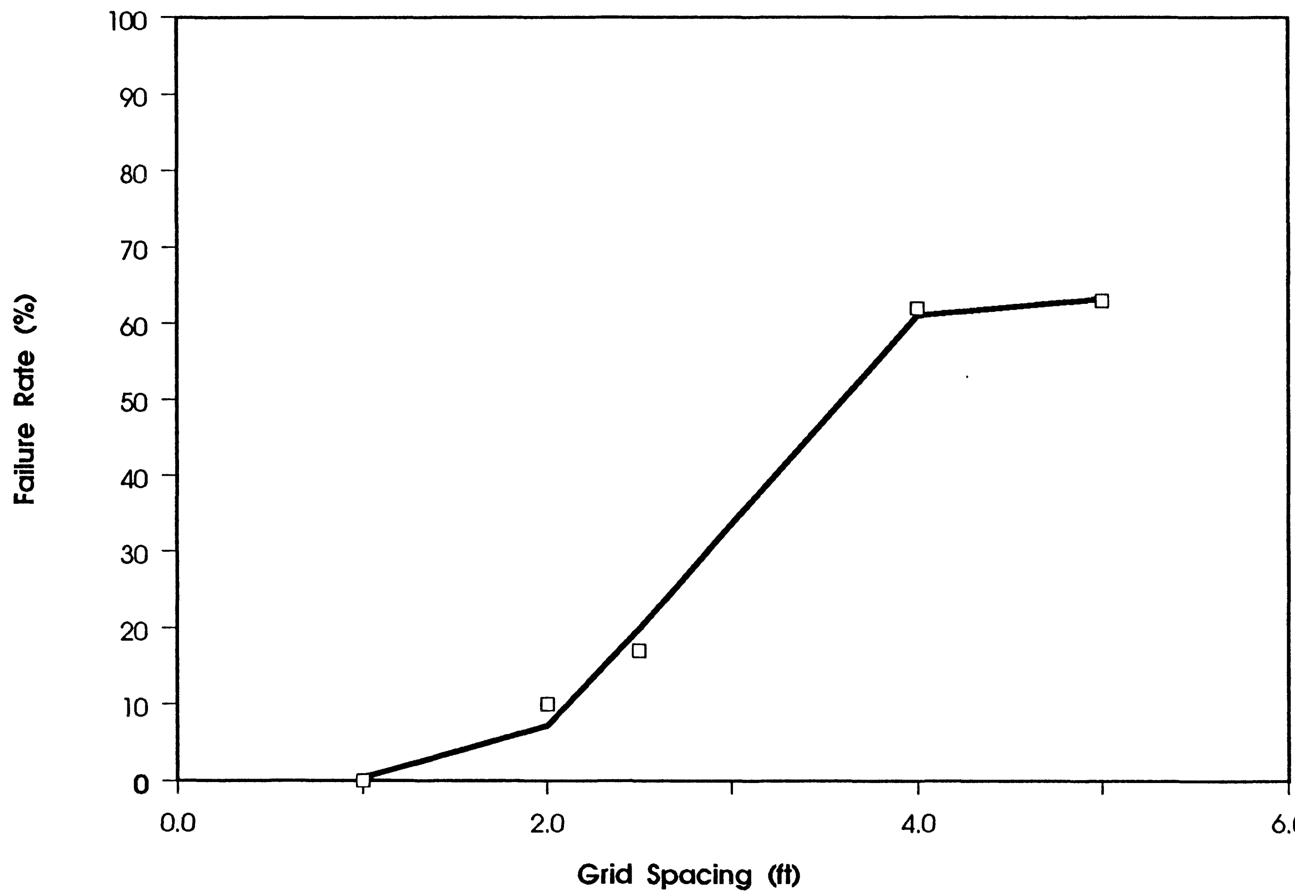

Figure 13. Effect of survey grid spacing used in half-cell potential surveys on the estimated failure rate for locating corrosion-induced deterioration in concrete bridge decks.

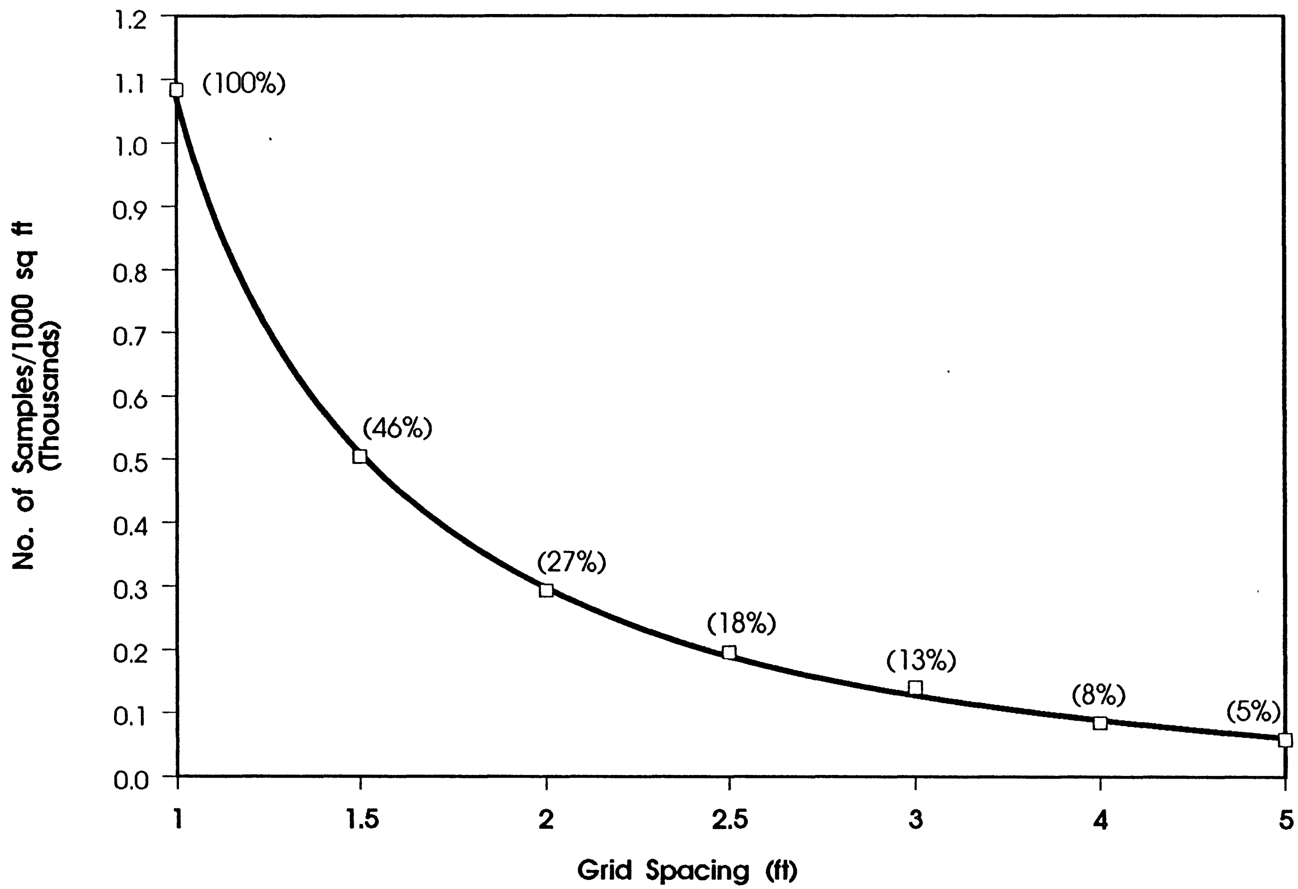

Figure 14. Number of sample locations in a survey grid used for a half-cell potential survey as a function of square grid spacing.

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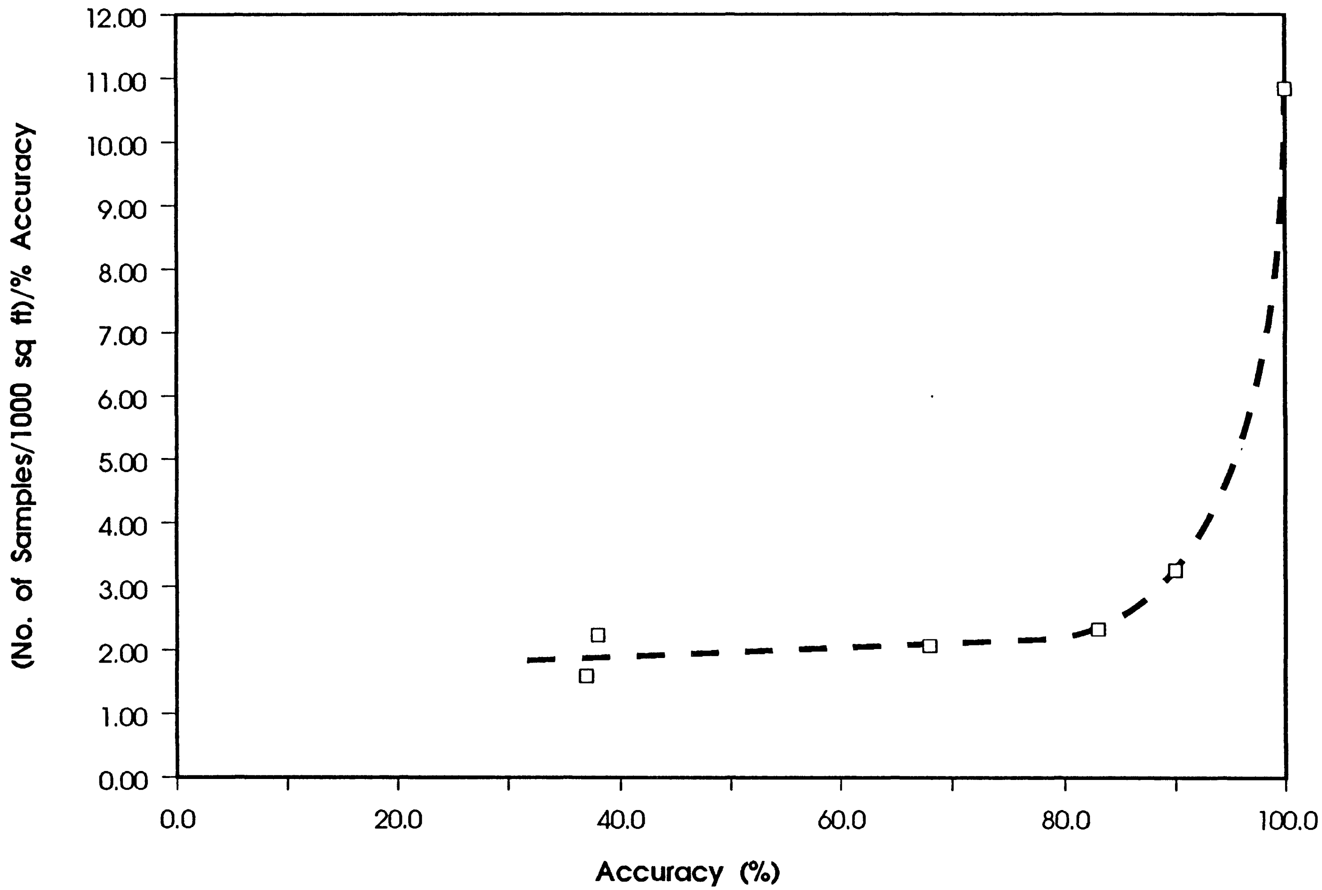

Figure 15. Influence of desired accuracy on the number of half-cell potential measurements required per 1,000 sq ft of concrete deck per percentage of accuracy.

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Table 1

COMPARISON OF TIME REQUIRED TO CONDUCT
A HALF-CELL POTENTIAL SURVEY ON A BRIDGE DECK*

	No. of Personnel	Time (min)	No. of Manhr	Grid Points	Manhr/ Point
By Present Survey Procedures					
1. Marking of 5-ft survey grid (laying of grid lines, marking of grid points on each grid line)	3	90	4.5	99	
2. Measurement and manual recording of potentials at all grid points	2	149	5.0		
3. Manual analysis of data	1	180	3.0		
Total			12.5	99	0.126
By Survey Procedures Used in Study					
1. Laying of survey lines (paths)	2	30	1.0	546	
2. Simultaneous multiple measurements and automated logging of potentials at all grid points	1	195	3.3		
3. Computerized analysis and contour mapping of data	1	30	0.5		
Total			4.8	546	0.009

*Assuming three 42 ft x 52 ft spans.

To illustrate the cost savings that the survey procedures used in this study (as described in a previous section on survey methods and the appendix) would provide, a comparison of personnel and the time required by both the current and the proposed set of procedures is shown in Table 1. For a bridge deck with three 42 ft x 52 ft spans, the current procedures would require approximately 12.5 manhours to conduct; in contrast, the proposed procedures would require only 4.8 manhours. Since the latter would provide much more details (see Figures 1 and 2) about the condition of the deck (with 546 grid points instead of 99), the resulting unit cost of 0.009 manhour per grid point is even more favorable than that for the current procedures. Of course, an initial capital investment of approximately $5,000 to $7,000 would be required for each complete microprocessor-based potential survey system.

Influence of Concrete Variables on Half-Cell Potentials

Half-cell potentials reflect not only the condition of the rebars and the concrete, as the preceding discussion has shown, but also the electrical resistance of the layer of concrete between the rebars being measured and the $Cu/CuSO_4$ electrode. Since the resistance of the concrete at any location in a deck is determined to various degrees by the thickness of the concrete layer and the seasonally fluctuating variables of temperature and moisture content, half-cell potentials can fluctuate from survey to survey. The investigator believed that such fluctuation contributed to uncertainty concerning the interpretation of half-cell potentials.

Due to this fluctuation, it is important to consider the individual potentials measured in any survey not in terms of their magnitude or numerical value (as recommended by the interpretation guidelines in ASTM C 876) but rather in terms of their relation to the magnitudes of the potentials at the surrounding concrete, as shown in the earlier discussion.

This point can be easily demonstrated by superimposing contour maps from separate surveys of the same concrete deck area. Figure 16 shows the contour map in Figure 11 superimposed on another obtained a month earlier for the same concrete deck area. (The moisture content in the concrete deck may be relatively higher during the earlier survey because of rainfall in the area days prior to the survey.) With the exception of some minor differences, it is evident that the contour maps remained quite similar. More important, the general locations of the defective areas remained constant, despite some apparent differences in the histograms for the two sets of half-cell potentials (see Figure 17).

As the results show, as long as the overall condition of a concrete deck has not changed significantly between surveys, the resulting contour patterns should remain relatively unaltered although the numerical values of the two separate sets of half-cell potentials may vary. Therefore, this investigator argues that the numerical value of each measured potential by itself is a poor indicator of the condition of the rebars and the concrete; instead, a high potential gradient is a better indicator.

Variation of Rebar Corrosion Rate with Location

The accuracy of the corrosion rates as determined by use of the commercial 3LP device was uncertain. This is due to uncertainty on the appropriateness of the suggested values of some parameters[18] used in the calculation of corrosion rate, namely the Tafel constants (B_a and B_c) and the surface area of the rebar that was actually under the influence of the counter electrode during each measurement. Recent comparison with other devices indicated that the corrosion rate values obtained with 3LP devices may be relatively high.[21] Nevertheless, the calculated corrosion rates for each survey area were at least suitable for internal comparison.

As expected, it was found that the corrosion rates of rebars across a concrete bridge deck varied. This variation is illustrated in Figure 18, which shows the contour map of the rebar corrosion rates observed in the survey area of Structure 09302. These corrosion rates ranged from 0.11 to 4.5 mA/sq ft, or 0.054 to 2.2 mil/year. (Even higher corrosion rates have been observed in other decks.) This typical contour map shows the presence of several areas wherein rebars were corroding at

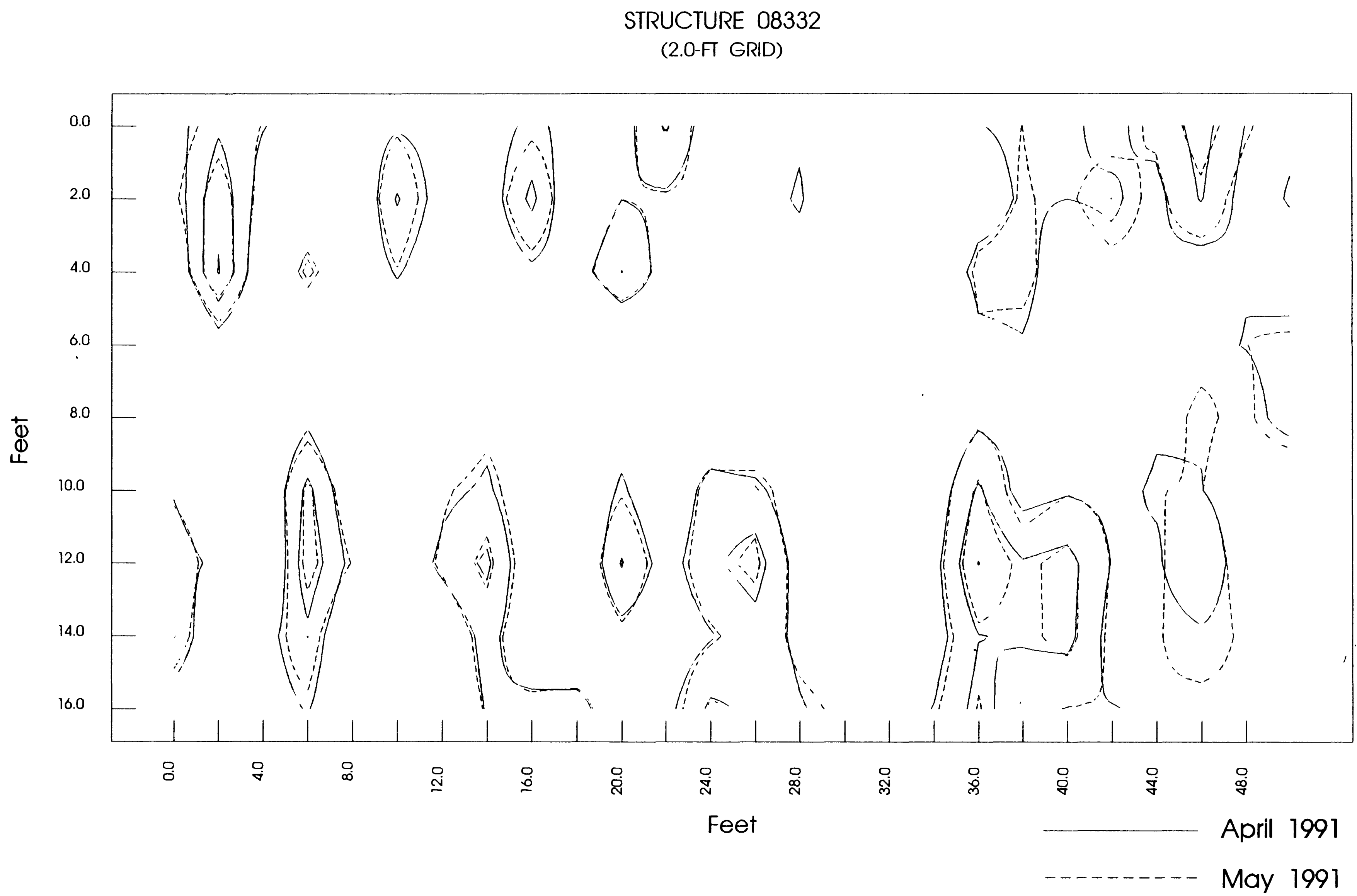

Figure 16. Contour maps of half-cell potentials observed during two separate surveys of a section of a concrete deck. (Iso-potential lines in 100-mV increments, starting at –300 mV.

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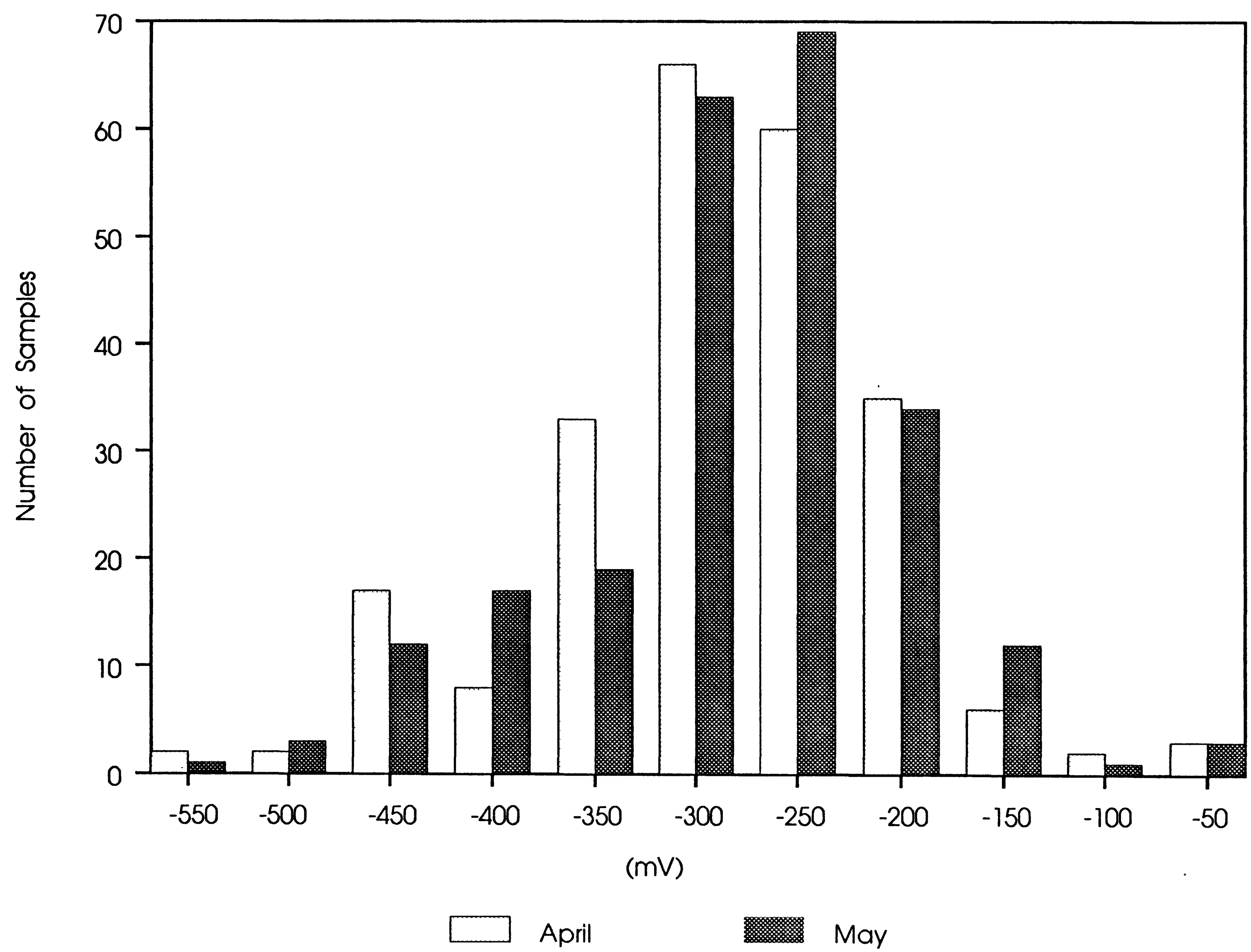

Figure 17. Frequency histograms of half-cell potentials observed during two separate surveys of a section of a concrete bridge deck.

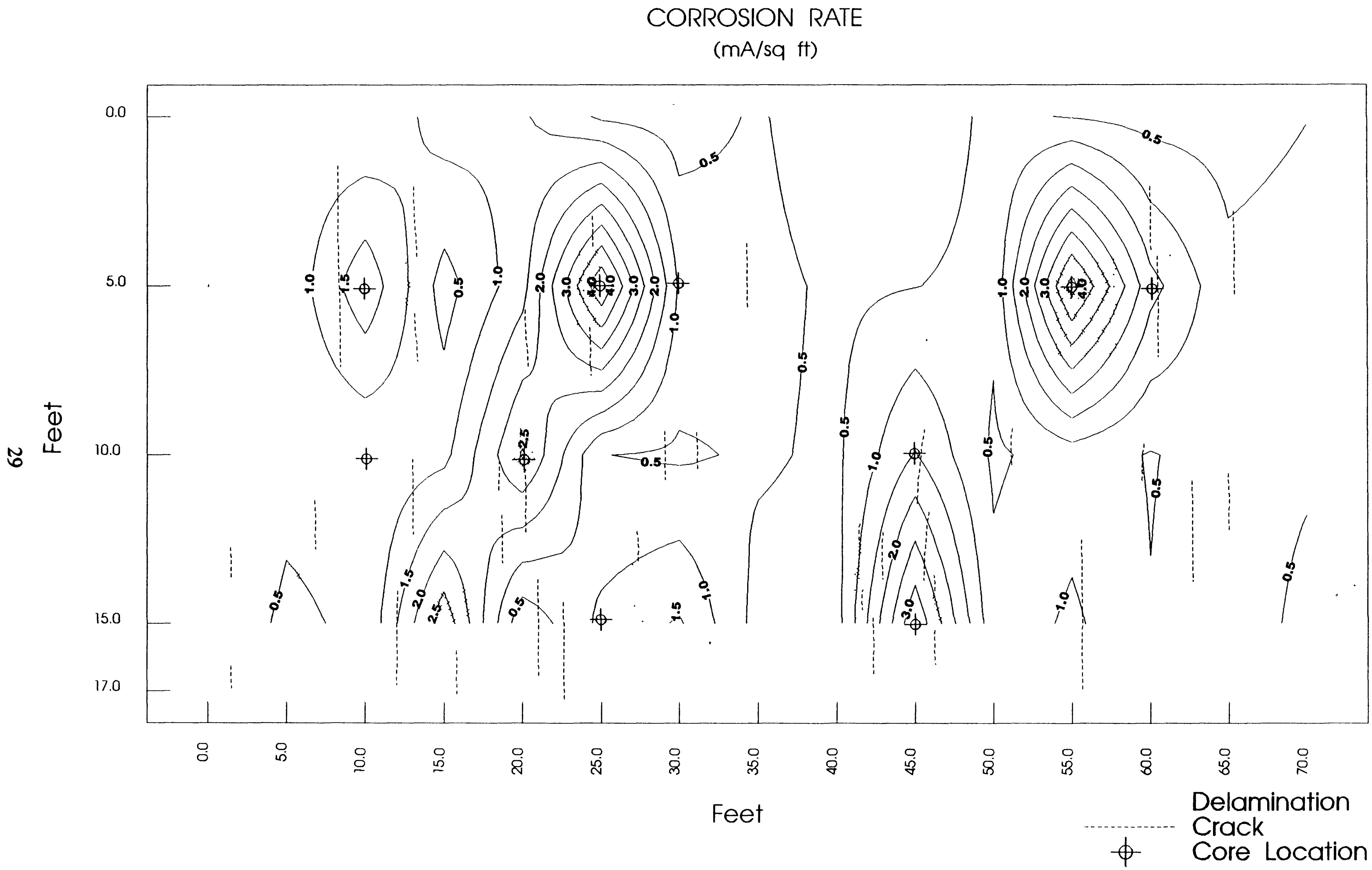

Figure 18. Contour map of rebar corrosion rates observed in a section of Structure 09302.

relatively high rates. The orientation of these "active" areas coincided with the orientation of the top transverse rebars of the deck, which are typically the ones to corrode first. As Figure 18 also shows, the concrete at many of these active areas was already deteriorated—as evidenced by the presence of transverse cracks and delaminations—which is noteworthy. This was not unexpected since the structure had been in service for approximately 20 years at the time of the survey and the rebars may have been corroding for at least 10 years.

From the standpoint of maintenance management, the only benefit of conducting a rebar corrosion rate survey is that it would allow an estimation of when a deck would reach a particular extent of deterioration so that rehabilitation or replacement would be necessary (if the relationship between the rate of rebar corrosion and the rate of concrete deterioration is known). If a concrete is already delaminated or cracked or its rebars are likely to be quite corroded, it follows that it is of no use to measure the rate of rebar corrosion there. Therefore, if a deck does not yet show any sign of corrosion-related damage in the concrete, then the survey of rebar corrosion rates should be conducted over the entire deck; otherwise, the survey should be limited to the portions of the deck that are still undamaged. As shown earlier, a half-cell potential survey of the deck can serve as a screening procedure to locate these damaged areas.

Once a survey has been completed, analysis of the observed corrosion rates would require the use of appropriate statistical parameters. As illustrated in Figure 19, which shows the frequency distributions for the study areas in Structure 09302 and two other decks, the rebar corrosion rates in a bridge deck tend to assume log-normal distributions instead of normal distributions. Half-cell potentials collected in this study have also been observed to assume log-normal distributions in most decks studied. Since a geometric mean is more appropriate for a population that assumes a log-normal distribution than an arithmetic mean, and the latter tends to be larger or equal to the former, it is necessary to use the appropriate statistical parameters (such as geometric mean instead of arithmetic mean) when analyzing rebar corrosion rates obtained from a bridge deck to avoid overestimation of the severity of the rebar corrosion in the deck.

Correlation Between Rebar Corrosion Rate and Metal Loss

The variation of rebar corrosion rates with location in a concrete deck, as clearly illustrated in Figure 18, also raises an interesting question: Is there any possible relationship between the instantaneous corrosion rates that are obtained during a survey and the current physical condition of the rebars? To provide an answer, it is necessary to examine the relationship between corrosion rate and severity of corrosion damage to a rebar or the extent of metal loss on the rebar.

The metal loss (*ML*) on a rebar is, simply stated, an accumulated damage that is a direct function of the individual durations (*Ti*) of corrosion (in, say, number of years) experienced by the rebar and the corresponding individual average annual corrosion rate (*Ri*), i.e.,

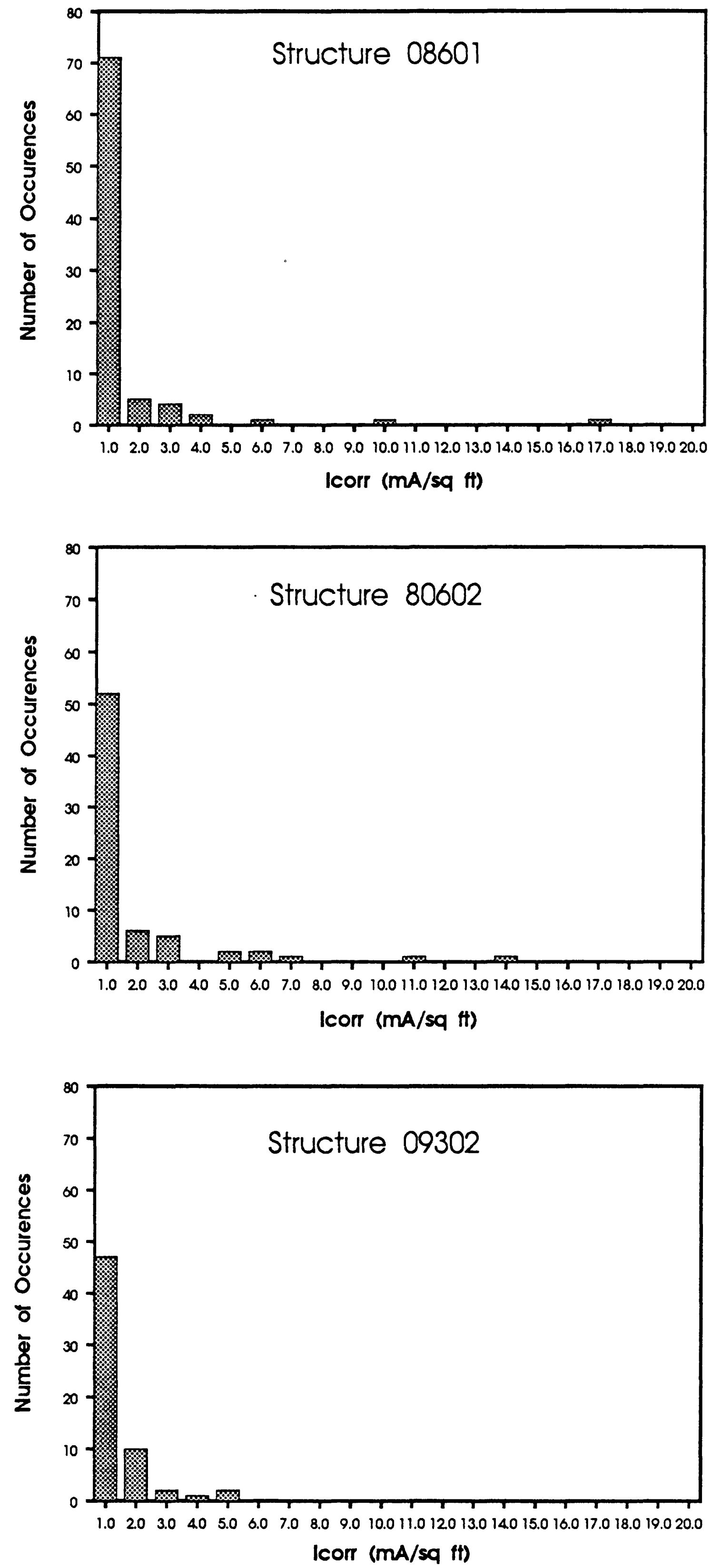

Figure 19. Frequency distributions of rebar corrosion rates observed in three concrete bridge decks.

$$ML = k \sum_i Ri \cdot Ti \qquad [4]$$

where k is the electrochemical equivalent of iron, in this case. If the annual corrosion rates were virtually uniform since the initiation of corrosion, then

$$ML = k\ R \sum_i Ti = k\ R\ Tt \qquad [5]$$

where Tt is the total duration of corrosion on the rebar. According to equation 5, if all the rebars in a deck started to corrode at different times and at different rates, the metal losses on the rebars at any time would likely not correlate with their corrosion rate alone.

However, correlations between metal losses and rebar corrosion rates alone appeared to exist for the bridge decks studied, at least for rebars from the same deck. For example, when the rebars at several selected grid points in the survey area in Structure 09302 were examined, it was observed that the severity of the damage on each rebar was visually directly related to the measured corrosion rate of the rebar (see Table 2). When the damage on the rebar was expressed quantitatively in terms of metal loss, it showed a significant degree of correlation (coefficient of 0.89) with the corrosion rate, as illustrated in Figure 20.

A similar correlation was found in the data for the other survey areas, although the degree of correlation appeared to vary from one deck to another (0.81 to 0.99). This implies that corrosion of at least the majority of the rebars in each deck studied started at virtually the same time, i.e., Tt was virtually the same for the majority of the rebars in a deck; therefore, any difference in the metal losses of the rebars from different locations in a deck was due mostly to differences in their (average) corrosion rates. This also indicates that it could be possible to predict future metal losses (and concrete damage) in statistical terms if the statistical distribution of corrosion rates in a deck could be determined through a survey.

Incidentally, when the data from all the deck areas were combined, the resulting correlation between rebar metal loss and corrosion rate was reasonably good, with a correlation coefficient of 0.85 (see Figure 21). Although such correlation was even less expected, it was not impossible since the ages of the decks were not very different (i.e., 19 to 23 years of service).

It is possible to equate metal loss and damage in a concrete, even in a very generalized manner, if a threshold metal loss at which concrete in bridge decks begins to fracture—due to the pressure of the corrosion products—is known. A range of typical threshold metal losses may have to be determined since the threshold is likely to vary between different decks due at least to differences in the strength of the concrete and the thickness of the concrete cover over the rebars. Examination of the condition of the concrete and the rebars in the limited number (30)

Table 2

RELATIONSHIP BETWEEN MEASURED CORROSION RATE AND REBAR METAL LOSS IN STRUCTURE 09302

Core Location	Rebar Condition	Corrosion Rate (mA/sq ft)	Rebar Length (in)	Final Weight* (lb)	Final Weight* (lb/in)	Metal Loss (lb/in)	Metal Loss (%)
(10, 10)	Very light corrosion in < 5% of surface	0.56	3.373	0.2817	0.08353	0.00137	1.6
(30, 5)	Very light corrosion in < 5% of surface	0.91	3.810	0.3241	0.08506	–0.00016	–0.2
(25, 15)	Light corrosion in 35 to 40% of surface	1.12	3.661	0.3036	0.08292	0.00198	2.3
(45, 10)	Light corrosion	1.53	2.869	0.2432	0.08476	0.00014	0.2
(60, 5)	Pitted in @ 40% of surface	1.68	3.844	0.3150	0.08196	0.00294	3.5
(10, 5)	Corrosion in @ 15% of surface	1.87	3.834	0.3170	0.08269	0.00221	2.6
(20, 10)	Pitted in @ 55% of surface	2.55	3.312	0.2685	0.08107	0.00383	4.5
(45, 15)	Severely pitted in > 50% of surface	3.44	3.804	0.3003	0.07893	0.00597	7.0
(55, 5)	Severely pitted in 80 to 90% of surface	4.21	3.883	0.3049	0.07852	0.00638	7.5
(25, 5)	Severely pitted in > 90% of surface	4.47	3.834	0.3062	0.07987	0.00503	5.9

*The average initial weight of each rebar is estimated to be 0.08490 lb/in.

of concrete cores extracted in this study indicated that the threshold metal loss ranged from 3 to 6 percent (by weight) among the four decks surveyed, which had an average concrete cover ranging from 1.49 to 2.14 in. (No determination of the strength of the concrete was attempted.) As Table 3 shows, this threshold metal loss was considerably higher than the metal losses equivalent to the threshold depths of attack of 16 to 32 μ reported by Hladky et al.[22] However, it is in agreement with those equivalent to the threshold depths of attack reported earlier by the same investigators.[22]

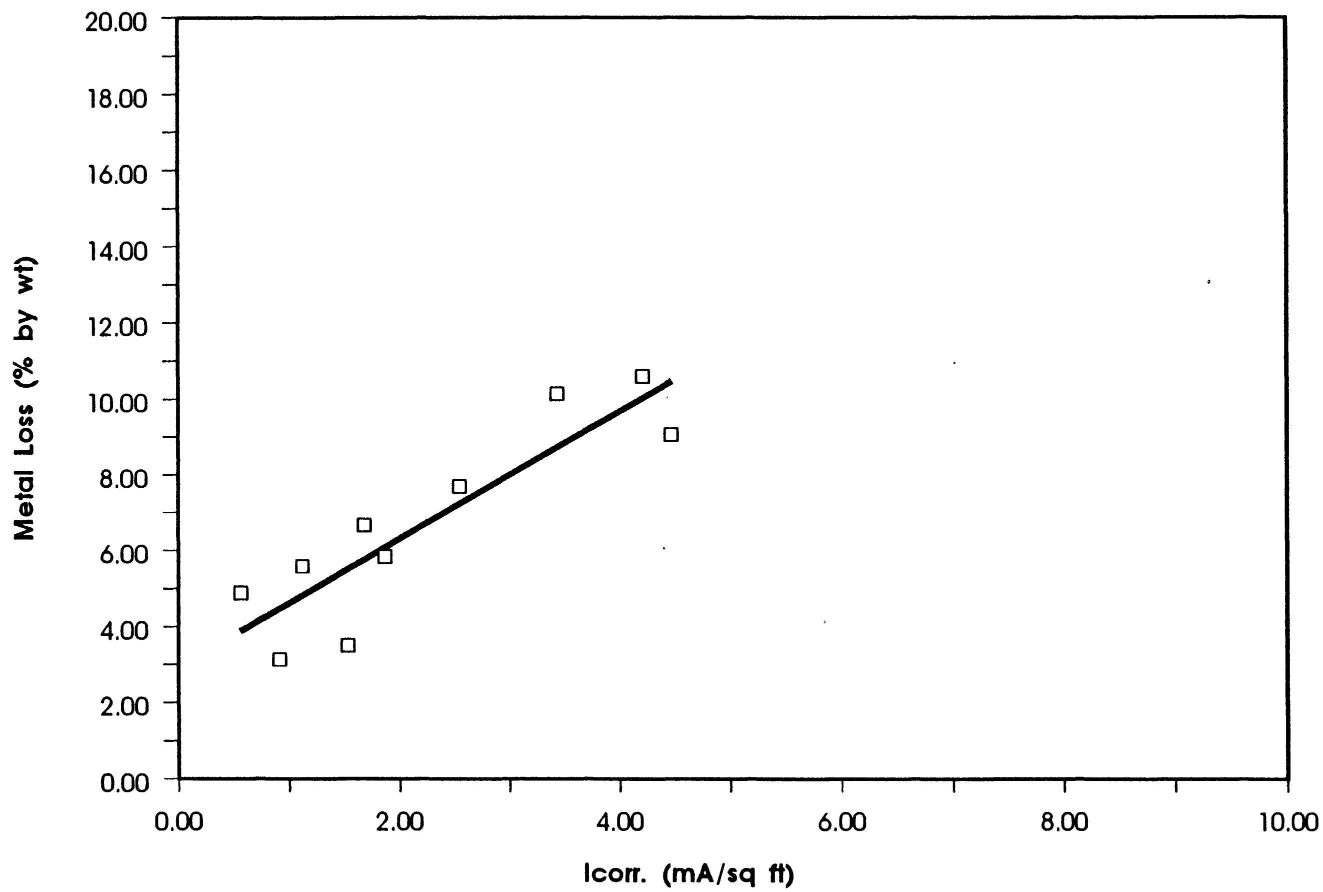

Figure 20. Correlation between metal losses observed on rebar specimens in Structure 09302 and their corresponding corrosion rates.

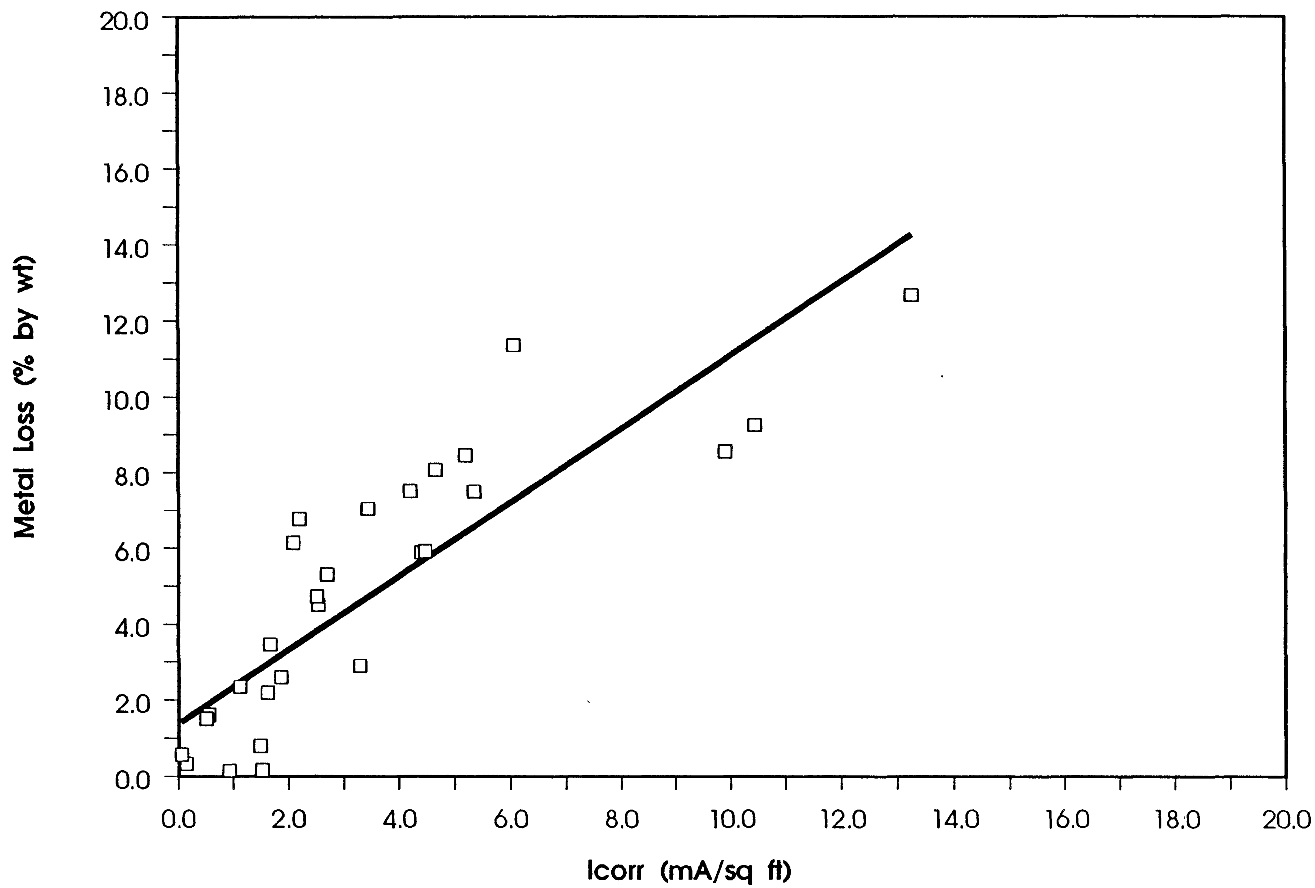

Figure 21. Correlation between metal losses observed on all extracted rebar specimens and their corrosion rates.

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Table 3

ESTIMATED THRESHOLD DEPTH ATTACK AND METAL LOSS ON REBARS TO INITIATE INTERNAL CONCRETE CRACKING

Reference	Depth of Attack (μ)	Metal Loss (% by weight)
14	150 to 300	1.9 to 3.8*
14	16 to 32	0.20 to 0.40*
Present study		3 to 6

*Estimated from reported depths of attack and assuming that the nominal size of rebars was No. 5.

Variation of Rebar Corrosion Rates with Time

As discussed earlier, the rebar corrosion rate measured at each location in a concrete deck represents only the instantaneous rate occurring during the survey. Therefore, to assume that the results of a single survey represent an entire year and the future years would likely lead to either underestimation or overestimation of the remaining service life of the deck. To avoid such error, a practical method needs to be developed for determining the annual average rebar corrosion rate of a deck without the need to repeat frequently the time-consuming survey. To facilitate the development of such a method, the influences of moisture and temperature on the corrosion rate of the rebars in three heavily salted concrete slabs were monitored for approximately 450 days, almost on a weekly basis.

As anticipated, the corrosion rates fluctuated considerably with the seasons, as shown in Figure 22. The fluctuations were no doubt in response to the combined influences of, at the least, fluctuating moisture and temperature in the slabs, as shown in Figure 23. There appeared to be more similarities between the profiles for the rebar corrosion rates of the concrete slabs and the profile for the average moisture content in the slabs than with the profile for the average concrete temperature; this probably indicates that moisture has a greater influence on the corrosion rate of rebars than temperature does.

Separate regression analysis of the data for the three concrete slabs indicated that the best correlation between rebar corrosion rate (I), concrete moisture (M), and concrete temperature (T) was expressed by

$$I = me^{[a + \frac{b}{M} + \frac{c}{T}]} + n \qquad [6]$$

where a, b, c, m, and n are constants. However, the corresponding correlation coefficients varied considerably between the slabs: with 84% for slab 1, 48% for slab 2, and 66% for slab 3 (see Figure 24).

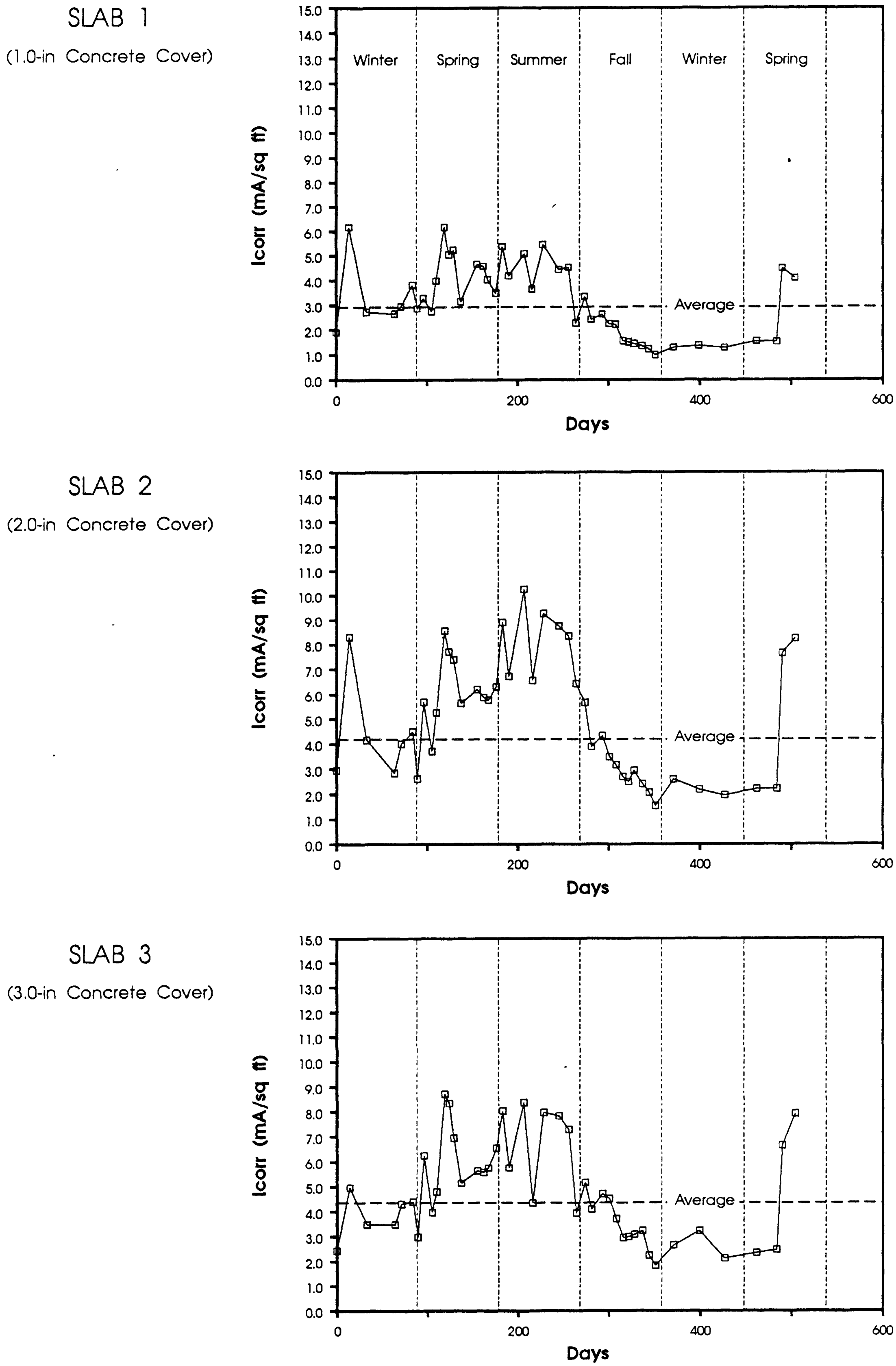
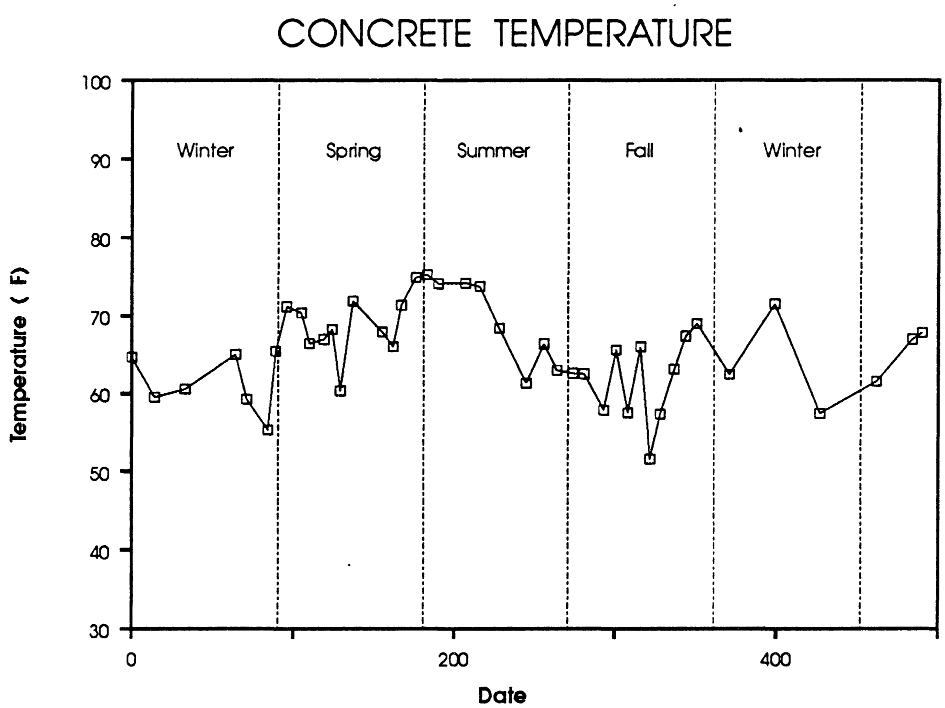


Figure 22. Observed average rebar corrosion rates in test-reinforced concrete slabs of varied concrete covers: (a) 1.0 in., (b) 2.0 in., and (3) 3.0 in.



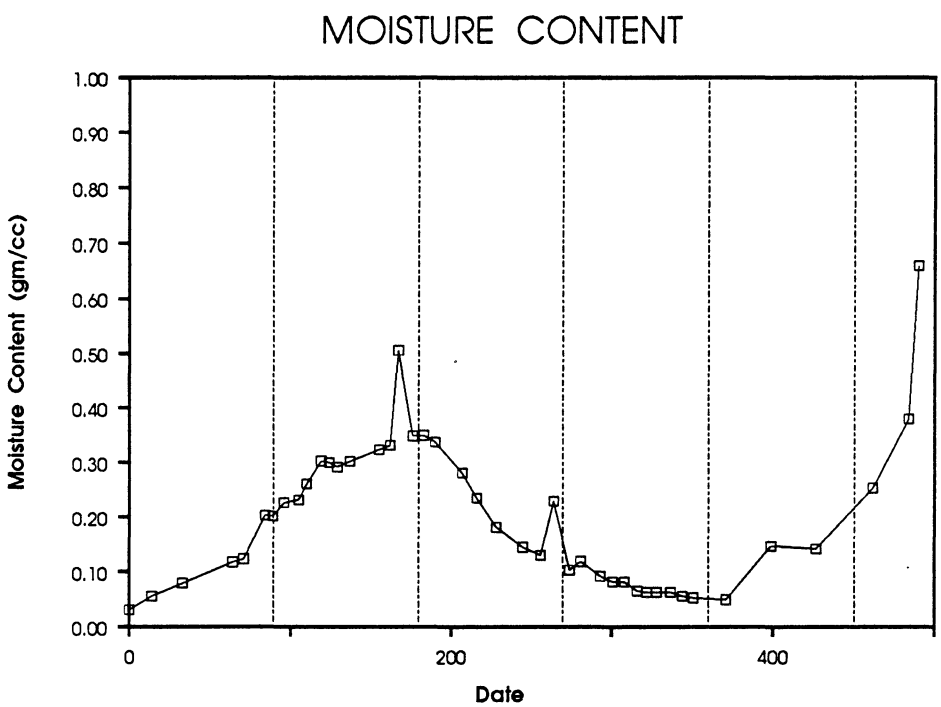


Figure 23. Fluctuations of average temperature and moisture content in test concrete slabs.

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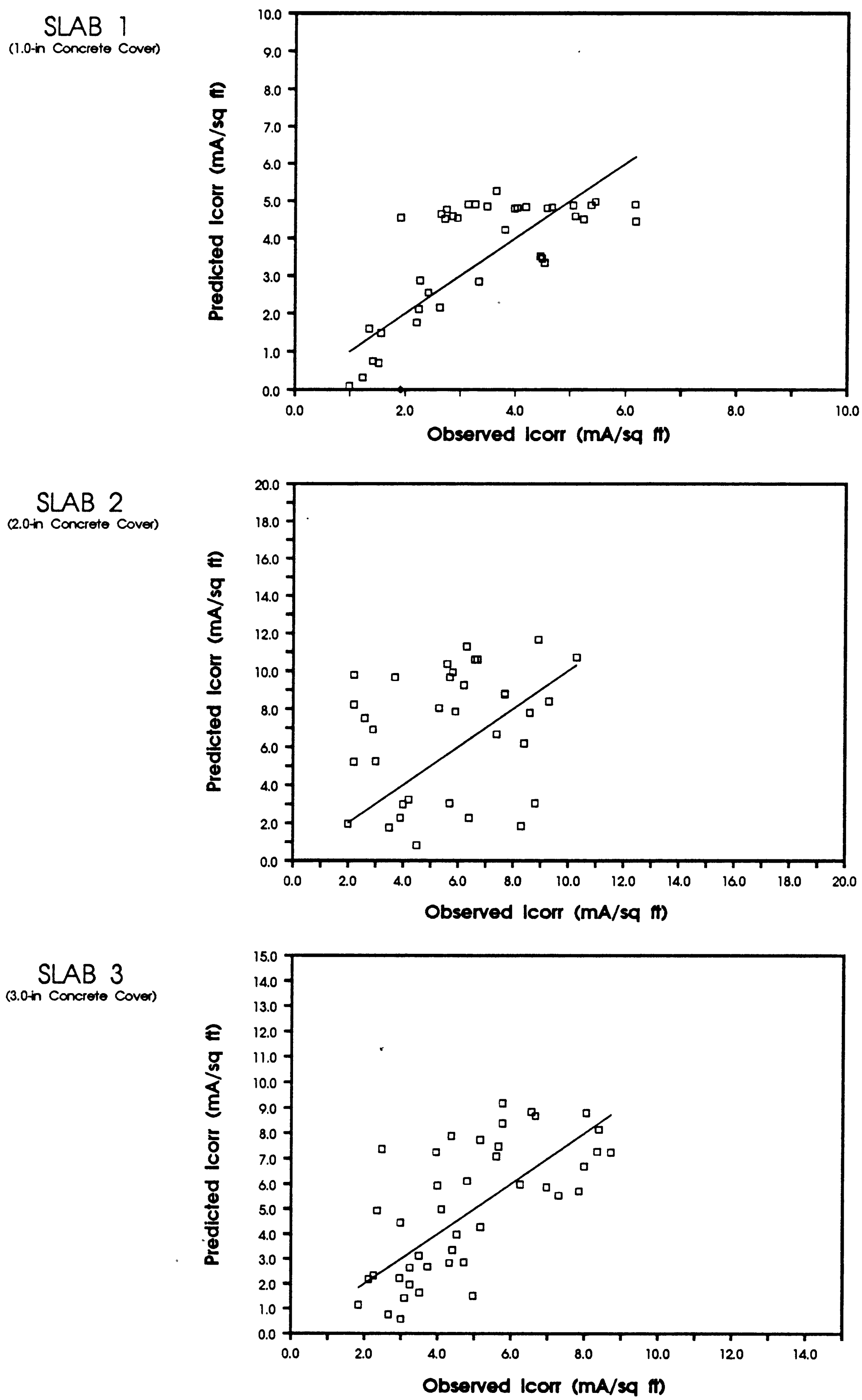

Figure 24. Comparison between observed corrosion rates and those predicted based on concrete moisture and temperature.

It also appeared that the corrosion rates of the rebars during the monitoring period also tended to assume a log-normal distribution (see Figure 25), just as corrosion rates for different locations in a deck. (This is not surprising since other weather-dependent physical variables, such as ambient concentrations of common air pollutants, also tend to assume a log-normal distribution.) This is important, as it indicates the appropriate statistical parameters to use when conducting a statistical analysis of data related to the spatial and temporal distributions of rebar corrosion rates for a deck.

In predicting the remaining service life of a concrete deck using the rebar corrosion rate, one can use (1) the average annual corrosion rate (at an average or the worst location in the deck), (2) the highest annual corrosion rate—for the worst case scenario; or (3) both the lowest and the highest annual corrosion rates—to estimate a range of possible remaining service life. Since the data presented confirmed that the corrosion rates of rebars in a concrete deck vary not only with location in the deck but also with time, determination of the average annual corrosion rate would require a proper statistical sampling procedure that takes into consideration both types of fluctuation, which would undoubtedly be time-consuming and costly.

The worst case scenario would be relatively less involved because it would require surveying a deck at a time when the rebar corrosion rates are likely to be at their maximum, which could be early summer or late spring, assuming that the patterns shown in Figure 22 are typical. A disadvantage associated with such a worst case analysis is that the resulting estimate of remaining service life may be unrealistically low.

The third alternative would be relatively simple, too, because it requires just an additional survey at a time when the rebar corrosion rate is typically at its lowest, which (according to Figure 22) could be midwinter. This analysis is, however, probably more practical than the other two alternatives since it yields a range of possible remaining service life. Although one approach may appear to be better in certain aspects than the others, it must be emphasized that the respective merits of these three alternative approaches to predicting the remaining service life of an existing bridge deck have not yet been studied; the data presented could serve as a starting point for such a study.

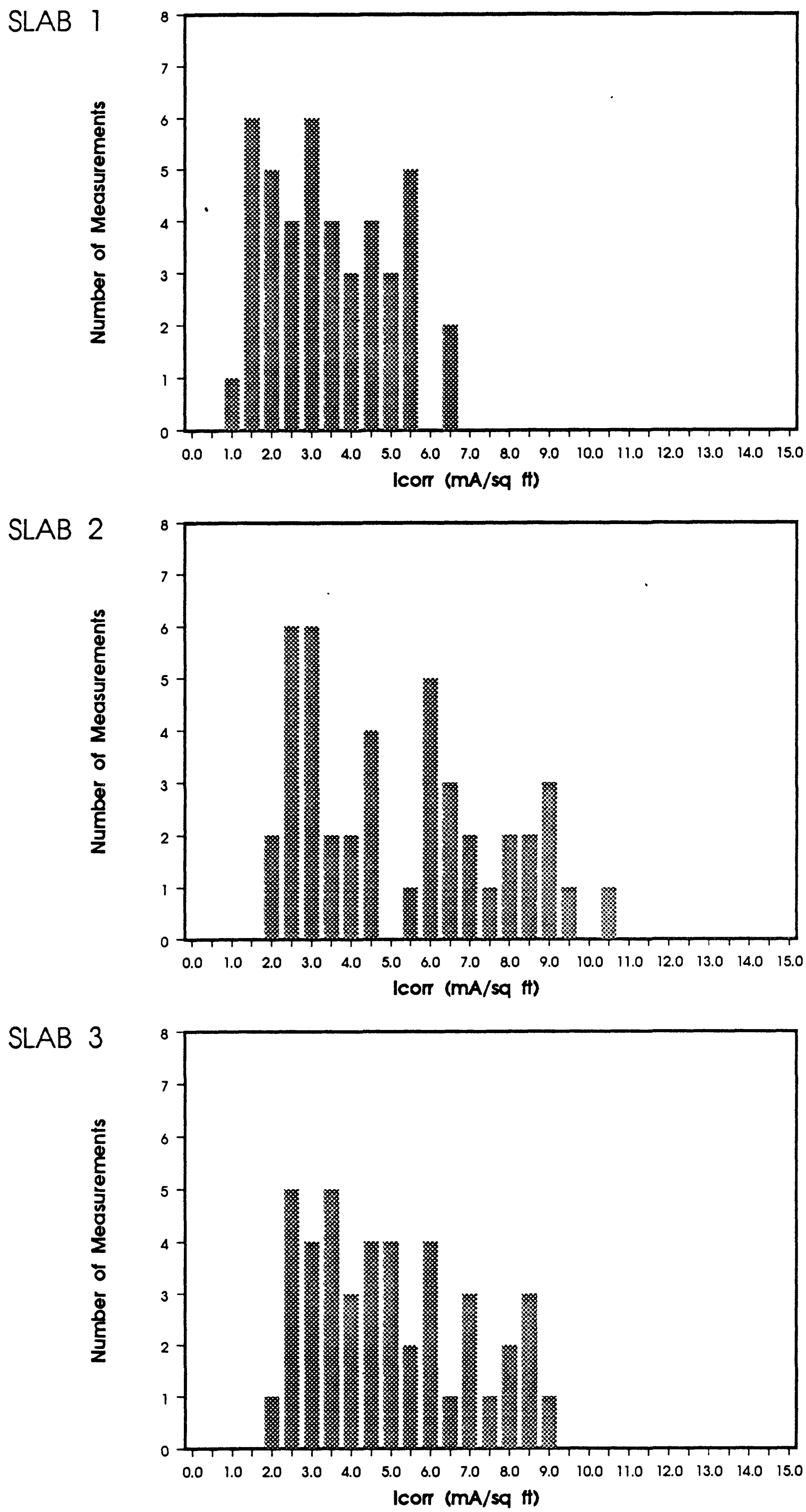

Figure 25. Frequency histograms of rebar corrosion rates observed in test-reinforced concrete slabs.

CONCLUSIONS

1. Because of the influence of fluctuating temperature, moisture, and oxygen in the concrete, the half-cell potentials in a concrete deck can fluctuate from one survey to another. Therefore, in contrast to ASTM C 876 guidelines, the numerical value of each measured potential should not be used by itself as an indicator of the condition of rebars or the surrounding concrete.

2. The localized nature of rebar corrosion (and the associated damage to the rebars and the concrete) lead to the manifestation of corrosion and its associated damage to the concrete in large potential gradients on the surface of a concrete deck. (Such potential gradients may range from –60 to –300 mV/ft.) Therefore, a high potential gradient would serve as a better indicator of the locations of actively corroding rebars and damaged concrete than the numerical values of individual potentials.

3. Because of the localized nature of rebar corrosion and the resulting damage, even the 4.0-ft grid spacing recommended in ASTM C 876 for half-cell potential surveys was found to be too large to allow locating all existing areas of corroded rebars and damaged concrete in bridge decks. Although a spacing of 1.0 ft is preferable, a spacing of no more than 2.0 ft would provide a reasonable balance between accuracy and required sample size.

4. A half-cell potential survey can be used to locate and reasonably quantify areas of active rebar corrosion and corrosion-induced damage in concrete in a concrete bridge deck when conducted using a grid spacing of no more than 2.0 ft and plotting the recorded half-cell potentials in contour maps.

5. Although its accuracy was not verified by another independent method, the 3LP device appeared to be a convenient tool for measuring the corrosion rates of rebars in concrete decks. However, the time necessary to make each measurement with this device (approximately 3 min) would make a complete survey of a bridge deck very time-consuming. Therefore, there is a need to improve the speed of this device or to develop a method that does not require physical contact of the sensing probe with the deck.

6. Rebar corrosion rates appeared to have a reasonable correlation with rebar metal losses. Based on the limited number of rebar samples involved in the study, the threshold metal loss that initiates delamination in concrete was estimated to be 3 to 6 percent, by weight.

7. Rebar corrosion rate varies not only with location in a concrete deck but also with other influencing factors in the concrete that appear to change with time. The frequency of corrosion rates, with respect to location and time, appear to assume log-normal distributions.

8. In view of these fluctuations, a practical survey method for determining the representative rebar corrosion rate of a concrete deck and an analysis method for relating this corrosion rate to the rate of future concrete damage or the

remaining service life still need to be developed. Until such methods are available, from the standpoint of bridge deck inspections and maintenance management, the benefits provided by field measurement of rebar corrosion rates would not be fully realized.

RECOMMENDATIONS

Based on the findings of this study, it is recommended that the current practice of conducting half-cell potential surveys for concrete bridge decks be improved in the following manner:

1. Use a survey grid spacing of preferably 1.0 ft or no more than 2.0 ft (instead of the 4.0-ft spacing suggested by ASTM C 876 or the 5.0-ft spacing used by many state transportation agencies).
2. To improve efficiency, use an array of multiple half-cells, in conjunction with an appropriate data logger and a distance measuring wheel (see the Appendix) to measure potentials simultaneously at multiple grid points at any time. (This would eliminate the most time-consuming and tedious procedure in the survey, which is marking a survey grid on the deck.)
3. Use an appropriate data analysis software to download the values for the half-cell potentials stored in the data logger to a desktop computer for subsequent analysis and plotting of iso-potential contour maps.
4. Identify the deck areas wherein the rebars are in active corrosion and/or the concrete is damaged by the locations of relatively high potential gradients in the contour maps (instead of using the interpretation guidelines in ASTM C 876). Use these results and those of sounding, if available, to estimate the quantity of necessary repair in a deck.

Regarding the use of rebar corrosion rates in projecting future damage to a bridge deck and estimating the remaining service life of the deck, it is recommended that further research be conducted to develop a statistically based survey method for estimating the future rebar corrosion rate in a deck that takes into consideration the variations in corrosion rate with location in a deck and time. To allow projection of future damage to a deck based on such projected future rebar corrosion rates, further research needs to be conducted to define precisely the relationships among concrete delamination, threshold metal loss, thickness of concrete cover, and the tensile strength of concrete.

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APPENDIX

Modified Procedures for a Half-Cell Potential Survey

The half-cell potential survey of a concrete bridge deck consists of marking the deck with a square survey grid, which is followed by the actual measuring and recording of the half-cell potential at each of the grid points. With the traditional equipment, which consists of one reference electrode and a voltmeter, the actual half-cell potential measurement is performed by a member of the inspection crew, who proceeds by reading on the voltmeter the potential at each grid point and so on. To allow speed, a second member records the reading at each point. This is obviously very time-consuming. However, as described earlier in this report, the time required to carry out these procedures can be reduced to 10 percent, conservatively, while reducing the number of inspection members to one by the use of the half-cell array and a data logger—which allow for simultaneous, automatic measurements and electronic storage of potentials at a maximum of four in-tandem grid points.

The marking process consumes an even larger portion of the entire survey time—even when using the large grid spacing of 5.0 ft that many state transportation agencies use. In addition, it requires as many as three inspection members to carry out—two to hold a measuring tape on the deck and a third to traverse the length of the tape and mark the deck at an interval equal to the chosen grid spacing—and repeat this procedure across the entire width of the deck, at intervals equal to the grid spacing.

This entire process can be simplified through elimination of the need to mark the individual grid points on the deck by adapting a distance-measuring wheel of sufficient resolution (±0.1 ft) onto the metal mounting bar for the array of $Cu/CuSO_4$ reference cells (see Figure A1). By using the wheel to monitor the location (at any moment during a survey) of the half-cell array on the deck, with respect to a deck joint, the entire survey procedure would be reduced to the following:

1. Starting at a transverse distance of $(n + 1.5S)$ ft from the curb, mark the longitudinal survey paths with a chalk line at intervals of $(4S)$ ft—assuming that the half-cell array has four reference cells—as shown in Figure A2 where n is the selected transverse distance of the nearest grid point to the curb and S is the selected grid spacing.

2. Starting from a deck joint, which would serve as the starting point on each deck span, roll the half-cell array along the first survey path—using the distance-measuring wheel that is attached to the mounting bar to measure the distance traversed. When the gauge on the wheel indicates m feet from the joint, lower the tips of the reference cells on the deck surface and read and store the half-cell potentials in a preprogrammed data logger.

3. Roll the half-cell array S ft further along the survey path, and again lower the reference cells on the deck surface and read the half-cell potentials.

4. Repeat step 3 until the entire length of the survey path is traversed.

5. Repeat steps 2, 3, and 4 along the remaining survey paths in the span.

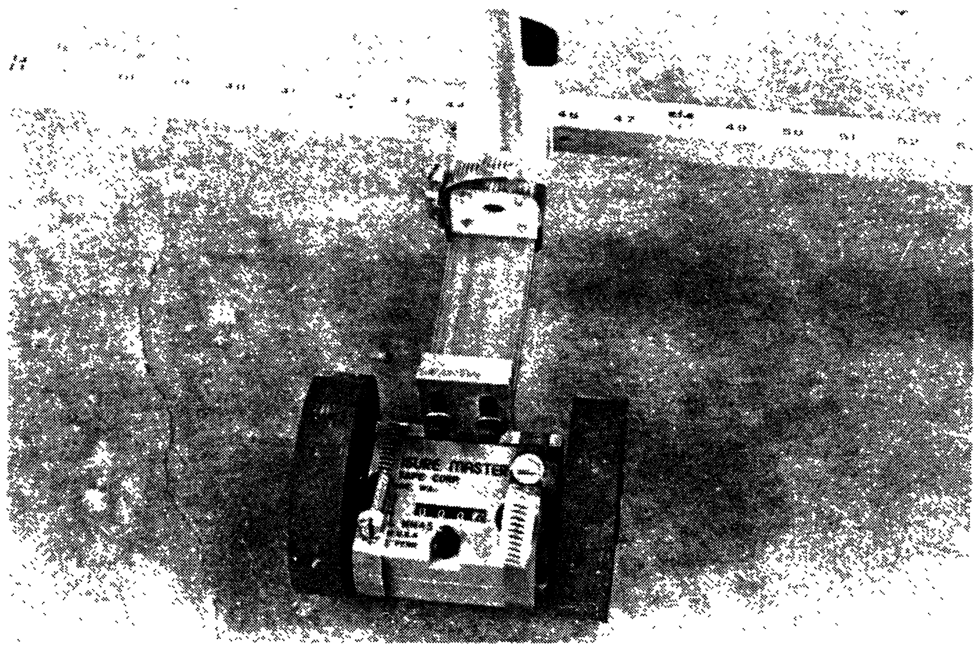

Figure A1. Mounting bar of the half-cell array adapted with a distance-measuring wheel.

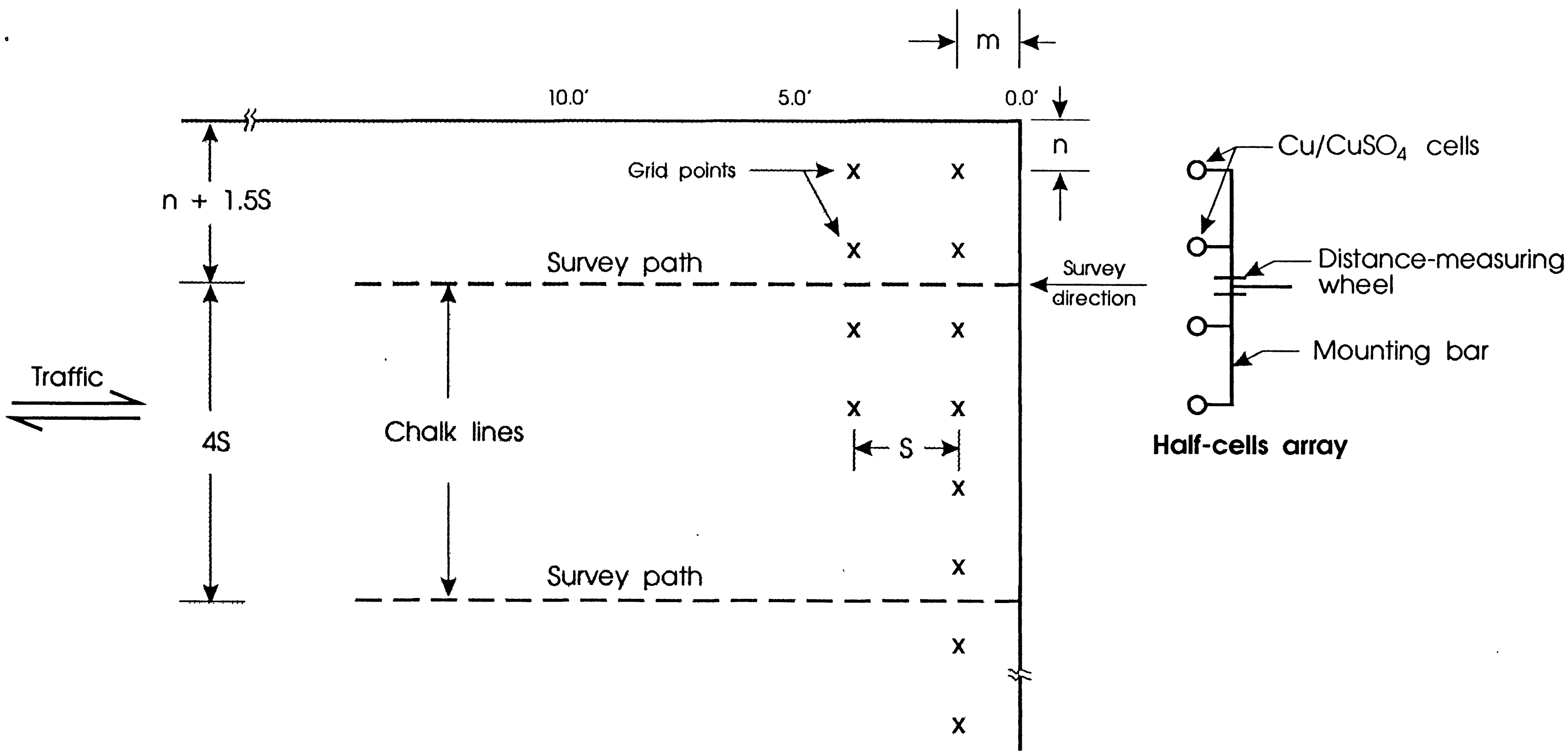

Figure A2. Rapid survey of half-cell potentials on a bridge deck, utilizing a half-cell array adapted with a distance-measuring wheel.

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Since this procedure would require only one inspection member after the survey paths are applied on each of the deck spans with a chalk line and would eliminate the need to mark the individual grid points on the deck, the accrued saving in terms of inspection time would be appreciable.