Inclusion of Rebar Corrosion Rate Measurements in Condition Surveys of Concrete Bridge Decks

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The benefits derived from measuring rebar corrosion rates in existing concrete bridge decks as part of condition surveys were examined using data collected from several bridge decks by traditional inspection methods. Rebar corrosion rates correlated reasonably well with metal losses observed in rebar specimens. Further, it appeared that a metal loss of 3 percent to 6 percent by weight may be the threshold metal loss necessary to initiate delamination in reinforced concrete decks. However, because rebar corrosion rates also vary with location in a deck and with temporal fluctuating conditions in the concrete, their use in estimating the remaining service life for concrete decks will be questionable until an appropriate methodology for determining the representative rebar corrosion rate for a deck and relating such to service life has been developed.

Corrosion of embedded rebars is the primary cause of premature concrete deterioration in concrete bridge decks. Therefore, a combination of inspection methods, such as visual inspection for concrete cracks and spalls, sounding for delamination by chain drags or other techniques (*1*–*4*), measurement of half-cell potentials (ASTM C876), chemical analysis of concrete samples for chloride contents (*5*,*6*), and use of a pachometer for measurement of concrete cover, are used in condition surveys of bridge decks. Together, the first three inspection methods provide reasonably thorough information on the extent of the concrete deterioration already existing in a deck. The information provided by the remaining methods allows the bridge maintenance engineers only to guess what may happen to the rest of the deck in the near future. To eliminate such guessing by allowing the engineers to predict, with reasonable certainty, the amount of possible future corrosion damage to the rebars and the concrete would require 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, FHWA 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 (*7*). Through adaptation of 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 (*8*) 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 (mA),
(dI/dE) = slope of polarization plot (mA/mV),
B_a = anodic Tafel coefficient,
B_c = cathodic Tafel coefficient, and
k = formula constant.

To facilitate its routine use in the field, this prototype 3LP device was modified and made available commercially by Clear (*9*).

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 (*10*). 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 rate.

To predict future damage, the ability to measure the rebar corrosion rate in bridge decks alone is not adequate because, similar to half-cell potentials, the corrosion rate can be expected to vary with both location and time. Because of nonhomogeneity 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 an increase of chloride concentration in the concrete with time to 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

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fluctuations of rebar corrosion rates in bridge decks when attempting to estimate future corrosion damage in terms of projected metal loss in the rebars and its resulting damage to the surrounding concrete.

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

EXPERIMENTAL PROCEDURES

Deck Surveys

In conjunction with another study that involved a half-cell potential survey of bridge decks, five concrete bridge decks were used in this study. During the survey on each deck, a preliminary sounding was conducted to allow selection of a survey area that included numerous delaminated areas. A square survey grid of 2.5-ft spacing was then marked on the selected area for detailed inspection through the use of various traditional inspection methods.

Traditional Inspection Methods

The traditional inspection methods used included visual inspection, sounding with chain drags and a hammer, and measurement of half-cell potentials. Visible concrete distresses such as cracks (especially those above the transverse rebars) and spalls were recorded and carefully mapped to within 2 in. Chain drags and a hammer were used to detect concrete delaminations, which were similarly mapped to within 2 in.

Survey of Rebar Corrosion Rates

The modified 3LP device was used to measure rebar corrosion rates in each deck. Because of the amount of time (at least 2 min) required to perform the measurement at each location, or grid point, a grid spacing of 5 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. 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, described by Clear (*9*), essentially entailed incremental polarization of the rebar to achieve potential shifts of 4, 8, and 12 mV (in 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 by Clear (*9*), the corrosion current was calculated from Equation 1. To allow comparison, the calculated corrosion current (in milliamperes) was instead expressed in terms of corrosion current density (I_d) (i.e., current per unit area of rebar, which was under the influence of the counter electrode), by using the following formula:

$$I_d = \frac{I_{corr}}{A} = \frac{367\ I_{corr}}{BK} \tag{2}$$

where

I_d = corrosion current density (mA/ft^2),
A = surface area of rebar (ft^2),
B = equivalent bar size (number of eighths of an inch), and
K = rebar length beneath the counter electrode (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 concrete cover and accumulated metal or weight losses as a result of 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.

The following procedures were used to determine the metal loss on each piece of extracted rebar:

1. The rebar was thoroughly cleaned by careful chipping with chisels and light sandblasting to remove corrosion products and the portion weakened by pittings.
2. The curved ends of the rebar were cut off to provide straight ends that facilitated 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:

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

where

ML = metal loss,
$(W/L)_f$ = final weight per unit length (lb/in.), and
$(W/L)_i$ = estimated initial weight per unit length of the rebar (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 had no visible sign of corrosion. The major sources of errors in each estimate of metal loss included the cleaning of the rebar specimens (overcleaning or undercleaning) 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 the Concrete Slabs

Three reinforced concrete slabs (3 ft × 3 ft), each with a different thickness of concrete cover (1, 2, or 3 in.) over the top-mat rebars, were fabricated according to the plan shown in Figure 1. The concrete mixture used was a Virginia Department of Transportation Class A4 concrete (*11*). 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 lb/yd^3 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 two locations shown in Figure 1.

Monitoring of Slabs

The slabs were allowed to cure outdoors for at least 60 days before the corrosion rate of the rebars, the concrete temperature, the moisture in each slab, and the outdoor air temperature were monitored. Measurements of the rebar corrosion rate in each slab were restricted to the two same rebars (see Figure 1) 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

The accuracy of the corrosion rates as determined by the commercial 3LP device was uncertain. This is due to uncertainty on the appropriateness of the suggested values of some parameters (*9*) used in the calculation of the 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. A recent comparison

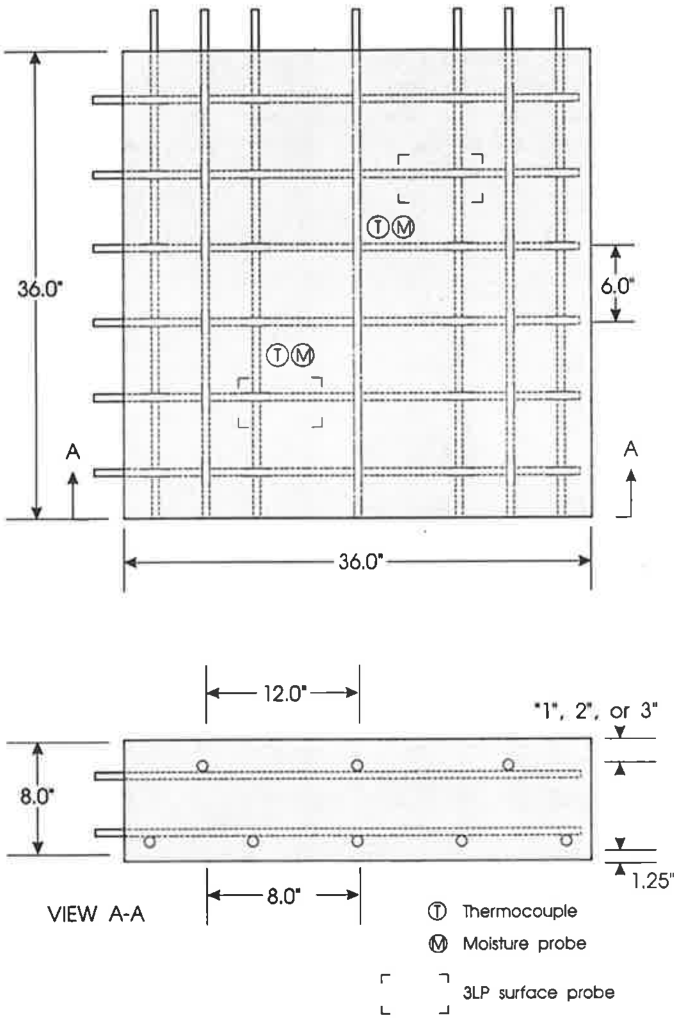

FIGURE 1 Outdoor-exposed concrete slabs used to monitor variation of rebar corrosion rate with seasons.

with other similar devices indicated that the corrosion rate values obtained with 3LP devices may be relatively high (*12*). Nevertheless, the calculated corrosion rates for each survey area were at least suitable for internal comparison.

Variation of Rebar Corrosion Rate With Location

As expected, the corrosion rates of rebars across a concrete bridge deck varied. This variation is shown in Figure 2, 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/ft^2, or 0.054 to 2.2 mil/year. (Higher corrosion rates have been observed in other decks.) This typical contour map shows the presence of several areas wherein rebars corroded at relatively high rates. The orientation of these active areas coincided with the orientation of the transverse top-mat rebars of the deck, which are typically the ones to corrode first. As Figure 2 shows, the concrete at many of these active areas was already deteriorated, as evidenced by the presence of transverse cracks and delaminations. That is not unexpected. 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.

This variation in rebar corrosion rate with location has implications for how a survey of corrosion rates should be conducted for a bridge deck. Because the benefit of such a survey is to allow prediction of when the rebars will reach the threshold corrosion necessary to damage the concrete, it is obviously of no use to measure the corrosion rates in concrete that is already damaged. Therefore, if a deck does not yet show any sign of corrosion-related damage in the concrete, 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. A combination of sounding and a survey of half-cell potentials can be used to screen out the damaged areas before a survey of corrosion rates (see other paper in this Record by Clemeña et al.).

Once a survey has been completed, analysis of the observed corrosion rates would require use of appropriate statistical parameters. As shown in Figure 3, 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 a log-normal distribution instead of a normal distribution. Similarly, half-cell potentials in concrete bridge decks have been observed to assume log-normal distributions, with rare exceptions (see other paper in this Record by Clemeña et al.). Therefore, the use of statistical parameters appropriate to a log-normal distribution (such as a geometric rather than arithmetic mean) would avoid an erroneous estimation of the severity of the rebar corrosion rates in a bridge deck.

Correlation Between Corrosion Rate and Metal Loss

The variation in rebar corrosion rate with location in a concrete deck, as shown in Figure 2, 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

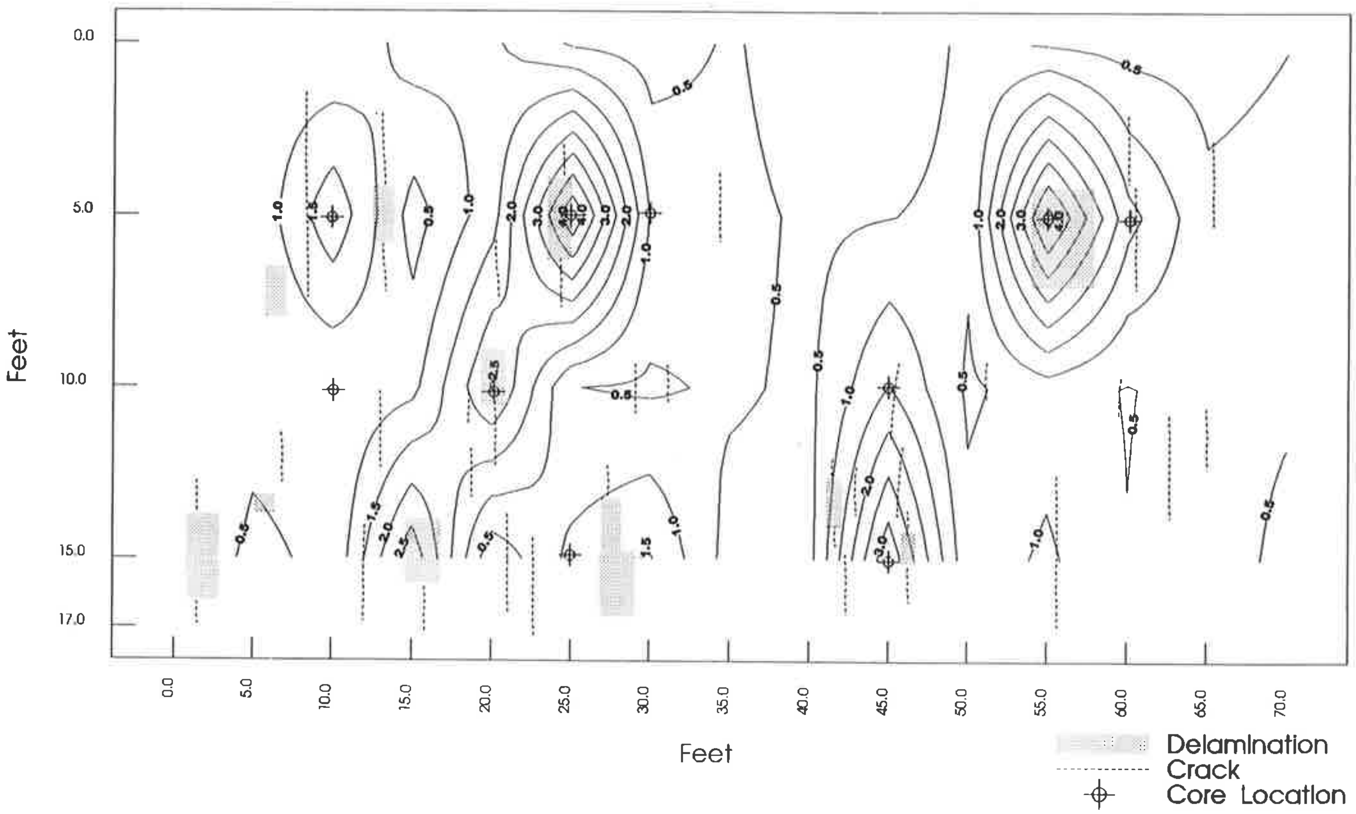

FIGURE 2 Contour map of rebar corrosion rates (mA/ft^2) observed in a section of Structure 09302.

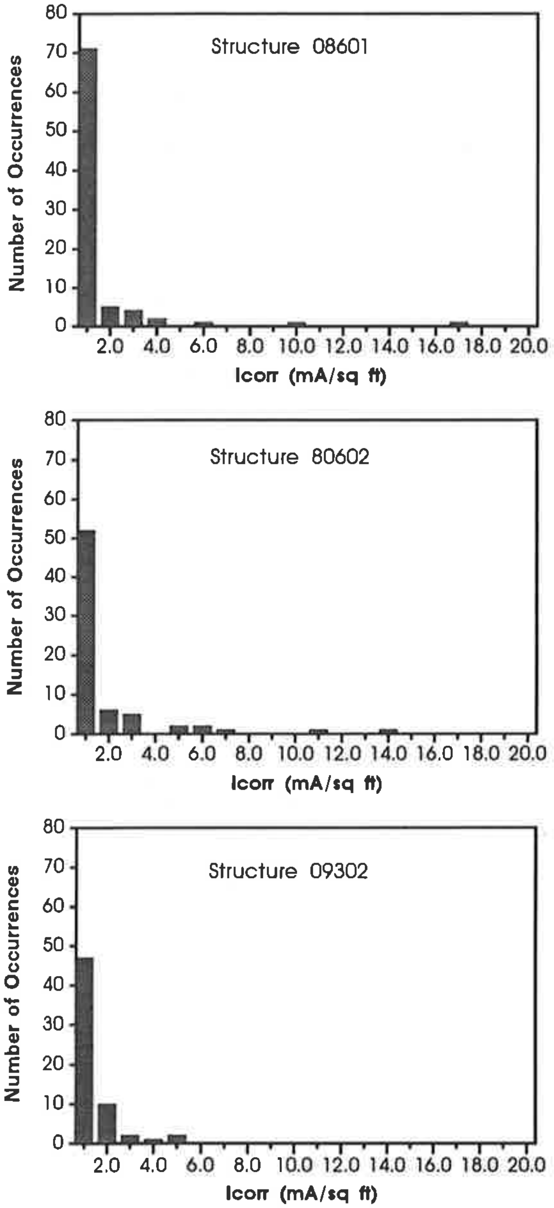

FIGURE 3 Frequency distributions of rebar corrosion rates observed in three concrete bridge decks.

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 (e.g., in number of years) on the rebar and the corresponding individual average annual corrosion rates (*Ri*).

$$ML = \sum_i Ri \times Ti \qquad (4)$$

If the annual corrosion rates were assumed to be practically uniform since the initiation of corrosion, then

$$ML = R \times \sum_i Ti = R \times Tt \qquad (5)$$

where *Tt* is the total duration of corrosion experienced by the rebar. Equation 5 states that, 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 not likely correlate with their corrosion rates 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 1). 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 shown in Figure 4.

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 about the same time. That is, *Tt* was practically the same for the majority of the rebars in a deck; therefore, any difference in the metal losses of the rebars at different locations in a deck was due mostly to differences in the (average) corrosion rates. This also indicates that it could be possible to predict future metal losses (and concrete damage) in statistical terms if the distribution of corrosion rates in a deck could be determined.

Incidentally, when the data from all the deck areas were combined, the resulting correlation between rebar metal loss and corrosion rate was reasonably good and had a correlation coefficient of 0.85 (see Figure 5). Although even less expected, such correlation was not impossible because the ages of these decks were similar (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 because the threshold is likely to vary among decks as a result of 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) of concrete cores extracted in this study indicated that the threshold metal loss ranged from 3 percent to 6 percent (by weight) among the four decks surveyed, on which the average concrete cover was from 1.49 to 2.14 in. (No determination of the strength of the concrete involved was attempted.) As Table 2 shows, this threshold metal loss was considerably higher than the metal losses equivalent to the threshold depths of attack of 16 to 32 microns reported by Hladky et al. (*13*). However, it is in agreement with those equivalent to the threshold depths of attack reported earlier by the same researchers (*13*).

Variation of Rebar Corrosion Rates with Time

As discussed earlier, the rebar corrosion rate measured for each location in a concrete deck represents only the instantaneous rate occurring during the survey. Therefore, to as-

TABLE 1 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.

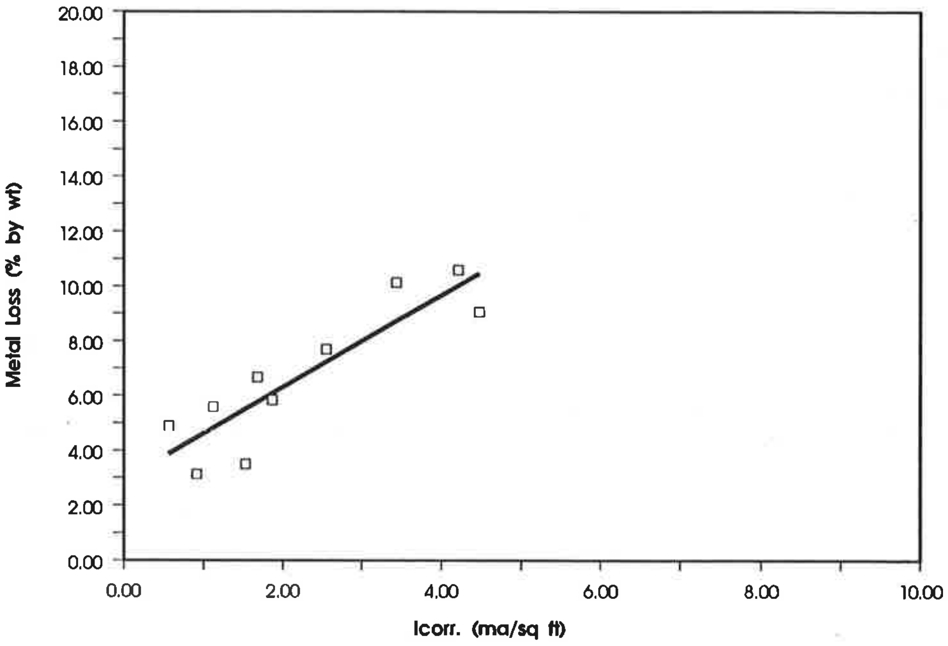

FIGURE 4 Correlation between metal losses observed on rebar specimens in Structure 09302 and their corresponding corrosion rates.

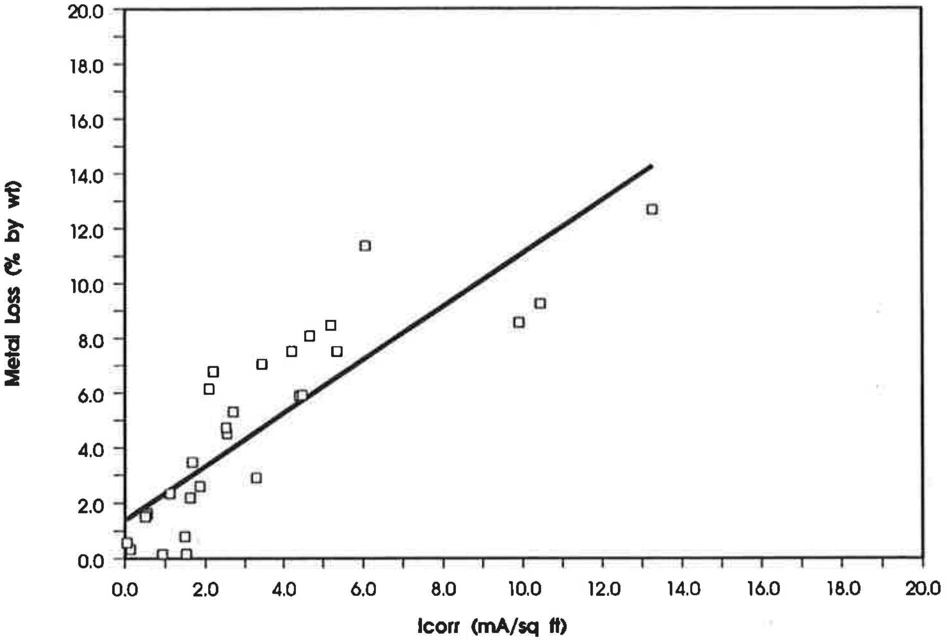

FIGURE 5 Correlation between metal losses observed on all extracted rebar specimens and their corrosion rates.

TABLE 2 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.

sume that the results of a single survey represent an entire year and the future would likely lead to either underestimation or overestimation of the remaining service life of a deck. To avoid such error, a practical methodology for determining the annual average rebar corrosion rate of a deck, without the time-consuming frequent repetition of the survey, must be developed. To facilitate the development of such a methodology, 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 of rebars in the concrete slabs fluctuated considerably with seasons, as shown in Figure 6. The fluctuations were no doubt in response to the combined influences of, at the least, fluctuating moisture and temperature in the slabs, shown in Figure 7. There appeared to be more similarities between the profile 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 does temperature.

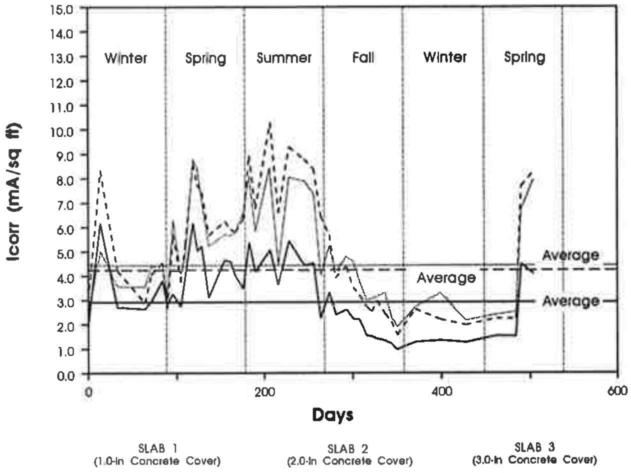

FIGURE 6 Observed average rebar corrosion rates in test reinforced concrete slabs of varied concrete covers: (a) 1.0 in.; (b) 2.0 in.; and (3) 3.0 in.

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

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

where a, b, c, m, and n are constants. However, the corresponding correlation coefficients varied considerably between the slabs: with 84 percent for Slab 1, 48 percent for Slab 2, and 66 percent for Slab 3 (see Figure 8).

It also appeared that the corrosion rates of the rebars observed during the monitoring period also tend to be distrib-

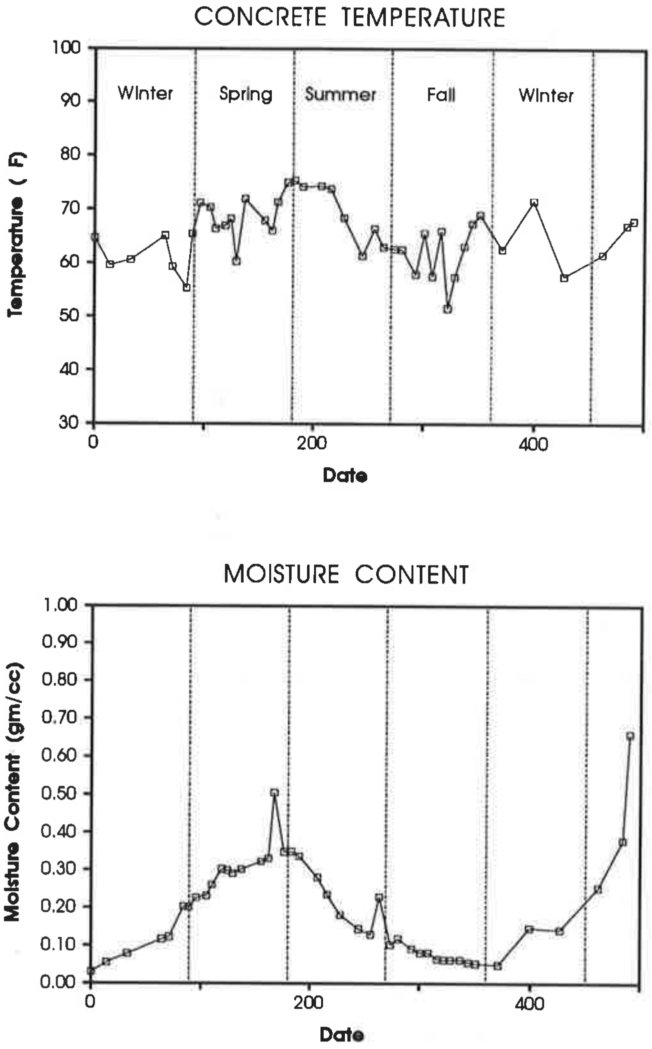

FIGURE 7 Fluctuations of average temperature and moisture content in test concrete slabs.

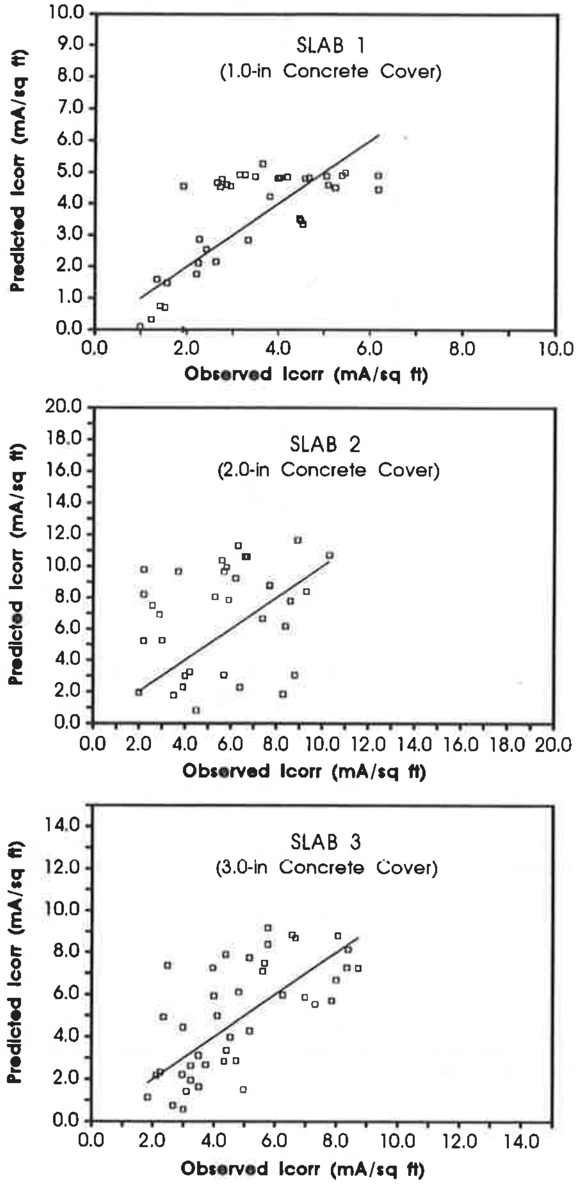

FIGURE 8 Comparison between observed corrosion rates and those predicted based on concrete moisture and temperature.

uted log normally (see Figure 9), just as corrosion rates for different locations in a deck were observed to assume. (This is not surprising because other weather-dependent physical variables, such as ambient concentrations of common air pollutants, also tend to assume log-normal distributions.) This is important because 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 from the rebar corrosion rate, one can use (a) the average annual corrosion rate (at an average or the worst location in the deck), (b) the highest annual corrosion rate (for the worst case scenario), or (c) both the lowest and the highest annual corrosion rates (to estimate a range of possible remaining service life). Because the data presented confirmed that the corrosion rate of rebars in a concrete deck varies 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 account both types of fluctuation, which undoubtedly would 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 rate is likely to be at a maximum, which could be early summer or late spring, assuming that the patterns shown in Figure 6 are typical. A disadvantage associated with such worst case analysis is that the resulting estimate of remaining service life may be unrealistically low.

The third alternative could be relatively simple, too, because it probably requires just an additional survey—at a time when the rebar corrosion rate is typically at its lowest, which (according to Figure 6) could be midwinter. This analysis is, however, more practical than the other two alternatives because it yields a range for 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

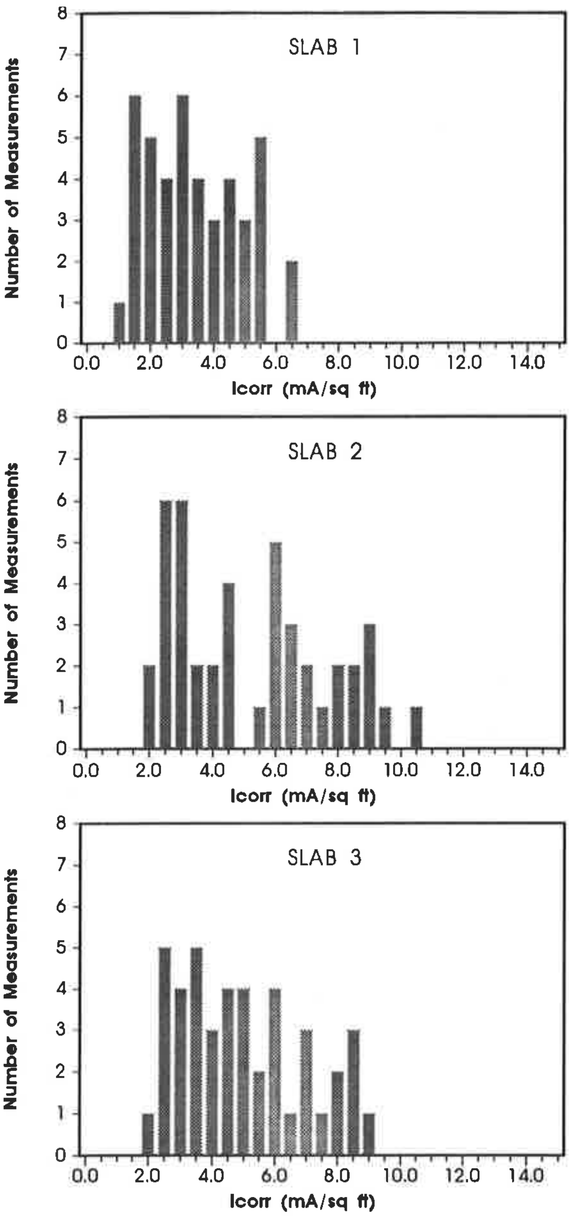

FIGURE 9 Frequency histograms of rebar corrosion rates observed in test-reinforced concrete slabs.

have not yet been studied; the data presented could serve as a starting point for such a study.

CONCLUSIONS

1. Although its accuracy was not verified by another independent method, the 3LP device appeared to be a convenient tool for measuring the corrosion rate of rebars in field concrete. However, the time necessary to make each measurement (approximately 3 min) would make a complete survey of a bridge deck time-consuming.

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

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

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

ACKNOWLEDGMENT

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