

CORROSION AUTOPSY OF A STRUCTURALLY UNSOUND BRIDGE DECK

Richard F. Stratfull, California Division of Highways

An investigation was performed on a 12-year old salt-contaminated, reinforced concrete bridge deck that had to be replaced because of its deteriorated condition. In this investigation, the electrical half-cell potential measurements and effect of chlorides present seem to be related to some threshold amount that changes the steel from a passive to an active state. Beyond this point, the amount of salt present has little or no effect except as it might influence the area of corrosion involved. The chaining or sounding of the deck to locate delaminated concrete performed the function very well but did not necessarily locate the corroded steel. From the observation of the type of cracking, it appeared that the final mode of distress was concrete fatigue. An investigation of actual concrete cover disclosed that there was reinforcing steel corrosion at depths greater than 3 in. It was determined that estimating the pit depth of steel by visually estimating the thickness of rust is not a very useful inspection technique. In this highly salt-contaminated bridge deck, no relation was found between variations in the chloride content of the concrete and the relative severity of the corrosion of the steel.

•AFTER approximately 12 years of service, a highway bridge deck needed replacement because of the corrosion of reinforcing steel. The corrosion was associated with other factors, such as thickness and quality of cover and presence of salt and moisture, as will be discussed. Concrete cracking had advanced to the degree that falsework was necessary to prevent structural failure. As a result, it was decided to make a comprehensive investigation of the condition. This consisted of making electrical potential measurements (1, 2), chaining (3) or sounding (4) the concrete for delamination, measuring the chloride content of the concrete, determining the relation between amount of rusty steel and metal loss, and measuring the water absorption and strength of the concrete.

Although there are many reports in the literature (5) concerning bridge deck deterioration, there are no known reports that describe a comprehensive corrosion evaluation of a bridge that became structurally unsound and required deck replacement.

STUDY SUMMARY

Electrical Potential Measurements

The electrical potential measurements were effective in indicating that the corrosion of the steel was of far greater extent than that which would be indicated by the measured area of concrete delamination. This type of measurement should always be included in any investigation of the corrosion of steel in a bridge deck.

The electrical potential measurements are not considered a reliable indicator of the rate or amount of corrosion of the steel. However, in general terms, the more extensive the area of active potentials, the more probable is the greater amount of corrosion.

This is because it takes time for a large area of corrosion to develop. Therefore, with a longer period of time for corrosion to be active, it is reasonable to assume that the corrosion loss would be greater. The half-cell potentials can be plotted on statistical distribution paper and used to "log" the condition of the bridge according to the percentage of active potentials. For example, for the bridge deck replaced, 94 percent of the potentials were measured to be active, whereas, for an adjacent bridge in which about 2 percent of the deck was repaired, only 30 percent of the values were found to be active.

Concrete Delamination

The chain (3) proved to be an effective and workable sounding device for locating undersurface fractures or concrete delaminations. However, care must be exercised so as not to misinterpret the hollow sound as always representing corrosion-caused concrete delaminations. In this investigation, one location that was thought to be a corrosion-caused concrete delamination actually turned out to be a location of disbonded epoxy membrane overlay. Delamination might also be the result of freeze-thaw action or separation of a grout layer from the underlying concrete due to poor bond. In this bridge, just prior to replacement, approximately 20 percent of the total concrete deck surface was found to be delaminated.

Probable Cause of Deck Unsoundness

Although the cause of the structurally unsound deck is considered to have been accelerated by a somewhat porous concrete, it appears that the principal factors leading to the unsoundness are as follows:

1. De-icing salts were absorbed by the concrete, which resulted in the corrosion of the steel.
2. The concrete was highly absorptive, which apparently resulted in some degradation of concrete strength, perhaps by a combined effect of salt crystallization within the concrete and freeze-thaw action. High water-cement ratios can greatly increase absorption and decrease strength of otherwise good concrete.
3. Corrosion of the steel caused concrete delamination or undersurface fractures and thus reduced the structural section of the deck to a point below that of the top mat of reinforcing steel.
4. As evidenced by the "alligator" type of cracking on the underside or soffit of the deck, the final stage of deterioration appeared to be fatigue failure of the slab, which is a result of "live" loading.

Rust Rating and Pit Depth

An attempt was made to determine how well the actual corrosion pit depth would correlate with a visual observation of the quantity of rust on the steel. On a scale of 0 (no rust) to 5 (Table 1), based on visual observation of rust scale and metal loss of the deformations on the reinforcing steel, only a poor correlation could be made. Therefore, metal loss should be measured, not estimated by rust thickness.

Chloride Content

The chloride content of the concrete in the range encountered was not a positive indicator of deterioration as evidenced by the comparison of the similar amount of deterioration in the two spans that had different salt contents. The significance in a chloride analysis seems to lie within the spectrum of simply determining that its concentration is sufficiently great so as to cause steel to change from a passive to an active state and thus be susceptible to corrosion. Beyond this point of chloride concentration, the control on the incidence of rust or corrosion rate will depend mostly on the moisture content of the concrete (1). Even if there are massive quantities of salt in the concrete, the steel will not actively corrode unless sufficient moisture is present. However, it is assumed that the extent of repairs would depend on the overall level of the chloride

content. For example, in this structure, the chloride content was as great as 21 lb of chloride ion per cubic yard of concrete. Therefore, the use of a membrane to inhibit further intrusion of the salt into an already highly contaminated concrete would be useless.

Concrete Cover

The specified concrete cover over the steel for this bridge was 1½ in. By actual measurement, it was found that the average concrete cover was about 2.0 in. with a range of 1.5 to 3.0 in. Corroding deck reinforcing steel was found not only in the top mat of steel where the concrete cover was as great as 3 in. but also in the lower mat of steel where the concrete cover was greater than 6 in. This could occur as the result of salt water reaching the steel by flowing downward through cracks.

Concrete Compressive Strength and Absorption

As given in Table 2, the compressive strength of the concrete was somewhat less than the strength level normally expected for 12-year-old concrete, and it was variable. The relatively low concrete strength values coupled with the relatively high volumetric absorption of the concrete indicate that the deterioration of the structure was accelerated by a concrete quality initially inadequate for a freeze-thaw and salting exposure. This mechanism for the deterioration seems to be confirmed by the fact that span 3, with its lower strength concrete, showed distress; however, falsework prevented actual failure. Span 1, with an equal amount of concrete delamination, did not show distress. However, it is not necessarily true that, even with a better concrete quality, the service life could be greatly extended.

General Conclusions

No one investigative technique was found to be adequate to answer all questions concerning the cause or extent of the deterioration. In specifically dealing with the corrosion phenomenon, the electrical half-cell potentials and the chloride analysis of the concrete appear to be the most important for determining the extent and level of corrosion activity. The electrical potential measurements have been previously found to be useful for predicting the locations of new concrete spalls (1). Conversely, for determining the area and location of advanced structural damage resulting from a concrete delamination, the drum-like sound of the chain when dragged over delaminated concrete was found to be the easiest and most rapid technique for locating undersurface fractures or spalls.

Because corrosion was found in all previous repairs, the epoxy mortar repairs did not inhibit further corrosion of the steel. Because corrosion was found when the concrete cover was as great as 3 in., it is apparent that an even greater depth of cover would be required for the concrete quality found in this bridge for the protection of the steel from salt.

BRIDGE HISTORY

The Truckee River Bridge is a three-span bridge of approximately 100 ft per span with a traveled way of 28 ft curb to curb. It carries two lanes of eastbound traffic, and the deck slopes to the right. It is a welded steel composite girder structure with four girders per span and reinforced concrete pier and wing abutments. All of the reinforced six-sack concrete was designed to contain 4 to 6 percent entrained air. The concrete aggregate was blended from a single source; the coarse aggregate (1½-in. maximum size) had a water absorption of about 2.8 percent by weight, whereas the fine aggregate had a water absorption of about 4 percent by weight.

The average 28-day concrete strength for the 11 bridges being constructed at the same time was 3,800 psi. There was only one test report for one concrete cylinder that could be positively identified as coming from the Truckee River Bridge. The 28-day compressive strength for the one concrete cylinder was 2,940 psi.

The cement used was an ASTM Type II of low-alkali content meeting California highway specifications, which in various shipments either was ground to ordinary range of fineness or was finely ground to provide high early strength. The records are not clear as to when the types of cement were used, but it is assumed that the fine type was used during periods of low ambient temperatures to accelerate early strength gain.

The bridge is located in the Sierra Nevada Mountains at an elevation of approximately 5,500 ft above mean sea level. The average rainfall is reported as about 24 in. Most of the precipitation is in the form of snow; the average annual snowfall is about 170 in. The temperature range in the area is from about +95 F to -41 F as measured in 1949. The frost penetration in the soil was anticipated to be at a 4-ft depth. As a consequence, the bridge and roadway are heavily salted during the winter season due to snowfall and frost. This bridge was completed in 1959 and was inspected at least annually by an engineer as part of a regular inspection program. Additional inspections were made as warranted. A review of the reports of inspections draws an interesting picture of the progressive deterioration of the deck of the structure.

In the report dated September 1960, it was observed that several large transverse cracks have opened up in all three spans as well as numerous other smaller cracks in the deck. In September 1961, it was observed that the transverse cracks in the three spans were of medium size in span 3 and of small size in the other two spans. Random pattern cracking was of medium size in all spans.

In September 1963, it was reported that, after only four winters, there was considerable pattern cracking of the soffit or the bottom of the deck of span 3 between the third and fourth girders. The following summer, the bridge was overlaid with an epoxy-sand seal. In August 1967, the soffit cracking in span 3 was described as severe. Leaking of water through cracks in the deck extended for more than 20 ft, and cracking was spreading to the deck areas between the second and third girders. The engineer estimated that this area of the deck would require replacement in 1 to 2 years. It was also noted in the 1967 report that seven areas of the deck had spalling and that the soffit of span 1 was starting to show cracking.

In May 1968, the soffit cracking in span 3 had increased to the point that its structural soundness was in question, and the placement of timber supports was recommended. The placing of the timber supports as well as the filling of 35 new deck spalls with an epoxy mortar and the placing of a new epoxy-sand seal in the slow traffic lane were completed during the summer of 1968. The epoxy mortar was made with a ratio of 5 parts pea gravel to 1 part epoxy by volume (1).

Additional deck spalling occurred and was repaired prior to the October 1969 report, in which it was recommended that the 11 additional spalls be repaired and the bridge deck be scheduled for replacement.

FIELD WORK

Initially, a reference grid was laid out on the deck on a 4-ft square pattern. These points were spotted on the deck using spray paint. The deck was then chained to delineate the areas of unsound or delaminated concrete. A solid ringing sound is normally heard as the chain is dragged on sound concrete, but there will be a dead or drum-like sound when delaminated areas are encountered. The delaminated areas were outlined on the deck using spray paint, and then they were plotted on cross-section paper for correlation with other operations. The existing deck patches of epoxy mortar and asphalt concrete, as well as the areas of the deck being supported by timber falsework, were also plotted on this sheet. Figures 1 and 2 show portions of spans 1 and 3 with delaminated areas being shaded and patched areas crosshatched.

Electrical potential measurements using a saturated copper-copper sulfate half-cell were taken on the 4-ft grid pattern on the top of the deck with additional readings at anomalies. These readings were reduced to contours and overlaid on the plot showing the delaminated areas. Figures 1 and 2 show these contours as a dashed line.

Twenty-two 4-in. cores were taken through the deck at various locations. The core locations were chosen to include all stages of deterioration and all ages of epoxy mortar patches. These locations were further chosen to include deck reinforcing. All cores

were identified and their locations recorded. They were then analyzed for compressive strength, absorption, and chloride content.

The first phase of the deck removal consisted of removing the concrete from both sides of both exterior girders by striking the concrete with a pneumatic-mounted hydro-hammer. This operation exposed the reinforcing steel from the curb lines to about 2 ft out into the traveled way. It was observed that a fairly large percentage of this steel showed from minor to extensive corrosion. However, the chaining had indicated very little concrete delamination along these areas even though the half-cell potential readings indicated active corrosion.

During the concrete removal, the actual amount of concrete cover over the steel was measured. Along the left (facing the direction of traffic movement) edge of the deck, it was found that the average concrete cover over the steel was 2.36 in. with a range of 2.0 to 3.0 in. The concrete cover over the steel along the right edge (lower) of the deck averaged 2.25 in. with a range of 1.5 to 2.75 in. These measurements were made with a ruler.

Along the middle of the deck, a "pachometer" was used for measurement, and the indicated average amount of concrete cover over the steel was 1.78 in. with a range of 1.63 to 2.0 in.

Three selected slabs, approximately 7 by 12 ft, from specific areas of the deck being removed were set aside for recovery of the reinforcing steel. The slabs were from (a) an area of little delamination or cracking of the concrete, (b) an area that showed severe cracking on the soffit and epoxy patches of different ages on the top of the deck, and (c) the timber-supported area that was structurally unsound. Upon removal, the location of the reinforcing steel was identified. A detailed corrosion evaluation of the steel was made. The locations of these three test slabs are shown in Figures 1 and 2.

The abutments and piers were examined visually for evidence of corrosion-related deterioration. Pier 3 showed the only visual evidences of corrosion in that there was corrosion-caused spalling of concrete on the top and bottom of the pier cap. The top of the pier cap sloped with the cross slope of the deck, and corrosion-caused deterioration was found only on the low end of the cap, where salt-contaminated deck drainage water would flow. Electrical potentials were taken on the top of the cap. At one location the average of four readings was -0.24 V, and there was no visible evidence of corrosion. At another location, the average potential was -0.53 V, and corrosion of the steel and concrete spalling were observed.

Samples of concrete were taken in the area of high potentials for chloride analysis. The results of this analysis showed that the concrete at the top of the cap contained the equivalent of 31 lb/yd³ of chloride ions and at the bottom contained 10 lb/yd³.

CONCRETE QUALITY

Twenty-two 4-in. diameter cores were taken through the deck for laboratory analysis, 10 cores from span 1 and 12 from span 3. Ten cores were in areas of deck spalling that had been repaired with epoxy mortar. All 10 cores in patched areas and 8 cores in unpatched areas were taken so as to include deck reinforcing. The reinforcing was found to be corroding at all epoxy patches. When making epoxy repairs, it is required that the steel be sandblasted; therefore, it is assumed that any corrosion products present must have been generated as a result of corrosion occurring after the time of repair.

Four cores, two each from spans 1 and 3, contained neither epoxy patches nor reinforcing steel. They were tested for 28-day absorption using California Test Method 538-A. These four cores were then checked for compressive strengths (Table 2), which appear to be lower than would be expected for 12-year-old concrete. Table 2 gives the 28-day volumetric absorption of the concrete between 14.09 and 16.61 percent. In previous testing, the author has generally observed that, in similar concrete mixes, the volumetric absorption would be in the range of 13.5 to 15.0 percent. The greater absorption of the bridge deck concrete is probably influenced by the absorptive aggregates.

The 22 cores were then analyzed for chloride ion intrusion. Each core was cut into 1-in. thick disks that were pulverized and analyzed using a "wet" analysis for control and X-ray diffraction (Table 3).

The data given in Table 3 show that a significant quantity of chloride ion has been absorbed by the concrete. Even though the chloride content was greater in span 1, there was no difference in the area of delamination between this span and span 3, which was structurally unsound.

In previous work (1), it was observed that, as long as the salt content was sufficient to support corrosion, then the presence or absence of corrosion depended on the moisture content of the concrete.

The threshold value of salt as measured for previous work was found to be about 1.5 lb of chloride per cubic yard of concrete. Localized distribution can of course vary greatly.

It seems clear that, once the minimum level of salt has been reached to cause corrosion, even greater concentrations play no further significant role in the corrosion of the steel. This is emphasized by comparing the chloride concentrations in the deck of this bridge, about 13 lb, to a previously reported (1) bridge that was corroding and eventually removed and only contained an average of about 1.5 lb of chloride ion per cubic yard at the level of the steel.

HALF-CELL POTENTIALS, CONCRETE DELAMINATION, AND RUSTING STEEL

In previous work (6), it was shown that, for half-cell potential values more negative than about -0.35 V to the copper-copper sulfate half-cell ($\text{Cu} \cdot \text{CuSO}_4$), the steel in concrete appears to be active as a result of salt intrusion. The value range of -0.30 to -0.35 V seems to be an inconclusive area, whereas for values less negative than -0.30 V ($\text{Cu} \cdot \text{CuSO}_4$) the steel is passive or chemically inhibited from corrosion.

To determine the relation among the half-cell potentials, rusting steel, and concrete spalls or delamination, the bridge was completely surveyed by electrical potential measurements, chained to find loose or delaminated concrete, and visually observed to locate any evidence of rusty steel.

Figures 1 and 2 show that the high potential measurements on nearly the whole deck surface indicated a massive area of active corrosion of the steel. Although not shown in Figures 1 and 2, visual observations of the steel during the concrete removal show significant areas of rusting steel even though the concrete in these areas showed no indication of spalling or delamination.

For the entire bridge deck about 94 percent of all of the half-cell potentials of the steel were in the active or corroding range. In contrast, approximately 20 percent of the deck area was found to be delaminated or to have undersurface concrete fractures at the top layer of the steel.

Figure 3 shows the electrical potentials and the concrete delamination that were measured on the relatively good slab. All of the half-cell potentials are in the active range of -0.35 to -0.55 V and denote the likelihood of significant corrosion of the steel. In Figures 3, 4, and 5, the locations of steel that had visible rust were plotted as solid lines. If the top mat of reinforcing had no rust, it was not shown.

Figure 3 shows an undersurface delamination that apparently is in a location of un-rusted steel. Because this area of concrete still had the epoxy-aggregate membrane on the surface, it is believed that the hollow sound that caused a recording of a concrete delamination might have been the result of disbonding between the epoxy membrane and the underlying concrete surface.

As shown in Figures 3, 4, and 5, the relation between rusted steel and concrete delaminations can be poor. This is not to say that the corroding steel does not cause delaminations, but it does indicate that the amount of rust that forms on steel causing a spall can be highly variable. It is obvious that the sounding of concrete only relates to the condition of the concrete and not necessarily to the condition of the steel.

Figure 6 relates the half-cell potential measurements to the observed condition of the bridge by showing the cumulative frequency distribution on three bridges. The upper

Table 1. Rusting in test deck slabs.

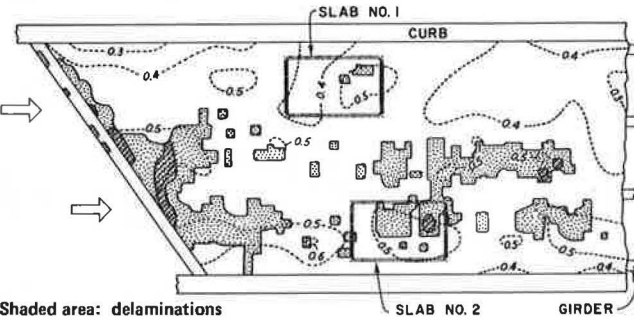
Slab Number	Rust Rating (percentage of length of steel)					No Rust
	1	2	3	4	5*	
1 (good condition)						
Top transverse bars	13	3	4	5	4	71
Top longitudinal bars	<1	1	-	-	-	99
Bottom longitudinal bars	-	-	-	-	-	100
Bottom transverse bars	<1	1	-	-	-	99
2						
Top transverse bars	12	3	4	3	37	41
Top longitudinal bars	8	2	1	1	7	81
Bottom longitudinal bars	1	<1	1	1	3	94
Bottom transverse bars	9	3	2	<1	<1	86
3 (unsound)						
Top transverse bars	7	2	6	4	62	19
Top longitudinal bars	5	3	5	8	32	48
Bottom longitudinal bars	1	1	2	3	14	79
Bottom transverse bars	18	5	9	11	7	50

*Rust rating 5 represents heavy rusting.

Table 2. 28-day absorption-compressive strength of 4-in. concrete cores.

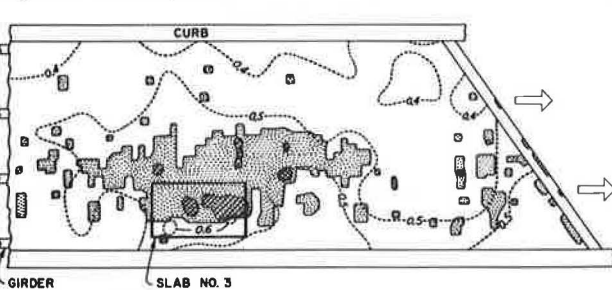
Location	Core Number	Compressive Strength (psi)	28-Day Absorption (percent/volume)
Span 1	2	3,690	16.61
Span 1	3	4,080	16.38
Span 3	11	3,020	14.42
Span 3	20	2,695	14.09

Figure 1. Part of span 1.



Shaded area: delaminations
 Crosshatched area: epoxy patches
 Dashed line: electrical potential contours

Figure 2. Part of span 2.



Shaded area: delaminations
 Crosshatched area: epoxy patches
 Dashed line: electrical potential contours

Table 3. Average chloride-ion distribution as a function of slab depth.

Span Number	Depth (lb chloride-ion/yd ³ concrete)							
	0 to 1 In.	1 to 2 In.	2 to 3 In.	3 to 4 In.	4 to 5 In.	5 to 6 In.	6 to 7 In.	7 to 8 In.
1	23.5	13.2	4.4	1.3	1.4	0.9	1.3	0.5
3	18.3	12.4	2.9	0.9	0.6	0.7	0.3	-

Figure 3. Slab showing little delamination.

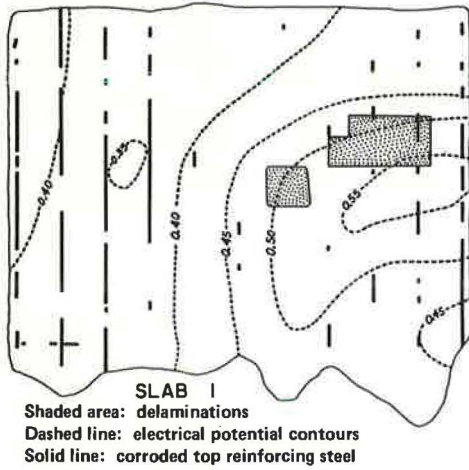


Figure 4. Slab showing severe cracking of soffit and epoxy patches.

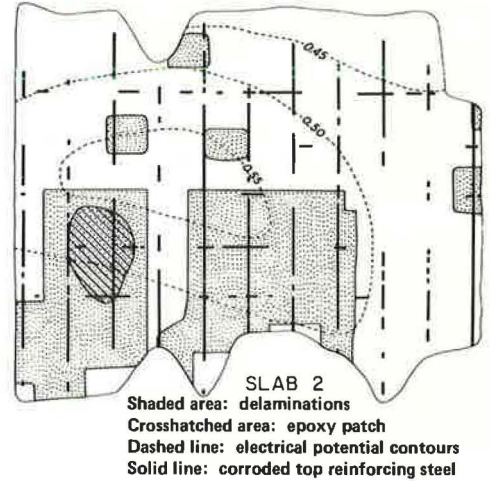


Figure 5. Slab from area of timber supports.

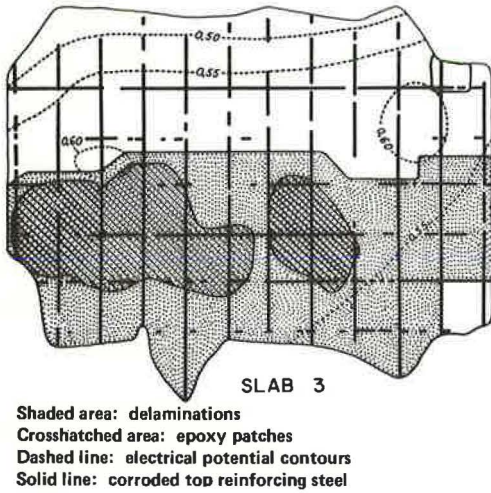


Figure 6. Distribution of half-cell potentials.

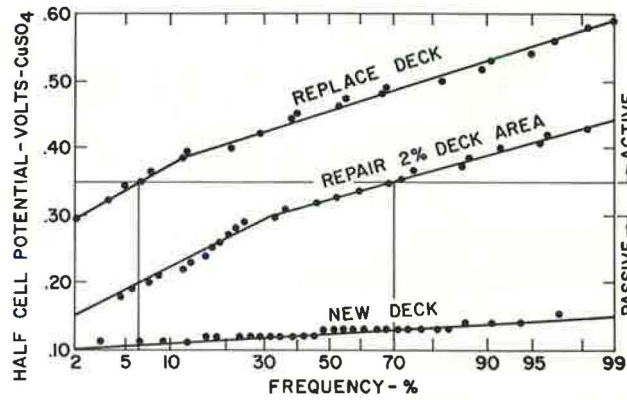


Table 4. Visual rusting and actual pitting of steel and concrete.

Rating	Pit Depth (mil)		Visual Condition
	Range	Avg.	
1	0 to 46	15	Trace to light rust
2	0 to 57	17	Medium to heavy rust
3	3 to 58	18	Very heavy rust
4	5 to 57	22	Light pitting and corrosion of deformations
5	21 to 97	52	Heavy pitting and corrosion of deformations

Table 5. Average potential values found in test slabs.

Characteristic	Slab		
	1	2	3
\bar{X} (V)	-0.44	-0.47	-0.53
Active potential (percent)*	91	98	100
Concrete delamination (percentage of total area)	4	34	58
Top bar rust (percentage of total length)	29	50	81
Rust of all bars (percentage of total length)	19	28	55

*Percentage of total measurements that exceed -0.35 V, Cu-CuSO₄.

distribution curve shown in Figure 6 is for the bridge under investigation; the middle distribution curve is for another bridge. For the bridge deck that required replacement (lower curve, Fig. 6), about 94 percent of all of the potential measurements made showed that the steel was active (more negative than -0.35 V, $\text{Cu} \cdot \text{CuSO}_4$) or corroding. For the deck that was repaired, only 30 percent of the measurements showed active half-cell potentials, and the new deck had no measured active potentials.

The data shown in Figure 6 may reflect a means for "logging" the condition of a bridge according to the percentage of active or passive potentials. However, the distribution curve can have a break; therefore, one must exercise caution in mathematically calculating a mean or standard deviation for the half-cell potentials.

REINFORCEMENT

The reinforcing steel removed from the three slabs recovered from the deck was evaluated as to degree of corrosion on a visual rating of 1 to 5 (Table 1). The slabs contained four layers of steel, two layers of top steel, the upper layer being transverse and the bottom being longitudinal, and two layers of bottom steel, the upper being longitudinal and the bottom being transverse. Truss bars were considered top steel when in the top plane and bottom steel when in the bottom plane.

The data given in Table 1 show that the amount and severity of rusting increased with the severity of the original physical condition of the test slabs. Slab 1 was from the area in better physical condition, whereas test slab 3 was from the area supported by falsework. In test slabs 2 and 3, there is a significant length of bottom reinforcing that was found to have significant rusting.

The relation of a visual rating of rust to the actual amount of metal loss was determined by sandblasting random pieces representing each visual rust rating and measuring maximum pit depths. It became obvious that estimating and logging degrees or amounts of rust can be misleading. Pit depths for 20 samples of each visual rating varied as given in Table 4.

The amount of metal loss to produce a visual volume of rust is highly variable as given in Table 4. This further illustrates the lack of a significant relation between delaminated concrete and rusted steel. In concrete of low absorption where small amounts of rust will cause high disruptive pressure (no absorption of the rust products by the adjacent concrete), the relation between rusty steel and concrete delamination should be of a high order.

Table 5 gives the average potential, \bar{X} , the percentage of the potentials that were active, the area of concrete delamination, and the percentage of rusted steel, which all increased in value commensurate with the increase in proportion to the distress observed in the slabs. The half-cell potentials were a better indicator of the area of rusted steel than was the area of concrete delamination. This is emphasized in the data from slab 1, where 91 percent of the half-cell potentials measured were found to be in the active range, whereas only 4 percent of the concrete was delaminated by rusting of the top steel, which was coated with rust for 29 percent of its length.

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The contents of this report reflect the views of the author, who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the State of California or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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DISCUSSION

Bernard Erlin and W. G. Hime, Erlin, Hime Associates

The author has presented an excellent paper on the use of electrical measurements for the determination of hidden corrosion. Our comments are initiated by the title of the paper, which appears to indicate a more complete "postmortem" than was undertaken.

We feel that, although the efforts to detect those concrete areas where corrosion had occurred were probably complete, the determination of the basic underlying cause—namely, the initial character of the concrete and its subsequent condition—is mandatory to an autopsy if a full understanding of cause and effect is to be attained. A more detailed study of the concrete would thus have been warranted.

Such a study, centered around a complete petrographic examination of the concrete, could have provided a wealth of information that might resolve the following questions:

1. Did freeze-thaw damage occur, and, if so, was it a contributor to, or a cause of, steel corrosion (through initiation of microcracks and consequent rapid migration of chloride ion)? Was the concrete actually air-entrained, as had been required by specifications?
2. Was the water-cement ratio of the concrete mix high? Were there features that would indicate the reason for the deep penetration of chloride and the reported variations of chloride concentration with depth?
3. Were there differences in concrete properties that would explain the relatively dissimilar performances of spans 1 and 3?
4. Because the high pH of concrete ordinarily protects steel except in the presence of extremely large amounts of chloride, could the observed corrosion be related to carbonation and/or chloride levels?

It would be of value for the author to elaborate on the chloride analyses because of the varieties of methods available. For example, if the chlorides were extracted with water, the amount of chloride extracted might be up to 60 percent incomplete, depending on the age of the concrete. We note that the author stated that he also used X-ray diffraction for such analyses. Current X-ray diffraction methods are entirely unsuitable for quantizing chlorides in concrete. On the other hand, X-ray spectrographic methods may be suitable. We believe, therefore, that, if an X-ray method was used, then it was X-ray spectrography.

AUTHOR'S CLOSURE

The comments by Erlin and Hime are appreciated; however, they appear to believe that the corrosion of reinforcing steel is an unusual circumstance that is prompted by either an initially poor or subsequently deteriorated concrete. Such is not necessarily the case.

The reason that a petrographic analysis was not performed is that the results are not directly applicable to the corrosion behavior of steel in concrete. This is not to say that steel will not corrode in concrete that has suffered distress due to an environmental attack, but steel can and does corrode in the best quality concrete without any degradation of the concrete itself. We have ample evidence of corrosion of reinforcing steel where there is no possibility of freeze-thaw damage to the concrete.

The following are the answers to Erlin and Hime's specific questions:

1. Yes, test procedures on the construction project verified that the concrete was air-entrained as required by the specifications. Visually, there was no evidence to indicate any significant freeze-thaw damage of the concrete. Microcracking by any cause is not necessary for the penetration of chloride ion into concrete.
2. It is regretted that construction records cannot be located that confirm the actual water-cement ratio of the concrete mix. However, based on an average 28-day concrete compressive strength of 3,800 psi for the total of 11 bridges built at the same time, it would seem that the water-cement ratio was not above normal limits. A review of the literature will show that the chloride ion readily penetrates all normal concrete and the concentration decreases with depth. Even in well-controlled concrete, the absorption characteristics to concrete vary greatly from point to point, especially on decks, which could result in highly variable saltwater penetration with time.
3. The only significant difference between spans 1 and 3 was that the lower strength concrete in span 3 resulted in apparent earlier fatigue failure in the latter span.
4. I have never observed carbonation of the concrete at a depth of 2 in. below the surface at the location of the reinforcing steel. Moreover, I am not aware of any reports by others claiming to find evidence of carbonation at such depth with the type of concrete used on this project.

The relation between chloride ion and corrosion that was found in this study was simply that a sufficiently great concentration of chloride ion penetrated to the surface of the steel and caused the steel to corrode. No relation was found between various concentrations of chloride and the degree of corrosion.

In addition, let me add that a modified Volhard method with an acid extraction was used. Also, I do not use X-ray diffraction, but the X-ray secondary fluorescence method, which is commonly called X-ray emission or X-ray spectrography.