

# HIGHWAY RESEARCH RECORD

Number 204

Metal Corrosion

3 Reports

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## Foreword

This RECORD contains three papers which should be of interest to materials and structural engineers concerned with highway improvements and maintenance.

Lindberg applied a method of making electrical polarization measurements, which was developed in the corrosion laboratories at the National Bureau of Standards, to determine the corrosion of buried metal, for estimating the amount of corrosion on full-sized metal culverts installed under highways. The method can be used to compare performance of culverts already in place, or applied to samples buried at points of interest along a proposed route, to determine the most suitable culvert materials. This is an excellent example of applying the results of basic research developed in the laboratory to a specific engineering problem in the field.

Beaton and his associates of the California Division of Highways provide information on the performance of reinforcing steel in concrete pilings which were continuously submerged in seawater for 37 years. Little has been published on this subject heretofore and thus the report contains much of interest. The investigators observed serious corrosion of the reinforcing steel which was attributed to a high buildup of chloride content in the concrete. The chloride content determined in the continuously submerged pilings averaged 25 pounds per cubic yard with a maximum of 34 pounds per cubic yard. The results indicate that emphasis should be placed upon understanding the mechanism of chloride accumulation in concrete, so as to devise ways to prevent buildup to levels that result in corrosion of the steel.

The New Jersey State Department of Transportation reports on an analysis of atmospheric corrosion test data from an extensive literature search and an attempt to apply these data, especially with respect to the use of low-alloy steels, to highway bridges. Also discussed is a test program under consideration for the construction of several unpainted low-alloy steel bridges by the New Jersey State Department of Transportation.

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# Method of Estimating Corrosion of Highway Culverts by Means of Polarization Curves

R. I. LINDBERG, Research Scientist, Reynolds Metals Co.

The advent of new materials into the corrugated metal culvert field initiated efforts to develop a comparative corrosion test. The work of W. J. Schwerdtfeger at the National Bureau of Standards proved the usefulness of polarization curves for determining the corrosion of buried metal.

This work describes the application of the Schwerdtfeger method to full-sized metal culverts installed under highways. The preferred method of obtaining curves is to use four auxiliary electrodes symmetrically placed near the ends of the culvert and two reference electrodes placed over the centerline of the culvert near the edge of the roadway.

The method is simple and inexpensive and yields curves that are not difficult to interpret.

•A NUMBER of years ago corrugated metal culverts became available in several different metals and the highway engineer was faced with a problem of selection. One of the important points to consider was the corrosion resistance of the culvert when installed under a roadway. This was a difficult thing to evaluate short of digging up the pipe and having a look at it. We sought a way around this problem and discovered the work done by W. J. Schwerdtfeger (1) at the National Bureau of Standards.

Schwerdtfeger ran polarization curves on buried specimens of two metals and reported that, when large weight losses were encountered, the predicted weight loss was less than 6 percent from the actual weight loss.

At this point a word about polarization curves is in order. When a metal is in contact with an electrolyte such as moist earth, it will exhibit a pipe to soil potential. This potential is a voltage measured relative to a standard reference electrode (half cell). If for any reason, corrosion or otherwise, a current flows into or out of the buried metal, changes in the environment at the interface between metal and soil will occur. These changes cause the pipe to soil potential to change and the direction of change will depend on the the direction of current flow. By plotting the change in pipe to soil potential against the logarithm of the current flowing, a curve of fundamental significance is obtained. Figure 1 is a schematic drawing of such a curve. Note the "break" in the curve where the two straight-line portions intersect. The current at the break points on the anodic and cathodic polarization curves are called  $I_a$  and  $I_c$  respectively. The corrosion current,  $I_{corr}$ , is found by Pearson's equation (2) to be

$$I_{corr} = \frac{(I_a)(I_c)}{I_a + I_c}$$

This is the current flowing from the culvert due to corrosion, and by Faraday's Law, we can calculate the weight loss of metal represented by this current. We can then

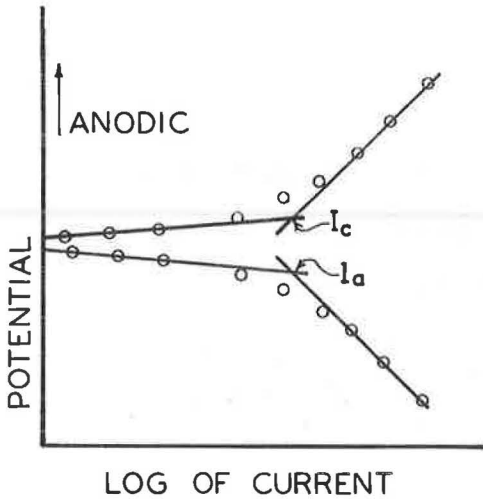


Figure 1. Schematic polarization curve.

the auger-type anchors used to chain dogs and is available inexpensively at pet shops.

Our first step was to bury specimens of both aluminum and galvanized culverts at one of our test facilities. Here 11-in. sections of 8-in.-diameter corrugated metal pipe were buried as well as 10-ft lengths of 24-in.-diameter culvert. Each sample had a lead wire attached which was brought above ground. After the culverts had been buried about a week in soil with a minimum specific resistivity of 9,000 ohm-cm, we ran our test. At this time, and every time since then, we have been able from our data

compare, by means of polarization curves, the corrosion resistance of two metal structures buried in the ground. This is the method proven accurate by Schwerdtfeger's work.

A possible drawback in Schwerdtfeger's work was the small size of his specimens—about  $\frac{1}{2}$  square foot of exposed area—and the question of applicability to larger structures was raised. Following discussion with Schwerdtfeger, we decided to follow his method exactly, even to duplicating his wiring hook up, as shown in Figure 2.

Figure 3 shows the wiring diagram translated to test equipment. This includes a galvanometer-potentiometer, bridge circuit, ammeter, copper/copper sulfate reference electrode, batteries, auxiliary electrode and lead wires. This is all standard equipment except the auxiliary electrode, which is conveniently made from

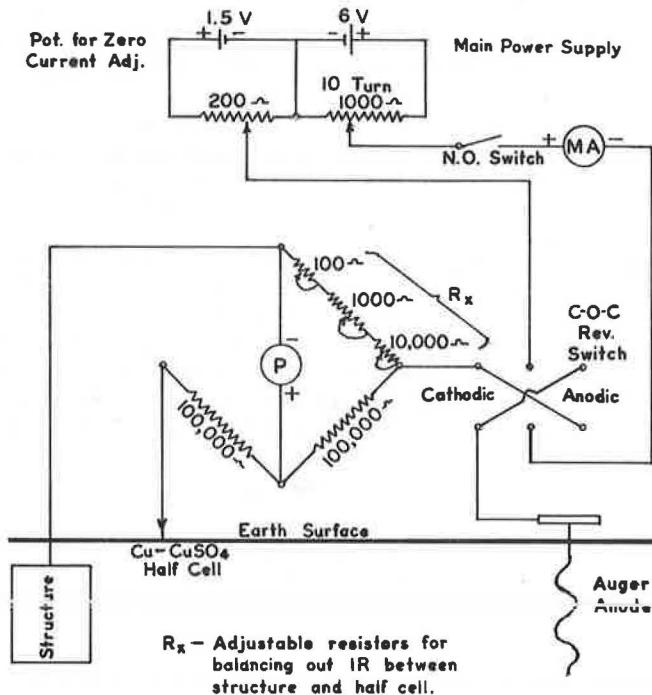


Figure 2. Polarization circuit.

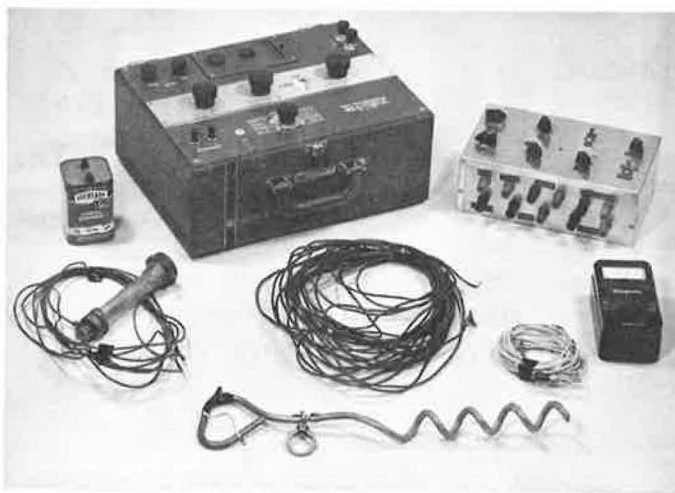


Figure 3. Equipment for polarization curves.

to obtain curves with good straight-line portions. A remote auxiliary electrode is used for all tests at this site and we position our half cell over the center of the culvert.

Encouraged by the results, we obtained permission from the Virginia Highway Research Council to try the method at the five sites they are now using for culvert tests. These are sites where full-size, generally 48-in. diameter by 25 to 30 ft long, culverts are placed under highways. Condition of exposures range from tidal flow to mountain stream, but in every case we obtained good breaking curves. It was about this time that we were invited to demonstrate the method in other areas where again the test produced good results.

In all these tests we used a single reference electrode placed at ground surface over the culvert and an auxiliary electrode placed a minimum of two or three pipe diameters away from the culvert and on the shoulder of the road which is the only area available. In discussing the test, a question was raised by a highway engineer as to the best place to put reference and auxiliary electrodes. To try to settle this question, a series of curves was run at the Virginia State Highway test site in Nansemond County where asphalt-coated galvanized steel and bare aluminum culverts are installed side by side. The pipes are 48 in. in diameter and 25 ft long. The installation is in a wooded area near the Dismal Swamp with a minimum specific soil resistance of 15,000 ohm-cm and a pH of 5.9 immediately over the culvert. A specific resistance of 14,000 ohm-cm and a pH of 4.3 were obtained about 20 ft away from the culvert. At the time the test was run, the culverts were flowing two-thirds full as a result of heavy rains.

The first configuration used included one auxiliary electrode placed 12 ft from one end of the culvert. One reference electrode was located over the center of the pipe midway out on the shoulder and another was placed beside and about 1 ft away from the auxiliary electrode. When IR drop was zeroed out for each reference cell, the rheostat (or potentiometer) position was marked so that it could be reset as needed. As soon as the test started, it was apparent that placing a reference close to the auxiliary electrode was a mistake because it gave potential shifts sixty times as high as the other cell and it was therefore disconnected. Figure 4 shows a cathodic polarization curve for the remaining reference cell, data for which are given in Table 1. Also shown is a curve obtained in 1964. It is interesting to note that the pipe to soil potential of this aluminum culvert was  $-0.64$  volts compared to  $-0.62$  volts in 1964. Break points on the curves were at 14 ma in 1964 and 7.2 ma today.

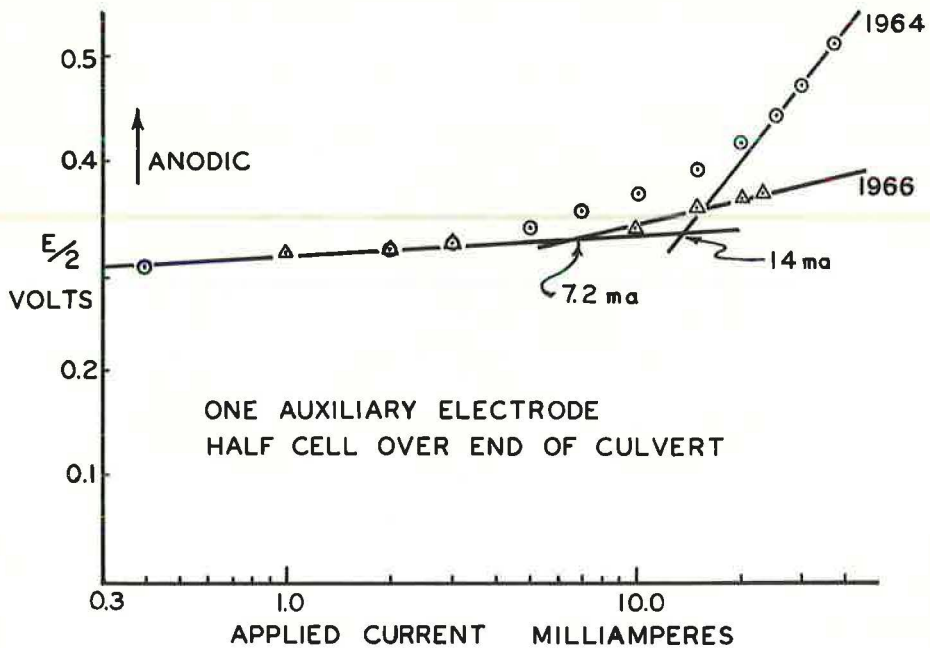


Figure 4. Cathodic polarization curve on bare aluminum culvert.

While the aluminum culvert was depolarizing, we moved to the asphalt-coated galvanized steel culvert and ran the next test. By thus alternating structures, we obtained both anodic and cathodic polarization curves on each structure using two half cells placed over the ends of the culverts as before. In these tests four auxiliary anodes were symmetrically placed about 4 ft from the edge of the culvert and midway out on the shoulder. These curves are shown in Figures 5 and 6 and data are given in Tables 2 and 3. Where one reference cell gave potentials substantially different from the other, the curve is a dashed line. It is interesting to note that when this occurs, the break point is not greatly different.

The abrupt increase in potential for the anodic reading of cell No. 1 at 5 ma in Table 3 is unexplained and this curve was not plotted in Figure 6.

A final test was run with a single auxiliary electrode about 4 ft from one end of the culvert. On the same side of the road, reference cells were placed both over the pipe,

TABLE 1  
CATHODIC POLARIZATION DATA—BARE  
ALUMINUM CULVERT<sup>a</sup>

Applied Current, ma	Pipe to Soil Potential, Volts/2 (as read)	
	Reference Cell	
	1 Over	2 Away
1	-0.316	-0.338
2	-0.318	-0.432
4	-0.322	-0.522
7	-0.322	Discontinued
10	-0.334	
15	-0.353	
20	-0.361	
23	-0.369	

<sup>a</sup>One auxiliary electrode 12 ft from culvert, two Cu/CuSO<sub>4</sub> reference cells, one over end of culvert and another near auxiliary electrode.



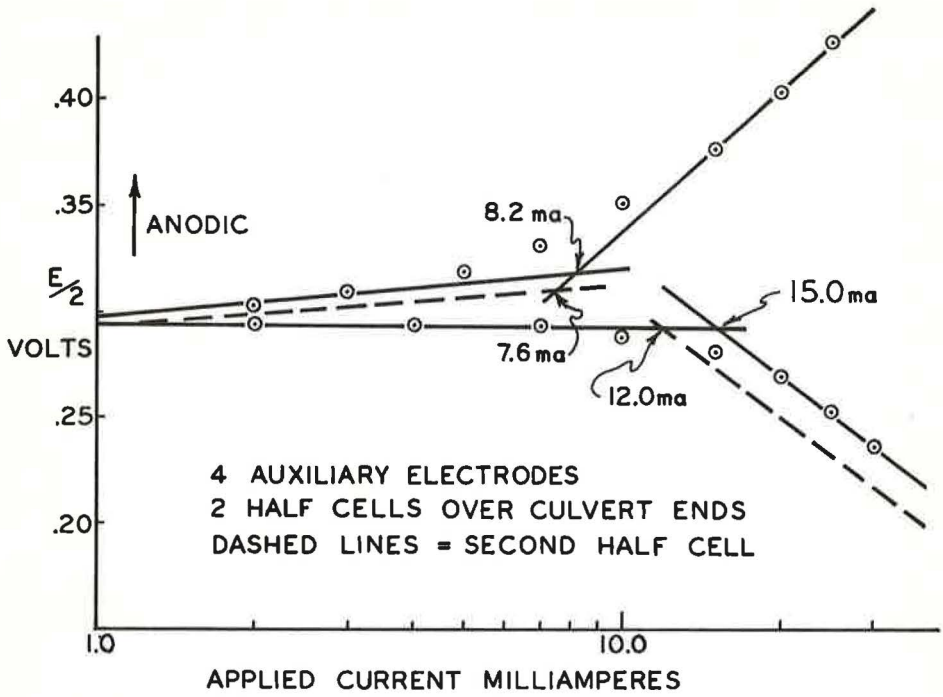


Figure 5. Polarization curves on bare aluminum culvert.

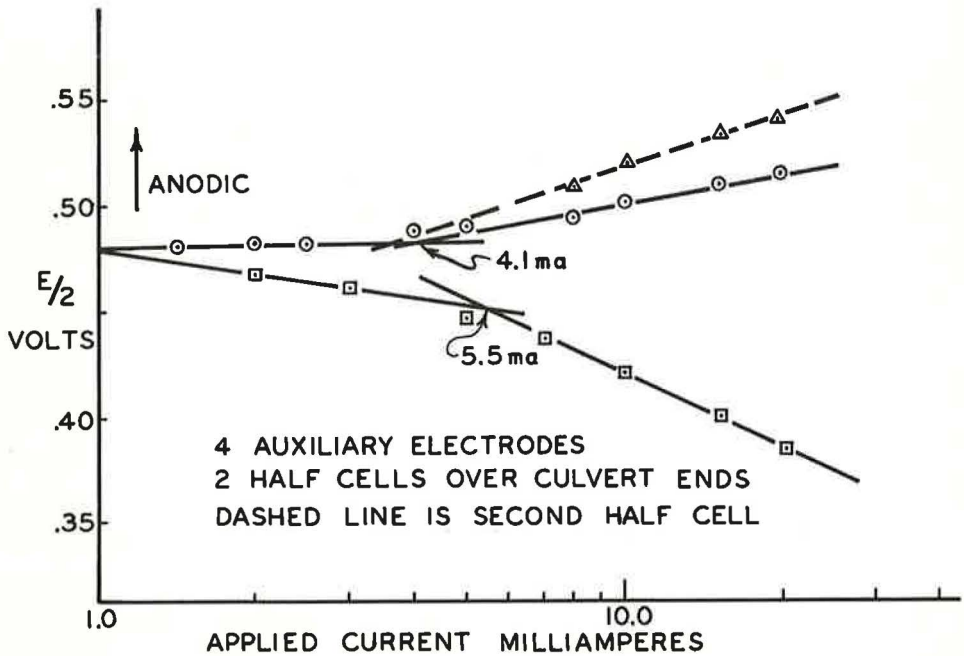


Figure 6. Polarization curves on asphalt-coated galvanized steel culvert.

TABLE 2  
POLARIZATION DATA—BARE ALUMINUM CULVERT<sup>a</sup>

Applied Current, ma	Pipe to Soil Potential, Volts/2 (as read)	
	Reference Cell	
	1	2
1	-0.297	-0.295
2	-0.298	-0.295
4	-0.306 (?)	-0.294
7	-0.280	-0.294
10	-0.279	-0.288
15	-0.265	-0.282
20	-0.252	-0.270
25	-0.234	-0.254
30	-0.218	-0.236
	Cathodic	
1	—	-0.293
2	-0.303	-0.298
3	-0.309	-0.305
5	-0.319	-0.316
7	-0.331	-0.326
10	-0.352	-0.353
15	-0.377	-0.380
20	-0.404	-0.408
25	-0.428	-0.430

<sup>a</sup>Four auxiliary electrodes 4 ft from ends of pipe, two Cu/CuSO<sub>4</sub> reference electrodes over opposite ends.

TABLE 3  
POLARIZATION DATA—ASPHALT-COATED GALVANIZED STEEL CULVERT<sup>a</sup>

Applied Current, ma	Pipe to Soil Potential, Volts/2 (as read)	
	Reference Cell	
	1	2
1	-0.474	-0.474
2	-0.460	-0.467
3	-0.451	-0.461
5	-0.463 (?)	-0.448
7	-0.456 (?)	-0.439
10	-0.442	-0.422
15	-0.425	-0.402
20	-0.410	-0.387
	Cathodic	
1.4	-0.481	-0.480
2.0	-0.481	-0.482
2.5	-0.482	-0.483
4	-0.488	-0.490
5	-0.490	-0.494
8	-0.495	-0.510
10	-0.502	-0.521
15	-0.511	-0.535
19.5	-0.515	-0.543

<sup>a</sup>Four auxiliary electrodes 4 ft from ends of culvert, two Cu/CuSO<sub>4</sub> reference electrodes over opposite ends.

and about 12 ft away from the culvert and on the side opposite the auxiliary electrode. The data were only taken in the cathodic direction and are given in Table 4 and plotted in Figure 7.

From these data and curves, we concluded that placement of the reference cell may make some difference in the potentials recorded, but that the break points in the curves are not materially affected. Good curves are obtained with either one or more auxiliary electrodes, but we found that the use of multiple auxiliary electrodes caused equilibrium conditions to be reached a little sooner. As a result, I prefer placing a reference over the culvert and one or more auxiliary electrodes off to the sides of the culvert.

### TEST PROCEDURES

After all connections have been made, the following preparations are required before the test can be run. With the ammeter (or milliammeter) on a low scale the current switch is flicked on and off. Generally some small current will be flowing due to galvanic effects between culvert and auxiliary electrode. This current must be bucked out by applying some current from the batteries and should be done quickly to minimize, if not prevent, any polarization of the structure. When this current is bucked out, the bridge circuit must be balanced, that is, the variable resistances must be made equal to the soil resistance between reference electrode and structure. This is done by opening the push-button switch and deliberately setting the rheostats so that when the switch

is closed, a small current on the order of 0.1 ma will flow between auxiliary electrode and the culvert. The galvanometer is now balanced and with the galvanometer key depressed, the current switch is flicked on and off. The galvanometer will deflect depending on the amount of IR drop. Now the variable resistances in the bridge circuit are adjusted to put some resistance into the system. The switch is again flicked on and off and adjustment continues until the galvanometer does not deflect when the current switch is momentarily closed. The system is now ready to oper-

TABLE 4  
POLARIZATION DATA—BARE ALUMINUM CULVERT<sup>a</sup>

Applied Current, ma	Pipe to Soil Potential, Volts/2 (as read)	
	Reference Cell	
	1 Over	2 Away
1	-0.306	-0.326
2	-0.327	-0.331
4	-0.349	-0.337
6	-0.372	-0.341
8	-0.396	-0.349
10	-0.436	-0.356
15	-0.496	-0.368

<sup>a</sup>One auxiliary electrode 12 ft away from culvert; two Cu/CuSO<sub>4</sub> reference cells, one over culvert, one 12 ft away and opposite auxiliary electrode.



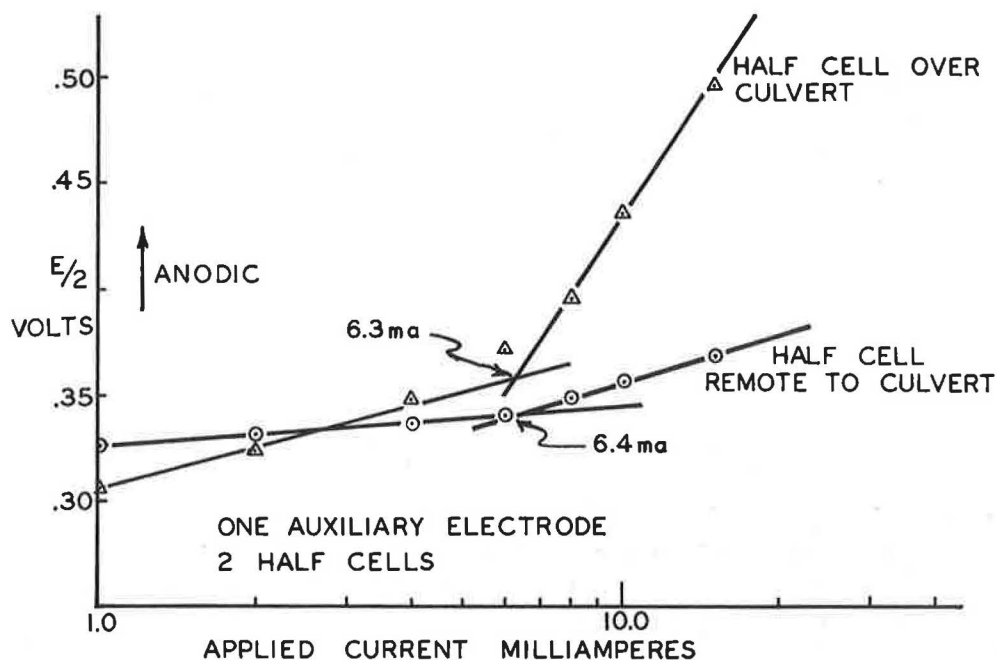


Figure 7. Polarization curve on bare aluminum culvert.

ate, but I usually wait 15 minutes to allow any polarization due to the brief intervals of current flow to dissipate.

With the system ready to go, you simply close the switch and, by adjusting the main power supply rheostats, put a little current between the auxiliary electrode and the structure in whichever direction (cathodic or anodic) you desire. After a couple of minutes, balance the galvanometer and read the pipe to soil potential. When equilibrium has been established (a steady reading), enter your data and add another increment of current. Continue this procedure until the curve is complete—most easily determined by plotting the curve on the spot.

If the second straight-line portion does not materialize, you have not applied sufficient current or been careful in zeroing out the IR drop and must repeat your work. The only other problem you will be likely to encounter is not having enough battery voltage. The resistance between auxiliary electrode and structure can be very high in dry climates; although this resistance does not enter into the bridge circuit, it surely can reduce the amount of current that flows. The answer is either to move the auxiliary electrode closer to structure or to connect several batteries in series with the latter being preferable.

The resistance between auxiliary electrode and soil can be reduced by pouring a little water over the electrode. A little water around the half cell also makes for better soil contact.

In this manner, polarization curves are determined and, as in all test work, the more carefully you work the better your data. From these curves you can estimate corrosion occurring at the time the data are obtained. By repeating them frequently, you can average out currents and calculate reasonable weight loss data. However, even occasional checks will allow comparison of materials at a given time under a given set of conditions.

The method can be used to compare performance of culverts already in place. It can also be used when applied to samples buried at points of interest along a proposed route to determine the most suitable culvert materials.

## REFERENCES

1. Schwerdtfeger, W. J. A Study by Polarization Techniques of the Corrosion Rates of Aluminum and Steel Underground for Sixteen Months. Jour. of Research, National Bureau of Standards, Vol. 65 C, No. 4, Oct.-Dec. 1961.
2. Pearson, J. M. Trans. Electrochem. Soc., Vol. 81, p. 485, 1942.

*Discussion*

W. J. SCHWERDTFEGER, National Bureau of Standards—In view of the field data obtained by Lindberg on relatively large culverts, it is gratifying to know that an electrical measuring technique might be useful in evaluating the comparative corrosion resistance of metals exposed to soils or in predicting the useful life of similar underground structures.

As Lindberg did not attempt to prove that he actually measured the corrosion on the outer surfaces of the culverts to which his data apply, it is believed that some comparison with National Bureau of Standards data (proven by weight loss) also obtained in the field might be of interest. After having exposed an aluminum pipe specimen at one NBS site, herein called site A, for 16 months (1), the stable corrosion current density based on the Pearson equation, using data from the breaks in polarization curves, was calculated to be about 8 microamperes/sq ft. This current density is based on the actual area of the bare high purity (1188) aluminum pipe (0.4 sq ft) exposed to a neutral soil having a resistivity of about 7500 ohm-cm, a relatively noncorrosive soil.

Lindberg obtained polarization curves on 25-ft lengths of 48-in.-culvert (315 sq ft of outer surface area) installed by the Virginia Highway Research Council. I calculated the corrosion current density based on a set of Lindberg's polarization curves and observed it to be approximately 17 microamperes/sq ft. The soil in the area where the aluminum culvert is installed might also be considered as being relatively noncorrosive, having a resistivity of 15,000 ohm-cm. Thus, by comparison with the NBS data (site A), Lindberg's data appear to be reasonable. Furthermore, the writer plotted Lindberg's data on rectangular coordinates and observed that breaks in the curves occurred at about the same values of current as indicated by the plot on semi-logarithmic coordinates. Lindberg showed that the breaks in the polarization curves occurred at values of current which were quite reproducible for a given environment.

It is believed that a demonstration of polarization curves plotted from data obtained at two sites, one very corrosive and the other noncorrosive, might also be of interest. Accordingly, some of the polarization curves from NBS site A on aluminum are shown in Figure 8 along with curves for an identical aluminum specimen exposed to a very corrosive soil (site B, 250 ohm-cm). For comparison, the curves are plotted on the same scale. For site A, it will be noted the currents at the breaks,  $I_p$  and  $I_q$ , decreased with length of exposure time, whereas the values of  $I_p$  for site B did not change. Also, note the difference in magnitude of the stable currents  $I_p$  for sites A and B at 16 and 32 months, respectively. The corrosion current densities calculated from the values of break current are about 8 microamperes/sq ft, as previously stated for site A, and 4,300 microamperes/sq ft for site B. Thus, the stable corrosion rates (weight losses) are in the ratio of about 500 to 1. There was remarkably good agreement between the calculated and actual weight losses of these specimens. Figure 9 shows the two aluminum pipe specimens (wall thickness 0.062 in.) after exposure and removal of corrosion products. There was some pitting (maximum 0.008 in.) on the one from site A but the specimen from site B was perforated in many places. In view of the low weight loss after 16 months on the specimen exposed to site A, its appearance probably would be little or no different after 32 months, the length of time the specimen in site B was exposed.

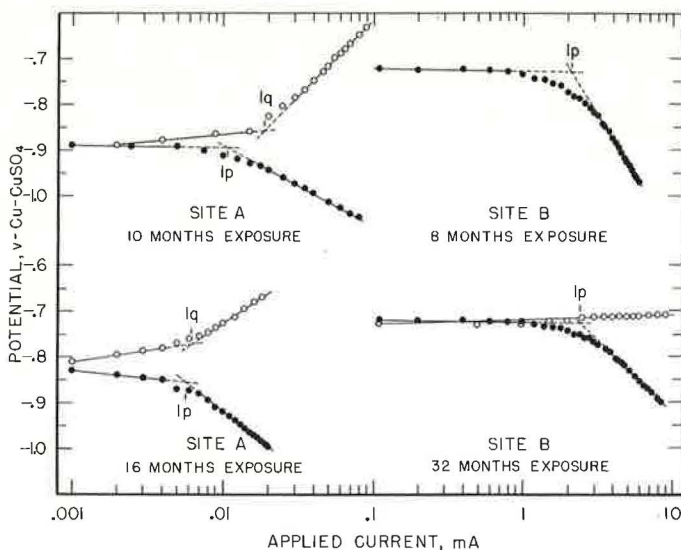


Figure 8. Polarization curves of aluminum (1188) pipe specimens during exposure at two underground sites: site A—soil resistivity, 7500 ohm-cm; site B—soil resistivity, 250 ohm-cm; ● cathodic, ○ anodic.

Now a few words about the measuring circuit and the electrical characteristics of the components. The battery voltage necessary is flexible and depends on the soil resistivity. The voltage required to complete a polarization curve can be reduced by increasing the exposed area of the auxiliary electrode (auger in Lindberg's paper). Values of  $R_x$  (balancing resistor) must be flexible to cover a range of soil resistivities. The push-button switch (used while balancing out the IR component which would otherwise be included in the measured potentials) is normally closed and, while helpful, is not



Figure 9. Aluminum specimens after exposure and removal of corrosion products: specimen A—exposure time, 16 months at site A; specimen B—exposure time, 32 months at site B.

absolutely necessary and may be replaced with a toggle switch. The magnitude of the current momentarily applied in balancing out the resistance in the potential measuring circuit will depend on the value of the soil resistivity. Corrosive soils (generally those of low resistivity) will require larger currents. The balancing is performed (one time) before beginning the polarization curve, and only the minimum current necessary should be applied in order to avoid or minimize polarization.

It is believed that the polarization technique can be applied to all metals commonly used underground. Lindberg has applied it to aluminum and to galvanized iron and recently I have found it to be applicable to copper (3) exposed to a very corrosive soil. In the laboratory, it has been successfully applied in measuring the instantaneous rates of corrosion of iron-chromium and iron-nickel alloys in a salt solution.

Mr. Lindberg is to be complimented for his practical application of this polarization method to a specific problem.

#### Reference

3. Schwerdtfeger, William J. Cathodic Protection of Copper in a Severely Corrosive Soil. IEEE Trans., Industry and General Applications, Jan. -Feb. 1967.

R. I. LINDBERG, Closure—Our thanks to W. J. Schwerdtfeger for his discussion as well as for his cooperation and helpful assistance during the initiation of this work.



# Corrosion of Steel in Continuously Submerged Reinforced Concrete Piling

J. L. BEATON, Materials and Research Engineer;  
D. L. SPELLMAN, Assistant Materials and Research Engineer—Concrete; and  
R. F. STRATFULL, Senior Corrosion Engineer, California Division of Highways,  
Materials and Research Department

The report describes a portion of a broader corrosion study and examines in some detail the aspect of steel corrosion in portions of piling which were continuously submerged for a period of approximately 37 years in seawater. As part of a contract for a new bridge as a replacement of the 37-year-old San Mateo-Hayward Bridge which was located in San Francisco Bay, the existing piles from the old bridge were removed. Seventeen of these piles were inspected for this study by exposing the reinforcing steel. In addition, samples of concrete were obtained so as to determine the contained salt content.

Under continuously submerged conditions it was found that 8 out of 17, or approximately 47 percent, of the pilings had heavy corrosion of the steel. The metal loss of the heavily corroded sections of the steel was in the character of irregular-shaped, broad pits that ranged in depth between 0.017 in. and 0.260 in. The average of the maximum pit depths was 0.114 in. for the pits having lengths up to approximately 6 in.

The chloride content found in the continuously submerged concrete ranged between 13 and 34 lb/cu yd, and averaged 25 lb/cu yd. On the basis of adsorption measurements made of the concrete, it was calculated that there could be a 10 percent chloride solution in the concrete, while the bay water contained 1.7 percent chloride.

•IN COOPERATION with the Bureau of Public Roads, the California Division of Highways, Materials and Research Department, has been continuing its studies of the causes of corrosion of steel embedded in concrete (1, 2, 3, 4, 5). This portion of the overall study is concerned with determining the level of chlorides in concrete that can cause the corrosion of the embedded steel and ascertaining the existence of corrosion of steel in continuously submerged portland cement concrete facilities.

Because of the difficulty of obtaining and inspecting submerged sections of portland cement concrete structures, little data seem to be available in the literature that described the long-time corrosion performance of such facilities. Recently there was an opportunity to inspect and sample the reinforced concrete piling which was being removed as part of the demolition of the then approximately 37-year-old San Mateo-Hayward Bridge. Because of modern-day traffic requirements, plus the continuing excessive cost of maintenance (1) of the 7.04-mi-long old bridge, a new bridge is being constructed to replace the old structure. The structure is located across the southerly arm of the San Francisco Bay.

The study is primarily a report on the findings at one bridge site. The data presented are not comprehensive or complete because this study does not report on the incidence of corrosion nor the chloride content in concrete structures elsewhere. It merely adds knowledge in an area of corrosion that is now sparse.

REINFORCED CONCRETE PILING HISTORY

There are many reports in the literature which acknowledge the corrosion performance of reinforced concrete piling. Lea and Watkins (6), in their laboratory studies of partially immersed, simulated reinforced concrete piling, concluded that the primary cause of concrete deterioration was corrosion of the reinforcement. The results of their laboratory-type studies are amplified by the full-scale studies reported by Tyler (7). He reported that corrosion of the reinforcing steel in the piling somewhat obscured tests being made primarily to evaluate the performance of various cements.

There have been many reports which have described the corrosion of reinforcing steel in atmospherically exposed sections of piling (1, 4, 8, 9, 11). It has also been reported that there has been satisfactory performance over a period of 20-30 years even when piling have been badly cracked as a result of driving (10).

There has been general agreement in the literature that the passivity of steel in concrete is destroyed by chlorides, and corrosion is usually the result of a galvanic type of corrosion cell (1, 5, 12, 13, 14, 15, 16, 17). One could assume that there would be a uniform moisture and chloride content of a submerged pile. However, Copenhagen (14) found differentials in the salt content of a pile submerged in seawater for at least a year.

The corrosion of steel in concrete has been reported when there are differentials in the salt content within the concrete (1, 12, 14). However, corrosion of the steel is not known to normally occur in relatively salt-free concrete in California bridges. Figure 1 shows that at chloride contents in excess of approximately 0.1-lb per sack of cement in a 5.4-sack concrete, the degree of deterioration of the concrete as a result of corrosion of the steel is not directly related to the chloride content (1). These data were obtained from atmospherically exposed beams of a bridge. The chloride contents shown in Figure 1 are averages for 1-in. -thick disks cut from 2-in. -diameter cores.

For this same bridge the data indicate (Fig. 2) that once the chloride content of the atmospherically exposed concrete is sufficiently great, the resistivity of the concrete in the anodic areas is a controlling factor in the deterioration (1). Within certain limits, the resistivity of a concrete is inversely related to its moisture content (1). Therefore, it would seem that if a concrete contained a high level of chloride-ion and was submerged so that its resistivity was low, corrosion of the steel could occur. It is assumed that the rate or even the incidence of visible results of corrosion would probably be controlled by polarization effects, the relative surface areas of the anode and cathode, and also time.

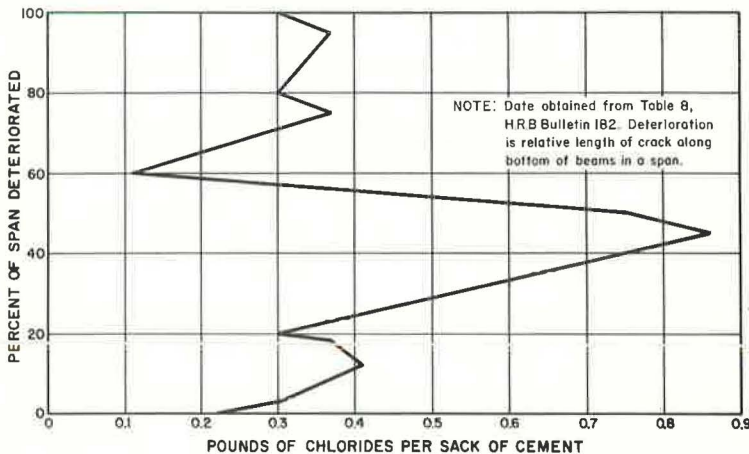


Figure 1. Deterioration of a structure vs chloride content.



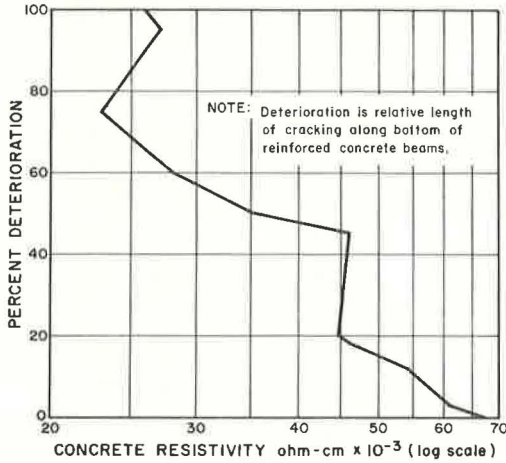


Figure 2. Deterioration vs resistivity.

Based on the grading of the aggregate and the slump of the concrete, it was estimated that the water content was about 40 gal/cu yd. From the cement factors as reported, water-cement ratio for the piles was computed to be about 5.7 gal per sack of cement.

The water of San Francisco Bay at the bridge site contains approximately 17,000 ppm of chloride-ion, or about 86 percent of the concentration of average seawater (1). Recommendations of the American Concrete Institute for exposures comparable to that at the bridge site, are water-cement ratios not in excess of 4.5 gal per sack in piles (18).

### Characteristics of the Concrete San Mateo-Hayward Bridge

Previous studies (1) were initiated to determine the degree to which the particular concrete used in this structure may have promoted corrosion of the reinforcing steel. Available construction records show that the average cement content of the concrete in the piles was 6.9 sacks per cubic yard.

The slump of the concrete used in casting the piles was reported as varying from 3 to 6½ in. but was generally indicated as within the range of 3 to 5 in. Gravel used as coarse aggregate, nominally of 1½-in. to No. 4 size, had a high percentage passing 1 in. Construction records pertaining to grading of the sand are meager, but analysis of hardened concrete samples from the bridge show it to be well graded with a fineness modulus of the order of 2.8.



Figure 3. Excavating mud from base of piers—superstructure removed.



Figure 4. Appearance after removed pile. Note line of demarcation between exposure to free water and mud immersion.

The water-cement ratios used in the bridge are, therefore, about 1 gal per sack higher than would be recommended for this exposure today.

Compressive strengths (28-day) of 170 test cylinders made during construction of the piles were reported to be in the range of 4000 to 5000 psi.



Figure 5. Removing concrete with jackhammer. Note rust stain (dark area) immediately adjacent to bit of jackhammer.





Figure 6. Corner of pile after removal of reinforcing steel.

### SAMPLING METHOD

In general, the method for removal of the piling was first to excavate approximately 5 ft of mud below the base of the pile, then repeatedly bend it until it broke. Figure 3 shows the piling in place while excavation of the bay mud from the base of the piles was in progress. In many cases, concrete was stripped from the pile in such a way as to twist the steel reinforcement. Only the steel in those piles exhibiting the least amount of removal damage at the mud line was sampled.

After a pile was removed and placed on the deck of the barge, it was photographed. Then by means of a jackhammer, the concrete was removed to expose the steel. Fragments of the removed concrete were identified and retained for further laboratory analysis. All sampling was performed within minutes after the pile was removed.

Figure 4 shows the typical line of demarcation between the mud and free water exposure of the piles.

Figure 5 shows the appearance of a pile just before removal of the concrete for sampling. In some cases, the corrosion products (rust) from the steel had penetrated to the surface of the concrete.

Figure 6 shows a pile after the reinforcing steel had been removed. A torch was used to cut the steel. All samples of the steel were approximately 6 ft long, extending approximately 3 ft in each direction from the mud line.

### Results—Corrosion of the Steel

The steel was stripped from one corner of each of 17 piles and any evidence of corrosion was immediately noted by visual evaluation and later measured by a micrometer.

Figure 7 shows the appearance of a removed oval-shaped and approximately 4-ft-long reinforcing bar. The locations of the mud or water in contact with the concrete are noted.



Figure 7. Condition of 1.10 x 1.37-in. oval reinforcing steel. Notations of "mud" and "water" are where the pile was in contact with those environments.

Figure 8 is a close view of the corrosion of the same steel bar which is shown in Figure 7, and shows a fragment of the concrete which was in contact with the steel. The dark area on the surface of the concrete fragment is a rust stain. In Figure 8, the area of relatively noncorroded steel is the lightly colored area. The "spotty" or localized corrosion of the steel was typical.

Figure 9 shows a close view of the corrosion attack at a different location than that shown in Figure 8, but is on the same piece of steel which is shown in Figure 7. This location is where the concrete was apparently in continuous contact with the bay mud. In all cases, the bay mud line on the piles was at least 10 ft below the low-tide water level at the time of inspection.

Of the 17 piles that were inspected, 8 or about 47 percent, were found to have significant corrosion of the reinforcing steel. The steel in the other 9 piles had what appeared to be a minor surface spotting of rust.

The maximum corrosion penetration of the steel in those piles having significant corrosion varied from 0.017 to 0.260 in. in depth and averaged 0.114 in.

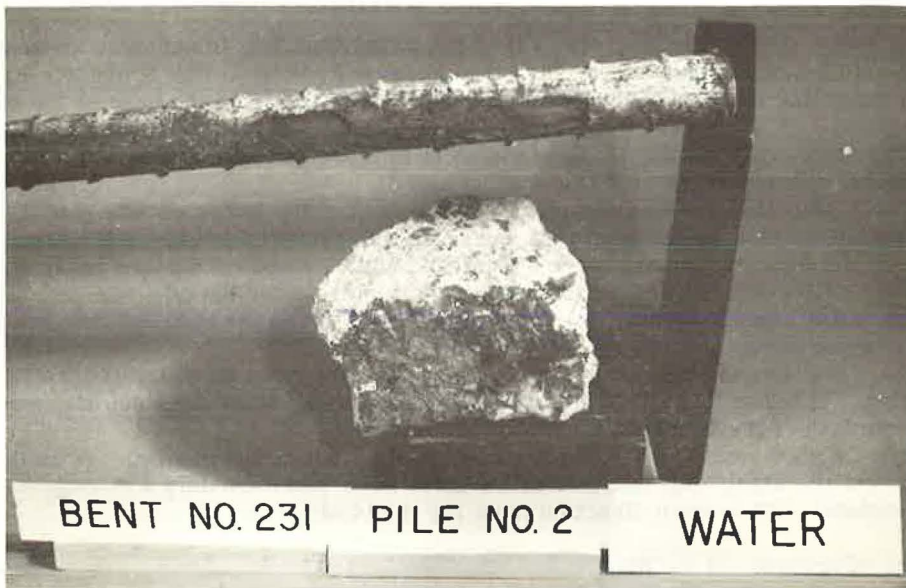


Figure 8. Close view of section of steel shown in Figure 7. Area where concrete was in continuous contact with bay water. Maximum metal loss in this area was 0.117 in. Dark area on concrete fragments is rust stain from the steel.

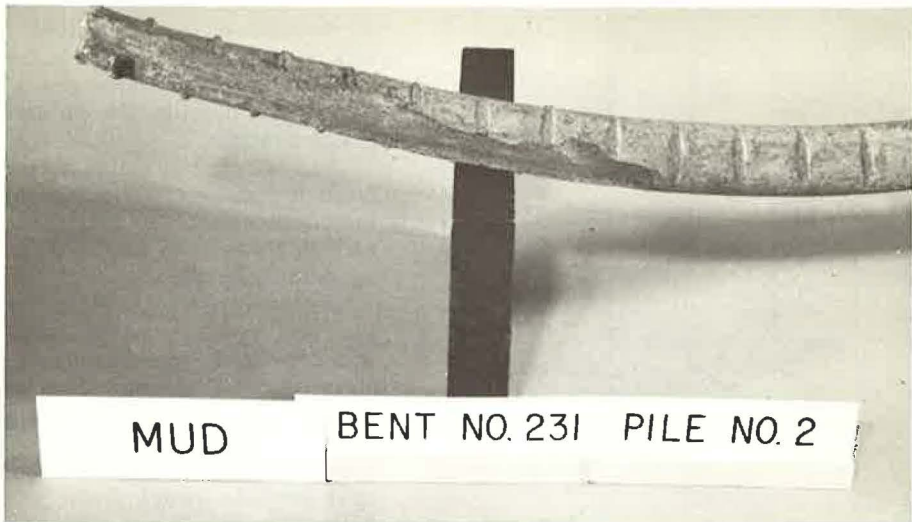


Figure 9. Close view of section of steel shown in Figure 7. Area where concrete was in continuous contact with bay mud. Maximum metal loss in this area was 0.210.

The maximum amount of measured metal loss for each pile is given in Table 1 along with other data on the concrete cover.

Table 1 shows that the maximum amount of metal loss was generally observed in the vicinity of the mud line on the surface of the piling. The reason that the maximum amount of metal loss was found near the mud line is not clear. However, as will be observed in Figures 7 and 8, significant amounts of corrosion loss of the steel were also found in the water contact zone of the piles.

#### Chloride-Ion in the Concrete

Concrete fragments obtained at the locations of corrosion were chemically analyzed for chloride content by means of an acid titration. The results of the chloride analysis (Table 1) are indicative of the average chloride content of the concrete for the total

TABLE 1  
CORROSION TEST RESULTS

Bent No.	Pile No.	Calculated Lb of Chloride Per Cu Yd	7-Day Water Absorption, Lb/Cu Yd	Concrete Cover, In.	Depth of Maximum Metal Loss, In.	Location of Maximum Metal Loss
231	2	24.7	248	1 <sup>3</sup> / <sub>4</sub>	0.210	Mud line
232	1	26.8	298	2 <sup>1</sup> / <sub>2</sub>	Minor rust	Mud line
	2	23.6	258	2 <sup>7</sup> / <sub>8</sub>	Minor rust	Mud line
	3	25.3	258	1 <sup>3</sup> / <sub>4</sub>	0.095	Mud line
	4	13.4	266	2	0.260	Mud line
233	1	25.9	283	2 <sup>1</sup> / <sub>4</sub>	Minor rust	Mud line
	2	26.2	273	2 <sup>1</sup> / <sub>4</sub>	Minor rust	Mud line
	3	27.3	256	2	Minor rust	Mud line
234	1	28.4	265	2	Minor rust	Mud line
	2	25.2	249	2	0.119	Mud line
	3	13.9	243	2 <sup>1</sup> / <sub>4</sub>	Minor rust	Mud line
	4	29.5	268	2 <sup>1</sup> / <sub>2</sub>	Minor rust	Mud line
245	3	27.0	244	2 <sup>1</sup> / <sub>4</sub>	0.017	1 <sup>1</sup> / <sub>2</sub> ft above mud line
	4	27.2	239	2 <sup>1</sup> / <sub>4</sub>	0.039	2 ft below mud line
246	1	33.6	241	1 <sup>1</sup> / <sub>4</sub>	0.097	Mud line
258	1	19.2	231	2 <sup>1</sup> / <sub>4</sub>	0.073	1 <sup>1</sup> / <sub>2</sub> ft below mudline
	2	24.0	227	2 <sup>1</sup> / <sub>4</sub>	Minor rust	Mud line
Avg.		24.9	255		0.114	



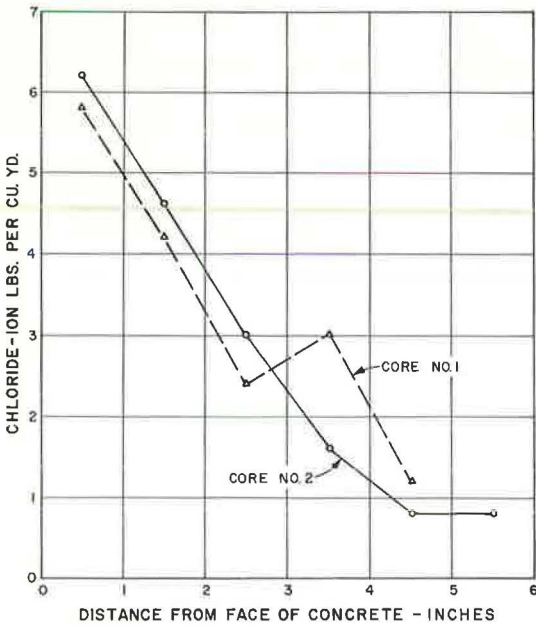


Figure 10. Chloride-ion in atmospherically exposed piling, San Mateo-Hayward Bridge.

depth of cover over the steel. Time was not available to permit the taking of concrete cores which could have permitted chloride determination at greater depths.

As indicated by Table 1, the calculated amount of chloride-ion found in the continuously submerged concrete ranged between 13 and 34 lb and averaged 25 lb/cu yd.

As a comparison, Figure 10 shows the chloride-ion concentration which was previously found in atmospherically exposed sections of piling in this same bridge after approximately 28 years of exposure (1). The chloride contents shown in Figure 10 are averages for 1-in.-thick disks cut from 2-in.-diameter cores. It was calculated that for a 2-in. depth of cover and 28 years of exposure to salt spray, the average chloride content in this concrete was approximately 5 lb/cu yd. However, an average of 25 lb of chloride-ion was found in the submerged (but different) piling in this structure after approximately 37 years of service.

As given in Table 1, the average 7-day absorption of the samples obtained from the piling in this study is equivalent to 255 lb of water per cubic yard of concrete, while the average chloride-ion content was 25 lb/cu yd. These results indicate that a calculated chloride content of approximately 10 percent by weight of the absorbed free water could be in this concrete. The chloride content of the bay water was found to be approximately 1.7 percent.

We have no construction records that indicate bay water was used as mixing water, or that calcium chloride was added, nor has any previous testing indicated that there were additions of chloride to the concrete mix.

### Concrete Absorption

In this investigation, the method used to measure concrete absorption was as follows:

1. The concrete was first oven-dried at approximately 230 F for 28 days.
2. Immediately after oven-drying, the concrete was immersed in tap water at approximately 72 F for a total of 28 days.
3. The weight gain of the concrete at 7 days was chosen to represent the absorption of the concrete.
4. The volume of the concrete under test was determined at the 28th day of soaking by the weight in air-weight in water method.

The results of the absorption tests are given in Table 1, and are shown as pounds of water per cubic yard of concrete.

We have outlined the method that we are presently using for measuring concrete absorption because thus far, for our purpose, it seems to show promise for obtaining reasonably reproducible results. Our investigation of a method for measuring concrete absorption is not complete, therefore we cannot comment at this time upon its absolute accuracy nor its relative value for distinguishing between concretes of various mix designs. However, the trend of some of the absorption data indicates that after 28 days

of soaking, the weight gain of some of the concrete samples has been within the range of 15 to 25 grams of the free water which this same concrete had contained even after 2 years of underwater curing. The 7-day absorption value for some of the concrete samples has been within 8 to 12 grams of the amount of water absorbed after 28 days of soaking. These weight values are for concrete cylinders that have a free water content that ranges between 350 and 400 grams at the conclusion of the various submerged curing times. Therefore, since our studies of concrete absorption are not complete, we have chosen at this time the 7-day absorption value—primarily because it is a common time element in concrete technology.

## DISCUSSION

Basically, this phase of the overall investigation was primarily concerned with two questions: (a) What chloride content of the concrete will result in corrosion of imbedded steel? and (b) Does steel corrode in continuously submerged piling?

With regard to the first question, it would not seem unusual to find corrosion of steel when the chloride content in the continuously submerged concrete is in the range of 25 lb/cu yd. What is surprising is that such a high quantity of chloride was found.

Recently Ost and Monfore (19) have reported that the migration of chlorides into concrete is greatly dependent upon the water-cement ratio of the original concrete mixture. Depending upon the test parameters, they found chloride contents in the concrete at an indicated range from 0 to about 30 lb or more per cubic yard after one year of testing.

Other studies have shown that concrete samples have absorbed up to approximately 45 lb of chloride-ion per cubic yard after 88 days of testing (5).

The mechanism of the corrosion of the steel in these piles was not investigated and is, therefore, subject to speculation. Previous work on the atmospherically exposed portions of piling from this bridge showed the presence of cathodic potential gradients in these areas. The potential measurements indicated that the anode was in the tidal water and splash zone of the piling. No potential measurements were made on the continuously submerged areas of the pilings. Also, no potential measurements were made during this later study.

Thus far, it has not been determined whether the corrosion in the submerged section of the piles on this bridge is sustained by a submerged cathode or by a cathode which is in the atmospherically exposed section of the pile.

Because of the high level of chloride found in the submerged sections of the piling, it would seem worthwhile to investigate the mechanism of chloride buildup in concrete. Previous work has indicated that chlorides may accumulate in concrete by assuming that chloride-containing water entered the concrete by absorption and left salt behind as a result of evaporation (4).

It appears that the mechanism of salt accumulation in concrete may not be simple. For example, in the laboratory, the placing of initially salt-free and air-dry reinforced concrete specimens in a saturated salt solution has resulted in visible evidence of corrosion of the steel in the range of 12 to 25 days (5). Current, but unreported and unfinished testing by us of reinforced concrete in saturated salt solutions, has shown that if the concrete is first saturated with "clean" water before being immersed in salt water, then it will take at least 8 months for corrosion of the steel to occur instead of 12 to 25 days (5). Therefore, it seems that the salt content and the time to corrosion for reinforced concrete may be significantly controlled by the amount and nature of the moisture in the concrete immediately prior to its being placed in an aggressive environment. For concrete that is saturated with "clean" water, it is speculated that chloride would penetrate by means of diffusion or by capillary action which could be the result of the evaporation of the originally contained water. It also appears that if the diffusion process was the controlling mechanism for chloride entry, then it would be expected that the calculated chloride content in the approximately 37-year-old concrete at this bridge would be no greater than that which is found in the bay water, since higher concentrations would tend to diffuse outward.



The results of this phase of the investigation indicate that emphasis should be placed upon understanding the mechanism of chloride accumulation in concrete, so as to devise ways to prevent buildup to levels that result in corrosion of the steel.

### SUMMARY AND CONCLUSIONS

The results of an inspection in the areas of continuously submerged portions of 17 reinforced concrete piles which were removed from the approximately 37-year-old San Mateo-Hayward Bridge, showed that 8 or approximately 47 percent, of the piles had significant corrosion of the reinforcing steel. The average maximum depth of corrosion pits of the steel was 0.114 in.

The average chloride-ion content of approximately 2-in. -thick fragments of concrete was found to be 25 lb/cu yd. Based on the 7-day absorption of the concrete, it is calculated that the chloride content of the water contained in the saturated concrete could average 10 percent by weight of the contained water. The San Francisco Bay water in the vicinity of the bridge has had a measured chloride-ion content of 1.7 percent.

The calculated difference in the chloride level of the concrete in areas of heavy corrosion and areas of light corrosion was quite small—on the order of 0.6 lb/cu yd. The difference does not seem to be significant. It appears that the corrosion phenomenon at these high levels of chloride concentration may be controlled by factors other than just the presence of salt.

This investigation did not determine the mechanism of corrosion of the steel in the continuously submerged sections of the piles. However, this phase of the overall investigation demonstrated that with this particular salt-contaminated concrete and approximately 2 in. of cover, corrosion occurs in continuously submerged sections of piling.

### ACKNOWLEDGMENT

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The work involved in this study was performed as part of a corrosion study in cooperation with the Bureau of Public Roads; however, the opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Bureau of Public Roads.

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# An Analysis of Atmospheric Corrosion Tests on Low-Alloy Steels—Applicability of Test Results to Highway Bridges

DIVISION OF RESEARCH AND EVALUATION, BUREAU OF STRUCTURES AND MATERIALS, STATE OF NEW JERSEY, DEPARTMENT OF TRANSPORTATION

The New Jersey State Department of Transportation will be constructing several unpainted, low-alloy steel bridges in the near future. To gather information for use in evaluating the performance of these structures a literature search was conducted. Some problems related to material specifications and atmospheric corrosion testing are discussed. The data contained in several reports on the atmospheric corrosion of many low-alloy steels are examined.

Sufficient data appear to be available in published corrosion tests to allow a reasonable estimate of the amount of corrosion which will occur to unstressed sample panels of many low-alloy steels exposed freely to the atmosphere, at many locations. These estimates can be made by comparing the chemical composition of a given steel with several tested chemical compositions to which it is similar in percentages of the major corrosion-reducing alloying elements. Any comparisons, such as these, must be made for similar environments and test conditions.

Test results from three different reports are compared for steels which were grouped in accordance with the percentages of the major corrosion-reducing alloying elements contained. Based on the information in these reports and other references, a tentative empirical method is proposed for predicting the depth of penetration of corrosion for bridge members. The empirical method assumes a linear long-term corrosion rate and includes an "exposure factor," "pitting factor," and a "safety factor." The reductions in several cross-sectional properties of two wide-flange beams and various sizes of plate are computed and it is suggested that further study of this problem should be made.

The possible effect of loads on corrosion and of corrosion on static and fatigue load resistance are briefly discussed, and "other factors" of relevance to the use of unpainted bridges are listed. A proposed test program for an experimental New Jersey bridge is briefly outlined.

- THE New Jersey State Department of Transportation will be constructing several unpainted, low-alloy steel bridges in the near future. These bridges have been classified as experimental, and an annual report on their performance will be submitted to the Bureau of Public Roads.

The Division of Research and Evaluation has established a project (1) entitled "Bridge Construction of Unpainted ASTM A-242 Steel," and is responsible for the observation, analysis, and evaluation of these experimental structures. This paper is a report on the first phase of this project—a literature search—and includes: (a) an analysis of some atmospheric corrosion test data, (b) an attempt to apply these data to highway bridges, (c) a listing of some related factors which appear to be in need of further study, and (d)



a brief description of the testing program being proposed for the experimental New Jersey bridges.

More than 25 years of industrial research has shown that many low-alloy steels are corroded at a much slower rate than ordinary structural steel when openly exposed to the atmosphere in the form of small panels on test racks. These steels claim no special ability to resist chemical attack (2), only atmospheric attack.

In this past research, the chief method used to evaluate the degree of corrosion was the determination of the amount of weight loss which occurred after sample panels were exposed to the atmosphere for various time periods and then cleaned of all corrosion products. This weight loss was then converted into an average thickness loss for each exposed surface. Thickness losses plotted against time produced time-corrosion curves which illustrated graphically a significant reduction in rate of corrosion of some low-alloy steels compared to ordinary structural steel in many types of atmospheres.

It has been proposed that some low-alloy steels can be used for structures in an unpainted condition, freely exposed to the atmosphere with little or no maintenance required (3, 5, 7, 9, 10). It has also been stated that these materials, when painted, cause increased paint life (4). It has been claimed further that the appearance of the rusted material is a desirable feature for some applications (3, 5). The question of appearance appears to be subject to debate, and adverse, as well as complimentary, opinions are available. The possibility of staining of adjacent materials by corrosion products carried off by rain must also be considered in this regard.

Some uses for which unpainted low-alloy steel has been suggested (3), (5) are bridges, building exteriors and open-type buildings, railroad cars, trucks, communication and power line poles and towers, highway guardrails, fences, light standards, railings, and sign supports. The obvious reason for using this material for these applications is the economy resulting from a combination of higher strength and low maintenance which, presumably, will more than offset the higher unit cost of material.

The suggested benefits to be gained by using high-strength, low-alloy weather-resistant steel have generated considerable interest in this material, especially in recent years. Many actual uses are described in the technical literature (3, 5, 6). The first major use of unpainted steel in bridge structures was on the John C. Lodge Expressway, Detroit, Michigan (6). The Michigan State Highway Department has reported that it is satisfied with the performance of A-242 steel to date. Their bridges have been open to traffic for more than a year.

The significant number of structures constructed of unpainted steel, even though many are considered experimental, indicates a considerable degree of confidence in its performance by both users and producers. It should be noted, however, that very few unpainted structures have been in existence for more than 5 years.

#### MATERIAL IDENTIFICATION AND SPECIFICATION

The materials which have been advanced as possessing sufficient resistance to corrosion to allow their use in unpainted structures, under some conditions, are generally classified as low-alloy, high-strength steels which are modifications of ASTM Specification A-242.

ASTM A-242 states: "These steels have enhanced atmospheric corrosion resistance equal to or greater than carbon structural steels with copper"; and, in a later paragraph, "If the steel is specified for materially greater atmospheric corrosion resistance than structural carbon steel with copper, the purchaser should so indicate and consult with the manufacturer."

The chemical requirements stipulated by A-242 for ladle analysis are carbon, 0.22 percent maximum; manganese, 1.25 percent maximum; and sulphur, 0.05 percent maximum. The tensile requirements are similar to ASTM A-441, and include a minimum yield point of 50,000 psi for Group I structural shapes and plates  $\frac{3}{4}$  in. and under in thickness. No tests for corrosion resistance are included in the specification.

A-242 also states: "If steel is purchased for welding, the suitability of the chemical composition for welding under the given conditions shall be based on evidence acceptable to the purchaser."

This specification does not contain any requirements which will insure that corrosion will be limited to any greater extent than it is limited in copper-bearing steel under similar conditions of use. In fact, the chemical requirements do not insure even this. Copper-bearing steel (0.20 percent minimum copper), under most conditions, is not usually considered for use in unpainted structures. The welding properties of the specified steel are, likewise, not defined by direct requirements.

In current specification practice, the following alternatives have been used by designers or suggested by steel producers:

1. Specify under A-242 and modify by using a corrosion resistance requirement of "four to six times that of ASTM A7 steel." Require manufacturer to submit evidence, satisfactory to the Engineer, that the material supplied satisfies this corrosion criteria. Require certification and evidence of weldability.
2. Specify under A-441 and modify by using the same corrosion resistance and welding criteria mentioned under alternative 1.
3. Specify by brand name either with or without an "or equal" clause.

The above alternatives are in need of improvement if they are to be continued to be used for nonproprietary construction specifications. For instance, the requirement of "four to six times the corrosion resistance of ASTM A7 steel" is not clearly defined, and hence, for specification purposes is meaningless. The amount of material loss which will occur to any steel due to corrosion is quite variable and depends on the type of atmosphere, the conditions of exposure, the exact chemical composition, the exposure time, and many other factors. When comparing the corrosion resistances of different steels, the conditions under which the comparisons are made must be clearly defined.

#### TESTING FOR ATMOSPHERIC CORROSION RESISTANCE

Atmospheric corrosion testing is characterized by a relative lack of standardization (2). The general types of tests which can be conducted may be classified as follows (13):

1. Accelerated tests—used for quality control and acceptance.
2. Laboratory tests—used to develop theory of corrosion mechanism or systematic study of individual factors (often accelerated).
3. Actual service tests—A built-in part of a structure or piece of equipment is usually removed for evaluation after a suitable period of use. (Most reliable and most expensive method.)
4. Field (or plant) tests—samples of material are exposed to the actual environment, although not under the conditions of actual use.

In regard to acceleration testing of low-alloy steels, the following quotes are taken from the literature:

"It has been established that so-called 'accelerated' corrosion tests do not yield reliable results" (2).

"Accelerated tests, such as salt spray and weatherometer tests, are virtually useless in evaluating the durability of the weathering steels. The safe way is to use long term corrosion curves" (6).

"At the present time no method has been found that will accelerate the formation of the rust coat" (6).

A somewhat more optimistic view is offered by Barton (22). In his paper, he defines a suitable accelerated testing method as one "which accelerates progress of the corrosion process, compared with corrosion under natural conditions, without changing the mechanism of the corrosion reaction and of the combination of factors controlling



its kinetics." He further states that a good knowledge of the mechanism of the corrosion reaction is necessary for this type of test. His paper then describes an accelerated method of test for metals in an industrial environment, but later states: "Since this method employs constant supercritical humidity it cannot be used for comparison of the resistance of such metals as low-alloy steels, for which the corrosion resistance in atmospheric environments depends, among other factors, upon differences in the hygroscopic qualities of their corrosion products. At present, we are trying to formulate conditions for accelerated corrosion tests of these materials, too. This is being done by a cycle combining the test conditions described above with a period of drying at relative humidities lower than the critical point. Preliminary trials show very promising results."

In regard to the mechanism of corrosion for low-alloy steels in the atmosphere, Horton states (10): "One of the major unresolved questions about rusting today is how these alloys are able to affect the atmospheric corrosion resistance of steel." He later remarks: ". . . there is reason to think that alloying effects occur in the rust layer rather than at the surface of or within the corroding steel."

It should be noted that attempts to explain improvements in corrosion resistance of steel by alloying have been made as early as 1913 up to the present. Further discussion of this problem can be found elsewhere (10, 12, 14, 22). In the words of Madison (6), "Just how the alloying elements produce more protective rust layers is not really known, but is under study."

In summary: accelerated tests depend on an understanding of the basic corrosion mechanism; and because this mechanism is not well understood for low-alloy steels, and because the corrosion resistance depends upon the rust film itself, accelerated tests are not practical at this time. The statements quoted above from Barton offer some hope for future development of a test of this type. Apparently, further research is required.

Inasmuch as acceleration of the protective coating development is not now practical, then it becomes necessary to take long periods of time to test a steel for atmospheric corrosion resistance. This means that quality control and acceptance tests which directly measure corrosion resistance are not possible at this time. However, sufficient data appear to be available, in published corrosion tests, to allow a reasonable estimate to be made of the amount of corrosion which will occur to unstressed sample panels of many low-alloy steels, exposed freely to the atmosphere, at many locations. These estimates can be made by comparing the chemical composition of a given steel with several tested chemical compositions to which it is similar in percentages of the major corrosion-reducing alloying elements. Any comparisons, such as these, must be made for similar environments and test conditions.

#### PUBLISHED ATMOSPHERIC CORROSION TESTS OF LOW-ALLOY STEEL

Copson (8), Larrabee and Coburn (L & C) (9), and Horton (10) report on the results of atmospheric corrosion tests conducted on a wide variety of low-alloy steels at a number of locations for periods up to 20 years.

In general, these tests were conducted on small sample panels, exposed freely to the atmosphere on test racks, and as mentioned previously, the degree of corrosion was evaluated by determining the loss in weight which occurred after all rusted material was removed. In one case, measurements of pit depths were made. For the most part, there is good agreement among the results of the tests discussed in these reports.

Some general conclusions taken from these and several other sources are listed below. These conclusions were either stated explicitly by the authors of the respective publications, or were obvious from their experimental data.

1. "Low-alloy steels, as a class, are not incorrodible, but under favorable conditions, as when they are exposed freely outdoors, they rust several times less rapidly than unalloyed mild steel." They have been described as "slow rusting steels" (11).

2. The kind and amount of alloying elements have a great effect on corrosion. Minor changes in composition sometimes are major factors in determining the atmospheric corrosion losses (8, 9, 10, 11).

3. The alloying elements which seem to have the greatest beneficial effect on corrosion resistance of steel are copper, chromium, phosphorus, nickel, and silicon (8, 9, 10).

4. The reduction in the amount of rusting due to a combination of alloying elements is not equal to the sum of the reductions due to each element acting alone. The addition of higher percentages of each beneficial element further reduces corrosion loss, but this is limited by economics (8, 9, 10, 11).

5. No one of the beneficial elements listed above is essential to obtaining high corrosion resistance (8, 9, 10).

6. The effect of varying the percentage of any given element is somewhat dependent on the percentages in which the other alloys are present (9, 10).

7. The type of atmosphere at a particular site has a great effect on the rate, magnitude, and uniformity (degree of pitting) of corrosion (8, 9, 12, 13). Atmospheres are usually classified as either rural, industrial, or marine. The adjectives, moderate and severe, are sometimes used to further describe atmospheres within the industrial and marine classifications.

8. The relative effect of the various alloying elements is dependent on the type of atmosphere (8, 9).

9. The conditions of exposure significantly influence the amount, rate, and uniformity of corrosion. That is, there is a difference depending upon whether the specimen is vertical or inclined, and whether it is sheltered from rain and sun or boldly exposed (11, 12, 13).

10. Other factors such as initial weather conditions, direction of exposure, and annual changes in air pollution also have an influence on atmospheric corrosion tests (12, 13).

#### EFFECTS OF VARIOUS ALLOYING ELEMENTS

To utilize existing corrosion test data for estimating the corrosion resistance of a given low-alloy steel, it is necessary to have knowledge of the relative effect of the various alloying elements. The reports cited above contain much information in this regard.

As noted previously, the most influential alloying elements appear to be copper, chromium, phosphorus, nickel, and silicon. It will be convenient in later discussions to refer to the percentage level of a particular element by a limiting adjective, such as low, medium, or high. The meaning of these adjectives, for purposes of this report, are defined below as they are introduced. It should be noted that the corrosion experts do not always agree on the relative merits of the individual alloying elements. The following discussion of these alloying elements is a fair summary of present thinking in this regard, for industrial atmospheres.

##### Copper

Copper is probably the most important alloying element. Even in the absence of the others, very small amounts of copper, in the range of 0.01 to 0.04 percent, cause a very large increase in corrosion resistance (9). After about 0.04 percent has been added, the rate of change of corrosion resistance with further increases in copper becomes less, but significant improvement continues up to about the 0.3 percent level beyond which the rate of change in corrosion resistance with additional amounts of copper becomes smaller (9).

Because most commercial low-alloy steels contain at least 0.20 percent copper, only two levels of copper content will be considered in this report; namely, medium copper (0.15 percent to 0.29 percent) and high copper (0.30 percent or more).

##### Chromium

Chromium is very beneficial in the presence of medium and high copper. For these levels of copper, the rate of increase in corrosion resistance with increases in chromium is greatest for amounts up to about 0.50 percent. In greater amounts, chromium continues to exert a strong beneficial effect on corrosion resistance although equal increases at the higher levels have a lesser beneficial effect than at the lower level (9, 10).



Because copper is present in good quantity in all the alloys shown in Table 1, chromium can be expected to be an important variable element. Three levels of chromium content will be considered; namely, high chromium—1.0 percent to 1.3 percent, medium chromium—0.5 percent to 0.8 percent, and low chromium—0.1 percent or less.

### Phosphorus

Phosphorus is a very powerful beneficial alloying element even in very small percentages (9, 10). For medium and high copper levels, it is probably only slightly lesser in importance than chromium. Three levels of phosphorus content will be considered; namely, high phosphorus—0.08 percent to 0.12 percent, medium phosphorus—0.04 percent to 0.07 percent, and low phosphorus—0.02 percent or less.

### Nickel

Nickel is also a quite beneficial alloy. The effect of varying nickel content for compositions in which all other constituents are held constant is approximately linear (9), at least, for percentages up to 1 percent. It does not seem to be as highly effective in combination with the other beneficial alloys as it is when used alone or with copper only (9, 10). The addition of higher percentages (2, 3, 4 percent) shows a continuing beneficial effect (8), but its use in these higher amounts in low-alloy steels for industrial environments appears to be limited by economics. The relative position of nickel improves when marine atmospheres (and presumably any salt attack) are considered. Reference will be made to three levels of nickel; namely, high nickel—approximately 1.0 percent, medium nickel—approximately 0.5 percent, and low nickel—less than 0.1 percent.

### Silicon

Silicon is a strong beneficial element. It appears to cause the largest incremental increases in corrosion resistance in amounts up to about 0.3 percent with lesser but continuing effectiveness at higher levels. Reference will be made to 3 levels of silicon; namely, high silicon—0.4 percent to 0.7 percent, medium silicon—0.15 percent to 0.3 percent, and low silicon—0.10 percent or less.

### Carbon

No indication was given in any of the tests studied that carbon is a significant element as far as corrosion resistance is concerned. Furthermore, the percentages of carbon are low and fairly uniform for the alloys given in Table 1. Carbon will be ignored in subsequent discussions.

### Manganese

Horton (10) states: "Manganese is of little benefit to the corrosion resistance of complex low-alloy steels." Copson (20) indicates that manganese is only slightly beneficial in industrial atmospheres. Manganese will be considered insignificant in the following discussions, although it does appear to have some beneficial effect.

### Sulphur

Horton (10) has indicated that sulphur can be a significantly detrimental element even in small percentages. The variations in the amounts of sulphur contained in the alloys given in Table 1 are considered to be small enough to make their effect negligible.

With regard to the effect of adding the various percentage levels of chromium, phosphorus, nickel, and silicon to a high-copper steel, the data shown in Figure 1 are informative. For more complete information on the effect of the individual alloying elements the reports referenced previously should be consulted.

It should be noted at this point that the curve of corrosion resistance for a particular steel with increasing amounts of any one of the beneficial alloying elements we are

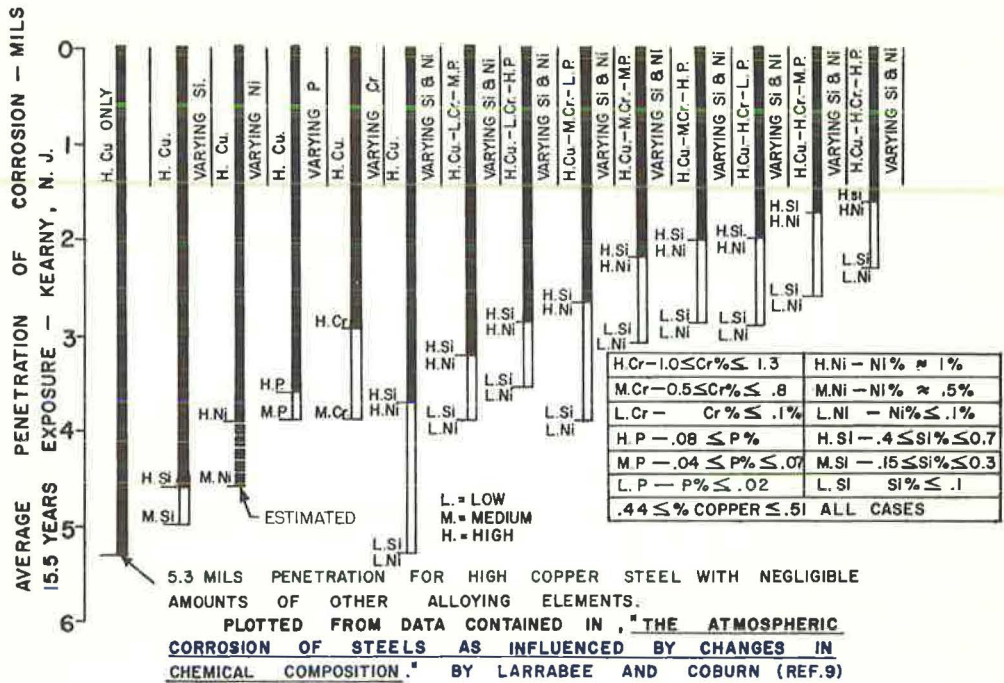


Figure 1. Effect of varying Cr, P, Ni, and Si on the corrosion resistance of high-copper-steel.

discussing here will be, generally, a continuous function (8, 9, 10). That is, it is not proposed here that the corrosion resistance changes abruptly as the percentage passes from medium to high phosphorus for instance; but, there will be a gradual change as the upper limit of the medium level defined previously is reached and the percentage passes into the higher level. The main purpose for defining the percentage levels the way they were defined is to make it easier to locate a number of compositions out of a large number of tested compositions which will have approximately the same corrosion resistance.

The data in Figure 1 show that, for high-copper steels in atmospheres similar to Kearny, N. J. (industrial), the relative effectiveness of the various percentage levels of the beneficial alloys is as follows:

1. High Cr
2. High P
3. Med. Cr or Med. P or High Ni
4. Med. Ni. or High Si
5. Med. Si

It was previously noted that the reduction in corrosion due to a combination of alloying elements is not equal to the sum of the reductions due to each element acting alone, and the effect of any given element depends upon the other elements present. This is also illustrated by Figure 1.

#### PREDICTED TIME-CORROSION CURVES FOR SEVERAL CLASSES OF LOW-ALLOY STEELS IN INDUSTRIAL ATMOSPHERES

In this report, the corrosion resistance of certain selected alloys in only industrial atmospheres (Kearny, New Jersey; Bayonne, New Jersey; and Pittsburgh, Pennsylvania) will be emphasized. The Kearny location is very close to, and should be well representative





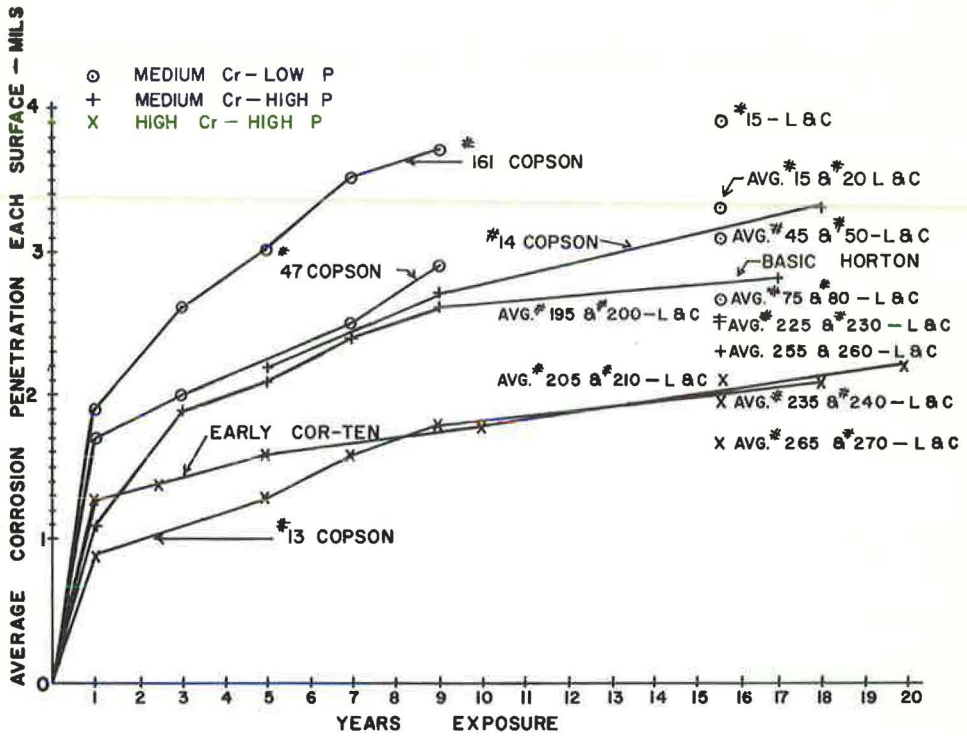


Figure 2A.

It is, of course, obvious that grouping by copper, chromium, and phosphorus is not a complete answer to predicting corrosion losses. No one alloy, even copper, is essential to obtaining corrosion resistance in the range exhibited by these alloys. Some alloys within the Table 1 groups can be made to have corrosion resistances equal to, or better than, an alloy in a "better" group by adding sufficiently large amounts of the other, less influential, but, nevertheless, beneficial alloying elements, such as silicon and nickel. This grouping procedure is recognized as being approximate, and after the groupings are made, it is necessary to look at the "typical" values and the ranges for the percentages of each element in a given chemical composition. Steels containing all the elements at the upper limit of the various percentage levels will no doubt have considerably higher corrosion resistances than steels with lower or intermediate values, and vice versa; and the ranges for the various elements are often quite large for commercial steels. Also, other properties of steels are influenced by these alloying elements and any changes in these properties, whether beneficial or detrimental, must be weighed.

It does appear, however, that reasonable estimates of corrosion resistance for a given type atmosphere can be made, for a given chemical composition, if it can be compared with several tested chemical compositions to which it is similar in percentages of the major corrosion-reducing alloying elements.

### Empirical Equations

In order to predict corrosion losses for the life of a structure it is necessary to extrapolate the available test data, most of which is for time intervals less than 20 years.

The following empirical equation is proposed to represent corrosion vs time curves (see Ref. 6 for a similar approach):



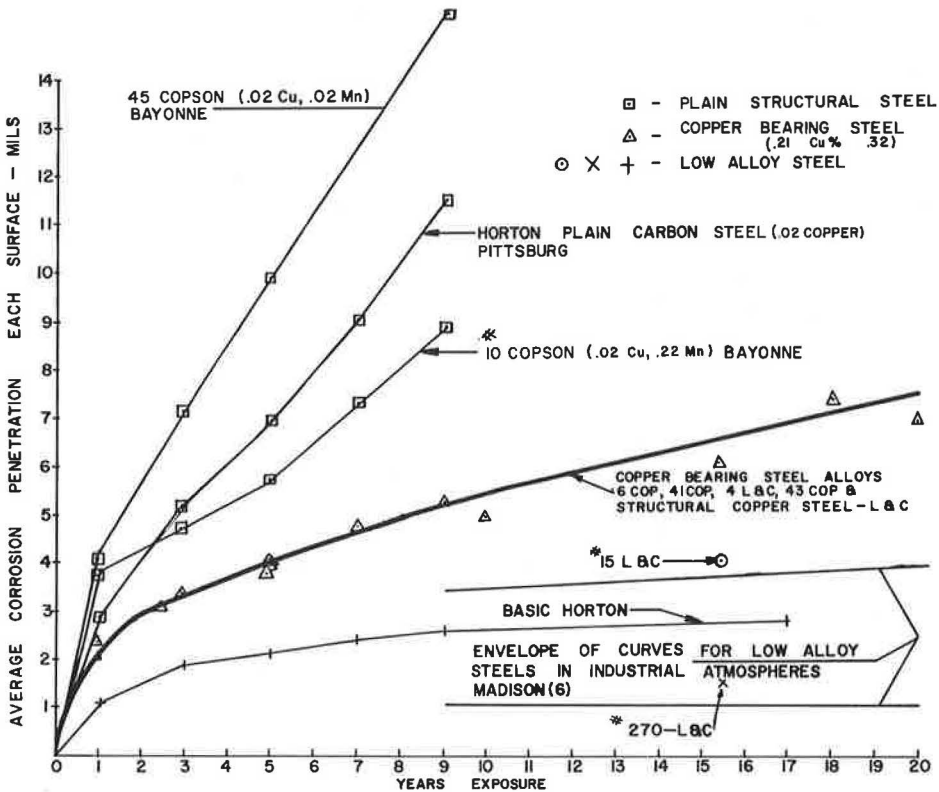


Figure 2B.

$$P = P_{10} + (N-10)R_{10-20} \quad (1)$$

where

$P$  = average penetration of corrosion, in mils, on each exposed surface, based on weight loss;

$P_{10}$  = average penetration after 10 years, in mils;

$N$  = total years of exposure; and

$R_{10-20}$  = rate of penetration with time for the time interval from 10 to 20 years, in mils per year.

It is assumed in Eq. 1 that the corrosion continues indefinitely at the same rate after 10 years of exposure. It may be that the rate of corrosion decreases further with time, but there does not appear to be sufficient data to warrant assuming this. There is no information which would indicate that the rate will increase with time, but this may be possible, under some circumstances, such as a significant worsening in the environmental conditions.

Examination of the experimental data for the steels given in Table 2 suggests the following relationships:

For the high-chromium, high-phosphorus group,

$$P = 2.0 + (N-10) 0.04 \quad (1a)$$

For the medium-chromium, high-phosphorus group,

$$P = 3.0 + (N-10) 0.06 \quad (1b)$$

No equation is listed for the medium-chromium, low-phosphorus steel since no data on the corrosion rate after 10 years were observed in the information studied. It is interesting to note that a recent Bureau of Public Roads correspondence (24) contained the following statement with regard to a steel somewhat similar to those in this group:

"General approval is given for the use of this material on Federal-Aid projects when economically justified, subject to the comments of this memorandum and the enclosed specification. Although this material has atmospheric corrosion-resistant properties, these are not considered sufficient to permit its use in an unpainted condition."

The ranges of or the chemical composition of the steel to which reference is made in the above paragraph are copper, 0.22 to 0.36 percent; chromium, 0.43 to 0.66 percent; silicon, 0.16 to 0.27 percent; carbon, 0.15 to 0.20 percent; manganese, 0.91 to 1.25 percent; and vanadium, 0.038 to 0.07 percent. Using the percentage level limits stated earlier, this particular steel can be described as: medium to high copper, medium chromium, low phosphorus, low nickel, medium silicon.

The corrosion rate ( $R_{10-20}$ ) is an important factor when long periods of exposure are anticipated, and the available data on it are sparse. More extensive study of this factor should be made. The equations proposed above are, of course, to be considered temporary approximations. The previous comments regarding the need for studying each composition within a group are applicable when deciding whether or not it can be represented by an empirical equation. The degree of accuracy of these tests is also an unknown factor although it has been stated that good agreement was found among replicate specimens for some of the tests (8, 12); for the other tests no mention of this was made.

#### Corrosion Losses for Bridge Members in Industrial Atmospheres

These values of penetration are based on the results of corrosion tests conducted under certain conditions. In order to obtain total loss of thickness for a bridge member, some modifications seem necessary.

For a two-sided plate (or beam flange or web) the following equation is suggested:

$$t_N = P_N \times 2 \text{ sides} \times E. F. \times P. F. \times S. F. \quad (2)$$

where

$t_N$  = total loss of thickness, in mils, after N years of exposure for a two-sided element;

$P_N$  = average penetration of rust, in mils, after N years estimated from empirical equation;

E. F. = exposure factor;

P. F. = pitting factor; and

S. F. = safety factor.

#### Exposure Factor

Horton states in his report (10): "These losses are for vertical surfaces openly exposed to the washing action of rain. Underside surfaces, sheltered from rain and slower drying, may corrode about 50 percent more."

Copson states (8): "It should be pointed out that shelter, crevices, undrained areas, and other factors, can increase the corrosion."

Larrabee and Coburn state (9): "For steel specimens thus exposed in industrial and semi-rural atmospheres, the ratio of the weight losses of the skyward surface to the groundward surface has been shown to be about 38 to 62."

This last statement indicates that corrosion for specimens not exposed to rain washing and direct sunlight would be about 25 percent greater than the average values used in time-corrosion curves.

In an earlier report (12), Larrabee observed that specimens exposed facing southward on a vertical rack, some sheltered, some unsheltered, had "50 to 100 percent higher corrosion losses than duplicate specimens that were exposed southward at 30 degrees to the horizontal on regular test racks" (unsheltered). Among the specimens supported vertically, he found variations in corrosion between the sheltered and unsheltered specimens of up to 20 percent. In some cases, the exposed specimens corroded more, and in some cases less, than the unexposed specimens. It was indicated that these variations were affected by the directions in which the specimens faced. All of the above was based on four years of exposure.

Also, in this same report (12), some tests on steels exposed in tunnels were discussed. After one year, the amount of corrosion of a steel alloy similar to the high-copper, high-chromium, high-phosphorus group, Table 2, of this report was 11 times as much in a continuously damp tunnel as it was, for the same steel, exposed on the roof of a building in an industrial atmosphere. After 4 years' exposure in a tunnel with dry walls, the amount of corrosion for this same steel was approximately 8 times the corrosion which occurred on the roof. This low-alloy steel showed little or no advantage over copper-steel in the tunnel exposures.

Another work (11) comments as follows: "To make a broad generalization, slow-rusting steels show to maximum advantage when they are freely exposed to the open air in industrial environments; it is doubtful whether, from the corrosion aspect, their use is worthwhile under sheltered conditions of atmospheric exposure, or where immersion in natural waters, or burial in the soil, is involved."

In substantiation of this, test results are listed for copper-steels in a tunnel in England. After 5 years' exposure, no significant change in rusting rate with varying copper content was found. Also, "as a further example of the ineffectiveness of low-alloy additions in slowing down rusting under sheltered conditions, tests by BISRA in indoor atmospheres have failed to reveal any substantial difference in the rusting of a chromium/copper-steel and of an ordinary mild steel in most of them. The tests sites covered a wide range of domestic and industrial conditions, from bathrooms to locomotive sheds" (11).

In attempting to explain the mechanism by which rusting of low-alloy steels proceeds, Copson (14) concluded with the following remark: "Variation in weather affects corrosion, but may affect different steels differently. This is because rainfall has a dual role. Moisture must be present for steel to corrode, but washing away of soluble material is beneficial. Dew, fog, high humidity, and shelter would be expected to be harmful."

The statements presented above indicate that:

1. The position of a test specimen, i. e., whether vertical, horizontal, or inclined, has an effect on the amount of corrosion.
2. The direction in which a test specimen faces, with respect to rain, wind, and sunlight, has some effect on the amount of corrosion.
3. Steel specimens, sheltered from rain and sunlight, will generally corrode more than steel boldly exposed to the elements.
4. The conditions under which the tests, for which the time-corrosion curves in Figure 2 were drawn, were such that the tests probably show the corrosion under relatively favorable exposure conditions.

To account for possible variations from test values of corrosion, it seems clear that an exposure factor should be used as a multiplier when estimates of corrosion for a structural member, such as an interior bridge beam, are being made.

Based on the previous discussion, it appears that this factor could range from 1.25 to as high as, possibly, 10. The higher factor would apply to steel that is continuously wet, a more severe condition than normal bridge environments. The lower figure applies to steel exposed under favorable conditions.

Conversations with steel producers' representatives have indicated that they expect very little difference in the amounts of corrosion between interior and exterior bridge members.



On the experimental bridges, it will probably take at least 5 years to detect any significant change in long-term corrosion between the two locations. In fact, the corrosion in the early years will probably be greater for boldly exposed steel.

It is hoped that a more reliable determination of exposure factors will result from further research. At this time, it is proposed that a factor of 2 be used for bridge members exposed in industrial atmospheres and subjected to only atmospheric moisture, and that careful study be continued. This may be slightly conservative under the best conditions or, perhaps, grossly unconservative for more severe cases such as long periods of dampness. In any case, it appears that careful observation of this effect on an actual structure must be continued for a relatively long time.

### Pitting Factor

Most publicized corrosion test results are presented in a form which shows the average depth of penetration of rust into an exposed surface. The question arises as to how much greater penetration above the average, can be expected, and what effect this might have on the strength of a structural member. Copson's report (8) includes some data on pitting, as follows:

AVERAGE PIT DEPTHS, MILS<sup>a</sup>  
(Industrial Atmosphere, Bayonne, New Jersey)

Sample	1 Yr	5.1 Yr	7.1 Yr	9.1 Yr	18.1 Yr
#13 Copson	6.0	10.5	9.9	10.4	13.0
#14 Copson	—	10.4	—	12.3	12.4
#47 Copson	8.0	—	10.4	11.6	—
#161 Copson	9.0	11.8	11.9	12.3	—

<sup>a</sup> Average of 4 deepest pits on skyward and 4 deepest pits on groundward surface.

As can be seen in Table 1, these steels represent each of the 3 groups studied previously. The above data show that there is very little variation among the groups in the depths of the deepest pits after long exposures. The deepest pits were on the order of 3 to 4 times the average corrosion penetration calculated from weight loss. No data on the total amount of pitting were given in the report (8); and the other reports (9, 10) studied made little or no mention of pitting.

Pitting will certainly reduce static strength due to loss in cross-sectional area; but this effect may be very slight, and will be considered as undetermined for the present. Pits will also act as stress-raisers and will tend to result in a somewhat lower fatigue strength for rusted steel. The general problem of the effect of repetitive, fatigue-type loadings on bridge materials is, itself, not well understood; but it is well known (15) that roughening a surface, whether due to corrosion or some other cause, reduces the fatigue life of a member.

In view of the above, consideration of even a temporary quantitative factor to account directly for pitting will be postponed until further study is made.

### Safety Factor

In light of the many uncertainties mentioned, the inaccuracies inherent in atmospheric corrosion testing, the irregularities from test conditions which are likely to occur on an actual structure, variations in chemical compositions from stated typical values, differences in atmospheric conditions, and of the uncertainty involved in extrapolating test data, it seems reasonable to apply a safety factor to estimates of material loss due to corrosion.

Madison (6) suggests a design factor of 3, but does not provide any special factor to account for exposure variations. In this report we have suggested a factor of 2 to

account separately for exposure. An additional safety factor of at least 2 seems reasonable for bridge structures with a life expectancy of about 50 years. This factor is suggested with the idea that careful inspections of unpainted structures will be made on a regular basis until more definite estimates of corrosion can be made.

This safety factor is considered to be independent of, and in addition to, the safety factor already incorporated in structural design criteria to account for other factors such as mill tolerances, strength variations, and overloads.

#### ESTIMATED THICKNESS LOSSES FOR BRIDGE MEMBERS AND REDUCTION IN THEORETICAL CROSS-SECTIONAL PROPERTIES

Calculated values of  $P_N$ , from Eqs. 1a and 1b, are shown below multiplied by 4 to account for the exposure and safety factors.

$$\begin{aligned} \text{Eq. 1a: } P_{25} &= 2.6 \times 4 = 10.4 \text{ mils} \\ P_{50} &= 3.6 \times 4 = 14.4 \text{ mils} \\ P_{100} &= 5.6 \times 4 = 22.4 \text{ mils} \end{aligned}$$

$$\begin{aligned} \text{Eq. 1b: } P_{25} &= 3.9 \times 4 = 15.6 \text{ mils} \\ P_{50} &= 5.4 \times 4 = 21.6 \text{ mils} \\ P_{100} &= 8.4 \times 4 = 33.6 \text{ mils} \end{aligned}$$

Using handbook dimensions and cross-sectional properties, the approximate percentage reductions in moment-of-inertia and web shear area, for two wide-flange beams, were calculated for various values of corrosion penetration,  $P$ . The percentage reduction in thickness for several sizes of plate was also calculated. This information is given in Table 2.

For purposes of discussion, in this report, a reduction in these cross-sectional properties of 5 percent, or more, has been arbitrarily selected as being sufficiently large to designate as "significant change."

In summary of Table 2: For steels whose time-corrosion curve is approximated by Eq. 1b, after 25 years—significant change for plates less than about  $\frac{7}{16}$  in. thick; after 50 years—significant change for plates less than about  $\frac{3}{4}$  in. thick; and after 100 years—

TABLE 2

EMPIRICAL EQUATION	SECTION OR PLATE THICKNESS	YEARS EXPOSURE	$P_N$	PERCENT REDUCTION IN MOMENT OF INERTIA STRONG AXIS	PERCENT REDUCTION IN WEB AREA OR PLATE THICKNESS
1a	36 WF 194	25	10.4	2.1	2.7
		50	14.4	2.9	3.7
		100	22.4	4.5	5.8
	36 WF 150	25	10.4	2.6	3.3
		50	14.4	3.6	4.6
		100	22.4	5.6	7.2
	1/2 " Plate	25	10.4	-	4.2
		50	14.4	-	5.8
100		22.4	-	9.0	
1 " Plate	25	10.4	-	2.1	
	50	14.4	-	2.9	
	100	22.4	-	4.5	
1b	36 WF 194	25	15.6	3.1	4.1
		50	21.6	4.3	5.6
		100	33.6	6.7	8.7
	36 WF 150	25	15.6	3.9	5.0
		50	21.6	5.4	6.9
		100	33.6	8.4	10.7
	1/2" Plate	25	15.6	-	6.2
		50	21.6	-	8.6
		100	33.6	-	13.4
	1" Plate	25	15.6	-	3.1
		50	21.6	-	4.3
		100	33.6	-	6.7

significant change for plates less than about  $\frac{15}{16}$  in. thick, and for I of 36WF194. For steels whose time-corrosion curve is approximated by Eq. 1b, after 25 years—significant change for plates less than about  $\frac{5}{8}$  in. thick; after 50 years—significant change for web of 36WF 194, I and web of 36WF150, and plates less than about  $\frac{7}{8}$  in. thick; and after 100 years—significant change for plates less than about  $1\frac{3}{8}$  in. thick, and for I and web area of both WF shapes.

It appears that Table 2 is evidence that further study should be made of the reduction in cross-sectional properties of unpainted structural members, and that revisions to design procedures may be necessary for some cases. There are, of course, many other shapes and sizes of structural members, as well as other important sectional properties and dimensional parameters which should be investigated. The particular properties and shapes discussed above were intended only to serve as an indication of the existence of a possible problem area, and were not intended to be comprehensive or conclusive. The properties which govern the design of a structural part are obviously dependent upon the type of member (tension, compression, beam, built-up girder, etc.), and the specifications which govern the design. It is noted that AASHTO states (section 1. 6. 14): "Metal exposed to marked corrosive influences shall be increased in thickness or specially protected against corrosion."

To account for material losses in design, one or more of the following provisions could be made:

1. Require a thickness of sacrificial material, over and above that required by normal design, equal to the amount of thickness loss predicted (see also 6).
2. Place a minimum thickness limitation on unpainted members, such that high percentage reductions in significant design parameters will not be expected.
3. Reduce the allowable stresses when design calculations are based on original cross-sectional properties; or, reduce the published values of cross-sectional properties by some percentage, or percentages, representative of the amount of corrosion predicted and the importance of the particular property.
4. Do nothing if the material losses can be tolerated as being insignificant.

The design provisions chosen must take into consideration the type of structure, the expected life, the environment, and the sensitivity to change of the governing parameters.

A thorough study of this matter is beyond the scope of this report. Further study is recommended.

In the foregoing discussion, we have been considering material losses due to environmental and exposure conditions somewhat similar to those at the sites of atmospheric corrosion tests. To account for some expected differences in exposure conditions for bridge members, and to account for other uncertainties resulting from somewhat incomplete test data and lack of long-term service experience, the test values were multiplied by an "exposure factor" and a "safety factor." It must be pointed out that the losses predicted on this basis, although apparently significant enough in some cases to require their consideration in design, are relatively small compared to the amount of material loss which may occur due to other factors as discussed in later paragraphs.

## EFFECTS OF LOADING

The discussions of corrosion resistance tests in this report have been centered, primarily, around three references (8, 9, 10), which report on the performance of unstressed steel specimens. In the previous section an attempt was made, based on test data from the above reports, to estimate the material loss and resulting loss in some cross-sectional properties due to corrosion.

In an actual bridge structure, the material will be subjected to stress and strain due to both static and dynamic loads. The possible effects of simultaneous corrosion and loading on the corrosion resistance and strength of bridge members must be considered.



Some questions in this regard have been formulated and are listed below. Quantitative information on their pertinence, in relation to the low-alloy steels being considered, does not seem to be readily available. Comments relative to each of these questions are given so that some estimate of their importance can be made. It is hoped that future studies will provide more complete answers to these questions.

Will the Amount of Corrosion Be Higher Than Indicated From Tests On Unstressed Specimens When the Material Is Subjected to Static or Dynamic Loading?

In other words: Will the protective coating remain intact, relatively impervious, and adherent; or, will the surface crack, "flake-off," or otherwise allow corrosion to continue at a high rate under the influence of stress and strain, either static or dynamic?

Relative Comments:

"It is well known that the presence of stresses in a material can accelerate the rate of its corrosion" (16). However, this effect is very small, and, "accordingly, only a few instances are known where the existence of uniformly distributed stresses can cause an increase in corrosion. . . . The instances where accelerated corrosion rates may be ascribed to the influence of stresses usually are the result of nonuniform stresses" (2). Also, "only in unusual circumstances do static stresses accelerate attack" (17).

With regard to another aspect of this question, Madison (6) states: "Because the rust coating of weathering steels is strongly bonded to the steel, it will not break off under stress, provided the member is not loaded beyond its yield point." He also notes that the coating is hard, resists scratching, and will reform itself if it is scratched. Whether or not small cracks develop under strains below the yield point, and whether or not cyclic loading destroys the bond, is not clear. It has been stated that the application of alternating stresses to metals during exposure strongly intensifies corrosion damage (17).

In view of the above, and since the coating will develop gradually after dead loads have been applied, it appears that the effect of static loads on the amount of corrosion is negligible in the absence of cracks in the base metal. The effect of cyclic loading on corrosion resistance, however, does not as yet appear to be resolved.

Does the Presence of Corrosion Affect the Static Strength of the Material to a Greater Extent Than Would Be Expected Due to Loss of Material Alone?

Relative Comments:

The detrimental effect of pits on static strength was discussed previously in this report under "pitting factor."

Another phenomenon of possible relevance to this question is that of stress corrosion cracking. The following statements are taken from the literature on this subject:

"Stresses play a much more dangerous part in the presence of corrosion in those cases when their action, combined with that of a corrosive medium, may lead to a brittle failure—the so-called stress corrosion cracking (or corrosion cracking)" (16).

"While there are many causes of stress-corrosion cracking, it is difficult to predict when this type of failure will actually occur. Each alloy is susceptible in only a few rather specific environments. The stress condition, structure, composition, heat treatment of the alloy, and duration of exposure all have a bearing on the cracking. Failures are less common than might be supposed, although the cracking can be drastic when it occurs" (17).

Some combinations of steel and corrosive environment which have been found to produce stress-corrosion cracking are as follows:



Description	Corrodent	Reference
Low-alloy steel	H <sub>2</sub> S	18
Steel	Hot nitrate solutions	19
Steel	Nitrates at room temp. <sup>a</sup>	19
Carbon and low-alloy steels	Aqueous solution of HCN (prussic acid)	16
Low-carbon-steels	Solutions of Na OH-NaSiO <sub>3</sub>	16
Low-carbon-steels	Solutions of nitric acid	16
	Salts (calcium, ammonium, sodium)	

<sup>a</sup>"Such failure occurred, for example, in 0.7 percent C steel cables of the Portsmouth, Ohio, Bridge after 12 years in service" (19).

Also, "to date, we have found far from all the corrosive media implicated in brittle corrosion-mechanical failure under tensile stresses" (16).

It is noted that some corrosion has always taken place on even-painted structures. There does not seem to be any direct evidence to show that stress-corrosion is a problem for steel bridges, either painted or unpainted, with the exception of the special combination of high stress and a particular corrodent which caused the bridge cable failure. Possibly, the presence of highway salts may be a condition conducive to stress-corrosion cracking.

It does seem prudent, to learn more about this phenomenon since all corrosion effects are likely to be magnified when a structure is left unpainted.

#### Does the Presence of Corrosion Affect the Ability of the Material To Resist Cyclic Loads?

Fatigue and its effect on highway structures is, itself, not clearly understood. A comprehensive survey of this problem is beyond the scope of this report. The brief discussion which follows is intended to indicate some of the ways in which cyclic load resistance is likely to be affected when structures are left unpainted. Reference was made in an earlier paragraph to the effect of fatigue on the corrosion process.

The fatigue behavior of steel, as determined by fatigue tests conducted under normal atmospheric conditions, is characterized by the existence of fatigue limits. When more severe corrosion is present, fatigue limits do not exist and the fatigue life is reduced. There is a difference in the influence which corrosion exerts on fatigue properties, depending upon whether corrosion precedes fatigue, or whether the two occur simultaneously (corrosion-fatigue).

Corrosion causes roughening of the surface and, therefore, reduces fatigue life. Pitting causes a reduction in cross section and acts as a stress-raiser, thus increasing the stress amplitude, which lowers the fatigue life. In short, "when corrosion precedes fatigue, a definite endurance limit is observed, as it is in the fatigue process, but its magnitude will be smaller owing to superficial or deep damage caused by prior corrosive action" (16).

Corrosion-fatigue, on the other hand, may be a still more serious problem. For example, "corrosion-fatigue can be defined as the type of failure which occurs when a component is subjected to cyclic stressing in a medium which is able to attack the material continuously if it becomes chemically exposed; in other words, if the material is capable of reacting with the environment in the absence of an oxide film or of a film of corrosion product, corrosion-fatigue is possible if not certain" (19).

Fatigue tests conducted in a vacuum indicate fatigue limits somewhat higher than fatigue tests conducted in the ordinary atmosphere. There is, also, a large decrease

in fatigue strength for steels tested in more corrosive environments, such as water of seawater. In fact, "the more corrosive the environment, the lower the fatigue strength."

Both ordinary fatigue and corrosion fatigue are dependent upon the frequency of loading. The effect of this variable seems to be more pronounced for corrosion-fatigue. "The lower the frequency, the lower the corrosion fatigue limit based on the same number of cycles" (16).

Most fatigue tests are conducted under normal indoor atmospheric conditions. It may be that these tests are not sufficiently representative of actual conditions on an unpainted bridge. The laboratory test environment, of course, differs from the actual bridge environment; but, more importantly, the time required for serious atmospheric corrosion to take place is much longer than the times used in these tests. The fact that lower frequencies are more harmful than the higher frequencies used is another reason for reevaluating the validity of the results of these tests for use on unpainted structures. Whereas, "for ordinary constructional steels, the use of effective coats substantially restores the existence of a true fatigue limit even under the conditions of corrosion fatigue" (16). It is, of course, well known that no protective coating system is 100 percent effective; but, since corrosion is invited on a wholesale basis when painting is eliminated, the problem of corrosion-fatigue seems even more important for unpainted structures.

While no quantitative data have been presented on the fatigue strength of unpainted steel bridges, it is important to note that: "Corrosion in any form is harmful to the fatigue life of a metal" (15).

Further study of this problem is recommended.

#### OTHER FACTORS INFLUENCING THE CORROSION OF STRUCTURES

This report has been concerned, primarily, with industrial environments. The performance of low-alloy steels in other environments is now briefly discussed.

##### Severe-Marine Environment

Description—material subjected to more or less continual salt spray. Test results indicate that the corrosion rate is constant with time. The time-corrosion curve does not "flatten out." Madison (6) suggests a corrosion rate of 1.25 mils per year per exposed surface as being representative. Deeper pits than those which occur in industrial atmospheres can be expected. Unpainted steel is not recommended for this type of environment.

##### Moderate-Marine Environment

Description—near seacoast, but no direct salt spray. No definite distance from the source of salt water is presently used to define the limits for this type of environment. The direction and magnitude of prevailing winds, and the frequency and severity of hurricanes and other storms must be considered in this regard. Table 3 includes some test results for two "moderate-marine" locations. The Kure Beach site was located 800 feet from the surf, while the Block Island site was on a bluff overlooking the ocean. It is obvious from the data that moderate-marine locations can cause a significantly larger amount of material loss than industrial environments. Deeper pits can also be expected. Marine environments are corrosive due to the presence of salts in the atmosphere; and in some cases, to a lesser degree, to the abrasion of the metal surfaces by wind-borne sand particles. Madison (6) suggests that a corrosion rate of 0.3 mils per year per side is representative of this type environment. This is about 5 or 6 times the long-term corrosion rate for industrial atmospheres. It is felt that unpainted bridges should not be constructed in marine environments at this time. It is noted, again, that the distance limits on moderate-marine environments are not well defined; thus it may be difficult to determine whether a given New Jersey site is industrial, rural, or marine. Further study of this problem is recommended.

TABLE 3

AVERAGE DEPTH OF PENETRATION OF CORROSION IN MILS FOR VARIOUS ATMOSPHERES AS DETERMINED BY WEIGHT LOSS

GROUP	ALLOY IDENTIFICATION	INDUSTRIAL *			MODERATE		MARINE		SEMI-RURAL
		KEARNY, N. J.	BAYONNE, N. J.		KURE BEACH, N. C.		BLOCK ISLAND, R. I.		SOUTH BEND, PA.
		15.5 YRS.	9.1 YRS	18.1 YRS	7.5 YRS	15.5 YRS	9.1 YRS	17.1 YRS	15.5 YRS
High Cu Med Cr Low P	15 L & C	3.9	-	-	-	8.2	-	-	5.1
	161 Copson	-	3.7	-	5.5	8.1	7.3	-	-
	20 L & C	2.7	-	-	-	5.9	-	-	3.2
	45 L & C	3.6	-	-	-	8.0	-	-	5.4
	47 Copson	-	2.9	-	3.2	4.8	4.3	-	-
	50 L & C	2.6	-	-	-	5.1	-	-	3.3
	75 L & C	3.0	-	-	-	5.7	-	-	4.2
	80 L & C	2.3	-	-	-	4.2	-	-	2.7
High Cu Med Cr High P	195 L & C	2.9	-	-	-	6.0	-	-	3.6
	200 L & C	2.2	-	-	-	4.6	-	-	2.6
	225 L & C	2.8	-	-	-	5.5	-	-	3.7
	14 Copson	-	2.7	3.3	3.1	4.7	4.4	-	-
	230 L & C	2.2	-	-	-	4.0	-	-	2.4
	255 L & C	2.6	-	-	-	4.7	-	-	3.2
	260 L & C	2.0	-	-	-	3.5	-	-	1.9
High Cu High Cr High P	205 L & C	2.3	-	-	-	5.4	-	-	2.7
	210 L & C	1.9	-	-	-	3.9	-	-	2.1
	235 L & C	2.1	-	-	-	4.6	-	-	2.6
	240 L & C	1.8	-	-	-	3.8	-	-	1.8
	265 L & C	1.7	-	-	-	3.5	-	-	1.7
	13 Copson	-	1.7	2.0	2.3	3.3	3.3	-	-
	270 L & C	1.6	-	-	-	3.2	-	-	1.3

\* Repeated from TABLE 2 for Comparison

### Rural and Semi-Rural Environment

It is generally recognized that rural and semi-rural atmospheres, which contain only small amounts of salts and pollutants, are less corrosive to ordinary steel than industrial or marine environments. It is somewhat surprising, therefore, that the test results shown in Table 3 indicate weight losses for South Bend, Pa. (classified as semi-rural), that are often larger than (by as much as 50 percent) the losses which occurred to similar specimens exposed, for the same length of time, in Kearny, N. J. (classified as industrial). One possible rationalization for this is that the rust coating forms more quickly and becomes relatively impervious sooner in the industrial environment than in the semi-rural. It is expected that the corrosion rate will be higher in the first few years and lower in the later years for the industrial location compared to the semi-rural. If nothing else, these data give further indication that the corrosion of unpainted steel is a complex phenomena, and that a safety factor should be used when attempting to translate test results into design criteria.

It is worthy of note in Table 3, that for all test sites, the more highly alloyed steels, as a group, exhibited the best corrosion resistance, although there was considerable overlapping among the groups. It must be remembered, however, that these groups were formed on the basis of the importance of the various alloys in industrial atmospheres, and that the effect of these alloys may vary considerably in other environments.



### Submersion in Water

When these materials are continuously submerged in water, the protective rust film is not able to develop, and there is no increase in corrosion resistance over ordinary steels.

### Burial in Soil

Low-alloy steels offer no apparent advantage over ordinary steels for this type of environment.

The following factors may cause more serious corrosion than that which might be expected from examination of atmospheric corrosion test results.

### Highway Deicing Salts

As indicated by the high amounts of corrosion which occur in marine environments, very serious corrosion damage may result if road salts or salt-laden water is able to leak through deck joints, drain onto bridge members, or otherwise come in contact with the unpainted steel. Special care in design and inspection of unpainted bridges will be necessary to guard against this possibility.

### Water Pockets

Since serious corrosion occurs when low-alloy steels are submerged in water, special care must be taken in detailing so that water does not become trapped and allowed to sit on the steel for long periods of time. It is, of course, good practice to avoid this condition on all structures, but it becomes even more important when steel is left unpainted.

### Unusual Concentrations of Atmospheric Corrosives

Atmospheric conditions within a given geographical area may vary widely due to the presence of local sources of contaminants, such as chemical plants. Each site should be surveyed with this in mind, and those making future inspections of unpainted bridges should be on guard against the possibility that new sources of contamination may be introduced near the site.

### Locomotive Blast and Exhaust

Bridges constructed over railroads are generally subjected to serious corrosion conditions. It is felt that unpainted steel bridges should not be recommended for this type of usage.

### Effects of Welding

Manufacturers' recommendations should be followed in selecting welding electrodes and procedures. Future inspections should call for careful examination of corrosion in the vicinity of welds.

### Fretting Corrosion

Relative movement of surfaces in contact may cause serious corrosion, and the contact surfaces should be protected.

### Dissimilar Metals in Contact

For unpainted structures it is desirable that all metal, including fasteners, be of the same material. Galvanized metal should be satisfactory (6) as long as the zinc coating remains intact. When contact between dissimilar metals cannot be avoided, then both surfaces should be painted or otherwise protected.

The importance of these factors will depend upon the actual conditions on, and at the site of, a particular structure. In many cases, they can be minimized by proper design,



detailing, and specifications. In some cases they will preclude the use of unpainted structures.

### TEST PROGRAM

In developing the testing program for the evaluation of one experimental bridge to be constructed in the Newark, N. J., area, the following factors were felt to be in need of clarification or of verification for a given chemical composition in a given type of atmosphere:

1.  $t_x$  = the number of years of exposure required for the time-corrosion curve to become essentially linear (assuming linearity to be the actual case);
2.  $P_{t_x}$  = the depth of corrosion penetration into an exposed surface after  $t_x$  years;
3. The corrosion rate after  $t_x$  years;
4. The "exposure factor";
5. The "pitting factor";
6. The degree of reproducibility of results;
7. Effect of static loads on corrosion rate;
8. Effect of cyclic loads on corrosion rate;
9. Effect of prior corrosion on static and dynamic strength;
10. Possible effect of corrosion-fatigue and stress corrosion;
11. Effect of "other factors";
12. Appearance of rusted steel; and
13. Rust-staining of adjacent surfaces.

Several sources (13, 17, 21, 23, and ASTM A224-46) were used as guides in developing the details of the proposed tests. The major proposals are as follows:

1. Weight loss and loss in tensile strength tests for sample panels of steel, similar in strength and chemical composition to the bridge steel, exposed on or near the experimental bridge in an unstressed condition.
2. Exposure of a number of unstressed sample panels of the bridge steel for use in future tests after a protective coating has formed.
3. Careful periodic inspection of the bridge members.
4. Photographic record (in color).

The proposed weight loss and loss of tensile strength test is intended to supply better information on factors 1 through 6, listed above, for the first experimental bridge.

This test program will not take into account the possible effects of: cyclic load on corrosion rate, stress-corrosion, or reduction in fatigue strength. As noted earlier, these factors are thought to be areas in need of further study. No testing is proposed at this time to account for these factors, but it may be possible to conduct some tests of this nature on the future experimental bridges. Additional information relative to these factors will be sought.

Careful inspection of the actual bridge members will be made on a regular basis. A detailed procedure for conducting these surveys will be developed. Visual examinations supplemented by ultrasonic and manual thickness measurements are being considered.

### OBSERVATIONS AND CONCLUSIONS

1. Since general corrosion is invited on unpainted structures, it is necessary to give greater emphasis to corrosion effects than is usually required for painted structures.
2. There is no evidence in the data studied that rust penetration stops completely, but there is a great reduction in the rate at which this penetration proceeds.
3. The detrimental effects of atmospheric corrosion may not become significant until relatively long periods of time have elapsed. Since most existing unpainted structures have been in use for less than five years, this practice should be considered

unproven, especially for highway bridge applications. All structures using unpainted steel should be considered experimental and should be kept under careful observation.

4. Low-alloy steels can be, and have been, formulated which will provide the highway designer with a material which is very resistant to atmospheric corrosion. The potential economy which may result if maintenance painting of structures were not required, makes further consideration and exploration of possible uses justified.

5. There is no one "magic" alloying element or combination of elements which is necessary to produce a material having corrosion resistance in the range being considered. Instead, there are a virtually infinite number of possible combinations of the major corrosion-reducing alloying elements which might be considered for use in an unpainted condition for some applications, in some environments. Economics of manufacture and the effect of the alloying elements on other properties of the steel will somewhat restrict the number of possibilities.

6. Even if it is assumed that all steels which might be offered for unpainted use will contain at least 0.20 percent copper, there is still a wide spectrum of possible time-corrosion curves, the nature of which depends on the exact chemical composition, the type of environment, and the exposure conditions. For a given type structure, with a given design life, in a given environment, some criteria must be established for determining whether or not a given chemical composition is suitable for the unpainted use being considered.

7. There are much data available on the results of atmospheric corrosion tests of a wide variety of low-alloy steels. It is felt that these data are sufficient to allow a reasonable estimate to be made of the time-corrosion curves for many low-alloy steels in industrial environments by comparing the percentages of the major corrosion-reducing elements which they contain with the percentages of the tested alloys.

The amount of material loss to be expected for a given chemical composition in a given environment can be estimated, in many cases, by empirical equations, based on a study of test data, multiplied by an "exposure factor" and "safety factor" appropriate to the particular conditions of use. For interior bridge beams, in industrial environments, assuming a life expectancy of about 50 years, an exposure factor of 2 and a safety factor of 2 are suggested.

8. Determinations of what is "sufficient corrosion resistance" will be dependent upon: the type of structure, the expected life of the structure, the environment at the site, and the amount of reduction in cross-sectional properties that can be provided for, or tolerated, in design. Further study of the effect of losses of material on the dimensional parameters used in design will be necessary before more general recommendations can be made in this regard.

9. The design provisions referred to above could take the form of: an additional thickness of sacrificial material, a minimum thickness requirement, reduction in allowable stresses, reductions in published values of cross-sectional properties for structural shapes and plates, or nothing, if the material loss can be tolerated.

10. It appears that weldability can be taken care of adequately by specified chemical composition limitations, and a requirement for evidence of weldability, for the steels now commercially available for unpainted bridges. Special care is necessary, however, with regard to this important property, because alloys formulated to obtain high corrosion resistance may contain larger than usual amounts of chemical elements which are generally unfavorable to weldability.

11. It is recommended that further information be obtained in regard to: accelerated corrosion testing, corrosion rate after long periods of exposure, "exposure factors," "pitting factors," effects of loading on amount of corrosion, reduction in fatigue properties due to prior corrosion, stress-corrosion, and corrosion-fatigue. It is recommended that obtaining an authoritative evaluation of the pertinence of stress-corrosion and corrosion-fatigue to unpainted highway bridges be given high priority, since failures due to these phenomena occur suddenly and are difficult to predict and arrest beforehand.

12. There are other factors such as: welding, water pockets, deicing salts, local chemical plants, locomotive exhausts, dissimilar metals, relative movements between



contact surfaces, and other environments which may cause more serious corrosion than might be predicted from examinations of atmospheric corrosion test results. The danger of serious damage due to corrosion by salts seems to be a particularly important possibility for highway bridges. In many cases, these factors can be minimized by proper design, detailing, and specifications, but they require special consideration, and in some cases will preclude the use of unpainted steel.

13. Existing specifications are in need of improvement if they are to be used for nonproprietary construction specifications. For instance, the requirement of "four to six times the corrosion resistance of ASTM A7 steel" is not at all clear and hence, for specification purposes, is meaningless. The amount of material loss which will occur to any steel is quite variable and depends on the type of atmosphere, the conditions of exposure, the exact chemical composition, the exposure time, and many other factors. When comparing the corrosion resistances of different steels, the conditions under which the comparisons are made must be clearly defined. There are other problems, such as the rather large variations in percentages of the alloying elements which are used (or which may be used) by different steel producers, and the lack of a suitable acceptance test for corrosion resistance, which make the preparation of specifications a difficult task.

Future specifications for unpainted steel should be based on chemical composition. This can, probably, best be done by establishing a list of chemical compositions which have been prequalified as having sufficient corrosion resistance for the particular use intended.

If a requirement such as "four to six times the corrosion resistance of ASTM A-7 steel" is incorporated into a specification then the conditions under which the comparison is to be made should be clearly defined.

14. Despite the possible need for conservative revisions in design procedures, and the existence of several areas of considerable doubt, it is felt, at this time, that unpainted steel can be used safely for bridge structures in industrial environments as long as they are given careful consideration in design and are kept under close study. If it is found that the material is not performing satisfactorily, or if further research fails to clear up the doubts, then the structure can be painted.

The intention of the literature search, on which this paper reports, was to lay a foundation for our future studies and eventual evaluation of the experimental New Jersey bridges. The purpose of this paper was to bring to the attention of highway engineers the data that were found, the tentative conclusions we have drawn, the factors that are felt to be in need of consideration, and the plans we are making for our future studies in the hope that further discussion and additional information might be generated.

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