Research Report
UKTRP-84-14

CORROSION OF CABLE SUSPENSION BRIDGES

by

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Federal Highway Administration
U.S. Department of Transportation

May 1984

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Corrosion of Cable Suspension Bridges		5. Report Date May 1984 6. Performing Organization Code
7. Author(s) Theodore Hopwood, II and James H. Havens		8. Performing Organization Report No. UKTRP-84-14
9. Performing Organization Name and Address Kentucky Transportation Research Program College of Engineering, University of Kentucky 533 South Limestone Lexington, Kentucky 40506-0043		10. Work Unit No. (TRAIS) 11. Contract or Grant No. KYHPR-84-95
12. Sponsoring Agency Name and Address Kentucky Transportation Cabinet State Office Building Frankfort, Kentucky 40622		13. Type of Report and Period Covered Interim 14. Sponsoring Agency Code

15. Supplementary Notes

Study Title: Special Problems of Metal Bridges

16. Abstract

This report discusses corrosion problems encountered on the cables of suspension bridges. A historical review is given of past cable suspension bridge corrosion and related laboratory work.

Findings of inspections of suspension bridges at Maysville, KY, Covington, KY, and Portsmouth, OH, are discussed. Recommendations are presented.

17. Key Words Bridges, Cable, Corrosion, Fatigue, Galvanizing, Suspension, Steel, Wires	18. Distribution Statement

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

SUMMARY

Suspension bridge wire is inherently susceptible to corrosion damage. The main cables of suspension bridges, which are composed of many of these wires, are usually fracture-critical (non-redundant) structural members. Therefore, consequences of corrosion of wires of suspension-bridge cables can be very severe.

At least seven different corrosive mechanisms may be involved in bridge-wire corrosion and cracking problems: uniform corrosion, pitting corrosion, crevice corrosion, fretting, stress corrosion, hydrogen cracking, and corrosion fatigue. Of these, stress corrosion, hydrogen cracking, and corrosion fatigue are cracking processes. The other forms of corrosion may be contributory or prerequisites for the cracking process.

Suspension bridge wire corrosion and cracking problems have not been infrequent since 1900. In the US, three wire-cable suspension bridges collapsed during that period, with some loss of life. Worldwide, at least 13 cable suspension bridges have experienced corrosion or cracking damage sufficient to warrant major repairs or complete cable replacement.

Laboratory tests of bridge wire were first performed in 1935 on specimens of heat-treated wire from the Mt. Hope and Ambassador bridges. In those tests, the Bureau of Standards attempted to replicate service fractures. In 1940, Pollard (Bureau of Standards) performed laboratory tests to induce stress corrosion fractures in wire specimens from the first cables of the Portsmouth Bridge. In 1978, Battelle Laboratories of Columbus, Ohio, performed failure analyses on wires from the second cables of the Portsmouth Bridge. Further chemical and mechanical tests were performed on that wire by the Federal Highway Administration in 1978. In the early 1970's, Boeing Commercial Airplane Company performed a series of stress corrosion tests on bridge wire for the Federal Highway Administration. A report on the Portsmouth Bridge second corrosion problem was published by the Ohio Department of Transportation in 1980. The National Physical Laboratory of Great Britain has recently reported on its work related to cracking problems in prestressing wires.

The Kentucky Transportation Research Program has conducted several corrosion inspections of bridge wires since 1974, when a wire rope (suspender), removed from the Ohio Bridge at Covington, was sectioned and examined. Cables on the Portsmouth, Covington (Ohio Bridge), and Maysville suspension bridges have been inspected. The Portsmouth Bridge was inspected during the period it was closed for cable replacement (1978-80). The corrosion problem that led to that work was caused by failure of the corrosion protection system (paint and wire wrapping) and the subsequent entrainment of large amounts of water.

Deterioration of galvanized helical strand was observed to occur in four stages. In Stage 1, the strand wire were shiny with random signs of zinc corrosion. During Stage 2, the zinc would be partially corroded, revealing a white corrosion product, but no ferrous rust would be present. In Stage 3, the zinc would be depleted, with occasional spots of ferrous rust. During Stage 4, the zinc corrosion product would be largely displaced by ferrous rust. Cracking was possible during Stage 3 and could be expected during Stage 4.

Inspection of the Maysville Bridge revealed that the corrosion protection system (paint and wire wrapping) was gradually deteriorating.

Some water and signs of moisture damage were detected on cable wrapping and at cable bands. The condition of the wires was assessed by placing several inspection ports on the cables and by examining the cable strands. The condition of the exposed wire varied from Stage 2 to early Stage 3 deterioration. The strands in the anchorages were satisfactory. No broken wires were detected.

Inspection of the Ohio Bridge at Covington disclosed several potential problem areas. The original cable strands were subject to water damage from roof leakage inside the anchor chambers. Also, some signs of minor corrosion damage were evident on secondary cable strands in the Ohio anchorages. The worst potential problem appeared to be posed by thick deposits of pigeon droppings on the cables and stays at the tower peaks. Several wire breaks were detected during the inspections (though most of those predated present concerns).

Two small unstiffened suspension bridges in eastern Kentucky also were inspected. The exposed wire strands on those bridges showed no signs of corrosion damage.

Work on suspension bridges in Kentucky yields a somewhat differing view of cable corrosion and cracking problems than would be surmised from existing literature. In part, this seems to be due to a general preoccupation with stress corrosion in past research efforts. Most work is well done, but does not relate suitably to the physical situations observed at the Portsmouth Bridge with regard to the second cable corrosion problem.

The salient features of the Portmsouth Bridge second cable corrosion problem were 1) the presence of uniform corrosion at all fracture sites, 2) the lack of specific stress corrosion-related corrodants other than sulfur and hydrogen, 3) the lack of unusual or damaging concentrations of corrodants in the Portsmouth area, 4) the differing fracture morphologies on the Portsmouth Bridge cables, and 5) the cyclic loading of the Portsmouth Bridge cables.

Many fractures on the second Portsmouth Bridge wire were more typical of corrosion fatigue cracking than stress corrosion cracking. Also, Boeing researchers were unable to produce sulfur-induced stress corrosion cracks in bridge wire. Combined with previously enumerated evidence, those facts indicated corrosion fatigue probably was the most important fracture mechanism affecting the Portmouth Bridge wires.

The importance of corrosion fatigue, compared to other forms of corrosion cracking, is that high concentrations of corrodants are not required. The only requirement is for bridge wire to be in long-term contact with moisture (and possibly a nominal amount of common atmospheric pollutants).

It is recommended that inspection ports be installed on the Ohio Bridge (KY 17) at Covington and to continue corrosion monitoring on the Maysville Bridge (US 68). Research related to bridge wire corrosion problems and their prevention should be encouraged.

INTRODUCTION

This is the second of three reports concerning suspension bridges and corrosion. The first report (1) contained background information on suspension bridges that may be useful to persons not well acquainted with those structures. The final (2) report contains suggestions for assessing the condition of suspension bridge wires and for providing necessary remedial actions.

This work was initiated in 1978 after the closure of the General U. S. Grant Bridge (termed the Portsmouth Bridge) (US 23) over the Ohio River between Portsmouth, Ohio, and South Shore, Kentucky. Of major concern to the Kentucky Department of Highways is the structural condition of the two large suspension bridges owned by Kentucky, the Ohio Bridge at Covington, Kentucky (1,010-foot main span), and the Maysville Bridge at Maysville, Kentucky (1,060-foot main span).

The basis for this report was a series of inspections conducted on the Portsmouth Bridge during the cable replacement operation (1978-79). Thereafter, a series of inspections was performed on the Ohio and Maysville bridges. Some brief laboratory examinations were performed on wire specimens taken from the Portsmouth Bridge. Also, an extensive review of relevant, available literature was conducted.

Perhaps the most interesting, and at times, the most puzzling fact addressed during this study is that the Maysville Bridge has remained in service since 1930 without showing major signs of cable deterioration. However, just 50 miles upstream on the same Ohio River, the Portsmouth Bridge cables have succumbed to corrosion twice since its erection in 1927. An investigation, which lead to this report, was deemed necessary to determine the contributory and mechanistic factors involved in suspension bridge corrosion problems and to establish the applicability of those factors to the Kentucky-owned suspension bridges.

CORROSION OF BRIDGE WIRE

Suspension bridge wires expose 10 to 20 times the surface area of normal structural steel (based on a ratio of surface area to cross-sectional area). Therefore, wire damage by corrosion may result in a much greater loss of strength than equivalent corrosion damage to common structural members. Another fact to consider is that most suspension-bridge main cables are fracture-critical members. The trusses of most suspension bridges will not support their own weight. If a main cable of a suspension bridge should fail, the entire structure might collapse in a catastrophic manner, with the possible exception of the towers (unless rocker towers were employed). Therefore, consequences of corrosion damage to suspension bridge cables are more severe than corrosion damage to most structural members of other bridge types.

Many corrosion problems are unique and must be examined on an individual basis due to the complex electrical-chemical-mechanical interaction that takes place between the metal, corrodant(s), and environment (3-5). The following discussion is a brief outline of corrosion, specifically the types of corrosion that may be experienced by suspension bridge wires. Corrosive mechanisms described are believed to be relevant, based on field and laboratory observations in Kentucky. The applicability of some of these mechanisms, based on field and laboratory investigations, will be detailed in another section of this report.

At least seven types of corrosive attack may be involved in bridge wire corrosion: uniform corrosion, pitting corrosion, crevice corrosion, fretting, stress corrosion, hydrogen cracking, and corrosion fatigue. While only two or three of those mechanisms may cause wire cracking, the other types may be contributory. Indeed, some of those other mechanisms are probably necessary prerequisites to corrosive attack mechanisms that cause wire fracture.

Uniform corrosion of iron or steel usually involves an interaction between iron, hydrogen, and oxygen. This usually produces three layers of iron oxides. The inner layer consists of white to greenish-black ferrous oxide ($FeO * nH_2O$) on ferrous hydroxide ($Fe(OH)_2$), which acts as a diffusion barrier between the atmosphere and the iron surface. A hydrous ferrous ferrite ($Fe_3O_4 * nH_2O$) forms a black intermediate layer. The outer layer consists of hydrous ferrous oxide (Fe_2O_3), which is orange to brown in color and is common rust.

Contact with aqueous solutions is required to maintain aggressive uniform corrosion of iron. The resulting corrosion product is a thick, scaly layer of ferrous oxide (rust). When iron is not in contact with an aqueous solution, it will form an intermediate, stable ferrous oxide. Oftimes this oxide will be passive (corrosion inhibiting). In aqueous solutions that are nearly neutral, the corrosion rate of iron is dependent upon the amount of oxygen dissolved in the solution. This diffusion-controlled uniform corrosion of iron increases with temperature. The corrosion rate may be doubled for every $55^{\circ}F$ rise in temperature. When hydrogen is evolved, the corrosion rate may be more than double.

Above a pH of 10, the corrosion rate decreases, as the iron surface becomes passive. In the pH range between 4-10, the corrosion rate is independent of pH and is controlled by diffusion of oxygen to the metal surface. Below a pH of 4, the ferrous-oxide film dissolves and the corrosion rate depends upon the reaction rates of hydrogen and oxygen at the metal surface (6).

In the pH range of 4-10, oxygen concentration, temperature, and disposition in an aqueous medium determine the corrosion rate. Cold-worked steel corrodes at the same rate as annealed steel in water. The corrosion rate of wrought iron or steel in that pH range is similar. In the pH range of 0-4, the percentage of metal impurities such as carbon, nitrogen, sulfur and phosphorus can have a marked effect on the corrosion rate. In acids, the corrosion rate of cold-worked steel is several times greater than annealed steel (7, 8). The disposition of cementite, the carbon-rich second phase of steel, has a strong influence on the uniform corrosion rate in an acid environment.

The atmosphere is a major factor in many corrosion problems. It can provide moisture, a susceptible thermal environment, and aggresive corrodants. Moisture is required for the electrolytic corrosion of metals. When a metal is exposed to the atmosphere, the time-of-wetness is not only dependent on periods of rainfall, but also the daily evaporation-condensation cycle. This is a complex interaction of temperature, air movement, and relative humidity. Large thermal gradients, such as loss of heat from the suspension bridge cables in the evening, will promote condensation of moisture on strands. More condensation may be expected on the exterior strands as the thermal gradients are higher at the outer portions of the cable. Since more moisture will be present on the exterior strands, uniform corrosion there will be worse (9-11).

Common atmospheric pollutants include SO_2, H_2S, NH_3, NO_2 and various salts (12). These act as corrodants when in contact with steel in an aqueous environment. Airborne solid particulates (dust) deposited on the metal surface may promote corrosion by acting as sites for the condensation of water vapor. When a corrosion product such as rust forms on the metal surface, it will absorb atmospheric moisture hydroscopically under conditions where moisture will not be deposited on a clean metal surface.

In situations where atmospheric moisture is retained, the ferrous surface may corrode more rapidly than when frequently washed (contacted by flowing water). This is usually due to the formation of H_2SO_3 from SO_3; however, under other circumstances, washing or periodic wetting greatly increases the corrosion rate (13).

A large amount of vehicle-related pollution is generated around bridges. Therefore, bridges in rural locations may be subject to higher concentrations of corrodants than the typical surrounding environment. Also, solid particulates such as sulfides generated by smokestacks may be carried airborne for very long distances. For instance, Lexington, Kentucky, has no substantial SO_2 output. Yet, at one time, the daily particulate sulfate concentration was relatively high (10-40 micrograms per cubic meter). Research revealed the pollution level was caused by smokestack output from power plants up to 226 miles away (14).

The corrosion of galvanizing (zinc coating) is usually linear with time in a given atmosphere. When zinc reacts with moisture, zinc hydroxide is formed and hydrogen is released. The zinc hydroxide reacts with CO_2 to form insoluble zinc carbonate. When the contacting moisture is acidic, the zinc will dissolve rapidly. As the reaction proceeds, resulting products will form a basic salt, usually a carbonate, which precipitates. This coating acts as a temporary inhibitor. Researchers suspect atmospheric pollutants, especially SO_2, form acids that attack the protective zinc oxide and carbonate films. Those films are soluble in acids (such as H_2SO_4) and eventually will be dissolved and redistributed over the metal surface with successive wetting cycles. The redeposited oxide is porous and does not act as a good diffusion barrier. Therefore, underlying steel will eventually rust (15, 16).

Pitting is a localized form of corrosive attack. The result of this form of corrosion is formation of small deep pits on the surface of the metal. Pitting occurs when small areas become anodic to the bulk of the surface, or when highly localized changes in the corrodant in contact with the metal accelerate the local attack. The corrosive rate of attack at pits may be 10 to 100 times as great as the uniform corrosion rate (17, 18). Pitting in galvanizing is usually associated with mild atmospheres (19). A localized failure of a protective oxide film may lead to pitting. When the galvanizing on wires is depleted, the zinc corrosion product may still be protective. However, if that film is chipped (possibly due to expansion of some underlying rust), the exposed ferrous surface may be subject to pitting. The most important effect of pitting is its effect on local chemistry in the pits, not its effect on stress intensity.

Crevice corrosion occurs due to differential aeration (differences in the concentration of oxygen in an aqueous corrodant, for example).

Another cause may be concentration of soluble contaminents, such as sulfur, in water. The aqueous corrodant penetrates crevices like those between wires in a strand (Figure 1). Initially, anodic dissolution ($M = M^+ + e^-$) of the metal and cathodic reduction ($O_2 + 2H_2O + 4e^- = 4OH^-$) occur uniformly over all metal surfaces including the crevices. Oxygen in the crevice is consumed, causing cessation of the anodic reduction in the crevice. Mobile negative ions, SO_4^{--} for example, migrate to the crevice to maintain charge balance. The resulting metal sulfate will dissociate in the aqueous solution to form metal ions and acid radicals. The sulfate ions and low pH accelerate the corrosion in a manner similar to autocatalytic pitting, while the reduction reaction cathodically protects the exterior surfaces (20).

Fretting is a wear phenomenon occurring between two surfaces having relative oscillatory motion of small amplitude (<25 micrometers). Fretting corrosion is a form of fretting in which the chemical reaction predominates. When wire rope or helical strand are loaded in tension, the wires in the strands rotate and translate slightly in relation to each other. Slight bending movement also may occur at tower saddles and cable bands. At those points, the strands bear against each other with forces in the range of 3,000 psi (21). Fretting action can erode the galvanized coating, wearing notches in adjacent wires (which act as sites for localized corrosive attack). Fretting also rapidly removes corrosion products and thereby accelerates corrosion (22).

Stress corrosion cracking (SCC) is a localized form of corrosive attack that occurs due to the concurrent application of 1) corrosive attack and 2) a static tensile stress (applied or residual). Both factors must be present for cracking to occur. Two theories exist which explain SCC. The electrochemical theory states that galvanic cells are created between metal grains and different phases. When an alloy is stressed in tension, localized electrochemical dissolution of metal creates a crack. The stress-sorption theory states that SCC proceeds by weakening of the cohesive bonds between atoms through absorption of damaging substances (23-25).

Stress corrosion cracks are usually characterized by branching and a transverse orientation to the applied tensile stress. One important characteristic of this phenomena is that low- to moderate-strength steels tend to undergo SCC only by the action of specific environments. High-strength steels do not appear to require specific corrodants (26). The SCC environments for steel are shown in Table 1. The magnitude of stress required to propagate a stress corrosion crack may be well below the design stress for bridge wires. In most cases, SCC is intermittent inasmuch as time is required for corrodants to diffuse to localized points of high stress and to damage the metal.

Several ions and substances known to cause SCC of steels similar to bridge wire may be found in normal industrial atmospheres (27). Some process(es) is (are) usually required to achieve a suitable concentration of those atmospheric pollutants to affect the material. Crevices are a good site for the concentration of impurities by leaching, decomposition, and corrosion. Condensation-evaporation cycles are another way for high corrodant concentrations to be achieved. On each drying cycle, pollutants are retained on the material surface and are available for intensified attack on the next moisture renewal cycle.

Factors affecting the rate of SCC include nature and magnitude of the applied stress, environmental temperature, humidity, pH of the

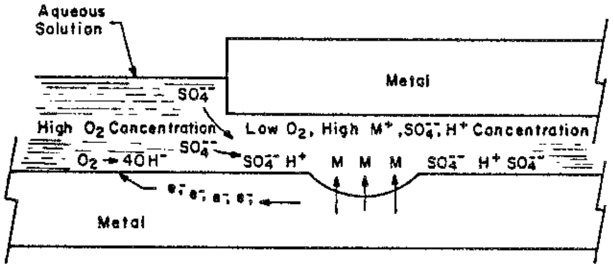


Figure 1. Crevice Corrosion Model.

TABLE 1. CORRODANTS SPECIFIC TO THE STRESS CORROSION OF STEEL AT ROOM TEMPERATURE (30, p 210)

CORRODANT	ALLOY
Cl_2 (gas)	High Strength-Low Alloy
HCl and HBr	High Strength-Low Alloy
H_2 (gas)	High Strength-Low Alloy
H_2S (gas)	High Strength-Low Alloy
Sulfide impurities in aqueous solutions	Medium to High-Strength Steels (hydrogen cracking)
MnS and MnSe	High-Strength Steels
Nitrates in aqueous solutions	Carbon Steels
Arsenic, antimony, and bismuth ions in aqueous solutions	High-Strength Steels (hydrogen cracking)

corrodant, concentration of the corroding solution, and physical nature of the corrodant. Geometric defects such as notches and pits may serve as stress raisers and increase the rate of SCC attack.

At low applied stresses (below K_{ISCC} (the stress intensity at the crack tip)), crack growth rate is proportional to increases in stress intensity. Above K_{ISCC}, the crack growth rate is unaffected by increases in stress intensity. The growth rate is probably controlled by rate-limiting processes such as diffusion. Crack branching occurs at this K-independant regime (28).

Metallurgical factors that make ferrous alloys more susceptible to SCC are grain size, amount of cold working, alloy content, and content of certain residual elements. Carbide-forming alloying elements such as Mo, V, and Ti help resist SCC. Alloying and residual elements such as C, Mn, S, and P increase SCC susceptibility.

Hydrogen stress cracking is a form of structural deterioration usually associated with SCC. It requires a stressed, high-strength steel and an environment that will furnish hydrogen ions. Unlike SCC, 1) there are no specific damaging solutions related to a given metal, 2) cathodic protection will not prevent cracking, and 3) a corrosion reaction is not required for hydrogen to enter a metal and cause cracking. Environments that may furnish hydrogen include water, moist air, hydrocarbons, acids, and H_2S gas.

Hydrogen stress cracking is similar to SCC described by the stress-sorption theory. Hydrogen, either present in the metal or furnished by other corrosion mechanisms, is diffused by stress gradients to points of attack, weakening metal bonds and thereby facilitating cracking. The presence of certain residual elements also promotes a susceptibility to hydrogen stress cracking. Bainitic and martensitic microstructures are very susceptible to hydrogen attack, even in the cold-worked condition. This is one reason hydrogen stress cracking usually takes place in steels with tensile strengths exceeding 150 ksi. A minimum stress is required to initiate hydrogen stress cracking, but this is well below practical design stress levels and may be associated with residual stresses. Hydrogen sulfide, H_2S, is the atmospheric pollutant most commonly associated with hydrogen stress cracking (29, 30).

Molecular hydrogen, H_2, has limited diffusion capabilities in steel, usually associated with very high temperatures. Steel charged with molecular or atomic hydrogen can exhibit slow strain embrittlement, which usually occurs at 40 percent of the yield strength. Maximum susceptibility is at room temperature (31). When molecular hydrogen is involved, the hydrogen must have been previously charged into the steel at elevated temperatures. Delayed failure, termed "static fatigue", is hydrogen-induced cracking under static load. Those cracks usually initiate in the interior of the material and will usually not show signs of crack branching. Corrosion-related hydrogen problems are caused by atomic hydrogen.

In electrochemical corrosion of steel, different areas of the metal surface become cathodic or anodic. At the cathode, one important reaction is

$$H^+ = 1/2H_2 - e^-.$$

Catalysts such as compounds of arsenic, phosphorous, or hydrogen sulfide may suppress that reaction. Hydrogen atoms generated by uniform corrosion may accumulate at the metal surface and subsequently diffuse into the metal, causing hydrogen stress cracking. Those compounds are called "cathodic poisons". In the case of steel exposed to an aqueous solution, only a few parts per million of H_2S is sufficient to act as a cathodic poison. Cold-working lowers the resistance of steel to that type of cracking. Aqueous solutions without much oxygen may be favorable to the evolution of hydrogen ions, as the lack of free oxygen prevents oxidation of those ions (32).

Local corrosion taking place within a notch or crack may produce pH and corrosion-potential conditions at the notch tip much different than those existing on the specimen surface. Hydrogen may be generated at those notches, though the surface would not reveal any unusual, gross, corrosion activity. Applied anodic potentials that cause pitting may produce hydrogen stress cracks due to hydrogen generation in the pits.

Corrosion fatigue is cracking caused by fluctuating stresses in the presence of a corrosive environment (33-35). The sum of the damage generated in such an environment is much greater than when one of the factors was missing or when they acted alternately. Like SCC, corrosion fatigue is capable of nucleating and growing a crack in a material until failure occurs. However, unlike SCC, a specific corrosive environment is not required. Corrosion fatigue may act in concert with general corrosion, pitting, hydrogen cracking, or SCC. The behavior of corrosion fatigue has been modeled by Wei and Simmons (36) as follows:

$$da/dN_c = (da/dN_{SCC}) + (da/dN_r) + (da/dN_{cf})$$

in which da/dN_c = corrosion fatigue crack propagation rate,

da/dN_{SCC} = contribution of the environment to attack,

da/dN_r = inert (no environment effect) fatigue propagation rate,

da/dN_{cf} = synergistic effect of fatigue and stress corrosion,

da = increment of crack growth, and

dN = increment of stress cycling.

At high loading frequencies and low stress amplitudes, the cycle-dependent term usually predominates. At low loading frequencies, slow loading rates, prolonged exposure at peak stresses, high stress amplitudes, and inaggressive environments, the environmental term usually has greater influence. The synergystic term is a result of the interaction of new crack surfaces produced by fatigue and the environment. The interaction may be thought of in two ways: corrosion-aided fatigue processes or fatigue-aided corrosion processes. If the material is subjected to a static tensile stress, the effect of a cyclic tensile load will be to open the crack, not only increasing the crack-tip stress intensity, but also increasing the likelihood that a corrodant will penetrate to the crack tip.

Corrosion fatigue will lower the cyclic life of steel compared to air-tested fatigue strength. A high-strength steel may lose up to 10 percent of its air-tested fatigue strength when subjected to plain moisture (37). In the presence of active corrodants, the loss of fatigue strength will be much greater. Steels subjected to corrosion fatigue do not show a fatigue limit. Nucleation or propagation of corrosion fatigue cracks is influenced by the corrosive environment, mainly bulk aqueous solutions or periodic vapor condensation.

In carbon steels, corrosion fatigue cracks usually (but not always) initiate at corrosion pits that are large and contain significant amounts of corrosion products. Cracks generated by corrosion fatigue tend to be transgranular and may show some crack branching. Multiple surface cracks are another sign of corrosion fatigue. River patterns on fracture surfaces, which are typically associated with normal fatigue failures, may be suppressed in corrosion fatigue.

HISTORY OF SUSPENSION BRIDGE WIRE CORROSION PROBLEMS

In his book, A Practical Treatise on Suspension Bridges (38), the noted suspension Bridge designer, D. B. Steinman, commented, "For the cables of the Portsmouth bridge, bright wire was adopted instead of the more customary galvanized wire, on account of the faith of the designing engineers in the adequacy of the protection afforded by modern cable wrapping. . . . The cables of the Williamsburg Bridge, made of bright wire, are in an excellent state of preservation. When Roebling's Niagara railway suspension bridge was taken down, . . . the bright wire composing the cables was found free of rust. Except possibly for coastal locations exposed to salt air, the author would unhesitatingly recommend the use of ungalvanized wire for bridge cables; with the proviso, of course, that the outside of the cables be suitably protected with tight wire wrapping."

Apparently, Steinman was unaware his statements about the Williamsburg and Niagara bridges were incorrect. This fact would become evident some 11 years after his book was published, when the cables of the Portsmouth Bridge cable first suffered corrosion problems. In defense of Steinman, it can be noted that other prominent bridge designers shared his view. Unfortunately, the general confidence held by those individuals did not reflect historical evidence about suspension bridge cable corrosion problems up to that time.

In the early to mid-nineteenth century, several French suspension bridges failed due to corrosion in the anchorages. Roebling's Niagara Bridge showed signs of corrosion during an inspection in 1877. Wires on the undersides of the cables were rusted and more severe wire corrosion was found at the entrance to the mortar-encased anchorage. Corroded wires in the anchorages were removed and replaced with spliced pieces of wire. In 1883, Roebling's Sixth Street Bridge over the Allegheny River at Pittsburg suffered cable corrosion at the anchorages. That problem was remedied by cutting out the corroded wires and splicing wire "hairpins" about the anchorage shoes (39).

In 1893, the mortar-encased anchorages of Roebling's Ohio Bridge at Covington, Kentucky, were opened to inspect the wire strands. The inspection was prompted by corrosion problems in the mortar-encased anchorages of several other Roebling bridges. Mortar in the anchorages was moist and, at points adjacent to the strands, covered with rust. The strands were encrusted in mortar. However, when cleaned, the outer wires of the strands showed considerable loss of section, though the wires inside the strands were in good condition. A number of broken wires, apparently severed by uniform corrosion, were found. Those were repaired with wire splices. It was estimated that corrosion had reduced the strength of the cables by about 12 percent. To remedy this, auxiliary eyebars were connected, running from friction collars outside the anchorages to the strand shoe pins (Figure 2). In each anchor house, the strands and anchorage assemblies were enclosed in a sheet-metal tanks filled with paraffin oil, and later wax, to protect the strands from further oxidation (40). Those tanks were removed at a later date.

In 1905, a suspension bridge at Charleston, West Virginia, failed, killing several people. It was later determined that one of the main cables parted due to corrosion in the anchorage. Three years later, a suspension bridge in Morgantown, West Virginia, was closed, due partially to corrosion of the cables (41).

The Kelso Bridge, over the Cowlitz River at Kelso, Washington, collapsed in 1923, with an attendant loss of 12 lives. The 300-foot span was built in 1907. Again, failure was due to corrosion of the cables in anchorages (42).

Rusting of the sheet-steel covers on the Williamsburg Bridge led to the wrapping of the cables in 1921. An inspection of the exterior wires showed rust discoloration. The corrosion was worse in the main span where rust stains were found in many places, generally occurring at the lower ends of panels and under the seizing at the lower part of the strand. Cable wires were wedged apart and an examination of the interior wires revealed no signs of corrosion (43).

In 1928, work was initiated on two large suspension bridges, the Mt. Hope Bridge at Providence, Rhode Island, and the Ambassador Bridge at Detroit, Michigan. Both bridges were to employ a new, untried, heat-treated wire that offered potential economic benefits over normal cold-drawn wire. Just prior to the completion of the Mt. Hope Bridge cables, broken wires were detected at tangent points adjacent to strand shoes in the anchorages. The number of breaks increased until the load on the strands was relieved. An attempt was made to repair the wires by splicing, but that was intended only as a temporary remedy. Similar wire fractures were found on the Ambassador Bridge before the deck structure had been completed. Additional breaks were found under cable bands of that bridge (44). The wires, which were galvanized, did not show signs of corrosion. Shortly after detection of those breaks, the cables of both bridges were removed, and the heat-treated wire was replaced with conventional cold-drawn wire (45, 46). Heat-treated wire was never again used on a suspension bridge.

The Portsmouth Bridge was completed in 1927. During an annual inspection in May 1939, cracks were observed in the main cables and the bridge was subsequently closed to traffic. The first breaks were located in anchor houses between the splay saddles and the anchor shoes. Most breaks were in the anchorages on the Ohio bank, which were previously inundated by flood waters. All anchor houses were damp. The dampness may have been partially attributable to the sand-filled anchorages. Wires in the anchor houses had been coated with asphalt for rust protection. However, that coating was ineffective. Asphalt wire coatings had been proven to be ineffective some 50 years previous (47). After removing the asphalt coating, many wires in the anchor houses were observed to be corroded and broken. Wrapping of the main cable was

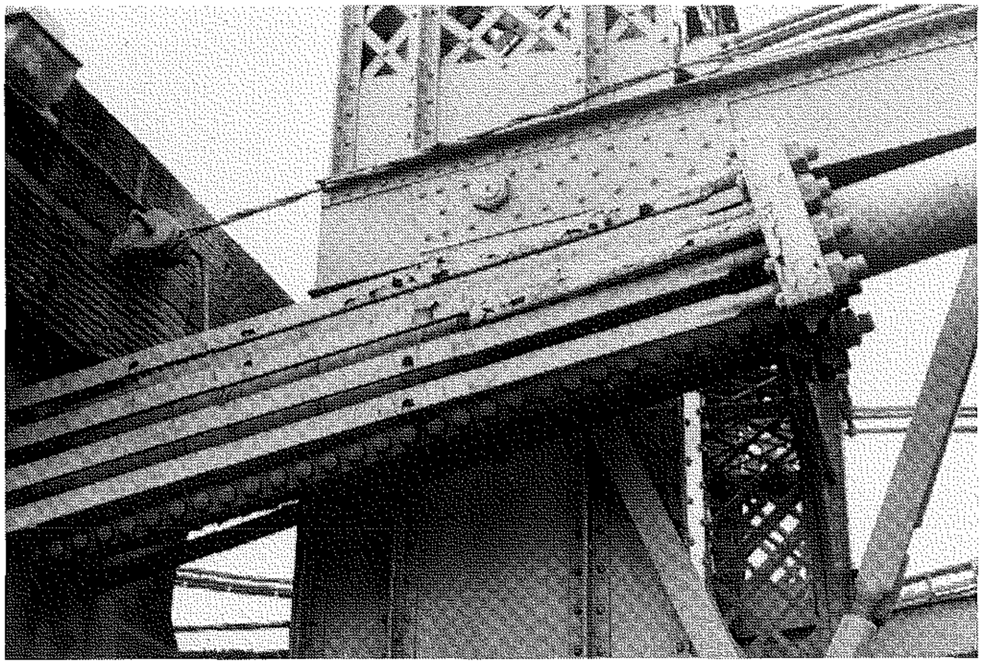


Figure 2. Friction Collar on the Ohio Bridge at Covington.

removed at many points, and the cables were plied apart by wedging to examine interior wires. The outer layer of cable appeared to be in fair condition; however, the interior was fused by dried paint and rust. Some wires were completely rusted through. Watchmen were placed in the anchor houses and reported that they could hear wires fracture on quiet nights.

Most breaks were found on the upstream cable; 198 occurring in the Ohio anchorage, and 110 in the Kentucky anchorage. On the downstream cable, 60 breaks were found in the Ohio anchorage and none in the Kentucky anchorage (48).

There was a slight imbalance in the main cable loading due to design of the roadway. The imbalance placed heavier loads on the upstream cable.

In 1940, the American Bridge Company, under the supervision of Modjeski and Masters Consulting Engineers, recabled the bridge. At that time, the bridge was owned by the Fullerton-Portsmouth Bridge Company.

To overcome flooding problems in the anchorages, the anchorage eyebars were extended and elevated outside the anchor blocks. The new cables consisted of 19 shop-fabricated, prestressed, galvanized helical strands, of which 13 were 1-3/4-inch diameter and 6 were 1-1/4-inch diameter. Each strand was pulled over the bents and towers by temporary sheaves on a hauling line and lowered into place. As with the Maysville Bridge, aluminum fillers were used to give the cables a circular cross section. The cables were then wrapped with a soft galvanized wire. The truss and suspenders, which were removed prior to demolition of the original cables, were reinstalled and new anchor connections also were constructed to accept the wire rope. A concrete deck replaced the initial wood-asphalt installation. The new cable had a total effective cross section of 31.4 square inches compared to 30.04 square inches of the original cables. After the first Portsmouth cable replacement, bright wire was never used in an American suspension bridge. At least four American suspension bridges that employed bright wire are still in service.

In November 1939, the 450-foot main-span Hidalgo-Reynosa Suspension Bridge, over the Rio Grande at Hidalgo, Texas, collapsed, killing one person (49). The failure was attributed to fracture of wires due to electrolytic corrosion in an anchorage. After the bridge was rebuilt in 1941, a series of tests was conducted to determine if ground currents were present in the cables. Those tests indicated electrolytic currents were flowing from each cable at the rate of 25-26 milliamperes at a potential of 0.4 volts. To neutralize that, the cable was given a cathodic protection system consisting of a two-volt wet-cell battery and a two-ampere charging unit. The system eliminated ground current (50).

In 1941, general remedial work was initiated on the suspension bridge over the Ohio River at Steubenville, Ohio. Examination of the concrete-encased wires in the anchorages revealed the wires (which were in direct contact with concrete) to be severely corroded in both anchorages. The bridge had been in service for 35 years.

To remedy the problem, temporary friction hitches were installed beyond the points of corrosion. The looped ends of the strands were cut beyond the points of corrosion damage. Socket fittings were attached to the newly cut strand ends. The other end of the sockets were connected to the anchorage eyebars by steel bar and U-bolt assemblies (51).

During 1967-1969, an inspection was conducted on the 2-11/16-inch diameter ropes of that bridge. Many of those ropes required replacement due to severe corrosion in the lower portion of the ropes. In other portions, the zinc coating was depleted. The worst corrosion was in the lowest portions of the ropes and at locations where the ropes had been wrapped with wire to prevent abrasion with a truss-guide casting.

The Portsmouth Bridge was operated from 1940 to 1974 by the Ohio Bridge Commission. Thereafter, it was owned by the Ohio Department of Transportation. Official reports stated the condition of the bridge was generally good up to 1973 (52). The bridge was painted last in 1967. However, a 1974 inspection disclosed cracked paint on the cables, light rust the on undersides of the cables, rust spots on the undersides of cable bands, and loose caulking at the cable bands. Free moisture also was observed at the bands.

In 1975, the cables were unwrapped at six of the lowest panels and inspected. The strands at the end bents were in good condition. However, at the center of the main span, the zinc coating was depleted and the wires were rusted. A few broken wires were found at that time.

In May 1978, a general rehabilitation program was initiated for the bridge. Included in this plan was work to remove the wrapping wire, clean any uniform corrosion, and rewrap the cable with a neoprene tape. However, as unwrapping proceeded on the downstream Ohio side, wires in the center section of the side span were badly corroded. Many breaks also were observed. Seventy-seven fractures were found in one panel (53). In June, a detailed investigation of the status of the main cables was conducted, examining several panels of the downstream side on several spans. On the side span, the greatest damage was detected in the central panels between the bent and the tower. Little corrosion could be found in the cables at the bent or towers.

Severe uniform corrosion was observed in the lower strands of cables. Wires in the upper strands appeared to be in good condition, with little corrosion of the zinc coating. No breaks were detected in the cable where galvanizing was not deteriorated. A heavy amount of corrosion products was observed in the lower wires. Cracking was detected only in severely corroded wires. The lowest section of the main cable, which had been unwrapped, was resealed with a protective tape. Several days after a storm, more than a quart of water was drained after the patch was punctured.

On completion of a report from Battelle Laboratories of Columbus, Ohio, the bridge was closed to traffic. The bridge cables were replaced with structural strand having Class C galvanizing, and the bridge was reopened to traffic in December 1979.

The 1,065-foot main-span Kohlbrand cable-stayed bridge at Hamburg, Germany, was completed in 1974. Signs of corrosion surfaced in 1976. An attempt was made to protect the cables (apparently bright wire) with a plastic coating. An official report stated the coating was ineffective and the cables suffered uniform corrosion due to the entrainment of moisture and salt kicked up from the roadway. The lower portions of the strands swelled due to the corrosion. Apparently, the plastic coating failed due to poor workmanship. The bridge was to be recabled with galvanized wire, and the work was scheduled for completion in fall of 1979 (54).

In 1978, an inspection was performed on the 130 year-old Wheeling, West Virginia, bridge. The four main cables were unwrapped at 13 locations. At one location, 15 broken wires were found. As the inspection revealed severe deterioration of the wrapping wire, the work was extended to unwrapping all main cables. At one point, 50 broken wires were detected. About 85 other wires showed 50-percent section loss (55, 56). The cables have been renovated. Work was completed in 1982.

As a result of the Federal Surface Transportation Act of 1978, inspections were conducted on bridges in the New York City area. Those inspections revealed, in part, corrosion on the stay cables of Brooklyn Bridge at entrances into the towers (57). On June 29, 1981, shortly after those inspections, two stays snapped, killing a pedestrian. The probable cause of failure was listed as corrosion due to atmospheric corrosion or bird droppings.

Those inspections also revealed severe corrosion and abrasive wear to wires of the Williamsburg Bridge (58). The problem was attributed in part to the unusual corrosion protection system (graphite and petrolatum). It has been suggested the graphite acted as an abrasive, exacerbating the fretting action of the cables (59). Possibly, the petrolatum interacted with moisture, forming acids promoting gradual deterioration of the wires.

In 1978, signs of wire breakage were detected in the suspenders of the Severn Bridge (60). This led to the replacement of 49 suspenders, with plans to replace the remainder. Failures were attributed to fatigue cracking that interacted with corrosion.

In 1978, severe corrosion damage was detected in the stay cables of the 16-year old Lake Maracaibo Bridge, Venezuela (61). Several months later, one stay broke. Failures were attributed to high humidity and the high salinity of the lake. All 2.9-inch diameter stay cables on the bridge were replaced over a two-year period at a cost of 50 million dollars.

On April 12, 1982, a two-truck collision on the Chiriqui River suspension bridge in Panama sheared several suspender ropes. The 45 year-old, American-built bridge collapsed. A subsequent investigation revealed the hanger cables had been significantly weakened by rust. One strand had lost two-thirds of its cross section due to corrosion (62).

LABORATORY TESTS OF WIRE

After failure of heat-treated wires on the Mt. Hope and Ambassador bridges, the National Bureau of Standards initiated a study, probably the first major laboratory analysis of suspension bridge wire failures. A formal report was issued by the Bureau of Standards in 1936, six years after the study was undertaken (63).

Wire furnished to the Bureau of Standards' investigators was from the Mt. Hope Bridge. Both the Mt. Hope and Ambassador bridges used similar wire from the same mill. The wire was made from basic open-hearth steel. After being cold-drawn, it was heat-treated, acid-cleaned, and galvanized. The resulting combination was not altogether satisfactory and 4,000 of the 30,000 coils of wire made at the mill were rejected. Mill rejections were due to poor surface finish after galvanizing, failure to meet specifications, or spontaneous fracture. Heat-treated wires had poorer surface finish than comparable cold-worked wire. Tensile test specimens sometimes failed with no reduction in area. The investigators believed shop fractures were due to hydrogen embrittlement. Wires that were not acid-dipped and galvanized did not break. The investigators stated that the effects of hydrogen embrittlement were transitory. Therefore, they felt that hydrogen did not cause field problems.

The heat-treated wire was stiff. In the field, the wires were bent around temporary shoes to form anchorage loops. The Bureau of Standards' investigators criticized workmanship in those locations. However, laboratory tests on bent-wire specimens did not show the bending operation to be critical to failure. Fatigue tests were conducted, using both heat-treated and cold-worked wire (64, 65). Fatigue limits of both types of wires was similar, though it was noted that galvanizing of either wire type reduced the fatigue limit. A special jig was arranged to fatigue wire hairpin specimens, one end of the wire was fixed and the wire looped about a jig, resembling an anchor shoe. The other end of the wire was fatigue cycled in tension. Heat-treated wire specimens had lower fatigue limits (though not substantially lower) than cold-worked wires, and the spread of results was greater for heat-treated wire. Some fracture surfaces of the heat-treated fatigue specimens resembled field failures. Based on those results, the Mt. Hope and Ambassador bridge problems were ascribed to fatigue by Bureau of Standards' researchers.

After the 1940 recabling of the Portsmouth Bridge, specimens of the original wire were sent to the Carnegie Institute of Technology for physical inspection. The wires had assumed the approximate curvature of the mill reels from which they were spun. All samples received from the Ohio side were badly pitted and rusted. Those from the Kentucky side were only slightly rusted. Specimens from the periphery of the main cables were only slightly rusted, while those from the interior showed severe pitting. Tensile tests of specimens distant from the field fractures revealed strengths between 242-265 KSI. Tests of specimens near the breaks had tensile strengths of 132-261 KSI (presumably based on the original wire cross-sectional area). Reductions in area of test fracture surfaces varied from 19-47 percent for specimens distant to the breaks and from 6-37 percent for specimens close to the field fractures.

All specimens having strengths less than the specified minimum (220 ksi) showed evidence of pre-existent cracks. Microscopic tests revealed both radial and longitudinal cracks in some specimens. Some wires showed numerous transverse surface cracks. The fractures exhibited branching and traveled on irregular paths into the wire. Crack paths tended to deviate from inward movement as greater depths of penetration were obtained.

The report (66) on the examination stated: "Only those (wires) at the anchorage shoes seemed dangerously brittle. At these locations, the wire appears to have been subjected to a combination of stress and corrosion resulting in stress-corrosion cracks. Specimens from other locations, in spite of corrosion, seem to have suffered no important deterioration of mechanical properties. These latter tests have not been extensive."

In August 1940, the National Bureau of Standards received samples of the original Portsmouth Bridge wire. Tests were performed on those wires in an effort to produce SCC in the wire using various components of industrial atmospheres. A microscopic examination of typical service fractures showed the failures to be transverse to the longitudinal axis for a short distance from the wire surface. The crack changed to a slanting fracture as it penetrated into the wire. Most wires had cracks on the concave side of the residual bow. That side would be in tension if the wire were straightened. The composition of the wire is shown in Table 2.

A series of SCC tests was conducted by the National Bureau of Standards by immersing specimens of the Portsmouth and heat-treated and cold-worked Mt. Hope wires in various aqueous solutions: ammonium nitrate, ammonium nitrite, ammonium sulfate, ammonium hydroxide, sodium nitrate and distilled water. Some bowed Portsmouth specimens were straightened and immersed in aqueous solutions. Other Portsmouth specimens were bent more severely than the residual set. All other specimens tested also were bent. The maximum tensile stress on the bent specimens was calculated to be in the range of 90-200 ksi at the outer fibers.

After 40 months in distilled water, the Portsmouth wire showed some general corrosion and severe pitting. At points near the water line, the wire showed a reduction in diameter up to 50 percent. Ammonium sulfate produced heavy general corrosion, reducing the wire diameter up to 60 percent. Ammonium nitrite and sodium hydroxide proved to be good inhibitors in the concentrations used (0.01 N).

Eleven of the 14 bowed Portsmouth wires immersed in 0.01 N solutions of ammonium nitrate and sodium nitrate developed stress-corrosion cracks and completely fractured. The maximum time for failure was 9.5 months. In each case, fracture initiated perpendicular to the wire surface, propagating transversely for a short distance in that manner and then becoming diagonal. The slanting fractures were intercrystalline. Most cracks occurred on the convex side of the bow. However, two cracks on the concave side were assumed to have been created in service. Two uncracked specimens suffered severe local pitting on the concave side of the bow, which led to failure. The lowest stressed specimen (90 ksi) did not fail and only developed pitting near the immersion line.

Pollard, author of the National Bureau of Standards report, concluded the cause of the Portsmouth Bridge corrosion problem was stress corrosion and felt the laboratory results confirmed the earlier tentative assumption of the Carnegie Institute of Technology (67).

A bowed Mt. Hope (heat-treated) wire tested in ammonium nitrate developed numerous stress-corrosion cracks. However, those remained transverse to the wire axis. A bowed cold-drawn wire of the type used to replace the original Mt. Hope cables did not fail after 39.5 months exposure to ammonium nitrate. However, it developed extensive pitting near the immersion line.

Laboratory tests of the Portsmouth Bridge second cables were made in 1978 by Battelle Laboratories of Columbus, Ohio. That work was preceded by a field inspection of the unwrapped Portsmouth Bridge cables by Battelle on June 22, 1978. Chemical analysis of the wire showed it conformed to original specifications. Visual inspection of specimens revealed substantial uniform corrosion, with the wires coated by a thick scale. Transverse cracks were found in some badly corroded wires. Those cracks penetrated halfway into the wires and changed orientation, 45 degrees to the longitudinal axis of the wires. Secondary cracks were observed in one specimen. That specimen showed consolidation of surface cracks, crack branching, and crack orientation transverse to the wire length. Scanning electron microscopic (SEM) analyses of fracture faces were not helpful due to gross corrosion on fracture surfaces. A black

TABLE 2. CHEMICAL CONTENTS OF TYPICAL BRIDGE WIRES (PERCENT BY WEIGHT)

ELEMENTS	MAYSVILLE BRIDGE	PORTSMOUTH BRIDGE (ORIGINAL)[2]	PORTSMOUTH BRIDGE (REPLACEMENT)[3]	MODERN BRIDGE WIRE[4]
Carbon	0.85 Max	0.69 - 0.84	0.85 Max[a] - 0.80[b]	0.65 - 0.78
Sulphur	0.04 Max	0.031 - 0.037	0.04 Max - 0.039	0.21 - 0.033
Phosphorus	0.04 Max	0.008 - 0.014	0.04 Max - 0.029	0.01
Magnesium	N/A	N/A	0.66	0.20 - 0.78
Silicon	N/A	0.03 - 0.14	0.23	0.10
Nickle	N/A	N/A	N/A	0.01 - 0.11
Chrome	N/A	N/A	N/A	0.10

1. From Maysville Bridge Specifications (Ref 88, p 88)
2. From Laboratory Analysis (Ref 67, p 438)
3. (a) From Portsmouth Bridge Specifications (Ref 68, p 12); (b) From Laboratory Analysis (Ref 68, p 14)
4. From Laboratory Analysis (Ref 72, p 49)

N/A - Not Available

corrosion product was found on the fracture faces. The only explicable impurity identified in the corrosion product was sulphur.

Payer and others, in the Battelle Report, assumed the sulphur was a residual byproduct of either SO_2 or H_2S. However, no definite corrodant was identified. In referring to the previous National Bureau of Standards' report, they concluded the second set of Portsmouth cables probably suffered SCC and recommended that those cables be replaced (68). A chemical analysis of the wire steel did not disclose any abnormally high concentrations of harmful elements. Traces of aluminum and lead detected in that analysis may have been due to their use in the zinc coating operation.

In 1978, at the request of the Federal Highway Administration, a series of tests on the Portsmouth Bridge second cable wire was performed by the Fairbank Highway Research Station. Included in the work were mechanical tests, chemical analyses, and fractographic analyses.

Surface chemical analyses, determined by SEM, revealed the presence of sulfur. A chemical analysis of the wire did not disclose any abnormally high concentrations of harmful elements. The as-received wire showed hydrogen concentrations of 212-220 ppm. The cleaned wire showed a hydrogen content of 0.6-0.7 ppm. Mechanical tests revealed a reduction in area of 28 percent, an elongation of 3 percent (on a 1.5-inch gage section), and an ultimate tensile strength of 236 ksi. Fracture toughness tests revealed low toughness values, averaging 38 ksi-inch$^{1/2}$. It should be noted that those fracture toughness tests did not meet any accepted standard.

Fractographic inspection by SEM revealed the fracture initiated at corrosion pits perpendicular to the longitudinal axis of the wire. The fracture surface was inclined at an angle of 40-50 degrees to the corrosion pit. No evidence of plastic deformation was evident at the fracture surfaces. The researchers concluded that the field fractures were typical of stress corrosion cracks. The report also suggested that hydrogen embrittlement, due to the galvanizing process, could be a contributory cause of failure (69).

A fourth general investigation, which dealt with the problem of SCC, was initiated by the Federal Highway Administration (70, 71). The performing contractor was the Boeing Commercial Airplane Company of Seattle, Washington. Tests were performed on various sizes of virgin bridge wire of both the bare and galvanized types. Specimens were prestretched to remove bending set imparted by the wire reels. Properties of the wires are shown in Tables 2 and 3.

Prior to testing, the wires were given a circumferential notch in the gage section. A plastic cup was placed around the notch, allowing it to be immersed in test solutions. Those assemblies were placed in test fixtures that allowed application of tensile loads. Aqueous solutions used in the tests were 50 ppm H_2S solution, 50 ppm SO_2 solution, and 5 percent calcium nitrate - 0.25 percent ammonium nitrate solution. Wire specimens were loaded to 40 percent of the ultimate tensile strength. Test exposures ranged from 40 - 59.5 weeks.

Extensive general corrosion was observed in bare wire in all tests. A white residue, believed to be a corrosion product of zinc, was noted on galvanized wires. However, no SCC was detected in any of the test specimens (72).

A narrative report on the second Portsmouth Bridge cable corrosion was compiled by the Ohio Department of Transportation in April 1980

(73). The report gives a detailed history of the bridge relevant to the recent corrosion problem. It includes field observations of the corroded cables by inspectors prior to the recabling operation. An overview was presented on relevant research related to the corrosion of wires.

An article in the May 14, 1982, Engineering News Record (ENR) commented on a draft report from Great Britain's National Physical Laboratory (NPL) on hydrogen embrittlement failures of prestressing strands (74). The ENR article referenced the failure of several wires on the Hood Canal Floating Bridge that the NPL attributed to hydrogen attack. The article mentioned that a bituminous film had been used unsuccessfully to protect the wires. The NPL report, in part, concerned exterior tendons in the Braidley Road Bridge at Bournemouten, a prestressed box girder bridge built in 1970. Wires began failing within a year of initial service; and within eight years, two complete tendons had broken. The tendons were coated with an aluminum-resin mix and wrapped in polyvinyl chloride. NPL stated that moisture penetrated the coating, causing corrosion by oxidation that, in turn, released hydrogen that made the wires brittle. The ENR article mentioned that cyclic traffic loads aggravated the cracks.

Another bridge covered by the NPL report was the 14-year-old Scotswood Bridge in Newcastle, England. The bridge was a tied-arch structure. The tie chords were box beams having tension strands in the boxes. In 1981, inspectors found 18 inches of water in the box beams. Two of the 28 strands were observed lying slack. The strands had been wrapped in a mastic-impregnated fabric tape. At the strand ends, congestion precluded wrapping and the ends were painted with red lead. Inspection revealed most of the wires failed in the painted zone though fraying was evident in three strands when the tape was removed.

The ENR article noted that several British corrosion experts, including the NPL, attributed both British bridge failures to hydrogen embrittlement. The article dispelled the notion that cold-drawn wire was not susceptible to hydrogen attack. The NPL report also stated that encasing prestressing wires in grout would prevent that problem. The ENR article also quoted another British official who was not confident of that statement.

The ENR article also noted some scientists had stated the Severn Bridge suspension cable cracking was aggravated by galvanized-induced hydrogen attack. However, others had contended the problem was due to corrosion and fatigue.

SUSPENSION BRIDGE INVESTIGATIONS IN KENTUCKY

Reserach with bridge wire dates to 1974, when a wire-rope suspender from the Ohio Bridge at Covington, Kentucky, was inspected for signs of corrosion damage. Initial field inspections of the Portsmouth, Covington (Ohio Bridge), and Maysville bridges were performed between May and December 1979. Laboratory examinations of wire specimens from the Portsmouth and Covington bridges also were conducted during that period. The most recent investigation was an inspection of the Maysville Bridge in November 1982. Most of that work has been reported previously (75). This section is a summary of investigations in Kentucky pertaining to the corrosion of bridge wire.

In early 1974, the Kentucky Department of Transportation removed a suspender rope from the Ohio Bridge. The rope was a central suspender (one of the original suspenders installed by Roebling). The suspender, approximately 15 feet long, was removed from the downstream cable of the Ohio side span. The rope had shown signs of swelling in the lower portion, adjacent to the end fitting. It was replaced by a solid 1.75-inch diameter steel rod. The suspender was 7 x 7 strand, No. 9 gage wire, having a nominal diameter of about 1.5 inches. The wire was made of wrought iron, drawn and fabricated by Roebling. Since those strands supported a third of the main span load, it was necessary to determine the corrosion damage experienced by that "worst-case" suspender.

The rope was sectioned by cross-cutting slices with a pipe saw. Interior sections were visually inspected. Slight swelling of the rope was evident near the lower end fitting; however, the wires showed no signs of gross corrosion (Figure 3). Interstices of the swollen section were filled with rust particles and quartz sand. Sand particles were probably embedded by a previous sand-blasting operation. It was concluded the suspender was in good condition and no further tests were performed (76).

Field inspections of the Portsmouth Bridge began in May 1979. Cables had been unwrapped prior to the winter of 1978. At the time of the first inspections, cable bands were still in place. During the inspections, a survey was made of the condition of the external strands (Figure 4).

Exterior strands were severely corroded in many locations. At one panel point or another, the exterior strands of both cables showed ferrous corrosion (rust). Uniform corrosion was much more severe on the main span and Ohio side span than the Kentucky side span. The downstream cable had more panels having gross uniform corrosion than the upstream cable.

Uniform corrosion was evident either on all exterior strands for an entire panel length (band-to-band) or concentrated in the lower exterior strands for the entire panel length. Severe rust was present for at least two consecutive panels, with the exception of Panel 66-67 on the Kentucky side span, downstream. Contrary to expectations, severe rusting was observed on both horizontal and inclined portions of the cable (Figure 5). Panels where cables were inclined usually had more exterior rust than neighboring panels of less inclination.

Only one panel (66-67, downstream) had severe rust on the Kentucky side span. Thirty-three of the forty main-span panels on the downstream cable were severely rusted. Twelve of the main-span panels of the upstream cables showed extensive rusting. Those were located on the lowest (shallowest) portion of the cable. Twelve panels of each cable on the Ohio side span showed severe rust.

All exterior strands showed some degree of zinc corrosion. In places where wires had rusted, the zinc coating obviously had failed by corrosive attack. Zinc corrosion was evidenced by its powdery white corrosion product deposited on the surface of the strands. On lightly corroded strands, the white corrosion product was tenaciously attached to the wires. In areas where heavy zinc corrosion occurred, sometimes with little or no visible rusting, the white corrosion product had a thick, fluffy texture. The corrosion product could be removed readily by light scraping with a finger nail. Usually, the steel wire, revealed

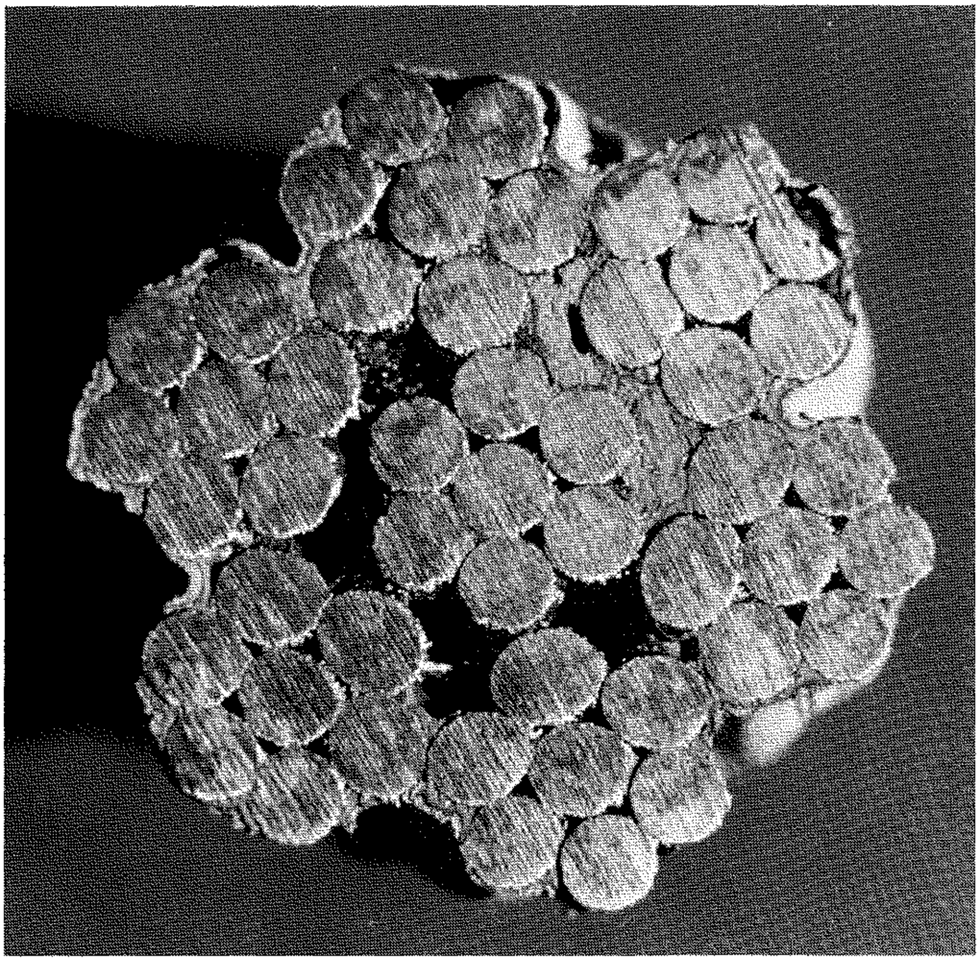


Figure 3. Swollen Section of Ohio Bridge Suspender Rope (1974).

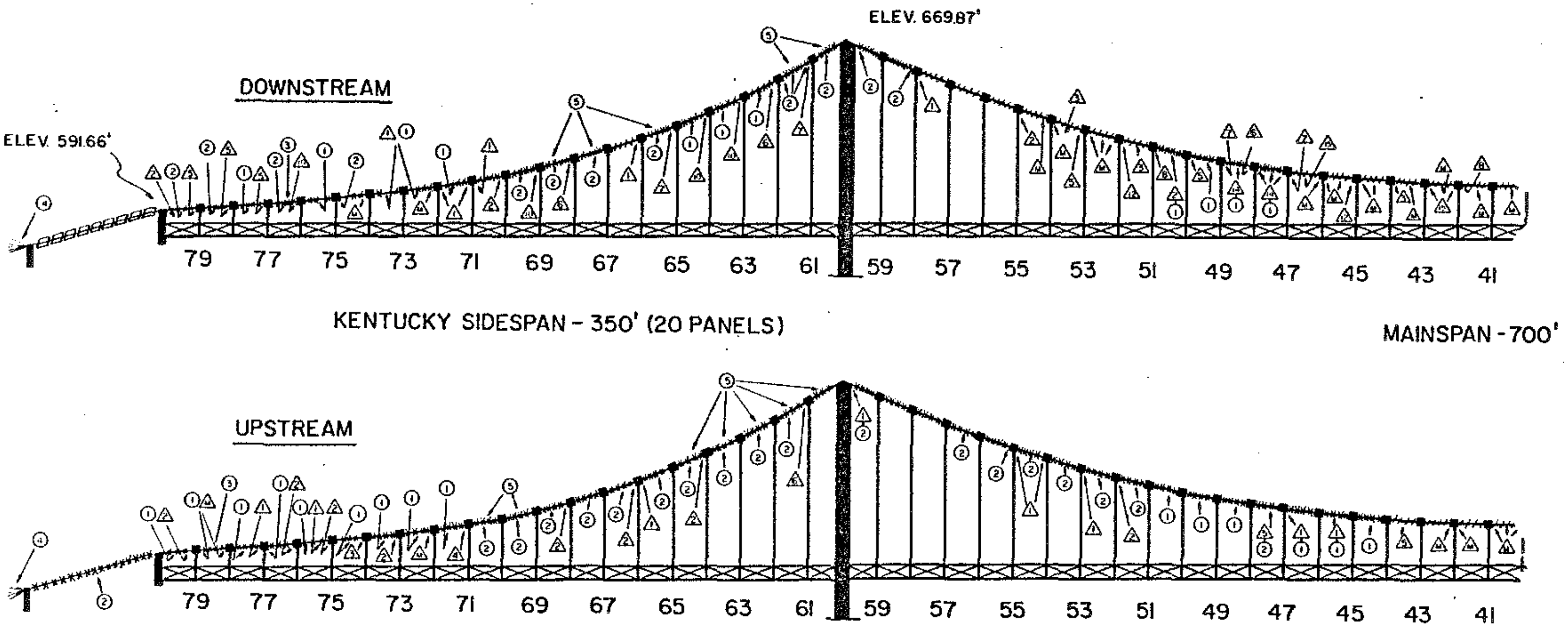
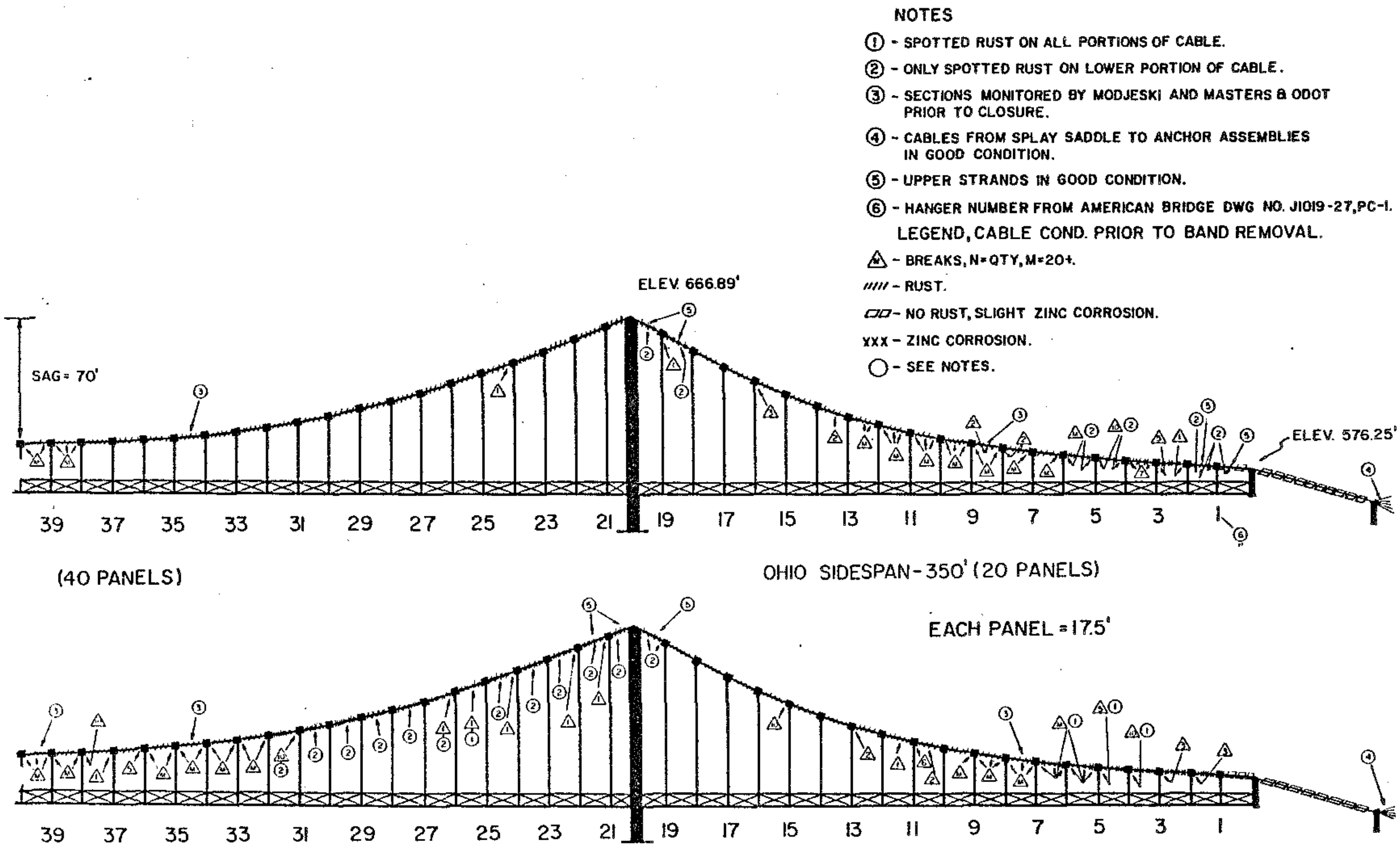


Figure 4. Corrosion Survey on the Portsmouth Bridge Prior to Recabling in 1979.



NOTES

① - SPOTTED RUST ON ALL PORTIONS OF CABLE.

② - ONLY SPOTTED RUST ON LOWER PORTION OF CABLE.

③ - SECTIONS MONITORED BY MODJESKI AND MASTERS & ODOT PRIOR TO CLOSURE.

④ - CABLES FROM SPLAY SADDLE TO ANCHOR ASSEMBLIES IN GOOD CONDITION.

⑤ - UPPER STRANDS IN GOOD CONDITION.

⑥ - HANGER NUMBER FROM AMERICAN BRIDGE DWG NO. J1019-27,PC-1.

LEGEND, CABLE COND. PRIOR TO BAND REMOVAL.

△ - BREAKS, N=QTY, M=20+.

//// - RUST.

▭▭ - NO RUST, SLIGHT ZINC CORROSION.

xxx - ZINC CORROSION.

○ - SEE NOTES.

Figure 4. (continued)

Figure 5. The Portsmouth Bridge, Downstream Cable, at the Kentucky Tower (Mainly Stage 2, Except Some Stage 3 on Lower, Outer Strands) (1979).

Figure 6. The Portsmouth Bridge, Downstream Cable, on the Kentucky Sidespan (Mainly Stage 2 Corrosion Damage)(1979).

under those locations, was lightly rusted and slightly pitted. Spotted rust was visible in most locations possessing heavy zinc corrosion. However, rust was not evident in areas of light zinc corrosion.

Nearly all portions of the cables not severely rusted exhibited large amounts of zinc corrosion. That included 19 panels each, on the downstream and upstream cables of the Kentucky side span (Figure 6). Seven panels on the downstream cable and 28 panels on the upstream cable of the main span showed heavy zinc corrosion, but no appreciable rusting. Seven panels on both the downstream and upstream cables also exhibited that behavior.

In certain locations along the cables, the aqueous corrodant had a washing action (Figure 7). In those areas, ferrous corrosion and pitting were severe. Those locations could be identified by the relative absence of zinc corrosion product on the surface. Most of those locations were where the inclination of the cable was relatively steep.

Portions of the cable from the bents to the splay saddles appeared in good condition, with the exception of the downstream Kentucky side span, which had some spotted rust. Most individually exposed strands, from the splay saddles to the anchor assemblies, were also in good condition, except for some severe zinc corrosion on the lower wires of a few strands. Panels 0-1 of both Ohio side-span cables were also in good condition.

Most breaks detected prior to removal of the bands were located in the lower portion of the exterior strands (Figure 8). Many broken wires were clustered where 1) most of the galvanizing was depleted by corrosion, 2) the wires were rusted severely, and 3) the inclination of the cables was shallow. In areas where little rust was evident, breaks were infrequent. But, a few were found, individually or clustered in groups up to ten, usually near cable bands (Figure 9). That characteristic was prevalent in many panels on the Kentucky side span and on the steeply inclined portions of the main span and Ohio side span. Few breaks were found in panels having steep slopes, even when uniform corrosion was severe.

Prior to band removal, some 300-400 breaks had been discovered on the cables. Portions of the cables under the bands appeared to be more corroded than adjacent portions (Figure 10). When the main cables were removed, between June 27 and July 18, 1979, hundreds of new breaks were found in the upper interior strands. Most of those were detected after the strands had been pulled through sheaves and grounded on the river bank; however, none of the strands were completely severed by breaks. From a rough count of breaks per strand made during the removal operation, it was estimated each cable had about 350 broken wires (77).

In early June 1979, 350 feet of the main-span suspended structure was removed as part of the cable replacement program. The remaining portions of the truss were supported by temporary stays and A-frames. Before dismantling the damaged cables, the suspender cables and bands were removed. When that was accomplished, several important discoveries were made.

Many drain holes, located in the packing of the vertically split bands, were improperly installed and did not allow water to drain from the cables. Though lead wool packing had been well driven into gaps between the band halves, water had frozen between them and three or four bands had suffered pop-outs. Many new wire breaks were detected at

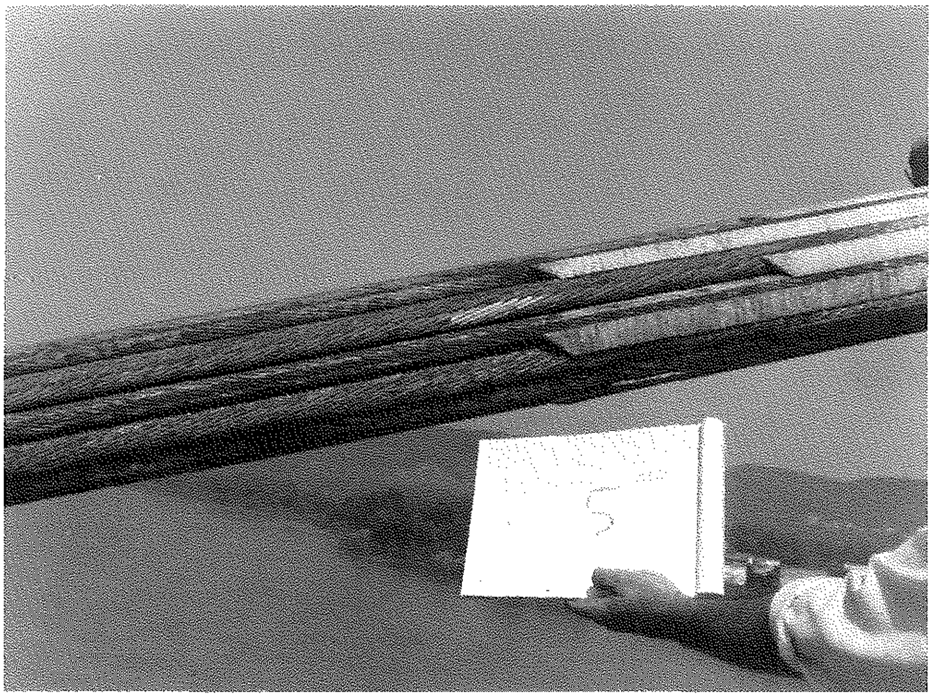


Figure 7. Stage-4 Deterioration on the Main Span, Downstream Cable, also Exhibiting Signs of Washing (1979).

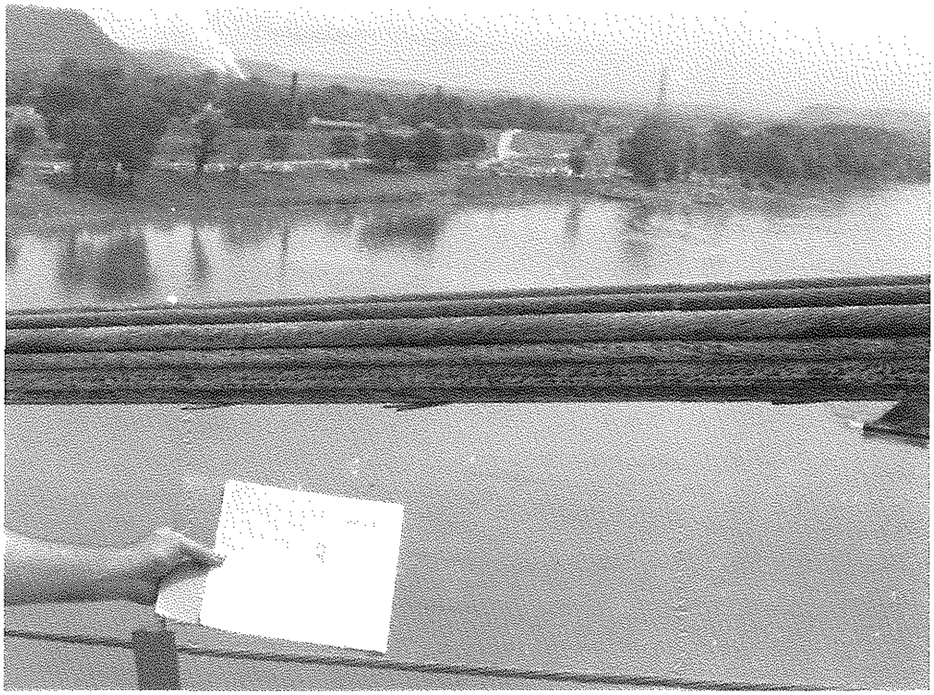


Figure 8. Breaks Clustered on Longer, Outer strands on Main Span, Upstream Cable, at Stage 4 Deterioration Site that Does Not Show Signs of Washing (1979).

Figure 9. Wire Fractures Adjacent to a Cable Band (Strands Show Signs of Early Stage 3 Deterioration)(1979).

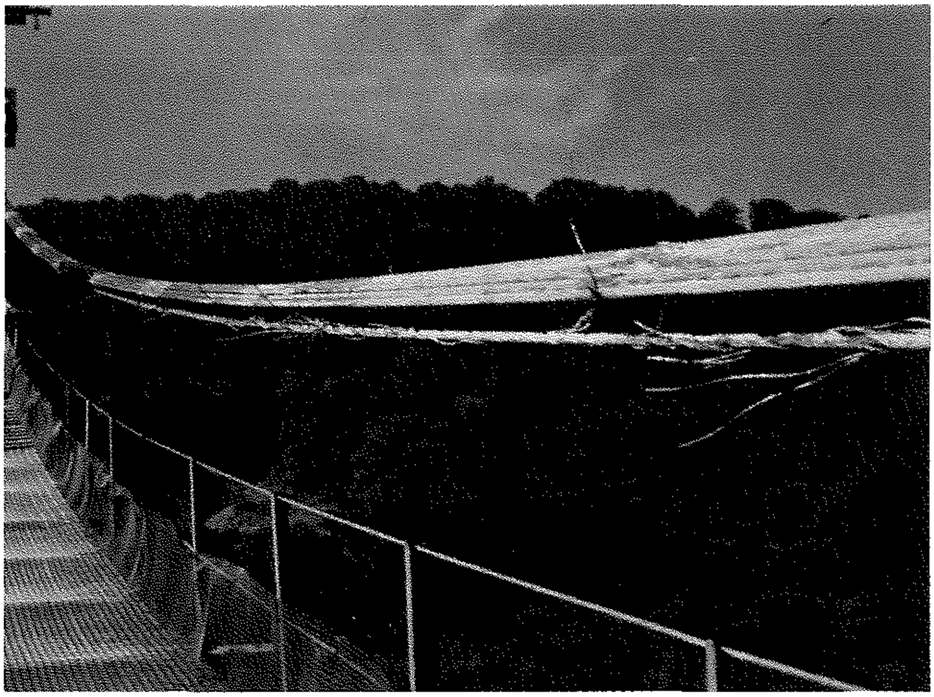


Figure 10. Portsmouth Bridge after Band Removal (Note Corrosion Marks at Band Locations)(1979).

points previously covered by bands. Many breaks were discovered on the upper strands at those locations. Prior to removal of the bands, no breaks had been observed in the upper strands. Under one band, breaks were noted on every exterior strand. Inspection of the interior strands at those points revealed extensive corrosion and new breaks (Figure 11). Unfortunately, the construction schedule precluded a detailed inspection of interior strands.

After the strands were removed, specimens were cut from strands for laboratory examination. The strand profile was maintained by banding the strand ends prior to sectioning. The cutting operation was accomplished with an abrasive cutoff disc mounted on a pavement cutter. That operation was difficult, as the strand segments tended to curl into loops. That was probably due to a plastic set acquired when the strands were mounted on spools, some 40 years ago, or when the strands were bent about a sheave during the dismantling operation. The wires also had a tendency to splay apart at the end when a strand was cut.

Specimens were removed from different strands and parts of given strands that showed various corrosion features. The specimens were taken to the laboratory, sectioned, and subjected to visual inspection. One wire rope suspender, also taken as a specimen, was subjected to only external visual inspection.

Several samples of strand that had no exterior corrosion were collected. On separating those specimens, no interior corrosion was noted, however, the faying surfaces of individual wires exhibited signs of fretting. That caused erosion of galvanizing and slight plastic deformation of the wire. The wires bore continuous longitudinal marks from contact with neighboring wires in the same layer and transverse stripes from contact with the adjacent layer of wires that had opposite lay.

Specimens of the Portsmouth strand also indicated occurrence of crevice corrosion. Some specimens appeared to be in good condition, having only slight zinc corrosion near wire interfaces. When splayed, those specimens revealed zinc corrosion and spotted rust on the backside of exterior wires. The second layer of wires was covered with corrosion products. The third layer was in good condition, as were the other interior layers (Figure 12).

From the inspections of the Portsmouth Bridge, both prior and subsequent to the recabling operation, it was determined that galvanized helical strand experiences four distinct stages of deterioration. During Stage 1, the strand is in "as new" condition (Figure 13). Zinc coating has a bright metallic appearance, though some slight spot corrosion of the zinc may be evident in the form of a thin, white powdery coating. The strand is in good condition during Stage 2. The zinc has a dull gray appearance. The white corroded-zinc film may be present near interfaces and on exterior surfaces of the wires. When the white film is removed by scraping, no rust is evident. As previously mentioned, the second layer of wires may be in worse superficial condition than the outer layer. But, as long as the outer layer of wires is stable, interior wires will probably remain structurally sound. Much of the strand is covered with a thick, white zinc corrosion product in Stage 3. Spotted rust is also visible on the wires. When the corrosion product is scraped off, the steel under the surface reveals some rust and pitting. Wire fracture by corrosion cracking is possible in that stage; however, breaks will not be clustered in large numbers,

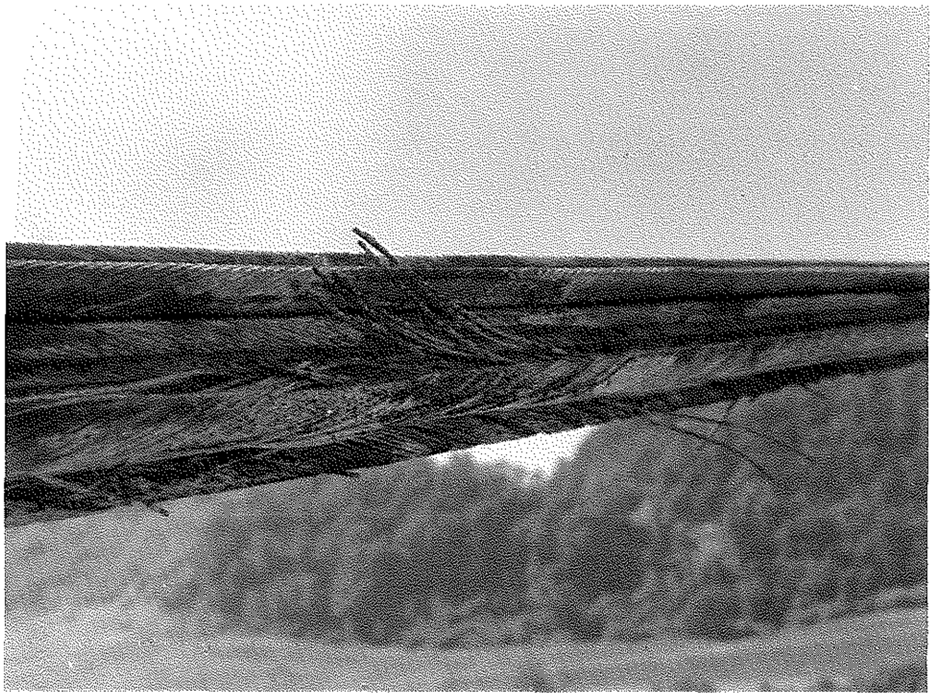


Figure 11. Portion of a Cable previously Covered by a Cable Band (Stage 4 Corrosion Damage)(1979).

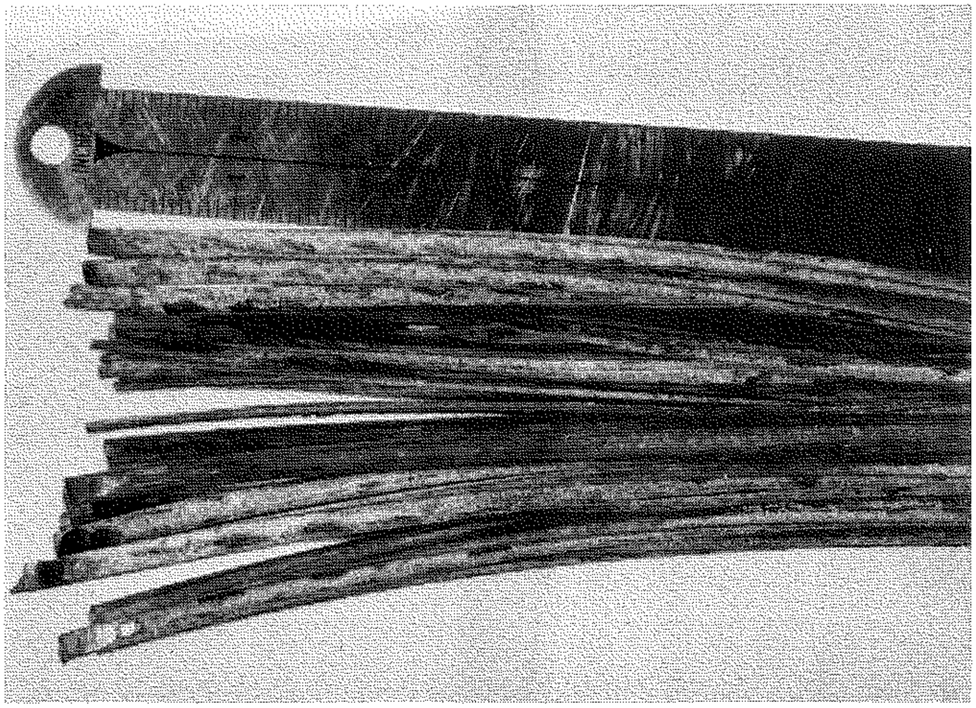


Figure 12. Separated Portsmouth Bridge Strand Showing Fretting Marks and Stage 2 Corrosion.

Figure 13. Stage 1 Condition on New Helical Strand.

except near points of high stress or aqueous corrodant concentration. In Stage 4, the strand will be severely rusted and pitted. Some zinc corrosion will be displaced by corrosion of the underlying steel (rust). The wires will have a speckled brownish-red and white appearance. When either the loading or corrosion conditions become severe, the strand will develop many corrosion cracks and readily fail in this stage.

Specimens exhibiting washing showed severe surface rust, pitting and little retention of the white zinc corrosion product on the exterior layer of wires. Despite that poor external appearance, the corrosion on interior wires was often no worse than in specimens previously described.

Severely corroded wires from locations of poor drainage were externally similar to the washed specimens, except for presence of a white zinc corrosion product. Inspection showed all the interior wires were severely corroded. Fretting marks were still visible on those specimens except at points where corrosion had depleted adjacent wires until they were no longer in close contact. Where close contact was maintained, neighboring wires wiped away the corrosion product, exposing the underlying bare metal.

The ends of broken wires exhibited three different fracture morphologies. Some wires had complete fractures transverse to the longitudinal axis of the wires. More commonly, a transverse fracture emanated from the surface and penetrated about halfway through the wire. Thereafter, the fracture reoriented itself 45 degrees to the longitudinal axis and penetrated through the wire. A few specimens had slanting fractures, typical of those examined by Pollard. None of the sectioned specimens revealed fractures in the internal layers that were not preceeded by fractures in the external layer of wires. Some broken wire specimens had a fracture spacing of less than three inches. All wire specimens having corrosion fractures exhibited complete zinc corrosion and, at best, some light surface rusting.

Inspection of wire rope suspenders in the field showed them to be in fair condition. The ropes did not show appreciable wear, even though they were 53 years old at the time of removal. The suspender ropes were galvanized and painted. The paint was chipping badly at the time of removal and replacement. The zinc coating of exposed wires had deteriorated in places and some rust was evident; however, the rusted wires were not pitted.

Inspection of the Maysville Bridge (Figure 14) began in June 1979. The bridge had been in continuous service for 49 years. It had no known history of corrosion problems. The first three inspections (June-August 1979) consisted of external inspections of wrapped bridge cables and splayed wire-rope strands inside the bridge anchorages.

Externally, the paint was in fair condition. The cables were last painted in 1974. Transverse cracks had developed in the paint on the upper portion of the cables. Paint had peeled from the lower surfaces of the cables in spots. This usually was observed where inclination of the cables was slight. Many points on the undersides of cables, adjacent to bands, showed signs of frequent peeling in the previous top coat of paint.

Suspender ropes were in good condition. Unlike conditions on the Portsmouth Bridge, which was designed to have drain holes in the lower packing, only two drain holes were provided per cable on the Maysville Bridge. Those were located in the lower packing of bands at the center

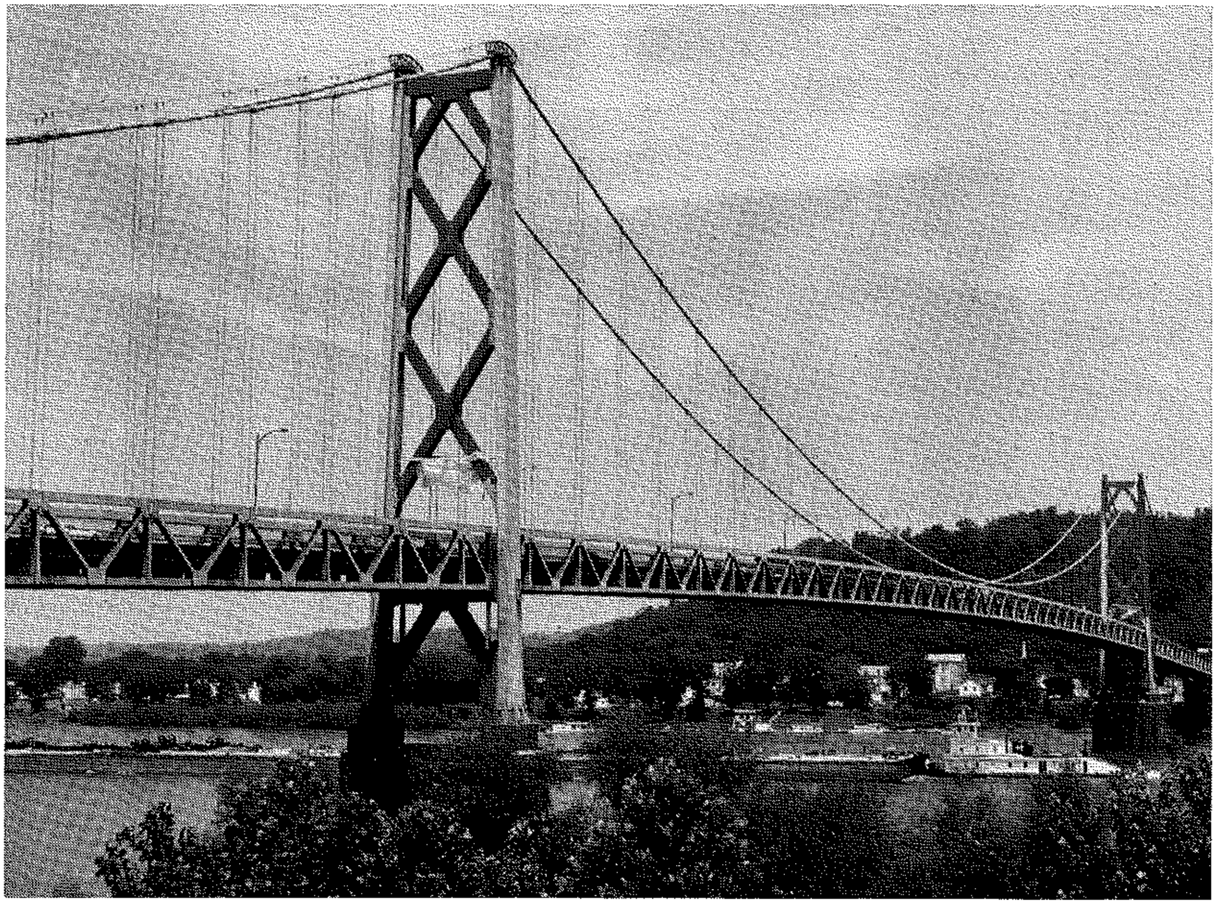


Figure 14. The Maysville Bridge (1977).

of the main span. The cable was horizontal between the drain-equipped bands.

Inspection of bands that had no drain holes revealed caulking (lead wool) on the underside of several bands had been forced out of place. Rust-colored stains were detected adjacent to many of those pop-outs on the cast-steel band halves (Figure 15). During an earlier inspection, after a rain, water was seen dripping from the stained portion of one band that had suffered a caulking pop-out.

Inspection of the strands in the anchorages revealed them to be in generally good condition. The strands had a base coat of red-lead primer with a top coat of green paint. The paint work appeared to have been poorly executed. Paint and primer had chipped off many strands. In spots, the zinc coating was depleted, leaving the usual chaulky white corrosion product, which was presumed to be zinc oxide. In closely inspected areas, no rust was detected under the corroded zinc. The end portion of strands in the sleeve bushings appeared to be in worse condition. There was much loose rust and chipped paint in the recesses of the sleeves. At the splay saddle in the Ohio upstream chamber, there appeared to be some washing of red lead, or rust, from the exterior cable that was visible on the lower strands.

In August 1979, bridge maintenance personnel installed seven inspection ports on the Maysville Bridge (Figure 16). Those were located at Panels 0-1, 7-9, 11-13, 59-61, 65-63 and 69-69 on the Ohio side span and main span (Figure 17). Ports were installed at points near bands having pop-outs and on panels where cable paint had deteriorated.

Exposed cables revealed that the Maysville Bridge strands, unlike those on the Portsmouth Bridge, were individually covered with a generous coating of red lead. Unfortunately, that coating was deteriorating, and in many locations, the zinc coating was depleted. At all inspection sites, corrosion of the zinc coating was worse on the lower strands. All exterior strands in Panels 0-1 and 69-69 on the downstream cable were in very good condition. The upper strands of the downstream cable, in Panels 11-13 and 65-63, were also in good condition; however, many bottom strands had extensive zinc corrosion. All external strands on the upstream inspection sites had zinc corrosion. Several dark rust spots were visible on the lower strands at Panel 59-61, downstream, and at Panel 65-63, upstream (Figure 18). The zinc coating in those areas was severely depleted. Except for a few large rust spots, no pitting or light rust was detected under the zinc oxide.

At several sites, a small amount of water seeped from the cables when the wrapping wire was removed. The red-lead coating was disintegrating from the lower strands in those locations and the layer of corroded zinc was thicker than on the upper strands. Some water had probably flowed to the lower portions of the cable between the wrapping wire and strands. The interior lower portion of the removed wrapping wire was corroded, having considerable spotted rust. Apparently, some retained moisture that settled on the inside of the wrapping had seeped into the lower portions of the wrapping wire and froze, thereby causing the paint to peel off.

On December 5, 1979, a follow-up examination of inspection ports was performed. Several days before the inspection, the Maysville area was subjected to a heavy rain. Water was dripping from popped-out packing

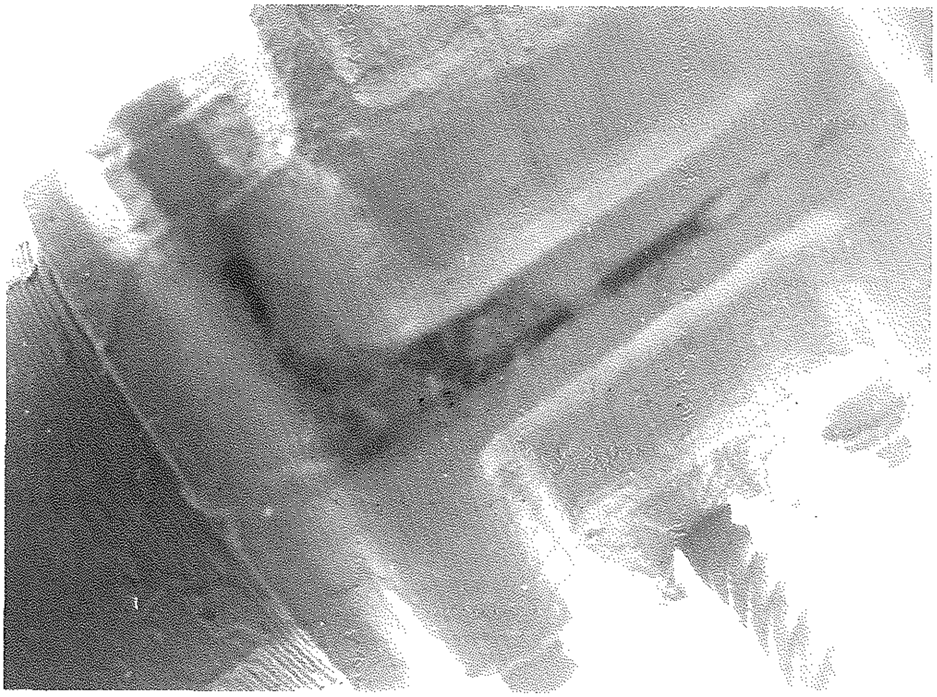


Figure 15. Pop-out and Rust Stain on Cable Band of Maysville Bridge (1980).

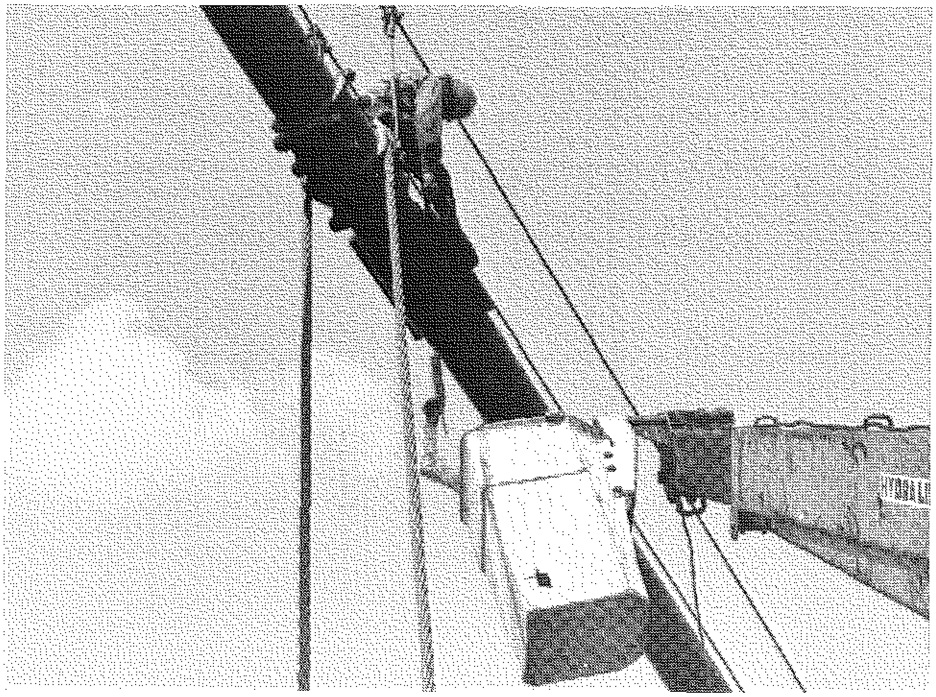


Figure 16. Installation of Inspection Port on the Maysville Bridge (1980).

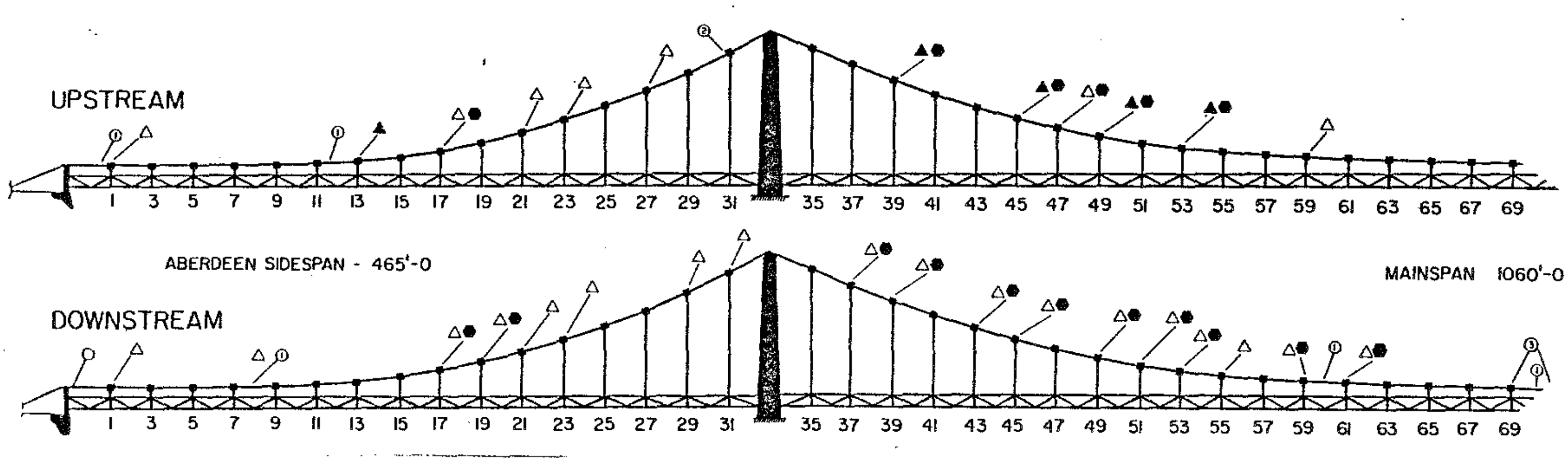
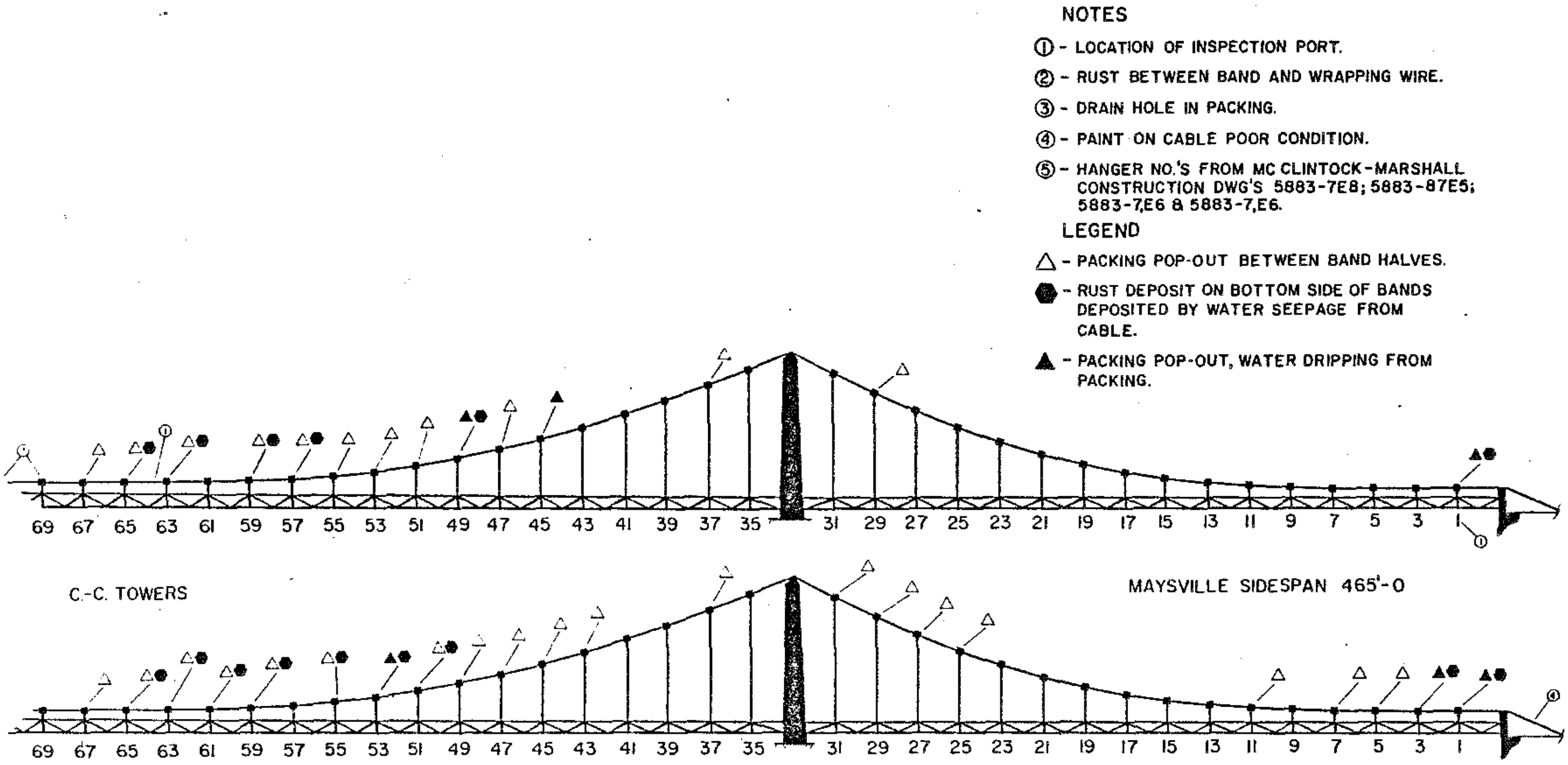


Figure 17. Exterior Survey of the Maysville Bridge (1980).



NOTES

① - LOCATION OF INSPECTION PORT.

② - RUST BETWEEN BAND AND WRAPPING WIRE.

③ - DRAIN HOLE IN PACKING.

④ - PAINT ON CABLE POOR CONDITION.

⑤ - HANGER NO.'S FROM MC CLINTOCK-MARSHALL CONSTRUCTION DWG'S 5883-7E8; 5883-87E5; 5883-7,E6 & 5883-7,E6.

LEGEND

△ - PACKING POP-OUT BETWEEN BAND HALVES.

● - RUST DEPOSIT ON BOTTOM SIDE OF BANDS DEPOSITED BY WATER SEEPAGE FROM CABLE.

▲ - PACKING POP-OUT, WATER DRIPPING FROM PACKING.

Figure 17. (continued)

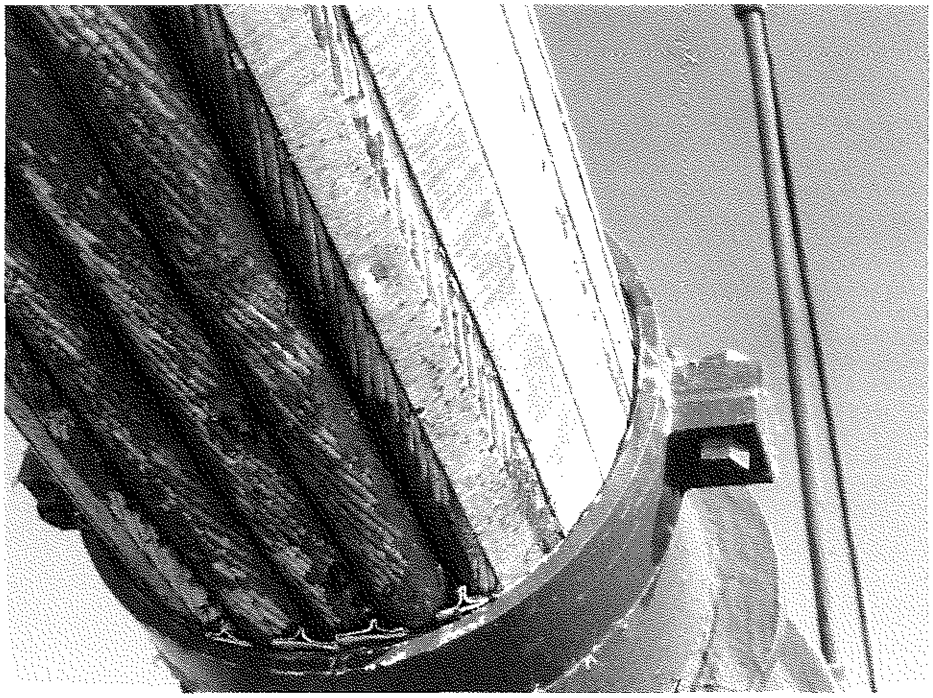


Figure 18. Rust Spots on Strands of the Maysville Bridge Cables (Early Stage 3 Deterioration)(1980).

at many band locations. When most of the inspection ports were opened, several quarts of water poured from the cable. Since the port caulking was in excellent condition at all locations, it was presumed water entered the cables at the bands or through the wrapping wire and settled in the ports. Red-lead residue was found on the inspection port covers located on the bottom face of the cables. That indicated a flow of water in the cables sufficient to cause washing. Because the closed inspection ports had collected water, drain plugs were left off the port cover plates on reassembly to minimize drainage.

Inspection of the caulking pop-outs was conducted from the sidewalk using binoculars. The frequency of band pop-outs indicated, despite wire wrapping, the cables are virtually porous. Presence of pop-outs near the towers revealed a significant amount of water may be entrained, either at wrapping discontinuities along a single panel, or at a sealing failure on a band. Seepage of water at a band, one panel lower than another band having a pop-out, may also indicate that water may be retained at the bands or by capillary action in the strands. Retention possibly may be aided by buildup or bridging of corrosion products and red lead near the pop-out.

On October 18, 1982, a second examination was conducted on the cables of the Maysville Bridge. One inspection port on the downstream cable (at Panel 69-69) and another on the upstream cable (at Panel 65-63) were opened. Previously, the cable exposed at Panel 69-69 had not shown evidence of severe corrosion. The condition of the cable, at that point, appeared to be unchanged at the time of the second examination. The upstream inspection port location previously had disclosed two rust spots of about one inch in diameter (1979). The second inspection revealed the rust spots had almost doubled in size. Also, new rust spots were visible along the lower exterior strands.

No wire breaks were detected on the Maysville Bridge during any inspection of the bridge cables. Presently, it would be reasonable to conclude the inspection sites chosen were representative of the entire bridge, with the possible exception of several locations on the Kentucky side span and at points under the cable bands.

Inspections were made of the Ohio Bridge at Covington in June and September 1979. The scope was similar to examination of the Maysville Bridge; however, inspection ports were not placed on any of the cables.

Superficial visual examination of the wrapped cables revealed no major physical defects. The wrapping and paint appeared to be in good condition. Cables were last painted in 1974. Only a slight amount of chipped paint was detected. That was in the original upstream cable at midspan where the cable borders the sidewalk. Rust-colored stains were visible at some cable bands. The nature of those stains could not be ascertained due to poor access. During the inspection, water was detected dripping from the lower upstream cables near the low point of the cable at midspan. The leakage was slight and emanated from a splice in the wrapping. Moderate rusting was detected on suspender ropes near the bottom sockets. Previous experience had indicated resulting corrosion damage to be slight.

An inspection also was made on the secondary cables at the Kentucky tower roller supports. The upper cables, located in the tower turrets, were covered with a thick layer of pigeon droppings. Stay cables also were covered with pigeon droppings (Figure 19). A bird screen was in place on the turret aperatures, but the pigeons had obviously penetrated

Figure 19. Pigeon Droppings on the Secondary Cables of the Ohio Bridge (1980).

the defense. One broken wire was found on the downstream cable at the roller support.

Anchorages were given the most thorough examination due to the history of wire corrosion at those locations. Inspection was made of all four original cable anchor houses. The upstream Kentucky secondary anchor house door opened directly onto the roadway. Traffic on the bridge prevented inspection of that anchorage.

Three original anchor houses (both upstream anchorages and the Kentucky downstream anchorage) showed evidence of water leaking from the anchor house roofs. Seepage into the Ohio downstream anchorage was slight, and anchor assemblies and strands were not damaged. In the other original anchorages, water had dripped onto the strands, eyebars, and anchor assemblies. That leakage caused deterioration of a whitewash wall-and-ceiling coating. Wrought-iron roof beams also were rusted. Dissolved whitewash was deposited as a white residue at spots where the leaks impinged on wires and anchor assemblies.

All metal surfaces in the anchorage were coated with a thick layer of red lead. That paint was in good condition in all places where water had not leaked onto the cables. The leakage had deteriorated the paint and severely corroded the underlying metal. Old wire breaks were observed at the three anchorages that had leakage problems. The number of breaks ranged from one in the Kentucky downstream anchorage to three in the Kentucky upstream anchorage (Figure 20). Most broken wire ends had been painted over, indicating those probably predate the present leakage problem. The broken unpainted ends of wires showed severe uniform corrosion and loss of section near the fractures.

Many wires in the lower strands contained splices that presumably date back to 1892. Most of those splices were located on the lower portion of the strands and were inaccessible. Splices were made by interlooping mating wire ends and wrapping each looped end to the parent wire with a tie wire (Figure 21). That type splice does not provide high joint strength and was not the type of splice employed by Roebling on the wires in the main cable. Many of the spliced wires were loose. It is questionable whether those wires were in tension.

Besides roof leakage, moisture was observed on strands adjacent to the splay saddle in the downstream Kentucky anchorage. That may be caused by leakage of water from the wrapped cable, or by sealing failure at the splay-saddle collar, located outside the anchorage.

The Kentucky secondary cables entered the anchorage through the roof of each extended anchorage. The strands and anchor assemblies employed the same paint system used on the original cables. The strands in the downstream Kentucky secondary anchorage house were covered with light dust. The upper surface of the strands were rough in a few locations, indicating possible presence of light rust under the paint. Excluding that observation, the paint and strands appeared to be in very good condition. There was no indication of water leakage onto the strands.

The Ohio secondary cables were housed in separate anchorages located under the Ohio approach. Cables entered through large open portals in the headwalls of the anchorages. One unusual feature was the slight splaying of the cables before they entered the anchorages. The splayed portion of the cables was sheathed in a sheet-metal jacket that extended down to the splay saddles. Concrete approach curbs were cast directly against the cable sheaths where the cables crossed the curbs into the anchorages.

Figure 20. Broken Wires and Water Leakage Stains in the Upstream Kentucky Anchorage of the Ohio Bridge (1980).

Figure 21. Repair Splice on Wires in Upstream Kentucky Anchorage of the Ohio Bridge (1980).

The Ohio secondary anchorages were very dusty. At the time of inspection, the bridge trusses were being repainted and the large open cable portals allowed the sandblast refuse to enter the anchor houses. Pigeons had frequented the upstream anchor house, depositing droppings and feathers on the strands. Anchorage roofs were part of the concrete approach roadway. Some white residue was detected on the upper portions of several strands, indicating some roof leakage may have taken place.

The strands and anchor assemblies used the same paint system employed in the other anchorages. At most locations, the paint appeared to be in good condition. There was no sign of paint cracking or peeling; however, the upper surfaces of some strands were slightly rusted. Rust was detected under a heavy layer of black sandblast material that lay on the upper surfaces of the strands. In both anchorages, several strands were rusted and swollen between the strand seizings (Figure 22). Those strands were connected to the lowest strand shoes on the anchor assemblies near the anchorage floor.

One break was noted in the upper strands of the upstream anchor house. The break was located near the cable portal. The broken end had been painted, so the cause of fracture could not be determined. One wire in the downstream anchorage ran between two adjacent strands. The wire was taut. Whether that was due to an impressed load or tension caused by the seizings could not be determined.

On November 19, 1982, two small, unstiffened single-lane suspension bridges at Tram and Sutton in Pike County (Figure 23) were examined. The main cables of those bridges were helical strands that were exposed to the atmosphere. The strands had served well over their 30-year service lives. The wires showed some light corrosion of the zinc at the wire interstices, but no ferrous rust was detected (Figure 24).

REVIEW OF BRIDGE WIRE CORROSION PROBLEMS

There was little impetus to investigate corrosion problems on bridges until the Silver Bridge collapse at Point Pleasant, West Virginia, in 1967. For that reason, there was a lapse in bridge wire corrosion research between the mid-1940's and the early 1970's; however, notable gains in the understanding of corrosion mechanisms occurred during that period.

In the 1970's, the principal work on bridge-wire corrosion was contained in the Boeing-FHWA reports on stress corrosion of highway bridge steels. That was followed by investigative reports on the Portsmouth Bridge second corrosion problem in 1979 and 1980.

The lack of research on bridge-wire corrosion may be due, in part, to a general belief that the Portsmouth Bridge corrosion problems were isolated phenomena caused by unusual local conditions. To dispel that view, a review of the elements of relevant literature is presented. Hopefully, this will also provide a unified view of suspension bridge-wire corrosion based upon literature that is sparse and somewhat disjointed.

The National Bureau of Standards' investigation of heat-treated wire, originally employed on the Ambassador and Mt. Hope bridges, bears note because it set a precedent for much of the following failure analyses of bridge wires. Researchers attempted to replicate the wire fracture process in laboratory tests. The six-year lapse between the

Figure 22. Swollen Strands in the Ohio Bridge, Secondary Anchor House (1980).

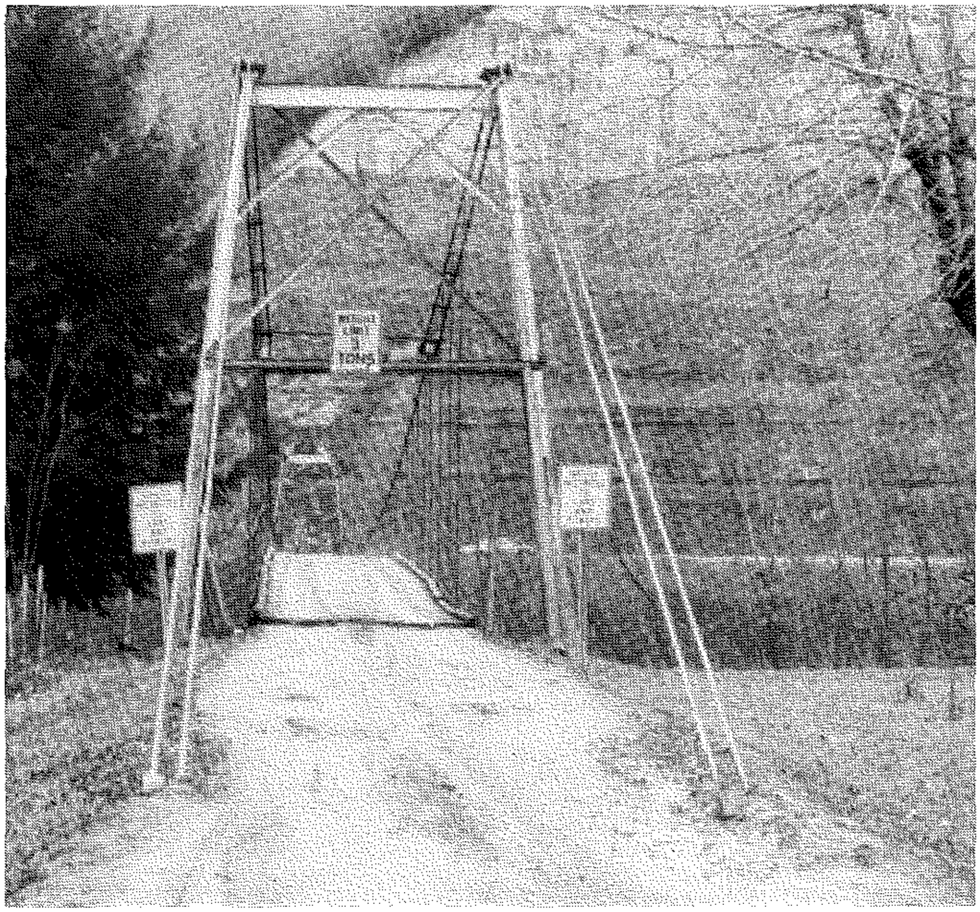


Figure 23. Unstiffened Suspension Bridge in Eastern Kentucky (1982).

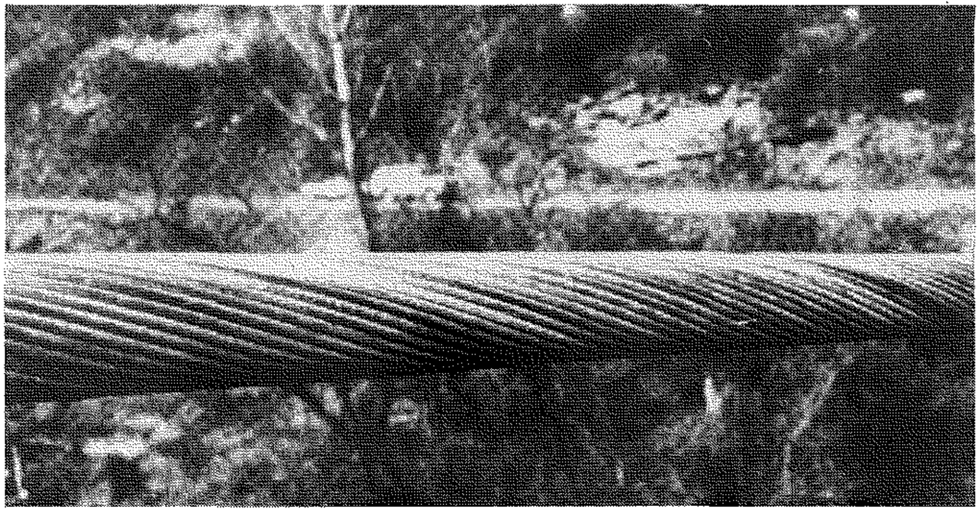


Figure 24. Exposed Cable on an Unstiffened Suspension Bridge (1982).

initiation of tests and publication of results indicated difficulty was encountered in that approach.

The investigators were cognizant of the multitude of problems encountered at the wire mill, and the association of those problems with hydrogen embrittlement. However, when Bureau of Standards' research was conducted, the investigators may have been unaware of all aspects of the damaging effects of retained hydrogen. The landmark paper on hydrogen problems by Zapffe and Sims (78) was not published until several years after the Bureau of Standards report.

The Bureau of Standards' investigators assumed the galvanized wire would outgas any residual hydrogen at room temperature in a short time and concluded the heat-treated wire problems were due to other causes. However, Townsend (79) has shown that hot-dip galvanizing traps hydrogen and will not permit outgassing even at elevated temperatures.

The Bureau of Standards' investigators performed a series of mechanical tests on both heat-treated and cold-drawn wire. Most of those tests were inconclusive. Even when the resulting static tensile failures of the heat-treated wires resembled hydrogen-induced fractures, the investigators failed to grasp the relationship.

Comparative fatigue tests between galvanized heat-treated and galvanized cold-worked wire revealed that heat-treated wire possessed a slightly lower fatigue limit. Galvanized wire of both types had significantly lower fatigue limits than uncoated wire. The investigators reported that fracture surfaces of the fatigued wires resembled field failures. Without enumerating those features or any further significant findings, the investigators concluded the field problems were fatigue-related.

Based on present knowledge, the heat-treated wire problems were probably related to classical hydrogen embrittlement. Most field fractures occurred at low construction stresses. The wires had been subjected to relatively few fatigue cycles at the time of fracture. Most field breaks occurred at locations where static stresses were high due to residual stresses caused by bending the wires (i.e., at the strand shoes and bands).

Laboratory fatigue tests revealed heat-treated wires were not noticably weaker than normal cold-worked wires when the surface roughness condition was taken into account. Since the wires tested were unnotched, fatigue crack initiation was probably more significant than fatigue crack propagation. The difference in fatigue limits between galvanized and bright wires may be explained by several facts: 1) the surface finish of galvanized wire is rougher than that of bright wire and 2) galvanized wire will tend to form small cracks at the zinc-steel interface. Both factors will decrease the number of load cycles required for crack incubation and lower the fatigue limit in an atmosphere of low corrosivity.

Fatigue testing is an unsatisfactory method for detecting residual hydrogen damage. Static tensile tests on field specimens indicated wide variation of wire strengths. Many field specimens, that failed at stresses lower than the design yield strength, had transverse fractures and exhibited little reduction of area at the breaks. Fractures encountered in the field were similar to the laboratory static tensile tests in those respects. The ability to produce such static tensile failures is a good sign that hydrogen embrittlement was involved. No mention was found of corrosion being detected on the wires of the two bridges. Wires of the Ambassador and Mt. Hope bridges were not exposed to the atmosphere for sufficient time for corrosion to have been a factor.

Wire from the first Portsmouth Bridge corrosion problem was originally analyzed by the Carnegie Institute of Technology. Specimen wires from the bridge were subjected to mechanical (tensile) tests. Wire specimens, from locations near anchorage shoes, failed at low stresses and elongations. Probably, the wires tested possessed undetected corrosion cracks that caused the low mechanical properties. Wires from locations outside the anchorages that were tested met existing materials' specifications. No significant cracking was noted in cables at points outside the anchorages. The Carnegie investigator opined that only wires near the anchor shoes were "embrittled" and that those wires had suffered from a combination of stress and corrosion, or stress-corrosion cracking. However, the investigator had not performed any work to verify that statement. It should be restated that knowledge of corrosion cracking was minimal at the time of that investigation.

Several years after the Carnegie report, the National Bureau of Standards performed stress-corrosion tests on bridge wire. The investigator, Pollard (80), stated:

> "The principal purpose of the supplementary investigation, therefore, was not to determine the cause of the original failures but to attempt to produce experimentally stress-corrosion breaks in the wire similar to those which occurred in service."

Pollard was able to produce stress-corrosion failures similar to those from the Portsmouth Bridge. He then concluded:

> "The results of the laboratory tests strongly support the tentative conclusion cited above, that the service failures of the Portsmouth bridge wires were due to the combined effects of stress and corrosion. The possibility of parts of the cables having been exposed to the attack of nitrate solutions of concentrations comparable to those used in the laboratory tests has been pointed out and the results of these tests show that failures can be caused by the corrosive action of such solutions in combination with imposed stresses comparable to those that could be produced merely by straightening the wire."

The conclusions are in contradiction to the aforestated aims of Pollard's research. Based on the facts, then existent, those conclusions were speculative.

Most Portsmouth test wires were bent into a smaller curvature than the reel set (approximately 25 inches). Specimens that did not fail in Pollard's tests assumed a smaller curvature than the reel set, showing that the wires were tested at yield-magnitude stresses. None of the straightened wires tested by Pollard failed. It is questionable whether the high stresses imposed during the Bureau of Standards' bend tests reflected service stresses on the bridge.

Another question arises regarding the choice of potential corrodants used in the tests. Pollard used ammonium compounds, nitrates, and sodium hydroxide. Of those, ammonium nitrite and sodium hydroxide are now considered corrosion inhibitors. Most corrodants Pollard tested were not those that would normally be associated with a highway environment. Perhaps his choice of potential corrodants reflects the limited knowledge of stress corrosion that existed at that time.

Mention has been made in other literature that trace amounts of nitrogen compounds were detected on the Portsmouth wires (81). To the authors' knowledge, no investigation has been conducted concerning the presence of harmful corrodants in the Portsmouth area. No nitrates were detected in the river, anchor house moisture, or on the surface of the wires. No mention of nitrates or nitrogen was indicated in the chemical analysis contained in Pollard's report. Also, it may be significant that the amount of sulfur Pollard detected in the wires was in the range of allowable sulfur content for bridge wire. It is not known whether the wires were cleaned prior to chemical analysis.

If trace quantities of nitrogen are detected in chemical analyses of steel, the source of that element should be determined. Trace quantities of ammonium salts on corroding wire may be formed from the nitrogen content of the steel (82).

Many recent researchers have mistakenly presumed that Pollard conclusively proved the first Portsmouth corrosion problem to be stress corrosion induced by nitrates. That is not true.

Most corrosion cracking on the original Portsmouth wire occurred at tangent points to the wire curvature at the strand shoes. Due to the location of the cracks, some engineers felt that a major factor in the cracking process was high stresses (residual and stress concentrations) at those points. Failure may have been more related to use of a water-permeable tar coating placed over the strand loops after the 1937 floods, which entered the Ohio anchorages. That coating would allow retention of water in close contact with the wires for extended periods of time.

Recommendations from the Battelle investigation of the second Portsmouth Bridge problem were based on limited information at the time of bridge closure. Subsequent examination of the cables, during dismantling, verified those recommendations, including necessity of replacing the bridge cables.

The Battelle researchers performed a field inspection on the Portsmouth Bridge that provided a better understanding of the corrosion processes. Also, the field inspection provided direction for laboratory work.

Laboratory work conducted by Battelle identified one element, sulfur, that may have been involved in the corrosion cracking process. That indicates the presence of sulfur-bearing compounds (i.e., H_2S and/or SO_2 from the atmosphere), in aqueous solutions, inside the cables. Sulfur compounds may form mild acids and increase the corrosivity of aqueous solutions, increasing the corrosion rate. Sulfur also may act as a cathodic poison and promote the entry of hydrogen into the wires.

Objectives of the Battelle work were to identify the general cause of the Portsmouth wire problems and recommend a course of action. The scope of work was limited by funding and time. The investigation did not address, in detail, the atmospheric conditions at Portsmouth and a causative pollutant or its source. Also, the laboratory work, while adequate to identify a general cause of wire failure (corrosion cracking), was not sufficiently extensive to definitely determine which specific corrosive mechanism or mechanisms were involved.

The Boeing reports are very extensive and cover the broad range of structural ferrous materials. However, as with most overview-type studies, there are possible exceptions or variances to statements or conclusions contained in the study material. The authors feel the Boeing report may contain several such contentious items.

The first Boeing report suggested that stress-corrosion problems due to water/bird-excrement reactions as being unlikely (83); however, there is no known work on that specific issue to completely verify that statement. Also, it should be noted the stay-cable fractures on the Brooklyn Bridge, detected in June 1981, were suspected to involve bird droppings. The buildup of bird droppings on the secondary cables of the Ohio Bridge was considerable. (Those areas have since been cleaned.) The presence of at least one broken wire in that area suggested caution should be exercised prior to accepting the Boeing conclusion in the absence of definitive tests.

Findings of the third Boeing report are seemingly in contradiction with events at Portsmouth, especially in consideration of findings of the Battelle report. The Boeing investigators were unable to create stress-corrosion cracks in notched wire specimens subjected to stresses exceeding the maximum wire design stress at the Portsmouth Bridge. To compound the contradiction, some Boeing tests used SO_2- and H_2S-saturated aqueous solutions. Those are typical of the sulfur-bearing compounds predicted by the Battelle investigators to have caused the Portsmouth bridge corrosion problems.

Both Pollard and Townsend were able to produce stress corrosion failures in wire under more severe conditions than those employed by Boeing. In the Boeing tests, wire specimens were subjected to continuous immersion at constant ambient temperatures. The Portsmouth Bridge cables were subject to periodic wetting and drying, heating and cooling, and freezing and thawing. Some of those factors may have interacted to increase the severity of the corrodant action and decrease fracture resistance of the wires. The method of load application (i.e., static tensile loading) and stress levels used by Boeing might require a long incubation period to initiate stress-corrosion cracking.

Despite use of acetic acid to prevent corrosion buildup, a layer of Fe_xS may have formed on the steel surface, inhibiting entrainment of hydrogen into the notch. Also, cathodic currents may have existed on the Portsmouth wires. Such currents might act to charge hydrogen into the wires at defective areas. The Boeing specimens were not polarized, anodically or cathodically.

Another contributory factor may be the chemical composition of the wires. As shown in Table 2, subtle changes in specifications have led to present-day bridge wire that is lower in carbon, phosphorous, and sulfur than earlier wires. The newer wire may be more resistant to corrosion cracking than the Portsmouth Bridge wire.

The Fairbank Highway Research Station (FHRS) report contained several interesting observations, including the possibility of hydrogen damage, low fracture toughness of wires, and a relationship between surface pitting and final fracture. The report concluded, on the basis of optical inspection, the fractures resembled stress-corrosion failures. However, as with the Battelle report, no detailed analysis was made to prove that fact. The FHRS investigators alluded that the wire was susceptible to stress-corrosion cracking and hydrogen embrittlement due to its mechanical properties. The investigators also suggested that the hydrogen content in the wire steel was due to the galvanizing process.

Townsend and others have shown that a hot-dip galvanized steel is more prone to hydrogen embrittlement and stress corrosion than the same steel in its uncoated state (84). He attributed that behavior to the barrier effect of the intermetallic coating existing at the interface between zinc and steel. Such research has shown that cathodic-protective coatings decrease resistance to hydrogen-related fracture mechanisms. That would seemingly support the FHRS report; however, the report contained several inferences that bear comment.

The first of those inferences concerned the general acceptability of the Portsmouth Bridge second cable wires. It should be noted that by the time the Portsmouth Bridge cables were first replaced, in 1940, over half of the modern American suspension bridges had been constructed. Wire making had reached a high degree of sophistication. The 1930 specifications for the Maysville Bridge wire were furnished by Modjeski and Masters. Mechanical properties in those specifications are shown in Table 3. Since Modjeski and Masters also were the consultant on the Portsmouth Bridge first cable replacement operation, specifications for the second Portsmouth cables would not be expected to be less strict. The Maysville Bridge specifications required that ten coils of wire from each heat be mechanically tested.

The FHRS report did not state the number of wires mechanically tested nor the original surface condition of those wires. However, the wires tested possessed sufficient tensile strength (236 ksi). FHRS values for elongation and reduction in area, measures of material ductility, were slightly below the specified Maysville Bridge values. However, the FHRS tensile tests employed an extremely small gage length (1.5 inches) compared to 10 inches as specified by Modjeski and Masters. That should readily account for the slight difference between the FHRS elongation (3 percent) and the Modjeski and Masters specified elongation (5 percent). The difference between the FHRS reduction in area (28 percent) and the Modjeski and Masters specified value (30 percent), though small, bears comment.

During the manufacture of the Portsmouth Bridge second cable wires, a problem was encountered in getting the first lot of galvanized wire to meet the reduction in area specifications (85). Smaller wires, on the average, were closer to specification requirements than were larger wires. Experiments showed the lead-bath and zinc-tank temperatures could be adjusted to meet the ductility criteria, but only at the expense of wire strength. Rather than accept that, the parties involved chose to accept a lower reduction in area. Specifications were changed to allow a reduction in area of 25 percent, with the added stipulation that every coil was to be inspected.

Tests indicated about five percent of the 0.198-inch diameter wire was rejected because of low ductility. About nine percent of the 0.205-inch diameter wire was rejected due to cuppiness (segregation).

That information indicated the wire, even though not within specification requirements, was not seriously deficient. The 30 percent reduction in area is, at best, a subjective criterion. The increased frequency of testing probably provided sufficient assurance of the suitability of the wire. That explains, in part, the mechanical properties of the wire determined by FHRS.

The FHRS chemical analysis of the wire reveals the Portsmouth wire was well within the limits of Modjeski and Masters specifications. The

TABLE 3. MECHANICAL AND PHYSICAL TEST PROPERTIES AND SPECIFICATIONS FOR BRIDGE WIRES

	MAYSVILLE BRIDGE[1]	PORTSMOUTH BRIDGE (ORIGINAL)[2a]	PORTSMOUTH BRIDGE (REPLACEMENT)	MODERN WIRE[3]
Ultimate Tensile Strength (ksi)	220 Min	242-265	220 Min	217[a]
Yield Point (ksi)	170 Min	167-185	170 Min	161[a]
Elongation (%)	5 (10-in. ga) Min	0.5-1.7	5 (10-in. ga)	10 (2-in. ga)[a]
Reduction in Area (%)	30 Min	19.3-47.0	30 Min	N/A
Mandrel Bend Size	1-1/2 Diameters[1]	N/A	N/A	1-1/2 Diameters[b]
Standard Preece Tests	4 One-Minute Immersions	N/A	N/A	5 One-Minute Immersions[b]

1. Specifications for Maysville Bridge (Ref 88, p 90)
2. (a) Laboratory data (Ref 67, p 483); (b) Specifications for Portsmouth Bridge (Ref 68, p 12)
3. (a) Laboratory data (Ref 72, p 49); (b) U.S. Steel Corporation, *The Making, Shaping and Treating of Steel*, Herbick and Held, Pittsburgh, PA, 1971, p 871.

N/A - Not Available

carbon content, as measured by FHRS, was 0.83 percent compared to 0.85 percent maximum specified by Modjeski and Masters. The FHRS sulfur and phosphorous analyses were very low, 0.022 and 0.012 percent, respectively. That was below specified maximums of 0.04 percent for both elements. Those two elements are especially important as they can act as cathodic poisons in stress-corrosion problems.

The second issue to be addressed is the hydrogen content of the wires. The FHRS-reported hydrogen content of 0.6-0.7 ppm in the wire is not considered a sign of defective (brittle) wire in either construction or industry practices. Townsend (86) mentioned that commercial steels generally contain 0.5 to 5.0 ppm of hydrogen without exhibiting hydrogen embrittlement. The damaging effect of hydrogen is not only related to its quantity but also to the steel microstructure and the disposition of the hydrogen in the material. Wires mechanically tested by FHRS did not show sufficient differences in ductility from normal specifications to indicate the Portsmouth wires were embrittled by hydrogen. Tests on ungalvanized outgassed wire should have provided some insight into the influence of hydrogen on those wires; however, present evidence does not strongly support any contention of embrittlement due to the presence of excessive manufacture-entrained hydrogen.

Use of hydrogen inhibitors, related to pickling of steel, dates back to at least the 1930's (87). Perhaps, due to problems with the Ambassador and Mt. Hope bridges, engineers of that period became aware of hydrogen-related problems that might arise in metal-cleaning operations prior to the galvanizing process. Those concerns were reflected in subsequent wire specifications (88) such as ones for the Maysville Bridge, which stipulated:

"295(c) After patenting, the rods shall be cleaned of all scale by immersion in diluted hydrochloric acid. Sulphuric acid shall not be used. Any coil or bundle in which defects are revealed by the pickling shall be discarded. All acid shall be removed from the rods by thorough and vigorous rinsing with fresh water and the coils shall be dipped into lime-water to neutralize any remaining traces of acid. The rods shall be baked for several hours to remove atomic hydrogen dissolved in the metal during pickling.

297(b) Hydrochloric acid shall be used for the pickling solution preceding the galvanizing. No sulphuric acid shall be used in any pickling process in the production of the wire. The pickling cycle shall include the proper use of inhibitors and approved means for eliminating retained acid and atomic hydrogen before galvanizing...."

Another possible inference from the FHRS report was that galvanized steel wire was prone to hydrogen-embrittlement. The aspects of retained-hydrogen problems in galvanized steel and possible avoidance measures were well covered by Townsend (89). Hydrogen retained from the hot-dip galvanizing process will not significantly contribute to cracking processes unless zinc corrosion charges hydrogen into the crack tip. No proof exists to support the idea that retained hydrogen is a major problem in wires, including the ones used on the Portsmouth Bridge. Horvick (90) notes there have been very few instances of hydrogen problems related to hot-dip galvanizing. Problems associated with electroplating-related hydrogen have also been well researched

(91).

On both suspension bridges (Ambassador and Mt. Hope) having heat-treated steel, wire fractures were detected shortly after erection. Those broken wires did not show signs of corrosion. If the Portsmouth Bridge second cable wires suffered from embrittlement due to manufacture-induced hydrogen, they would have failed early and would have been detected during installation. Such failures would likely occur at the plant, during the stranding operation, or in the field prior to cable wrapping.

To the authors' knowledge, broken wires encountered during the Portsmouth Bridge second corrosion problem were detected only at locations that had experienced corrosion of the zinc coating and, at least, limited corrosion of the underlying steel. If the wires had suffered fractures due to the galvanizing process, broken wires would have been detected in locations where no corrosion was evident. That would be especially true just prior to the recabling operation, after the cables had experienced their greatest deterioration and highest dead loading. None of the wires contained surface defects, such as checking or tearing, associated with retained hydrogen. Also, none of the wire-fracture surfaces examined exhibited signs of flaking.

The amount of hydrogen detected on surfaces of the as-received FHRS wire was not high for a corrosion product. That hydrogen was probably the result of cathodic reduction of hydrogen ions during the corrosion process. The presence of sulfur as a cathodic poison also may have promoted the entry of hydrogen atoms into the wire surface, in the event zinc coating was completely depleted in a given area. However, that type reaction may be more likely in a pit or crack.

Unfortunately, the FHRS report did not reference field locations of the wire specimens. As previously mentioned, corrosion activity differed between various portions of cable. The quantity of hydrogen in wires from different locations on the bridge would probably vary if corrosion were the source.

Burke's (92) report contained several valuable insights relative to the corrosion condition of the Portsmouth Bridge prior to the second recabling operation. His work noted difficulties in resolving results of previous reports relating events at Portsmouth. The Burke report contained several items that warrant discussion.

One concern is the fact the Ohio Department of Transportation report tacitly assumed the Portsmouth Bridge second corrosion problem was caused by stress corrosion. That could lead to the conclusion that an unusual corrosion condition existed in the Portsmouth area, which could presumably be either a specific corrodant or a severe corrosion condition due to typical atmospheric pollutants.

The first issue may be dealt with by the fact (noted by Burke) that the only stress corrosion-related element detected on the cables was sulfur. The relative abundance of that element in industrial atmospheres precludes the assumption of an unusual specific corrodant. Also, unusual historical sources of sulfuric acids in the immediate area of the bridge were not located.

The second issue is more difficult to dismiss. Admittedly, the Portsmouth area was, at one time, a heavily industrialized area (though much of that industry was downwind from the bridge). From about 20 years prior to the most recent closure of the bridge, the economy of area had changed. As a result, the atmosphere in and around the

Portsmouth Bridge was well within federal pollution standards and was similar to conditions in the Maysville area by the early 1970's.

Burke observed rapid uniform corrosion of recently sandblasted wire on the bridge when subjected to rain. However, contrary to statements about acid rain, that is not a sign of severe atmospheric corrosivity. Freshly blast-cleaned steel will rust quickly and copiously when exposed to moisture, regardless of whether the moisture is or is not corrodant-laden.

There were many more important factors contributing to the Portsmouth Bridge problem. Some are contained in the Burke report. Several facts tend to downplay the effect of atmospheric severity in any corrosion-cracking process that may have occurred at Portsmouth. First, research has shown that for stress-corrosion problems, the pH of a corrodant in a crack or pit may be very low, even when the pH in the bulk of the corrodant in contact with metal is nearly neutral. Also, very little sulfur is required to act as a cathodic poison in hydrogen-related stress-corrosion cracking. In fact, too much sulfur may prove detrimental to the process under some conditions. Within ordinary limits, the effect of concentration of atmospheric pollutants may not have an accelerating effect on the actual cracking process. However, it may affect durability of the galvanized coating that protects the wire from the onset of corrosion-cracking processes.

The report referenced one observer's comment that the number of breaks was roughly proportional to the amount of observable surface corrosion. That is only true in areas where flowing water (washing) did not occur and (or) stagnant water remained in contact with the wires over a long period. On steeply inclined portions of the main span and the Ohio side span, cables suffered much more severe uniform corrosion than the nearly horizontal portions. However, in the nearly horizontal portions of the cables, where water could stand, the number of wire breaks was much greater and the observer's comments were correct. A more revealing statement would be that breaks occurred only in areas experiencing at least some uniform ferrous corrosion on the wire.

The National Physical Laboratory stated that hydrogen-embrittlement problems arise from the uniform corrosion of wire. The NPL authorities also felt that grouting around wires would preclude the problem. In view of the problems with Roebling's grout-filled anchorages, some doubt should be cast as to suitability of grouting for use as a wire-protection procedure.

POTENTIAL CORROSION PROBLEMS

Oftentimes, failure analyses, especially those involving corrosion cracking, suffer from variances in terminology or sometimes lack of complete agreement by researchers about basic fundamentals. For instance, some researchers term loss of ductility due to the presence of molecular hydrogen (not connected with corrosion problems) as "hydrogen embrittlement". Others have broadly ascribed that term to both cracking and loss of ductility, regardless of whether atomic or molecular hydrogen was involved or whether or not an electrochemical reaction was involved. Some researchers have called corrosion-cracking problems involving hydrogen "stress-corrosion cracking" (SCC); others use more restrictive terms such as "hydrogen-assisted cracking" or "hydrogen stress cracking". Also, apparently there has not always been complete agreement about the exact role of hydrogen in corrosion cracking; nor, has there always been complete agreement about the salient features of the various forms of corrosion cracking, many of which may involve hydrogen to some extent.

To investigators of corrosion cracking who are not directly involved in basic corrosion research, those difficulties should temper any pronouncements concerning mechanisms of failure. In coping with corrosion cracking, contributory rather than mechanistic factors may eventually prove of more value.

In the case of the Portsmouth Bridge, the following facts relate to the possible mechanisms of failure:

1. Considerable general corrosion occurred on the main cables.
2. At least some uniform ferrous corrosion of the wire was detected at each fracture site. (This was first observed by the Battelle investigators.)
3. The frequency of wire fractures was not necessarily proportional to the severity of uniform corrosion.
4. Some wire fractures were clustered about cable bands.
5. No fractures were detected between splay saddles and strand end fittings, areas where strands were exposed to the environment.
6. No atmospheric SCC-specific corrodants were identified in unusual concentrations at Portsmouth.
7. The only possible damaging elements identified on the Portsmouth wire in laboratory investigations were sulphur and hydrogen (by the Fairbank Highway Research Station and Battelle).
8. Portsmouth wire specimens exhibited several types of fracture morphologies.
9. The bridge cables are subject to both corrosion and cyclic loading (low frequency and low stress), a combination about which little known research exists.

As previously noted, all Portsmouth wire failures contained some superficial zinc and ferrous corrosion. Also, no fracture surfaces inspected showed evidence of flaking. That fact tended to rule out classic molecular (H_2) hydrogen embrittlement. Also, the frequency and stress range of cyclic loading probably precluded cracking due to pure fatigue.

A more likely cause was stress-corrosion cracking. Considering the subject metal (steel), the only laboratory-identified corroding elements (hydrogen and sulfur) and the general corrosion evident on the cables, the anodic-dissolution SCC mechanism is improbable. If SCC was the prime cause of failure, the stress-sorption SCC mechanism would be the most likely cause of failure. That type of corrosive attack could be termed "hydrogen-assisted cracking", "hydrogen-stress cracking", or "corrosion-generated hydrogen embrittlement".

Townsend (93) produced hydrogen-assisted stress-corrosion cracking in aqueous solutions saturated with hydrogen sulfide (H_2S) using bare cold-drawn wire similar to bridge wire. He noted that cracks formed by the SCC mechanism tended to be aligned 45 degrees to the direction of tensile stress. That type failure was produced in both uniaxial tension and bending.

Usually, stress-corrosion cracks occur normal to the principal tensile stress. Townsend felt the unusual fracture alignment in cold-drawn wire was due to the wire-forming process. Cold-drawn wire is known to have a <110> texture. The <110> direction of the B.C.C. iron lattice has a greater than random possibility of being aligned parallel to the wire axis. As a result, the #100[cleavage planes for B.C.C. iron have a greater than random probability of being situated at 45 degrees to the wire axis, which was also the direction of tensile stress. Therefore, brittle fractures would be expected to be oriented normal to the cleavage planes or at approximately 45 degrees to the wire axis.

To prove that, Townsend heat treated cold-drawn wire specimens to remove the effect of texture. After the heat-treated wires were subjected to stress-corrosion cracking, he noted the SCC fractures were oriented normal to the wire axis.

Interestingly, that phenomenon was also observed some 30 years earlier in Pollard´s report. SCC fractures, produced in specimens of cold-drawn Portsmouth Bridge wire´s, failed obliquely to the wire longitudinal axis. Similar fractures created in specimens of heat-treated Mt. Hope Bridge wire were normal to the axis. Also, Thul exhibited a cold-drawn galvanized bridge wire (used on a cable-stayed bridge) that also had a stress-corrosion crack oblique to the wire axis (94).

Two facts tend to mitigate the role of stress corrosion cracking in the Portsmouth Bridge second corrosion problem. Those are: results of stress-corrosion tests of bridge wire performed by Boeing and the differing fracture morphologies observed on the Portsmouth Bridge wires.

The Boeing experiments, using circumferentially notched galvanized bridge-wire specimens showed stress corrosion could not be induced in wires at high stresses (120 ksi) in 5 ppm H_2S aqueous solutions (a much higher concentration than normal atmospheres). The corrodant concentration used by Townsend (95) was considerably higher than that used by Boeing. In a corrosion reaction involving iron and H_2S, one principal reaction is

$$xFe + H_2S = Fe_xS + H_2.$$

When the zinc begins to corrode, it acts as an anode and donates electrons to the exposed steel at the notch (Figure 25). The surface of the exposed steel becomes cathodic and attracts atomic hydrogen to low energy locations in the iron lattice (with sulfur promoting this action). However, when the rate of hydrogen charging or the applied stresses are too low, the corrosion product Fe_xS may be deposited in sufficient thicknesses on the notch to act as a diffusion barrier. That will prevent further charging of hydrogen into the exposed steel. In effect, the reaction is auto-extinguishing. The Boeing investigators used acetic acid to suppress formation of a Fe_xS diffusion barrier; however, that may not have been effective.

There are several explanations of why the Boeing SCC tests may not be applicable to stress-corrosion cracking on the Portsmouth Bridge. During the service life of the bridge, zinc and ferrous corrosion products could trap and store corrodants, increasing their effective concentration at the metal-corrosion product interface. Also, the massive amount of general corrosion and presence of other forms of corrosive attack on the bridge indicated that corrosion polarization may

A. Corrosion of wire initiates at 1 with the reaction x Fe + $H_2S \rightarrow Fe_xS$ + Hz.

B. Zn coating acts as an anode at 2 and furnishes e^- to exposed Fe. This, in turn, cathodically charges the exposed Fe with H^+ that diffuses to low-energy locations in the Fe lattice at 3 .

C. However, Fe_xS forms a barrier to H^+ trying to penetrate into the notch, and the embrittling reaction stalls.

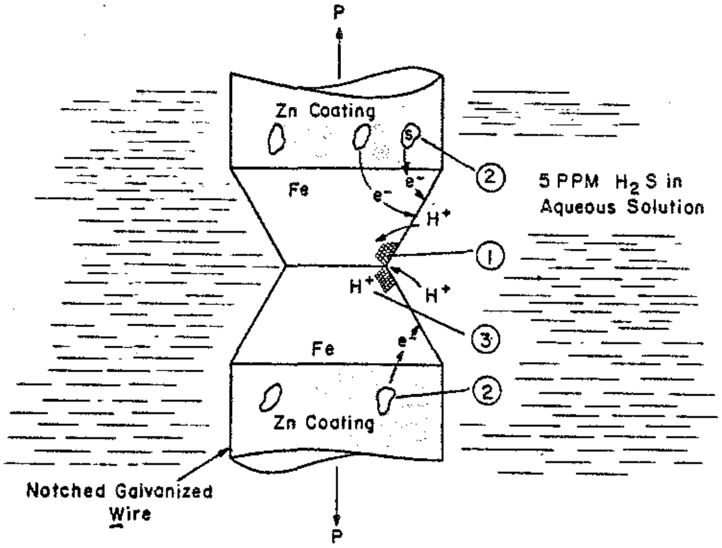


Figure 25. Hydrogen Charging at Notch due to Corrosion of Galvanic Coating.

have occurred. That would create high induced currents that could charge hydrogen into the wires and possibly suppress formation of the Fe_xS corrosion product.

Unfortunately, even those possibilities do not explain variances in fracture morphologies observed on the Portsmouth wire. None of the bridge wire fractures showed any signs of ductility (necking). Fracture types were 1) flat (transverse to the wire axis), 2) flat-and-invert (part of fracture transverse, the remainder oriented at an angle to the wire axis), or 3) spiralling (angled approximately 45 degrees to the wire axis. Most wire fractures observed at Portsmouth were of the flat-and-invert type. Those wires showed differing amounts of flat-and-invert fracture surfaces.

The best explanation of the first two fracture types appears to be corrosion fatigue. That phenomenon might also explain why the Boeing experiments failed to produce stress-corrosion cracking; yet under less severe conditions, corrosion cracking occurred at Portsmouth. As with plain fatigue, corrosion-fatigue cracks tend to be normal to the applied stress; or in this case, a transverse fracture perpendicular to the longitudinal axis of the wire.

Nield (96) observed such typical corrosion-fatigue cracks in a service failure of bridge prestressing wire. That wire had been exposed to water and the cracks had initiated in preferentially corroded zones close to the fractures. Multiple cracks were evident, most of which were mainly transverse.

Williams and Firth (97) described fatigue cracks on hoisting rope wires as transverse fractures exhibiting a sharp tip at the point of final fracture. However, those fractures had occurred at low nominal service loads. The authors were of the opinion those ropes had either been in extended service or had been abusively overloaded. The possibility of corrosion fatigue was not considered in the article.

Whitaker and Booker (98) produced corrosion fatigue cracks in mild, drawn sheet steel in distilled water. Those tests involved low-frequency loading and reverse bending. That work indicated aqueous contact may be an important factor where corrosion fatigue is involved, and also, participation or concentration of an attendant corrodant may not be as important as with stress-corrosion cracking.

The stress range of most suspension bridge cable cyclic loading is low. For the Maysville Bridge, the cable design live load was about 21 ksi, and it is doubtful the bridge is ever loaded that heavily. Larger suspension bridges may experience lower live loads than smaller bridges, especially in respect to their dead loads. Resultant stresses in the cables may be simple tension or combined tension and bending at the bands, depending upon rigidity of the stiffening truss (when one is present) and the cross section and curvature of the cables.

Suspension-bridge main cables are infrequently subjected to heavy live loads. Both the Maysville and Portsmouth bridges have heavy traffic intersections at the ends of the approaches. During peak traffic periods, the bridges are completely loaded, though usually with a mixture of cars and trucks. It is likely that most suspension bridges experience from three to less than one hundred major live-load applications per day.

Effects of wind loadings have been omitted in this report; however, that was covered at length in the Federal Highway Administration Report No. FHWA/RD-81/090, "Analytical Study for Fatigue of Highway Bridge

Cables."

Relatively small cyclic stress ranges and infrequency of significant load applications on main cables probably precluded normal fatigue problems. However, as Nield noted, the stress range required for corrosion fatigue to cause failure diminishes with time. Presence of a corrodant, even in low concentrations, may be sufficient, combined with low-range cyclic loads, to cause corrosion fatigue. While cylic loading of the Portsmouth Bridge was infrequent, peak loads were maintained for long periods (up to several hours). Also, many other corrosion mechanisms operant on the Portsmouth Bridge created a rough corrosion-product laden surface, which could facilitate corrosion-fatigue crack initiation, on the wires.

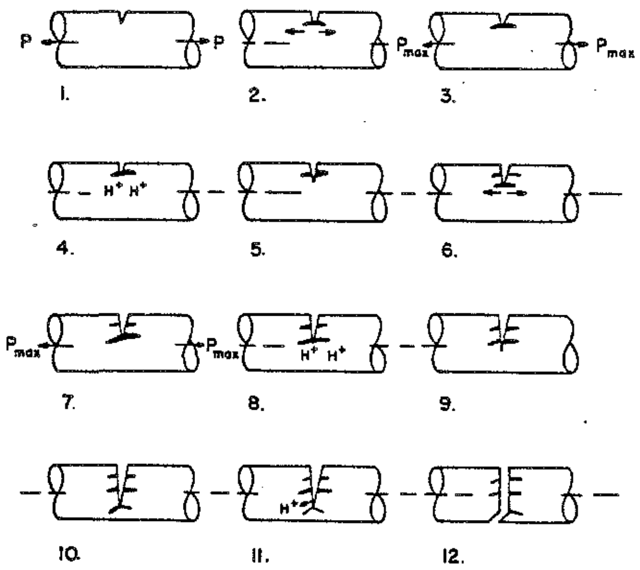
The type of corrosion-fatigue mechanism hypothesized for the Portsmouth Bridge wires initiates with formation of a crack on the wire surface (Figure 26). Cracks form on exterior surfaces of the wire on the outer layer of the strand. The crack initiation site may be a pit or possibly a localized rupture of the protective surface film caused by other corrosion processes. It should be noted that fatigue corrosion of steels in acids is not always related to pits. Crack arrests due to insufficient driving stresses and due to a corrosion-product barrier at the crack tip prevent extended corrosive attack at that location. Points along crack walls, which are exposed to the aqueous corrodant, may experience secondary dissolution along the elongated grain boundaries.

Application of another major cyclic load ruptures the brittle corrosion product (Fe_xS) or separates the corrosion product-metal interface. That allows the aqueous corrodant solution to contact the iron and initiate new corrosive attack. Due to the presence of sulfur (the cathodic poison) and cathodic currents caused by corroding zinc, hydrogen is charged into the iron and the crack extends into the wire. However, the depth of hydrogen damage may be limited; and the crack may rearrest due to the formation of another corrosion-product crack barrier at the extended crack tip.

The steps repeat upon further applications of other major cyclic loads. When the crack reaches a critical size, the stress intensity at the crack tip, K_{IX}, becomes greater than K_{ISCC}, the SCC critical stress intensity. Crack mechanism changes to the stress-corrosion cracking and the crack changes from a transverse to an oblique orientation. That process would be considerably shorter in duration than the preceeding corrosion-fatigue cracking.

The Portsmouth Bridge wire failures that exhibited flat-and-invert failures contained varying amounts of such fracture areas. That is probably related to variations of applied stresses acting on the wires. When dead-load stresses are low (i.e., few wires in the strand are broken or little cross section of the wire has been consumed by general corrosion), the corrosion-fatigue mechanism will control and most fractures will be flat and transverse to wire axis. When dead-load stresses are high (i.e., many wires in the strand are broken or a large loss of cross section has occurred), stress corrosion will control; and most fractures will be inverted (or spiralling).

The family of fractures evident on the Portsmouth Bridge wires represented a progression of failure of the strands and, eventually, the cables (Figure 27). Early failures were probably of the flat type and occurred at low dead-load stresses (where corrosion fatigue was the sole



1. Crack initiation and arrest at a point of H^+ damage.
2. Crack tendrils form as Fe_xS barrier forms.
3. Cylic load breaks barrier, corrodant contacts Fe.
4. Corrosion action charge Fe with H^+ at crack tip.
5. When sufficient H^+ is charged, $K_{IX} > K_{ISCC}$ and crack grows.
6. Crack stalls beyond area of H^+ damage and Fe_xS barrier forms.
7. Cylic load again breaks barrier.
8. Repeated H^+ charging into Fe.
9. Repeated crack growth in H^+ damaged area.
10. When the crack has grown to a sufficient size, $K_{IX} \gg K_{ISCC}$. The fracture mechanism changes. Crack reorients along texture plane.
11. Crack has opened sufficiently that no cyclic effects are required for wire fracture.
12. The wire fractures with the resulting flat and invert fracture surface.

Figure 26. Proposed Corrosion-Fatigue Mechanism for Wire Cracking.

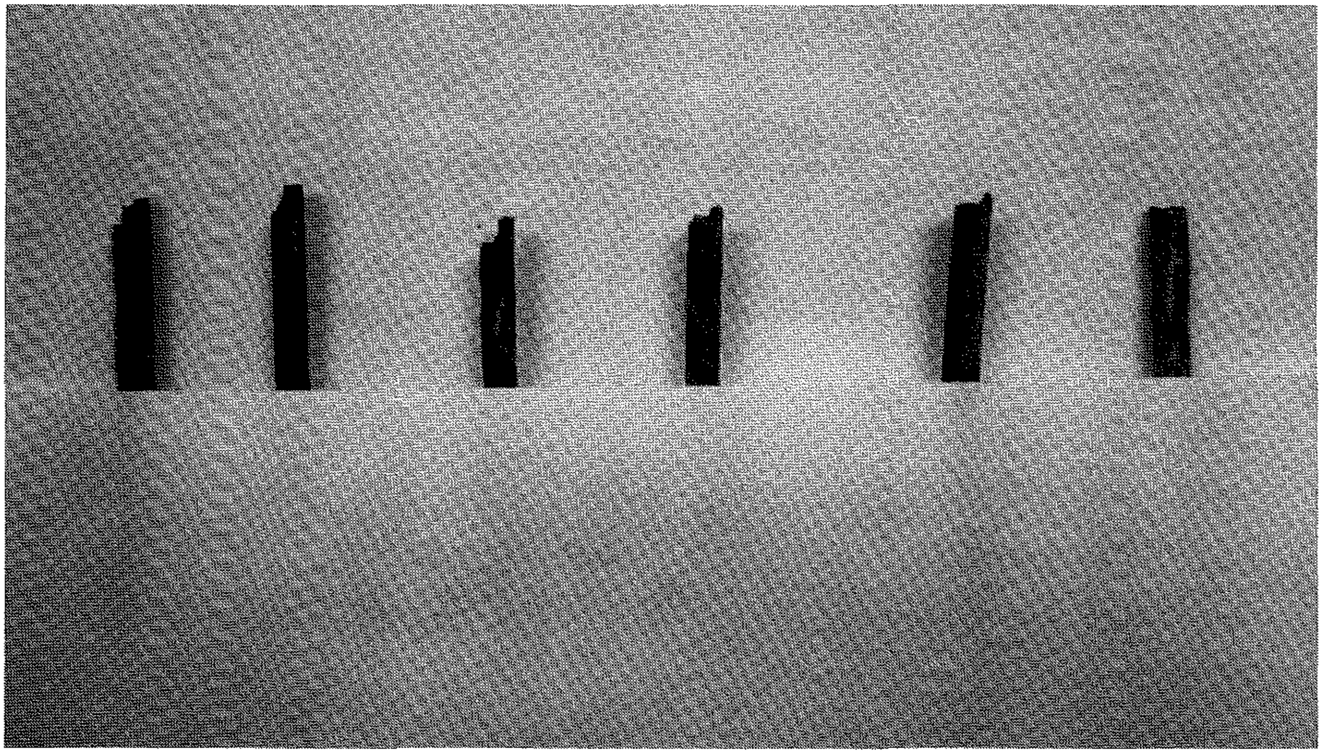


Figure 27. Progression-of-Fracture Using Wire Fractures from the Portsmouth Bridge Second Cables.

fracture process). As corrosion damage and dead-load stresses increased, fracture morphology changed from flat to flat-and-invert with increasing amounts of invert fracture surfaces. The most advanced failures were the spiralling-type at high stresses and are entirely SCC failures.

Corrosion-fatigue cracking and SCC may coexist as competing mechanisms in the same fracture. For example, a crack may initiate by SCC and propagate by corrosion-fatigue cracking. Conversely, corrosion fatigue may initiate a crack that will grow by SCC. Also, both processes may alternate during the crack-growth process. That would make failure analysis very difficult. Corrosion fatigue does not have to be an exclusive cracking mechanism.

Evidence to support the role of corrosion fatigue in the Portsmouth Bridge second corrosion problem is, at best, circumstantial. Research should be conducted to verify the effect of corrosion fatigue on bridge wire.

It also should be noted that strands failed from the outside to inside as did the wires. Residual stresses from the stranding operation caused broken wires to fray outward. The invert portions of wire fractures lay on the inner portions of wires. Therefore, mechanical notches caused by fretting did not contribute to fracture processes, as in plain fatigue. However, fretting probably contributed to the problem by facilitating other forms of corrosion.

Galambos (99) noted that German bridge engineers considered hydrogen embrittlement, not stress corrosion, the source of wire problems, which were associated with rough handling of wires and common corrosion-protection practices (probably meaning galvanized wires).

Thul's 1972 article on cable-stayed bridges in Der Stahlbau referenced several wire corrosion problems experienced on unnamed German bridges (100). He noted that bare wire succumbed to loss of section due to gross corrosion, while galvanized wires suffered cracks. Thul stated that cracks in galvanized wires occurred at tower mounting points. Also, he found that helical-type strand was not any more corrosion-prone than parallel-wire strand. Thul's report contained pictures of galvanized wires that had been subject to corrosion cracking. The failure(s) appeared to be of stress-corrosion cracking or hydrogen-assisted cracking type (spiralling).

One of the photographs exhibited a series of rough parallel marks on the surface of the uncorroded galvanized section. Thul termed that a "crocodile" appearance ("Krokodilhautartige"). Actually, the wire appeared to be subjected to checking that could be caused by hydrogen embrittlement (due possibly to defective wire or improper galvanizing or by the intermetallic layer cracking under the zinc coating). Townsend noted that hot-dip galvanizing of steel yields lower K_{ISCC} values than plain steel or steel galvanized electrically. Corrosion of galvanized wire investigated by Thul may have involved wire of lower quality than employed in America. Examinations of a large amount of galvanized wire in Kentucky have never shown "crocodile" type defects.

CLOSURE

Hypotheses and conclusions presented in this report concerning corrosion mechanisms are, at best, tentative and require further laboratory testing for confirmation. One corrosion-cracking mechanism should not be expounded as the sole contributor to past suspension bridge cable-wire fractures. Corrosion fatigue, however, possesses many relevant traits detected on the Portsmouth Bridge second cable wires and referenced in pertinent literature. Also, several forms of corrosion cracking may have interacted during the course of wire cracking, making it impossible to duplicate in laboratory failures using a single corrosion-cracking mechanism. The contribution of other forms of corrosion to the fracture process should not be disregarded. If those of mechanisms had not been active, the wires would not have failed.

At the present, it would be sufficient to state that long-term moisture contact with suspension bridge wire should be avoided. Failure of the Portsmouth Bridge second cables was not caused by unusual atmospheric problems in the area (though sulfur was probably present in typical industrial concentrations). Contributory factors were probably the design, construction, and maintainability of the wire corrosion-protection system, which allowed significant atmospheric moisture to be in long-term contact with the wires.

Atmospheric environments at Covington and Maysville can be as conducive to suspension bridge wire corrosion as the environment at Portsmouth. Events leading to closure of the Portsmouth Bridge indicated that, once a given level of wire degradation is reached, fractures can be expected in a surprisingly short time. Requirements for wire corrosion cracking appear to be 1) Stage 3 wire condition, 2) long-term contact of atmospheric moisture with the wire, and 3) normally anticipated bridge loadings (including occasional overloads). As the bulk of wire on both of the Kentucky-owned bridges (Maysville Bridge and Ohio River Bridge) is probably in better condition than Stage 3 deterioration, the main concern with those bridges is the performance of the cable corrosion-protection systems.

RECOMMENDATIONS

The following recommendations pertain to the two major Kentucky-owned suspension bridges.

1. Inspection ports should be installed on the Ohio Bridge (KY 17) at Covington.

2. Monitoring of the Maysville Bridge (US 68) inspection ports should continue.

3. Further research related specifically to bridge wires should be encouraged. That work should be in the areas of corrosion failure, wire-fracture susceptibility in various atmospheric environments, repair of corroded cables, prevention of corrosive attack on existing bridges, and the formulation of improved corrosion-prevention systems.

4. Inspection of the two Kentucky-owned suspension bridges, followed by the formulation of designs for modifications to those bridges to preclude or arrest early corrosion problems, should be considered. Those modifications should be implemented as soon as rehabilitation funding becomes available. Also, a maintenance manual

for each of these bridges should be prepared.

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