

NATIONAL COOPERATIVE
HIGHWAY RESEARCH PROGRAM REPORT

272

PERFORMANCE OF WEATHERING STEEL IN BRIDGES

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
REPORT

272

PERFORMANCE OF WEATHERING STEEL IN BRIDGES

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RESEARCH SPONSORED BY THE AMERICAN
ASSOCIATION OF STATE HIGHWAY AND
TRANSPORTATION OFFICIALS IN COOPERATION
WITH THE FEDERAL HIGHWAY ADMINISTRATION

AREAS OF INTEREST:

STRUCTURES DESIGN AND PERFORMANCE
CONSTRUCTION
GENERAL MATERIALS
MAINTENANCE
(HIGHWAY TRANSPORTATION)
(PUBLIC TRANSIT)
(RAIL TRANSPORTATION)

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WASHINGTON, D.C.

JULY 1984

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

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The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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The members of the technical committee selected to monitor this project and to review this report were chosen for recognized scholarly competence and with due consideration for the balance of disciplines appropriate to the project. The opinions and conclusions expressed or implied are those of the research agency that performed the research, and, while they have been accepted as appropriate by the technical committee, they are not necessarily those of the Transportation Research Board, the National Research Council, the American Association of State Highway and Transportation officials, or the Federal Highway Administration, U.S. Department of Transportation.

Each report is reviewed and accepted for publication by the technical committee according to procedures established and monitored by the Transportation Research Board Executive Committee and the Governing Board of the National Research Council.

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FOREWORD

*By Staff
Transportation
Research Board*

This report contains the findings of a comprehensive assessment of the performance of weathering steel in bridges based on a review of the literature, a survey of practice in highway agencies and other organizations, and contact with selected individuals knowledgeable on the subject. The report should be of value to structural engineers and others interested in the design, construction, and maintenance of steel bridges.

Weathering steel has been used in the construction of more than 2,000 bridges in the United States. Under the proper conditions, this material is expected to form its own protective oxide coating and to require no painting. It, therefore, offers the potential for considerable savings in life-cycle costs by elimination of the need for painting bridges, particularly bridges over major highways, electrified railways, or bodies of water.

Under some environmental conditions, corrosion in weathering steel bridges has continued at a rate more rapid than anticipated, and there are concerns regarding the long-term performance of this material. Hence, the use of weathering steel in bridges has been more limited than might be expected in view of the potential cost savings. Information on the performance of weathering steel continues to accumulate, and there is a need to assemble and evaluate the data in order to facilitate the decision-making process and place it on a more rational basis.

The objectives of NCHRP Project 10-22, "The Performance of Weathering Steel in Bridges," are (1) to assemble a systematic body of information on the performance of weathering steel, (2) to document and evaluate the current state of practice, and (3) to develop, where feasible, practical guidelines for design, construction maintenance, and rehabilitation of bridges using this material.

The findings of the first phase of research are presented in this report and relate primarily to the first two objectives. In Phase I, problems that have developed in bridges were identified, and other factors that have limited the use of weathering steel in the United States and abroad were analyzed. The topics considered included: corrosion, salt exposure, other environmental factors, pitting, fatigue, in-service inspectability, location within the bridge, structural details, shims and bearings, bolted connections, welding, adequacy of specifications, alloy content of the steel, initial cleaning and painting, remedial cleaning and painting, and adverse effects of corrosion on fatigue life. In addition, existing experimental and field performance data were assembled, and the current state of practice in regard to the use of weathering steel in bridges was evaluated.

Some progress was made in Phase I on the third objective, but more research is needed. A second phase of research, initiated in August 1984, has as its specific objectives (1) to fatigue test 8-year weathered A588 transverse stiffener specimens under both constant and variable amplitude loading in both air and aqueous environments, and (2) to develop practical guidelines for design, construction, maintenance, and rehabilitation of weathering steel bridges.

The findings contained in this report will be of interest to practicing engineers; however, the major recommendations based on both phases of research will be presented in the final report at the conclusion of Phase II. The information acquired in the first phase of research will serve, in part, as the basis for developing the Phase II guidelines, which are expected to be available in the form of specific recommendations to the AASHTO Subcommittee on Bridges and Structures by the spring of 1986.

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Snehal Munshi, Senior Vice President of Sheladia Associates, Inc., is the Coordinator and Project Manager on the project. Seymour K. Curn, Corrosion Engineer, retired, United States Steel Corp., and now

President of Corrosion Consultants, Inc., supplied the historical background on the development of the weathering steels, checked the accuracy of the exposure data, reviewed the discussions, and assisted in some of the interpretations. Richard W. Vanderbeck, Engineering Materials and Processes, Inc., contributed to the material on strengthening mechanisms and toughness. Kentaro Yamada, Associate Professor of Civil Engineering, Nagoya University, Japan, provided the Japanese experience and the bridge examples.

PERFORMANCE OF WEATHERING STEEL IN BRIDGES

SUMMARY

Weathering steel has been used nationwide in the construction of bridges. When the structural details are properly designed and the bridge is exposed in a suitable environment, weathering steel forms a protective oxide coating and should require no painting. Under these conditions, it offers the potential for considerable savings in life-cycle costs by the elimination of the need for maintenance painting. Under some environmental conditions corrosion has continued at a rate more rapid than anticipated, and there is concern about the long-term performance of this material. Hence, the expanded use of weathering steel in bridges has been limited despite the potential cost savings.

The objectives of this research are to: (1) assemble a systematic body of information on the performance of weathering steel to date; (2) document and evaluate the current state of practice; and (3) recommend where feasible practical guidelines for design, construction, maintenance, and rehabilitation of weathering steel bridges.

The findings presented in this report relate primarily to the first two objectives. Limited progress was made on the third objective, and a second phase of research, initiated in August 1984, has as its specific objectives (1) to fatigue test 8-year weathered A588 transverse stiffener specimens under constant and variable amplitude loading in air and aqueous environments, and (2) to develop practical guidelines for design, construction, maintenance, and rehabilitation of weathering steel bridges.

The information acquired in the first phase of research will serve as the basis for developing the Phase II guidelines, which are expected to be in the form of specific recommendations to the AASHTO Subcommittee on Bridges and Structures.

History

The weathering steels evolved in the 1930's from a steel base that was alloyed with copper. The first major application came in 1933, in the form of exterior painted coal hopper cars, and later in the decade, in electrified railway and trolley cars. In 1948, a nonpainted weathering steel member was installed in a galvanized transmission tower. This first application was followed by the construction of unpainted transmission towers in several states.

The first bridges and buildings were built from weathering steel in the mid-1960's. The first bridges were: on Iowa 28 over Racoon River; on Eight-Mile Road over U.S. 10, Detroit, Michigan; over Brush Creek, Dayton Ohio; and in Morristown, as an adjunct to the New Jersey Turnpike. The first two bridges are now, 20 years later, scheduled for remedial painting. More information on the history of weathering steels can be found in Chapter One of the report, under "Historical Development."

Specifications

The three weathering steel specifications ASTM A242, A588, and A709 were first issued in 1941, 1968, and 1974, respectively. The A242 Type 1 steels were mainly used for architectural applications. The low-phosphorus version, A242 Type 2 steel,

corresponds to the A588 and A709 Grade 50W steels. The A588 steels come in nine proprietary grades, each with a variation of the same basic chemical composition. A709 is a general specification for bridge steels. The two grades with enhanced atmospheric corrosion resistance, 50W and 100W, correspond to A588 steel and to a modified A514 alloy steel.

The weathering steels come in yield strengths of 50 to 100 ksi (345 and 690 MPa). According to the ASTM specifications, they have an atmospheric corrosion resistance of approximately two times that of structural carbon steel with copper. This corresponds to four times that of structural carbon steel without copper (0.02 percent max. Cu). Matching high-strength bolts and weld rod are available in chemical compositions that give the connectors the same corrosion resistance and weathered appearance as those of the base metal.

Few changes were made over the years in the weathering steel specifications. The most notable occurred in 1975, when the corrosion resistance of A242 steel was lowered from "at least" to "approximately" four times that of carbon structural steel without copper. This change aligned the corrosion resistance requirements for the A242 steel with those for the A588 and A709 steels (Chapter One, sections under "ASTM A588 Steels" and "ASTM A709 Steels").

The foreign weathering steel compositions have similar tensile properties and concentrations of those alloying elements that are added mainly to achieve corrosion resistance (Chapter One, "Foreign Weathering Steels").

Domestic Experience

There are about 1,800 nonpainted weathering steel bridges on the federal-aid highway system, excluding state, county, city, tollroad, and mass transit bridges not on the federal-aid system. Weathering steel bridges were built in 43 states and the District of Columbia. Of those, 31 states and the District foresee using weathering steel for new bridges, 2 states are reconsidering their position and 10 states have stopped using it in a nonpainted condition. Many states carefully select the sites, avoiding severely polluted industrial environments, high humidity and rainfall areas, heavily salted highways and marine environments along the coast and waterways. For aesthetic reasons, Illinois, Missouri, and Wisconsin employ weathering steel mainly for stream crossings that are out of sight; Wisconsin uses it for grade separations in remote areas. Other than the requirement by some states to paint the steel structure up to 10 ft (3 m) on either side of deck joints, the design and detailing practice for nonpainted weathering steel bridges in the United States is the same as that for painted steel bridges.

Of the 10 states that have stopped using weathering steel, Alabama, Indiana, Iowa, Michigan, Mississippi, and West Virginia have done so because contamination with highway deicing salts is accelerating the corrosion process. Alaska discontinued the use because of the cost of steel relative to concrete in that state, and because of the very humid environment along its southeastern coast. The long life of painting systems in the dry climate of Montana makes painted steel construction more economical than bare weathering steel construction. In North Dakota, concrete bridges are often more economical than steel bridges.

Seven states never used bare weathering steel. Their main reasons are as follows. Arizona, Nevada, and New Mexico have dry climates in which painting systems last indefinitely, and weathering steels would not for a long time develop an oxide coating of uniform appearance. Steel is expensive in Hawaii, relative to concrete. New Jersey and South Dakota question the corrosion performance, particularly in salt environments, and dislike the rust staining of the concrete substructure. South Carolina has not used weathering steel because of problems reported by others.

Many steel users, including those who have rejected bare steel construction, have specified for economic reasons painted A588 steel either outright or as substitute for A572 in thicknesses greater than 2 in. (50 mm). This is done when the savings resulting from reduced weight, in comparison to A36 steel, exceeds the premium paid for high-strength low-alloy steel.

The majority of bare weathering steel bridges built in the United States are performing in a satisfactory manner. But the application of weathering steel has not been without problems. In 1980, the Michigan Department of Transportation, following an extensive evaluation of their bridges, banned all uses of nonpainted A588 steel on the state highway system. They had found that traffic-sprayed runoff water contaminated with salt was severely corroding the bridges in rural and urban areas.

In 1982, under the auspices of the AISI, a task group inspected 49 weathering steel bridges in Illinois, Maryland, Michigan, New York, North Carolina, and Wisconsin and on the New Jersey Turnpike. They found that 30 percent of the bridges showed good performance in all areas; 58 percent exhibited moderate corrosion in some areas; and 12 percent showed heavy corrosion in some areas. Most bridges, the report concluded, did not need immediate attention or overall painting. Notable exceptions are most of the structures inspected in the Detroit, Michigan, area.

The corrosion problems arising from unsuitable applications of weathering steel are not limited to Michigan alone. There are known cases of bridges in Alaska, California, Iowa, Louisiana, Ohio, and Texas that have not performed to expectation. These bridges are not developing a protective oxide coating for two reasons: (1) salt contamination from any source; and (2) prolonged time of wetness. These bridges were not included in the AISI survey. Eight bridges in Ohio and one in Louisiana were remedially painted in 1983. Many others in Alaska, California, Iowa, and Michigan are scheduled for cleaning and remedial painting.

The domestic use of weathering steel and the experience with this material is described in Chapter Two, under "Domestic Use," and under, "Domestic Experience."

Foreign Experience

The experience with weathering steel construction in Europe is reflected in the specifications such as those issued by France, Federal Republic of Germany, German Democratic Republic, and Czechoslovakia. All specifications caution against the deleterious effect that air pollution, prolonged time of wetness, and salt contamination have on the corrosion performance. Several specifications describe the environments unsuitable for nonpainted weathering steel.

In 1979, prior to Michigan's action, the West German Department of Transportation virtually banned the use of weathering steel for bridges on the federal system. Only two exceptions were granted; both were for composite girder bridges over electrified railways to avoid having to restrict the railway traffic during maintenance painting had the bridges been built of structural carbon steel.

According to a survey conducted by the Japanese Society of Civil Engineers, 115 weathering steel bridges had been built in Japan by March 1979. Of those only 5 bridges were of nonpainted steel. The motive for using weathering steel prior to that date was mainly to prolong the life expectancy of the paint in industrial environments and coastal areas with relatively high humidity. The trend in Japan is shifting to bare steel construction, with some bridges receiving a sacrificial paint-like "weather coat" that helps the steel to form a protective oxide and prevents runoff water from staining the concrete substructure.

Japanese steel companies have built and operated within their plants weathering steel bridges with painted, weather-coated and nonpainted sections. This allows them to closely monitor and compare the performance of the three systems. On the basis

of the experience with these bridges, structural details for new bridges are being modified in ways that enhance drainage and inhibit accumulation of debris. These details are being implemented in new construction.

The higher latitude and the generally higher levels of atmospheric pollutants cause the weathering steel to corrode more in the central and northern European countries than in the United States. The lower angle of the sun reduces the intensity of the drying cycle and prolongs the time of wetness. The Japanese use weathering steel despite having extensive marine exposures with high relative humidity and much rain. They try to overcome the problem by careful design (Chapter Eleven, "Water and Accumulation") and selective use of a porous protective coating (Chapter Nine, "Rust Stabilizing Treatment").

A survey of weathering steel bridges in the Province of Ontario confirmed the observations of others, but gives more information on conditions existing within box beams.

More information on the foreign experience can be found in Chapter Two, under "European Experience" and "Japanese Experience, and in Chapter Eleven, under "Recommendations from Ontario Study."

Corrosion Mechanisms

The corrosion of metals proceeds by an electrochemical mechanism involving oxidation, reduction, an anode (corroding metal), a cathode (existing rust), and an electrical current through the electrolyte (water film). An oxidation reduction front separates the rust coating into an inner layer where rust is electrochemically reduced and an outer layer where magnetite is chemically oxidized to rust. Marked differences between the atmospheric corrosion behavior of structural carbon steel and weathering steel begin to show up only after a full rust coating has developed.

Weathering steels derive their enhanced atmospheric corrosion resistance from the inclusion of 2 percent, or less, of such common alloying elements as copper, phosphorus, chromium, nickel, and silicon. On the basis of weight loss studies of steels with systematic variations of those elements, the following was found after 15.5 years of ideal and bold exposure in rural, industrial, and marine environments:

1. Increasing the copper content of a steel from 0.01 percent to 0.04 percent increased the corrosion resistance more than the addition of any other element.
2. Further increases in copper were beneficial, but not to so great an extent as shown by the initial addition.
3. Relatively small single additions of nickel, silicon, or phosphorus improved the corrosion resistance, but the largest improvements were obtained by specific combinations of these elements.
4. The beneficial effects of several alloying elements were not additive.

Two mechanisms appear to explain the beneficial effects of these elements. One attributes the corrosion resistance to the precipitation of basic compounds of the alloying elements. The other has the alloying elements catalyze the precipitation of amorphous ferric oxyhydroxide. Both have several features in common: two-layer structure of the rust coating, electrochemical corrosion cycle, enrichment of alloying elements in the inner rust layer and basicity of the rust. Under prolonged periods of wetness and contamination with salt, a protective oxide coating does not form, and weathering steel corrodes at a rate comparable to that of structural carbon steel.

Predictive equations fitted to extensive exposure data indicate that, even if the concentration of each element (Cu, P, Cr, Ni, and Si) in any grade of weathering steel were kept at the lowest level specified by ASTM, which is unlikely, these boldly

exposed steels should still exhibit a 15.5-year corrosion penetration of not more than 6.4 mils (163 μm) in the rural, 7.1 mils (180 μm) in the industrial and 9.3 mils (236 μm) in the marine environments tested. The penetrations reported for some severely corroding weathering steel bridges exceed these values much earlier, some after one year. This degree of corrosion cannot be explained in terms of a lean composition. It is caused primarily by a misapplication of the steel in environments different from the desired bold exposure.

The corrosion mechanisms are presented in detail in Chapter Three.

Atmospheric Corrosion

The corrosion resistance of a steel is typically determined from multiyear exposures of standard size coupons mounted on racks facing south at a 30-deg angle from the horizontal. The average corrosion penetration per side is calculated from the weight loss of specimen after the oxide layer is chemically removed. The macroenvironments at the exposure sites are broadly classified as being rural, industrial, and marine. Urban environments are often included with industrial environments because both have similar atmospheric pollutants. The standardized exposure conditions yield comparable data for macroenvironments, but they are not typical of bridge service conditions (Chapter Four, "Test Method" and "Types of Environments").

Rural environments are usually free of aggressive pollutants. They contain relatively small amounts of sulfur oxides and carbon dioxide from combustion products. In such clean air, the rate at which steels corrode increases only when the relative humidity exceeds about 70 percent. Rural environments generally are not aggressive toward steel (Chapter Four, "Rural Environments").

The most potent causes of corrosion in industrial environments are the sulfur oxides and nitrogen oxides produced by burning automotive fuels and fossil fuels in industrial and utility power stations. The sulfur oxides dissolve in films of condensed moisture on the surface of the steel where they can be further oxidized to form sulfuric acid and attack the steel. This occurs at a critical relative humidity as low as about 60 percent. In the absence of moisture and condensing conditions sulfur oxides are not necessarily aggressive toward steel (Chapter Four, "Industrial Environments").

In marine environments airborne salt spray droplets, salt fog, and nightly condensate drops can keep the steel damp for long periods of time. On drying, salt crystals hygroscopically attract moisture at low relative humidities. Visible condensate, in the form of droplets, is not necessary for corrosion to proceed. The presence of chloride inhibits the role of sulfur oxides in forming a protective oxide film. Weathering steel may still resist corrosion provided that the surfaces are boldly exposed and rain frequently washes off the salt. However, the corrosion resistance is greatly diminished in severe marine environments, or when the surfaces are sheltered. Information gained from the performance of the weathering steels in marine exposure tests aid in determining the corrosion behavior of bridges contaminated with roadway deicing salt (Chapter Four, "Marine Environments").

While rural atmospheres are less corrosive than industrial atmospheres and much less than marine atmospheres, this broad classification by macroenvironment is not necessarily a reliable description of the aggressiveness of an atmosphere towards steel. The corrosiveness of an exposure site depends mainly on: (1) the time of wetness of the specimens, determined largely by the length of time during which the relative humidity exceeds a critical value; (2) degree of airborne sulfur oxide pollution; and (3) chloride contamination. The severity of these factors are greatly influenced by differences in the microenvironments of the exposure sites even though the macroenvironment may be the same.

For weathering steels exposed to rural and industrial environments, the curves of corrosion penetration versus time rise at first and then tend to level off. The curves for American sites fall within the scatterband of 2 to 4 mils (50 to 100 μm) corrosion penetration reported for Mayari R steels (Bethlehem Steel) with a broad range of chemical compositions, after 18 years exposure in different environments. The penetrations at some of the generally more corrosive German and English sites exceed 4 mils (100 μm), some after only 2 years (Chapter Four, under "Rural Environments" and "Industrial Environments").

For weathering steels exposed to east and gulf coast environments, the corrosion penetration curves continue to rise at a constant rate. Most exceed 4-mils (100- μm) penetration after 5 to 10 years exposure. Those for European sites rise even faster. The constant presence of salts in marine environments inhibits the formation of a protective oxide coating (Chapter Four, "Marine Environments").

When using bold atmospheric exposure data to estimate the corrosion performance of bridges built from weathering steel, the effects of microenvironment associated with local topography, climate, and service conditions must be carefully considered, along with the fact that bold exposure does not occur under a bridge deck.

Corrosion Resistance

The ASTM specifications A242, A588, and A709 state that the atmospheric corrosion resistance of weathering steels is about equal to two times that of carbon structural steel with copper, which is equivalent to four times carbon structural steel without copper (Cu 0.02 percent max.). But they do not define how the corrosion resistance of the weathering steel, relative to the reference steel, should be determined. Ratios of corrosion penetrations, rates, and times have been used for this purpose, depending on the viewpoint and application.

The ASTM results do not identify or define good and poor corrosion performance. Furthermore, the advantages of the more corrosion resistant weathering steels tend to be understated in the less corrosive environments. The observed anomalies come from tying the behavior of weathering steel to that of carbon and copper steels (Chapter Four, "Corrosion Resistance").

Consideration should be given to adopting design guidelines that state the requirements for the corrosion resistance of weathering steels in terms of the penetration, independent of the behavior of the reference steels. They should specify: (1) curves of allowable corrosion penetration per side versus exposure time in different environments; (2) conditions under which material thickness should be increased to compensate for net section loss; and (3) required increase in thickness. Some foreign specifications have taken this approach.

Service Corrosion

The microenvironment and the service conditions encountered at bridge sites can alter the corrosion behavior of boldly exposed weathering steel test specimens. The main factors are: initial climate, sheltering, orientation, angle of exposure, time of wetness, atmospheric pollutants, deicing salt, and debris. Depending on the service conditions and type of details, the steel may be subjected to the following forms of corrosion not previously discussed: poulitice, galvanic, pitting, and crevice attack.

Initial Climatic Conditions. The climatic conditions at initial exposure can produce large differences in corrosion penetration. These differences are much larger for carbon steel than for weathering steel. For neither type of steel, however, does the difference disappear with increasing exposure time. Evidently, the initial reactivity of the steel with its environment affects the long-term corrosion behavior (Chapter Five, "Initial

Climatic Condition"). For these reasons, initially contaminated weathering steel should be cleaned after installation. This was done in the case of the Luling Bridge, Louisiana, many of whose sections had been exposed to sea-salt spray during transport from Japan.

Orientation. Specimen surfaces that are exposed to the sun and rain corrode less. Those facing the ground, the north (in the northern hemisphere), and sources of contaminants corrode more (Chapter Five, "Shelter and Orientation").

Sheltering. The corrosion penetration of test specimens exposed under a bridge was found to be greater than that of control specimens boldly exposed. Similarly, because of better drainage and less debris accumulation, sheltered vertical specimens performed better than sheltered horizontal specimens (Chapter Five, "Shelter and Orientation").

Under conditions of bold exposure, vertical and horizontal specimens corroded more than specimens facing south at an angle close to the latitude of the site, a condition that maximizes sun drying and rain washing (Chapter Five, "Angle of Exposure").

Continuously Moist Condition. Alternate cycles of wetting and drying are essential to the formation of a protective oxide coating. When the weathering steels remain continuously wet, they do not exhibit corrosion resistance superior to that of carbon steel. Direct precipitation of rain, snow, or condensation is not needed for a steel surface to remain continuously wet. Moisture can be deposited by capillary action of the porous oxide coating, and absorption by corrosion products or salt deposits on the surface. A thin, invisible film of electrolyte can form on the surface at a relative humidity well below 100 percent.

The critical relative humidity, above which the corrosion rate increases sharply, depends on the nature of the atmospheric pollutants, surface contaminants, and corrosion products. Weathering steels with a dense oxide coating in a salt-free environment corrode beginning at relative humidities somewhat higher than 65 percent, while those with a loose oxide coating in a salt-laden environment can corrode down to 55 percent relative humidity.

The length of time during which the critical relative humidity is exceeded determines the time of wetness and greatly affects the corrosion penetration. Reducing the degree of contamination and the time of wetness is a requisite for improving corrosion performance (Chapter Five, "Continuously Moist Conditions").

Industrial Pollutants. The proximity to a source of industrial pollutants can alter the corrosion behavior of weathering steels. This depends on the type and concentration of atmospheric constituents emitted by the source. The sulfur oxides are the most widespread and potent cause of corrosion in industrial environments. However, with the advent of the EPA standards, the installation of scrubbers in fossil fuel plants and the use of low-sulfur coal, excessive levels of sulfur oxides are less likely to occur now and in the future.

Among the local sources that emit pollutants are the chloralkali plants, chemical plants, pulp and paper plants, fertilizer plants, oil refineries and petrochemical plants as well as coal-fired electrical generating facilities. To assess the corrosiveness of a localized area towards the weathering steels, specimens of carbon steel and weathering steel should be exposed for a minimum period of 18 to 24 months. Either the 30 deg to the horizontal or the vertical mode of exposure may be used.

Deicing Salts. Deicing salts spread on highways and bridge decks can create an aggressive localized environment that often exceeds conditions prevailing near the ocean. The corrosive deicing salt solution can leak through the bridge deck by way of expansion and construction joints and drain along girder and floor beam flanges leaving a corrosive salt residue that remains active throughout the year by absorbing moisture from the air. In the case of double deck bridges and overpasses, a salt mist

or spray from moving traffic can be generated such that it splashes overhead girder structures leaving a salt residue that remains active throughout the year. Likewise, dried salt residue from the highway can be blown into and on various bridge components where it can continue its aggressive action during humid periods. Salt water also moves upgrade on flanges and around "drip stops" or intended run-off points by that same capillary action. Evaporation of the water leaves salt deposits, leading to accelerated pitting of the surface.

Such structures should be high-pressure hosed or stream-detergent cleaned at the close of each winter season. Areas experiencing aggressive attack will require spot protection on a periodic basis (Chapter Five, "Deicing Salts").

Poultice Corrosion. The debris that accumulates on the horizontal surfaces of girder elements consists of road dirt, salt, leaves, rust flakes spalling from the web, and occasionally bird nests and droppings. By retaining moisture, the debris forms a poultice that keeps chlorides and sulfates in close contact with the steel and greatly prolongs the time of wetness (Chapter Five, "Debris").

Galvanic Corrosion. When two dissimilar metals are in contact in a moist environment, a galvanic couple forms and causes the anodic metal to corrode more than the cathodic metal. Steel is anodic to mill scale. Thus, mill scale can cause galvanic pitting of the steel along breaks in the mill scale.

Carbon steel is anodic to low-alloy steel, and structural steels are anodic to more noble metals such as bronze (Chapter Five, "Galvanic Corrosion"). To avoid galvanic corrosion in aggressive environments, bolts and weld filler metal should have compositions similar to the weathering steel. Bronze washers in pin-and-hanger connections must not be in direct contact with the steel.

Pitting. In all instances the chloride ion from deicing salt or seawater spray are the primary stimulants that accelerate pitting. Differential aeration cells resulting from the scattered deposits of debris, likewise, stimulate pitting of the surface of structural steels. Under these conditions, bridges covered with mill scale may develop numerous pits at cracks in the scale. Bridges that are free of mill scale can, however, develop deep pits beneath deposits of moist debris (Chapter Five, "Pitting").

Crevice Corrosion. Crevice corrosion is a form of localized attack of the steel surface at, or immediately adjacent to, an area shielded from bold exposure. Bolted connections, cantilever expansion joints, guardrail lap joints, and lap joints not welded all around are examples of details susceptible to crevice corrosion. The rate of corrosion attack in the crevice is higher than that on the surfaces outside the crevice (Chapter Five, "Crevice").

Fatigue

The available test data on fatigue of weathering steel structural details come from specimens that were exposed to the environment for a number of years and then fatigue-cycled to failure in dry laboratory air. This condition, termed herein weathering fatigue (Chapter Seven), reduced the life of most details and increased the life of a few. The observed reduction of life, attributed to rust pitting, was greatest for base metal and continuously decreased as the stress concentration in the detail increased. The observed gain in life of a few details resulted from weld-toe rounding by corrosion. The aforementioned behaviors were the same for weathering steels and carbon steel. It should be kept in mind that the latter would be coated and not subjected to pitting.

In addition to the effect of weathering, bare steel bridges are subjected to corrosion fatigue during their service life (Chapter Eight). All available corrosion fatigue and crack growth rate data show that cracks initiate earlier and propagate faster in aqueous environments than they do in air. There are no data that show a beneficial effect of

exposure to aqueous environments. Therefore, it may be assumed that weathering steel bridges have shorter fatigue lives than painted bridges.

The authors' estimates, accounting for the effects of both weathering and corrosion fatigue, give the following total losses in stress range: 34 percent for Category A base metal, 24 percent for Category B welded beams, 19 percent for Category C transverse stiffeners, 13 percent for Category C 2-in. (51-mm) attachments, and 11 percent for Category D, E, and E' attachments. The corresponding losses in terms of number of cycles are: 73 percent for Category A base metal, 60 percent for Category B welded beams, 47 percent for Category C transverse stiffeners, 37 percent for Category C 2-in. (51-mm) attachments, and 29 percent for Category D, E, and E' attachments (Chapter Eight, under "Estimation of Loss in Life" and "Fatigue Design Stresses").

The AASHTO fatigue specifications are based on fatigue test data obtained in clean and dry laboratory air. Current design practice applies these provisions to both painted steel and weathering steel bridges. The new drafts of the European codes for fatigue design of steel structures state that the "reduction in life due to corrosion" is not explicitly covered. Their design S-N curves are said to apply only to structures in normal atmospheric conditions with suitable corrosion protection. British fatigue specifications for offshore structures impose a penalty factor of two on the fatigue life (50 percent loss in life) for all steel welded joints exposed to salt water unless they are coated or cathodically protected (Chapter Eight, "Fatigue Design Stresses"). The environment of a bridge is different from offshore structures but, in some ways, not less severe because chlorides build up on the surfaces of salt-contaminated members that are sheltered and, therefore, not rain washed.

It is prudent to design bare weathering steel structures for fatigue to reduced allowable stress ranges. Additional fatigue tests of weathered steel members are currently being sponsored by the Maryland Department of Transportation. The second phase of research under NCHRP Project 10-22 also includes tests of 8-year weathered weldments. The results of these studies will be considered in developing fatigue design recommendations in the second phase of research.

Painting

Initial paint systems last longer on a weathering steel substrate, because the protective and dense oxide coating that forms in scratches and exposed areas does not undercut the adjacent intact paint film. With carbon steel, the expanding rust continuously destroys the adhesive bond between the paint and the steel surface. The procedures and standards for preparing the surface of new weathering steel are the same as those for carbon steel. With chlorides present, paints do not perform better on A588 steel than on carbon steels (Chapter Nine, "Initial Painting of New Bridges").

In comparison with shop coating new steels, the problem with the remedial painting of severely corroded and pitted weathering steels lies in removing all rust and salt trapped in dimples and pits. Unless this is achieved, the paint life is reduced. With proper surface preparation and paint application it is possible to protect A588 steel against corrosion, even if it has been exposed to a severely leaking bridge joint environment. Zinc-rich primer compositions appear to out perform other systems (Chapter Nine, "Remedial Painting of Existing Bridges"). Final recommendations for remedial painting of weathering steel bridges must await the results of on-going studies being performed by the Michigan Department of Transportation and the Steel Structures Painting Council.

Connections

Weathering steels are weldable with the use of good welding practice and the proper choice of filler metal as specified by the American Welding Society. Covered carbon

steel electrodes and bare carbon steel electrode-flux combinations may be used for: underlying passes of multiple-pass weldments, single-pass shielded metal arc welds to 0.25-in. (6.4-mm) size, single-pass submerged arc welds 5/16-in. (8-mm) size, and painted structures. Covered low-alloy steel electrodes and bare low-alloy steel electrode-flux combinations are used for bare applications requiring weld metal with atmospheric corrosion resistance and coloring characteristics similar to the base metal (Chapter Ten, under "Arc Welding" and "Specifications").

A325 and A490 Type 3 high-strength bolts have atmospheric corrosion resistance and weathering characteristics comparable to that of weathering steel. As is done in welds, matching the chemistry of the connectors and the base metal prevents accelerated galvanic corrosion of the connectors.

The crevice between the plies of a bolted weathering steel joint usually seals itself with oxide when the joint is appropriately tightened and stiff. If the joint moves, the surfaces should be coated with a protective material and filled with a suitable sealant to avoid progressive corrosion. Expansion forces induced by corrosion products in the crevices can deform the connected elements and cause large tensile loads on the bolts. Significant bowing of the plates between bolts or edge distortion can be controlled by limiting the pitch to 14 times the plate thickness or 7 in. (178 mm) and the edge distance to 8 times the plate thickness or 5 in. (127 mm), whichever is smaller (Chapter Ten, "High-Strength Bolted Joints").

Friction-type connections of weathering steel plates with mill scaled surfaces have much lower slip resistance than their carbon steel counterparts. The loss is attributed to the dense, hard and slippery nature of the mill scale on HSLA steels. Corrosion of the mill scale contact surfaces before the joint is high-strength bolted tends to increase the slip resistance (Chapter Ten, "Friction-Type Joints with Mill Scale Surfaces").

Friction-type connections with blast-cleaned surfaces have a slip resistance comparable to that of blast-cleaned carbon steel plates (Chapter Ten, "Friction-Type Joints with Blast-Cleaned Surfaces").

Loose and flexible joints are prone to crevice corrosion and packout bulging. Such joints must be protected against corrosion and provisions made for cleaning and maintaining them. Examples of such joints known to exhibit the aforementioned problems are pin-and-hanger connections and guardrail lap joints (Chapter Ten, under "Pin and Hanger Connections" and "Highway Guardrails").

Structural Details

The most important considerations when detailing weathering steel bridges are to divert the flow of runoff water from the structure, prevent ponding, and avoid the accumulation of debris that traps moisture containing chlorides. Unfortunately, leaking of expansion joints has been one of the major causes of extensive maintenance and costly remedial work on bridges. Experience has demonstrated that the performance of many joint systems is disappointing. To prevent the progressive corrosion of weathering steel bridges, several states have been painting the girder ends on either side of an expansion joint over a length varying from one beam depth to 10 ft (3.0 m).

Water ponds and debris accumulate on horizontal surfaces and in corners formed by horizontal and vertical plates. The most susceptible locations are: the top surfaces of lower beam flanges and truss chords, gusset plates for horizontal bracing, bolted splices of horizontal and sloped members, bearing and intermediate stiffeners, and the interior of nonsealed box sections. To avoid water ponding and debris accumulation it is important to: (1) slope horizontal surfaces longitudinally or transversely; (2) avoid re-entrant corners; (3) seal box sections; and (4) design details for easy discharge of water. To enhance the good performance of weathering steel bridges, careful attention

must be given to detailing. It is important to note that salty water causes the steel to develop a porous oxide coating with high capillarity. As a result, water migrates to places it would not normally reach. This may reduce the effectiveness of some details designed to remove water from the surface.

Chapter Eleven gives more information on structural details for weathering steel bridges and their implementation in two bridges.

Conclusion

The majority of the weathering steel bridges are in good condition, although localized areas of accelerated attack can be found in many structures. A number of bridges in several states are experiencing excessive corrosion because of salt contamination and/or prolonged time of wetness. Weathering steel can provide a satisfactory service life with limited maintenance if the structural details are designed in a manner that prevents accelerated attack, vulnerable areas are painted, and contamination with chlorides is avoided.

The report contains extensive information on the performance of weathering steel and its use in highway bridges. The practitioner can use this body of knowledge as a general guide for designing new bridges and for trouble shooting. The second phase of research under NCHRP Project 10-22 will develop specific guidelines for design, construction, maintenance, and rehabilitation of weathering steel bridges.

CHAPTER ONE

INTRODUCTION

PROBLEM AND OBJECTIVES

Problem

Weathering steel specimens boldly exposed under ideal conditions develop a dense and well-bonded oxide coating that shields the underlying steel base against penetration of moisture, oxygen, and atmospheric contaminants, and greatly reduces long-term corrosion. When properly designed and exposed in a suitable environment, weathering steel bridges are also expected to form a protective oxide coating and to require no painting.

Weathering steel offers the potential for savings in life-cycle costs. The initial cost of an unpainted weathering steel bridge is said to be about the same as the cost of a painted regular steel bridge. Because a weathering steel bridge should not need maintenance painting, its life-cycle cost would be lower than that of a painted steel bridge. Furthermore, by eliminating the need for maintenance painting, paint jobs of bridges over difficult terrains, electrified railways, and bodies of water can be avoided; heavy traffic lanes need not be closed; and safety hazards to motorists and painters alike are prevented. In an evaluation of 16 factors that optimize the cost of steel plate girders for continuous composite bridges with span lengths up to 200 ft (60 m), Knight [1983] cited the use of unpainted weathering steel as being one of the three most influential factors.

Despite the aforementioned advantages, weathering steel is recently being specified for new bridges less than might be expected. The main reason for this stems from the observation and the concern that some bridges are corroding faster than anticipated. This may result in a significant loss of metal not considered in design.

Weathering steels develop a protective oxide coating under the following conditions:

1. Atmospheric exposure to intermittent wet-dry cycles without prolonged wet periods.
2. Absence of heavy concentrations of corrosive pollutants, especially deicing salts.
3. Washing of the exposed surface by rain water.
4. Absence of detail geometrics that trap moisture, dirt, or debris and hence foster corrosion.

These conditions may not be met in some bridges. For example, the deck shelters the steel girders from the rain. Runoff water, mixed with roadway deicing salts, leaks through joints onto the steel. Salt-bearing sprays kicked up by trucks passing under a bridge settle on the girders. Roadway debris and rust flaking off the web accumulate on the flange and hold moisture. Water collects at poorly designed structural details. A lack of aeration and low clearance over bodies of water keep the steel

damp over long periods of time. When some of these conditions occur at a bridge site, corrosion problems can develop.

In addition to the previously described potential corrosion problems, there have been concerns about localized galvanic and crevice corrosion, reductions in life due to rust pitting and corrosion fatigue, and rust staining of concrete piers and abutments by oxide-laden water running off the steel.

Objectives

The objectives of this study are to: (1) assemble a systematic body of information on the performance of weathering steel; (2) document and evaluate the current state of practice; and (3) develop, where feasible, guidelines for design, construction, maintenance, and rehabilitation of weathering steel bridges.

Bridges are subjected to a variety of service conditions and many complex factors which influence the performance of weathering steel. Recognizing that all bridges cannot be designed for good performance with a general set of guidelines, the report presents data from which the designer can extract the information needed for assessing the performance under a given set of conditions. The information available in the open literature is adequate to evaluate the performance of weathering steel.

The findings presented in this report relate primarily to the first two objectives. Limited progress was made on the third objective, and a second phase of research, initiated in August 1984, has as its specific objectives (1) to fatigue test 8-year weathered A588 transverse stiffener specimens under constant and variable amplitude loading in air and aqueous environments, and (2) to develop practical guidelines for design, construction, maintenance, and rehabilitation of weathering steel bridges.

The information acquired in the first phase of research will serve as the basis for developing the Phase II guidelines, which are expected to be in the form of specific recommendations to the AASHTO Subcommittee on Bridges and Structures.

The remainder of this chapter traces the history of the development of weathering steels and reviews the applicable ASTM specifications. Chapter Two summarizes the domestic and foreign experiences. Chapters Three to Five deal with the corrosion performance, and Chapter Six with strength and impact properties. The fatigue behavior is examined in Chapters Seven and Eight. This is followed by Chapter Nine on initial and remedial painting, Chapter Ten on welded, bolted, and miscellaneous connections; and Chapter Eleven on structural details. Chapter Twelve includes the conclusions and an enumeration of areas needing further investigation.

HISTORICAL DEVELOPMENT

This brief history traces the development in the United States of what are familiarly known as the "weathering steels" of which the most promising application currently is in highway bridges. They were part of a family of steels termed "high-strength low-alloy" (HSLA). The products evolved from an iron and steel base. It was noticed in the early 1800's, and later reported by Karston in 1827, in Germany, that additions of small amounts of copper to foundry products resulted in a reduction in the corrosion rate in acid. Because of this observation, many people were trying to evaluate and predict the performance of different steel compositions for use in structural applications by the rate at which they dissolved in 20 percent sulfuric acid.

As a consequence of numerous reports of copper additions to iron during the 19th century, Williams conducted a simple experiment in 1900. He selected a series of copper-bearing and copper-free wrought irons and Bessemer steels and wetted them with water several times daily for 1 month while exposed outdoors. Only the copper-bearing products significantly retarded corrosion.

The subject of copper in steel and iron was frequently discussed during the first decade of the existence of the newly formed American Society for Testing and Materials (1900-1910). It remained for G.H. Clamor at the 1909 meeting to call attention to the work of Burgess and Aston (also members of the Society) in which they added single elements to pure electrolytic iron and exposed it in Madison, Wisconsin. They found that copper enhanced the corrosion performance of the iron. Clamor further reported that copper improved the ultimate strength of carbon steel. He suggested that Sudbury (Ontario, Canada) ores contained nickel and copper, and that such natural ores should provide a steel with improved properties without any further alloy additions.

The most authoritative review of the state of the art is contained in a report in the 1915 Yearbook of the American Iron and Steel Institute by D.M. Buck of the American Sheet and Tin Plate Company, a division of the United States Steel Corporation. He reported that he observed in 1910 the excellent performance exhibited by a 0.07 percent copper-bearing steel sheet. To confirm that observation, full-size 16-gage and 27-gage roofing sheets of carbon steel with varying copper levels were exposed in rural, industrial, and marine atmospheres.

Buck reported his findings in 1913 in a paper given before the American Chemical Society in which he stated that no difference exists between sheets with 0.15 percent and 0.34 percent copper content; whereas, sheets without copper rusted through during the first year of exposure in a coke oven atmosphere. The sheets with 0.06 percent copper took an intermediate position.

Buck, with the assistance of Handy, then director of research for the Pittsburgh Testing Laboratories, performed a more detailed exposure test in which they exposed around Pittsburgh full-size roofing sheets that varied in copper levels from 0.04 percent to 2.2 percent. In addition, they obtained a number of "pure" irons which contained from 0.014 percent to 0.018 percent copper. At the end of almost 1 year they found that the low copper level specimens lost weight at the rate of 0.31 oz/ft²/mo (95 g/m²/mo) in contrast to a loss of 0.12 oz/ft²/mo (37 g/m²/mo) for all of the copper-bearing specimens. Thus they were able to locate the point for effective copper levels in carbon steel.

Allerton Cushman of the Institute of Industrial Research in Washington, D.C., and a member of ASTM Committee A5, voiced objection to the facts reported by Buck and Handy at the American Iron and Steel Institute meeting in 1915. He insisted that a carefully supervised test be repeated under the sponsorship of Committee A5 of the ASTM with cooperation from the National Bureau of Standards. Furthermore, the exposure tests were to be located at protected sites such as Fort Sheridan, Illinois; the Naval Academy at Annapolis, Maryland; and at Fort Pitt in Pittsburgh, Pennsylvania.

During the preparation of the test specimens, it was pointed out (in the ASTM Committee A5 report in 1916) that the iron specimens have a high copper content because of the use of eastern Pennsylvania ores and that an effort would be made to

find ores substantially free of copper such as typified by the newly available Mesabi Range ores.

The Proceedings of the American Society for Testing Materials for 1919 contain Buck's report in which steels with copper levels varying from 0.01 percent up to 0.25 percent were exposed at the aforementioned test sites and inspected twice yearly. Committee members included Buck and Cushman or his representative. The findings following 18 months of exposure indicated that beginning with 0.04 percent copper the atmospheric corrosion rate slowed down considerably, and that a copper level of 0.15 percent was ample.

The results of this 1916 exposure test were summarized and presented again by Buck at a meeting of the American Iron and Steel Institute and published in their Yearbook for 1920. In that article Buck referred to the earliest use he could find for copper, namely, in 1627, wherein the physical properties of the iron were improved. He related that Richardson and Richardson, in 1916, reported that copper additions to steel resulted in greater improvement than equal copper additions to pure iron.

On the strength of their own atmospheric exposure tests, the New York Central Railroad purchased 620,000 copper-bearing steel tie plates weighing 4,000 tons and containing 0.2 percent copper based on their findings of a loss rate one-tenth that of plain carbon steel plates. Buck states unequivocally in his report that the superiority of copper-bearing steels is twice that of plain carbon steel and, in fact, his findings show it to be sometimes as much as three to five times better in certain aggressive environments.

The U.S. Steel Corporation, through its American Sheet and Tin Plate Division, began marketing copper-bearing steel sheets in 1911. At about this time, Byramji D. Saklatwalla came from England to Pittsburgh, Pennsylvania (1909) to investigate means for incorporating vanadium into carbon steel. He was not unaware of the activities of Buck and began some experiments in this field. By 1926, in cooperation with one of the U.S. Steel mills (Vandergrift, Pennsylvania), he obtained a patent covering the addition to carbon steels of several elements besides copper to both enhance atmospheric corrosion resistance as well as to increase the yield strength beyond that achieved by copper; namely, 32 to 36 ksi (220 to 250 MPa). Meanwhile, during the period 1920–1930 U.S. Steel was developing a family of high-strength low-alloy steels intended primarily for the railroad industry.

At the same time in Pittsburgh, Jerome Straus, paralleling Saklatwalla's work, also developed a high-strength low-alloy steel composition for which a patent was granted in 1935. U.S. Steel acquired the patent rights of both of these inventors to round out their product line of HSLA steels.

An article by Cone in *Steel Magazine*, in 1934, reviewed the dozen or so new proprietary HSLA steels then available and described the U.S. Steel family of steels, which included USS Cor-Ten steel.

Each alloying metal addition makes its own unique contribution to both corrosion resistance and strength. Larrabee and Coburn [1961] showed in a comprehensive study the contribution to corrosion resistance made by the elements copper, phosphorus, silicon, nickel, and chromium. They demonstrated the interplay of the various combinations by testing 270 steels with three variations of chromium, five of copper, three of phosphorus, three of silicon, and two of nickel.

Porter [1981] pointed out that the optimum balance of cor-

rosion resistance and strength, among many possible combinations, has resulted in a steel four to eight times more corrosion resistant than carbon steel in the bare state in a variety of aggressive environments. The steel has a minimum yield strength of 50 ksi (345 MPa) in 1/2-in. (12-mm) thick plates for composition containing about 0.10 percent carbon. Similar yield strengths were also achieved through 4-in. (100-mm) thick plates by increasing the carbon and manganese levels and adding vanadium.

USS Cor-Ten steel, the first and best known of the weathering type high-strength low-alloy steels were first marketed in 1933. Its acronym, Cor-Ten, was derived from the two properties that distinguish it from mild carbon steel and copper-bearing steel; namely, improved atmospheric corrosion resistance and higher yield and tensile strengths.

The first major application in 1933 was in the form of an exterior painted coal hopper car. This met with instant success, and since then over 1,000,000 such cars have been fabricated and placed in service worldwide. The next major application at that time came as a result of the Great Depression. It resulted in the standard design for an electrified railway or trolley car known as the PCC car. This stands for the President's Conference Car, a result of President Roosevelt's effort to improve the nation's transit system while providing an economic stimulus to the steel industry. Many such cars went into service in the 1936–1939 period with the remainder supplied in the 1946–1949 period. The city of Pittsburgh, Pennsylvania, is currently rehabilitating 45 such trolleys from the 1949 class. With the exception of dents, the cars continue in excellent condition, and are retaining the steel between the belt line and the roof.

The major patents covering Cor-Ten steel expired in 1940 and 1950, respectively. However, the trademark was retained and constitutes one of the major marketing tools for United States Steel Corporation. Just about all of the major steel firms developed a variation of the Cor-Ten steel composition for applications where atmospheric corrosion resistance was desired. The first ASTM specification entry was in 1941 with A242-41T for Low-Alloy Structural Steel. It covered steel suitable for welding or riveting intended primarily for use as main stress-carrying structural members. The scope was expanded in the 1942 edition to cover applications where savings in weight and atmospheric corrosion resistance were important. The degree of atmospheric corrosion resistance provided by A242 was quantified in the 1960's as being at least two times that of structural carbon steel with copper and four times that of structural carbon steel without copper (0.02 percent copper content).

Following the commercialization of the HSLA steels during the 1930's, a number of research reports and reviews described the superior atmospheric performance that can be expected from Cor-Ten steel over copper-bearing steel [Schramm 1934a; Schramm 1936; Jones 1937; ASM 1946; Pilling 1948]. The major application was, as indicated earlier, railroad coal hopper cars and coal barges. In 1948, an unpainted Cor-Ten steel angle member was installed in a galvanized carbon steel transmission tower in the Gary Steel Works of the U.S. Steel Corporation. Periodic inspections indicated that it was performing effectively. Encouraged by this example, a bare steel transmission tower line of Cor-Ten steel was built along Lake Michigan in the Gary Steel Works. An experimental tower also was erected by General Electric in Massachusetts. The first extended transmission line was erected by Veeco in Virginia followed by the erection of similar lines in Missouri and Georgia.

The most prominent competing steel was trade marked Mayari R (Bethlehem Steel Corporation), based on the fiery appearance of the natural nickel-chromium ore obtained from the Mayari District in northeastern Cuba. The "R" stood for enhanced atmospheric corrosion resistance. Youngstown Steel distributed its Yolloy brand of atmospheric corrosion-resistant steel.

Many foreign steel producers, as well as domestic producers wishing to take advantage of both the well-known Cor-Ten steel name as well as the enormous amount of production know-how available from U.S. Steel, took out a license for its manufacture and sale. It was in the early 1970's that Bethlehem Steel Corporation began referring to its Mayari R steel as "weathering steel." Use by the construction industry of the weathering steel phrase now makes it recognizable by the general public.

Because of the proprietary nature of the Cor-Ten, Mayari R and Yolloy steel compositions, and a number of variations similarly effective in exhibiting enhanced atmospheric corrosion resistance beyond that offered by copper-bearing steel, the basic compositions and mechanical properties are listed in the manner shown in ASTM Specifications A242, A606, A588, and A709. Specifications A618 and A714 deal with pipe and tubing.

Two outstanding and comprehensive papers mark the contemporary history of the HSLA steels and the efforts to place on a firm basis the necessary alloy content, as well as the possible operating mechanism and the influence of widely differing environments [Larrabee 1961; Copson 1960]. Works of this type led to a change in the A242 specification to include the enhanced atmospheric corrosion resistance and the need for the customer to discuss with the steel supplier his exact application so that the appropriate steel composition would be supplied.

The John Deere Administration Center, located on the outskirts of Moline, Illinois, includes a pedestrian bridge leading to a two-story exhibition building, which is attached to the seven-story administration building. Additional buildings have since been added. This complex represents the first instance in which unpainted Cor-Ten steel was used for structural members and all exterior columns, posts, beams, girders, and sun control devices. Prior specifying Cor-Ten steel, a 2-year exposure test was conducted to establish the performance of the steel in the Moline environment [Coburn 1965].

The first application of a weathering type steel in a high-rise structure took place in the heart of a metropolitan business district, with the erection of the 32-story Chicago Civil Courts Building in 1964–1965. It was erected with uncoated steel curtain walls consisting of fascia, column covers, and window frames. Test panels exposed about 1/2 mile (800 m) from the building had indicated that the environment was satisfactory and the terminal color would likely be reached in about 3 years. All exposed steel was fabricated from A242 Type 1 (Cor-Ten A) steel. The Picasso sculpture on the plaza was fabricated from A588 Grade A (Cor-Ten B) steel [Coburn 1965].

The group of steels designated as ASTM A588 in 1968 were part of a broader effort to develop HSLA steels capable of being welded in thick sections. The development effort began after World War II, when concern about the brittle long running fractures experienced in ships, pressure vessels, and such welded structures as bridges focused attention on the fracture toughness and the ductile-to-brittle transition temperature of plate steels. Corrosion-resistant HSLA steels could be produced economically by hot rolling, but the weldability generally decreased, as

strength and plate thickness increased and the transition temperature was raised.

The ensuing broad-scale research in the United States, Canada, Europe, and Japan on grain size effects, strengthening mechanisms, composition, and steel cleanliness resulted in the currently available tough and weldable HSLA steels. These steel products combine carefully controlled heating and rolling procedures with optimum use of alloying elements [Porter 1981].

In 1968, specification A588 was issued for a high-strength low-alloy steel with 50-ksi (345-MPa) minimum yield strength for plate up to 4 in. (102 mm) in thickness. Its composition was intended for use in welded bridges and buildings. Its atmospheric corrosion resistance is approximately equal to twice that of structural carbon steel with copper. It is available in several proprietary grades each based on minor variations of the same basic chemical composition.

The satisfactory experience with transmission towers and buildings, coupled with the possibility of avoiding or minimizing costly maintenance painting of the structures during their service lives, led to the construction in 1964–1965 of the first four unpainted weathering steel bridges. Iowa erected a bridge on Iowa 28 over Racon River. The Michigan Highway Department built four structures at the crossing of the Eight-Mile road over U.S. 10 in Detroit. Ohio replaced an old bridge with a new all-welded three-span steel bridge across the Brush Creek in Miami County north of Dayton. New Jersey erected a four-span composite bridge as an adjunct to the New Jersey Turnpike near Morristown. Since then about 1,800 bridges on state highway systems have been built of unpainted weathering steel. This figure does not include bridges owned by local government agencies. It is of interest that the first two built, in Iowa and Michigan, are now scheduled for remedial painting.

In 1975, ASTM issued a general specification A709 for Structural Steel for Bridges to cover carbon, HSLA and quenched and tempered alloy steels. Five grades are available in three strength levels: 36, 50, and 100 ksi (250, 345, and 690 MPa). Grades 50W and 100W exhibit enhanced atmospheric corrosion resistance.

The following three sections summarize the evolution of the specification requirements for the three domestic weathering steels, ASTM designations A242, A588, and A709. The summaries are broken down into scope, chemical requirements, tensile requirements, and supplementary requirements. Knowledge of the changes made in the scope and requirements over the years is helpful in assessing the corrosion performance of the older bridges in particular. A summary of some foreign weathering steel specifications concludes this chapter.

ASTM A242 STEELS

The ASTM Standard Specification A242 corresponds to the ASHTO Standard No. M161.

Since the first publication in 1941, 17 editions of the ASTM Standard Specification A242 for high-strength low-alloy structural steel have been issued. The current edition is dated 1981.

Scope

The 1941 edition covered "Low Alloy Structural Steel," suitable for welding and riveting, and intended primarily for use as main stress-carrying material of structural members. The ma-

terial was limited to thickness not under 3/16 in. (5 mm) and not over 2 in. (50 mm). No reference was made to type of product and corrosion resistance.

In 1942, the scope was expanded to cover structural members where saving in weight and atmospheric corrosion resistance were important.

In 1955, the specification title changed to "High-Strength Low-Alloy Structural Steel." Steel shapes, plates, and bars were included in thicknesses up to 4 in. (102 mm).

In 1960, bolted construction was added to the scope. The atmospheric corrosion resistance was quantified as being equal to or greater than that of carbon structural steels with copper. If the steel was specified for materially greater atmospheric corrosion resistance than structural carbon steel with copper, the purchaser was expected to consult with the manufacturer. Welding characteristics were said to vary with type of steel furnished.

In 1968, concurrently with the adoption of the new A588 specification, several changes were made in the A242 specifications. The atmospheric corrosion resistance was specified as being at least two times that of carbon structural steel with copper, which is equivalent to four times carbon structural steel without copper (0.02 percent max.). When required, the manufacturer had to supply evidence of corrosion resistance satisfactory to the purchaser. That left the A242 steels with a higher corrosion resistance than the A588 steels, which were specified to have atmospheric corrosion resistance approximately (not at least) four times that of carbon structural steel without copper.

Welding technique being of fundamental importance, the A242 specification presupposed that welding procedure would be in accordance with approved methods.

The scope was last changed in 1975, when the atmospheric corrosion resistance was reduced from "at least" to "approximately" four times that of carbon structural steel without copper.

Chemical Requirements

The first edition, A242-41 (note that the date refers to the year in which the specification was published), specified the maximum alloying content of carbon, manganese, and sulfur listed in Table 1. The manufacturer was free to use such alloying elements—with carbon, manganese, and sulfur within prescribed limits—as would give the specified physical properties and suitability for welding under the given conditions. He had to analyze each melt and report the percentages of carbon, manganese, phosphorus, sulfur, and any alloying elements used.

The 1942 edition stated the intent that the alloying elements should be chosen so as to materially increase the atmospheric corrosion resistance of the steel.

In 1955, the maximum carbon content was raised to 0.22 percent. For compositions with a maximum carbon content of 0.15 percent, the maximum limit for manganese was allowed to increase to 1.40 percent.

In 1960, the manufacturer had to determine and report the content of all alloying elements found by a ladle analysis for information purposes only, to identify the type of steel used. The purchaser had to indicate and consult with the manufacturer if he needed materially greater atmospheric corrosion resistance than that of structural carbon steel with copper.

In 1968, the same year when the requirement of atmospheric

Table 1. Chemical composition of A242 steel, 1941–1983.

Designation	Type	Composition, % (Ladle Analysis)				
		C, max.	Mn, max.	P, max.	S, max.	Cu, min.
A242-41	-	0.20	1.25	-	0.05	-
A242-55	-	0.22 ^b	1.25	-	0.05	-
A242-68	1	0.15	1.00	0.15	0.05	0.20
	2	0.20	1.35	0.04	0.05	0.20 ^a

a. If chromium and silicon contents are each 0.50% min., then the copper 0.20% min. requirement does not apply.

b. For composition with a maximum carbon content of 0.15%, the maximum limit for manganese may be increased to 1.40%.

corrosion resistance was raised to at least four times that of carbon structural steel without copper, the chemical requirements were changed in three ways: (1) A242 Type 2 steel with low phosphorus content (0.04 percent max.) was added; (2) maximum phosphorus and minimum copper contents were specified for both types; and (3) if chromium and silicon contents of Type 2 steel were each 0.50 percent min., the 0.20 percent min. copper requirement did not apply. The chemical requirements for Type 1 and Type 2 are given in Table 1. The elements commonly added in addition to those listed were chromium, nickel, silicon, vanadium, titanium, and zirconium; the first three mainly for corrosion resistance, the other three mainly for mechanical properties. Type 2 steel was intended for use when better impact properties were needed. The basic premise remained unchanged, even to date. That was to let the manufacturer choose and use the alloying elements, in combination with those prescribed within limits, that gave the atmospheric corrosion resistance and mechanical properties.

Tensile Requirements

Table 2 gives the tensile requirements for A242 steel that were specified in the original 1941 edition, for material thicknesses varying from 5/16 in. (8 mm) to 2 in. (51 mm). The basic values of 50-ksi (345-MPa) yield point and 70-ksi (485-MPa) tensile strength at the lowest thickness range are the same as the values specified today. In 1950, the minimum elongation in 8-in. (200-mm) gage length was reduced to 18, 19, and 20 percent for the three thickness ranges. In 1952, the requirement of 24 percent minimum elongation in 2-in. (50-mm) gage length was added for the thickness range over 1/2 in. (38 mm) to 2 in. (51 mm).

In 1955, the yield point and tensile strength at the two higher thicknesses were raised to the values that are still specified today, as indicated by the entries for A242-81 in Table 2. The elongation in 8 in. (200 mm) was lowered to 19 percent for the thickness range 1/2 in. to 4 in. (38 mm to 102 mm). This elongation was further reduced to 16 percent in 1964. Tensile requirements were also added for three groups of structural shapes. The values were identical to those for plates and bars of corresponding thicknesses.

Table 2. Original and present tensile requirements for A242 steel.

ASTM Specification	Structural Shapes	Plates & Bars Thickness mm (in)	Tensile Strength	Yield Point	Elongation in 200 mm (8 in)	Elongation in 50 mm (2 in)
			MPa (ksi)	MPa (ksi)	min, %	min, %
A242-41T	-	$8 \leq t \leq 19$ ($5/8 < t \leq 3/4$)	485 (70)	345 (50)	21	-
	-	$19 < t \leq 38$ ($3/4 < t \leq 1-1/2$)	455 (66)	310 (45)	23	-
	-	$38 < t \leq 51$ ($1-1/2 < t < 2$)	435 (63)	275 (40)	24	-
A242-81 Type 1&2	-	$t \leq 19$	485 (70)	345 (50)	18	-
	-	($t \leq 3/4$)				
	-	$19 < t \leq 38$	460 (67)	315 (46)	18	21
	-	($3/4 < t \leq 1-1/2$)				
	-	$38 < t \leq 102$	435 (53)	290 (42)	18	21
	-	($1-1/2 < t \leq 4$)				
	Groups 1 & 2	-	480 (70)	345 (50)	18	-
	Group 3	-	460 (67)	315 (46)	18	-
	Groups 4 & 5	-	435 (63)	290 (42)	18	21

In 1970, the elongations for all thicknesses of plates and bars, and all groups of structural shapes were set at a uniform 18 percent in 8-in. (200-mm) gage length and 21 percent in 2-in. (50-mm) gage length. With that last change, all tensile requirements had become identical to those of the A242-81 edition listed in Table 2.

Supplementary Requirements

The current edition of the A242 specification refers to the ASTM A6 specification entitled "General Requirements for Rolled Steel Plates, Shapes, Sheet Piling, and Bars for Structural Use" for standardized supplementary requirements for use at the option of the purchaser. Those considered suitable for use with the A242 specification are: S2 Product Analysis, S3 Simulated Post-Weld Heat Treatment of Mechanical Test Coupons, S5 Charpy V-Notch Impact Test, S6 Drop Weight Test, S8 Ultrasonic Examination, S14 Bend Test, and S15 Reduction of Area Measurement. The bend test requirement was mandatory from 1941 to 1974. It became optional in 1975, at the request of the purchaser. The other requirements were referenced beginning in 1975 (S2, S3, S5, S6, and S8) and 1979 (S15).

ASTM A588 STEELS

The ASTM Standard Specification A588 corresponds to the AASHTO Standard No. M222.

Scope

In 1968, ASTM adopted the Standard Specification A588-68 for "High-Strength Low-Alloy Structural Steel with 50-ksi (345-MPa) Minimum Yield Point to 4 in. (100 mm) Thick." This specification covers high-strength low-alloy structural steel shapes, plates, and bars for welded, riveted, or bolted construction, but intended primarily for use in welded bridges and buildings where saving in weight or added durability are important. The atmospheric corrosion resistance of A588 steel is approximately two times that of carbon structural steel with copper, which is equivalent to four times carbon structural steel without copper (Cu 0.02 percent max.). Welding technique is of fundamental importance. It is presupposed that welding procedure will be suitable for the steel and the intended service. This specification is limited to material up to 8 in. (200 mm), inclusive, in thickness. The scope has remained unchanged in the 12 editions that were issued from 1968 to 1982.

In 1969, the user was cautioned that compliance with the A588 standard may require the use of an invention covered by patent rights. This note was subsequently deleted in 1971.

Chemical Requirements

A588 steel comes in nine grades A to K, excluding I, each proprietary to an individual producer. Table 3 identifies the producers of each grade and the corresponding proprietary names of their steels. In some instances, several producers man-

ufacture a specific grade and many producers manufacture more than one grade of A588 steel. Grades A to G appeared in the original 1968 edition. Grades H, J, and K were added in 1969, 1970, and 1980, respectively. Grade G was deleted in 1983.

The chemical requirements, as specified in the latest A588-82 edition, are summarized in Table 4. The minor changes that were made in the compositions of grades A, B, D, and E over the years are given in Table 5. It shows for each grade and element what the content was in 1968, when it changed, and what it is today.

As will be shown in Chapter Three, silicon, nickel, chromium, and copper contribute most to improving atmospheric corrosion resistance. Various combinations of these elements provide essentially the same results. To illustrate the variability of approaches taken by the 10 producers, Table 6 approximately quantifies the content of these elements as low, medium, and high. These terms are relative to the range of content in all

Table 3. Grades, producers, and proprietary names of A588 steels.

Grade	Producer	Proprietary Name
A	U.S. Steel	Cor-Ten B
B	Bethlehem Steel	Mayari R-50
C	Stelco	Stelcoloy 50
D	Great Lakes Steel	NAX High Tensile
E	Youngstown Sheet & Tube	Yolloy High Strength
F	Republic Steel	Republic 50
G	Armco	High Strength A
H	Kaiser Steel	Kaisalloy 50
J	Jones & Laughlin	Jal-Ten
K	Republic Steel	Dura Plate 50

Table 4. Present chemical composition of A588 steels.

Designation	Grade	Composition, %												
		C	Mn	P max.	S max.	Si	Ni	Cr	Mo	Cu	V	Zr	Cb	Ti
	A	.19 max.	.80 1.25	.04	.05	.30 .65	.40 max.	.40 .65		.25 .40	.02 .10			
	B	.20 max.	.75 1.25	.04	.05	.15 .50	.50 max.	.40 .70		.20 .40	.01 .10			
	C	.15 max.	.80 1.35	.04	.05	.15 .30	.25 .50	.30 .50		.20 .50	.01 .10			
	D	.10 .20	.75 1.25	.04	.05	.50 .90		.50 .90		.30 max.		.05 .15	.04 max.	
	E	.15 max.	1.20 max.	.04	.05	.30 max.	.75 1.25		.08 .25	.50 .80	.05 max.			
	F	.10 .20	.50 1.00	.04	.05	.30 max.	.40 1.10	.30 max.	.10 .20	.30 1.00	.01 .10			
	G	.20 max.	1.20 max.	.04	.05	.25 .70	.80 max.	.50 1.00	.10 max.	.30 .50				.07 max.
	H	.20 max.	1.25 max.	.035	.040	.25 .75	.30 .60	.10 .25	.15 max.	.20 .35	.02 .10			.005 .030
	J	.17 max.	.60 1.00	.04	.05	.30 .50	.50 .70			.30 min.				.030 .050
	K	.20 max.	.50 1.20	.04	.05	.25 .50	.40 max.	.40 .70	.10 max.	.30 .50			.005 .05	

Table 5. Changes in Chemical requirements of A588 steels, 1968-1983.

Grade	Year of Change	Element	Previous Content, %	New Content, %
A	1977a	C	0.10 - 0.19	0.19 max.
		Mn	0.90 - 1.25	0.80 - 1.25
		Si	0.15 - 0.30	0.30 - 0.65
		Ni	none	0.40 max.
B	1970a 1977a 1980a	C	0.10 - 0.20	0.20 max.
		Ni	0.25 - 0.50	0.50 max.
		Mn	0.75 - 1.25	0.75 - 1.35
		Si	0.15 - 0.30	0.15 - 0.50
D	1977a	Cr	0.50 - 0.75	0.50 - 0.90
E	1977a	Si	0.15 - 0.30	0.30 max.
		Mo	0.10 - 0.25	0.08 - 0.25

Table 6. Relative contents of alloying elements in A588 steels.

Element	Grades							
	A,B,K	C	D	E	F	G	H	J
Si	med	low	high	low	low	med	med	med
Ni	low	low	-	high	high	med	med	med
Cr	med	med	high	-	low	high	low	-
Cu	med	med	low	high	high	med	med	high

Table 7. Tensile requirements for all grades of A588 steel.

Structural Shapes	Plates & Bars Thickness	Tensile Strength	Yield Point	Elongation in 200 mm (8 in)	Elongation in 50 mm (2 in)
	mm (in)	MPa (ksi)	MPa (ksi)	min, %	min, %
	t<102 (\bar{E} <4)	485 (70)	345 (50)	18	21
	102<t<127 (4<t<5)	460 (67)	315 (46)	-	21
	127<t<204 (5<t<8)	435 (63)	290 (42)	-	21
All groups		485 (70)	345 (50)	18	21

grades. For example, Grades A, B, and K combine low nickel with medium silicon, chromium, and copper contents. The largest contrast is apparent by comparing Grades D and E. Grade D is low in copper and high in silicon and chromium; it has no nickel. Grade E is low in silicon, and high in nickel and copper; it has no chromium.

It should be kept in mind that these elements also influence, in combination with others, the tensile and impact properties of the steel, not just the corrosion resistance. The producers balance the contents of all alloying elements to achieve the desired overall behavior.

Tensile Requirements

Table 7 gives the current tensile requirements for A588 steel. The tensile strength and yield point are the same as those specified in 1968, with one exception. In 1974, the tensile strength and yield point of Group 5 structural shapes were increased from 67 ksi (460 MPa) and 46 ksi (315 MPa), respectively, to the uniform values of 70 ksi (485 MPa) and 50 ksi (345 MPa) applicable today for all groups. In addition, the elongation of plates and bars in 8 in. (200 mm) was lowered from the original 19 percent to 18 percent; and the elongation of structural shapes in 2 in. (50 mm) was raised from the original 19 percent to 21 percent.

Supplementary Requirements

Similarly to Specification A242, the bend test requirement in Specification A588 was mandatory from the first edition to 1974. Thereafter it became optional at the request of the purchaser. The additional standardized requirements, for use at the option of the purchaser, were adopted in 1977 (S2, S3, S5, S6, and S14) and in 1979 (S15 and S18), all of which are listed in Specification A6.

ASTM A709 STEELS

Scope

In 1974, ASTM issued Standard Specification A709 for "Structural Steel for Bridges." This specification covers carbon and high-strength low-alloy steel for structural shapes, plates, and bars and quenched and tempered alloy steel for structural

plates intended for use in bridges. The five available Grades 36, 50, 50W, 100, and 100W are equivalent to A36, A572, A588, and A514, respectively. Grade 100W corresponds to A514 with atmospheric corrosion resistance. When the supplementary requirements are specified, the steels exceed the requirements of Specifications A36, A572, A588, and A514. Specification A709 conveniently allows the purchaser to specify a bridge steel with the desired tensile requirements, impact test requirements for several temperature zones, and atmospheric corrosion resistance. There are no grade distinctions based on proprietary chemical compositions as is the case with Specifications A588 and A514. The temperature zones are those chosen by AASHTO and are in relationship to the lowest ambient temperature expected for that area. Specification A709 has not changed significantly since 1974; however, a revised version to include complete chemical analysis requirements, including proprietary grades in A588 and A514, is being considered by ASTM Subcommittee A01.02 on Structural Steels.

Steel grades with suffix "W" provide atmospheric corrosion resistance about two times that of carbon structural steel with copper, which is equivalent to four times carbon structural steel without copper (Cu 0.02 percent max.). When required, the manufacturer must supply evidence of corrosion resistance satisfactory to the purchaser.

Chemical Requirements

Table 8 gives the basic chemical requirements for carbon, manganese, phosphorus, and sulfur of Grades 50W and 100W. Specification A709 contains some additional requirements for grain-refining elements (aluminum, columbium, vanadium, and

Table 8. Chemical composition of A709 steels with enhanced atmospheric corrosion resistance.

Grade	Thickness mm (in.)	Composition, % (Ladle Analysis)			
		C	Mn	P, max.	S, max.
50W	102 (4)	0.20 max.	1.35 max.	0.04	0.05
100W	102 (4)	0.10 - 0.21	0.40 - 1.50	0.035	0.04

titanium); as well as carbon, manganese and silicon depending on material thickness, toughness, and treatment. The choice and use of alloying elements, combined with carbon, manganese, phosphorus, and sulfur within the limits prescribed to conform to the tensile and mechanical properties or to enhance the atmospheric corrosion resistance, or both, may vary by manufacturer.

Tensile Requirements

The tensile requirements for Grades 50W and 100W, given in Table 9, are the same as those for A588 and A514 steels.

Supplementary Requirements

The purchaser may specify supplementary requirements regarding fine grain practice, Charpy V-notch impact energy absorbed, supplying evidence of atmospheric corrosion resistance, and ultrasonic examination.

FOREIGN WEATHERING STEELS

Tables 10 and 11, respectively, summarize the chemical and tensile requirements stated in the weathering steel specifications issued by Canada [CSA 1976], United Kingdom [BS 1969],

Table 9. Tensile requirements for A709 steels with enhanced atmospheric corrosion resistance.

Grade	Plate Thickness	Structural Shapes	Yield Point/Strength, min.	Tensile Strength,	Minimum Elongation, %		Reduction of area min. (%)	Brinell Hardness Number
	mm (in)		MPa (ksi)	MPa (ksi)	in 200 mm (8 in)	in 50 mm (2 in)		
50W	t ≤ 102 (4)	Groups 1-5	345 (50)	485 (70) min.	18	21	-	-
100W	t ≤ 64 (2-1/2)	-	690 (100)	760-895 (110-130)	-	18	40-50	285-293
100W	64 < t ≤ 102 (2-1/2 < t ≤ 4)	-	620 (90)	690-895 (100-130)	-	17	50	-

Notes:

- (a) The elements columbium and vanadium may be used singly or in combination up to the total of 0.10% max., except that if columbium is used singly or in combination with vanadium in plates thicker than 12 mm or shapes heavier than Group 1, the silicon content shall be 0.15% min. Aluminum may be used as a grain refining element without prior approval by the purchaser, and when so used, shall not be included in the summation of grain refining elements.
- (b) The combined content of chromium, nickel and copper shall be not less than 1.00%.
- (c) The combined content of chromium and nickel shall be not less than 0.40%.
- (d) At the purchaser's request or at the producer's option, the steel may be made with no minimum silicon content provided that the steel contains a minimum of 0.02% total aluminum content.
- (e) WR 508 may also be supplied in normalized condition.
- (f) Steels of Type 4, 83 and 84 shall contain at least one of the following grain refining elements in the indicated percentages: Al total ≥ 0.015%, Nb = 0.015 - 0.060%, V = 0.02 - 0.15% and Ti = 0.02 - 0.10%.
- (g) The steel shall contain nitrogen oxidizing elements in sufficient content to refine the grains.
- (h) Content to be agreed upon.
- (i) KT 45-2 steel shall be killed with aluminum, maintaining the relationship $Al/3 + V/4 \geq N$.
- (j) One or more elements among Mo, Nb, Ti, V and Zr shall be added. The total content of these elements shall not exceed 0.15 percent.

Table 10. Chemical requirements for foreign weathering steels for bridges.

Type or Grade	Composition, %														Comments	
	C	Mn	P	S	Si	Ni	Cr	Mo	Cu	V	Zr	Cb	Ti	Al		N
Canada: G40.21 [CSA 1976]																
50A	.20 max	.75 1.35	.03 max	.04 max	.15 max	0.90 ^C .90 max	0.70 ^C .70 max		.20 .60	a		a		a		Killed; fine grain practice
60A	.20 max	.75 1.35	.03 max	.04 max	.15 .40	.90 ^C max	.70 ^C max		.20 .60	a		a		a		Killed; fine grain practice
United Kingdom: BS 4360 [BS 1969]																
WR50 B&B1	.10 .19	.90 1.25	.04 max	.05 max	.15 .50		.40 .70		.25 .40	.02 .10				.01 .06		As rolled; killed, e
WR50C	.10 .22	.90 1.45	.04 max	.05 max	.15 .50		.40 .70		.25 .40	.02 .10				.01 .06		As rolled or normalized; killed
WR50C1	.10 .19	.90 1.25	.04 max	.05 max	.15 .50		.40 .70		.25 .40	.02 .10				.01 .06		As rolled or normalized; killed
France: A35-502 [AFNOR 1979]																
E24W 2,3&4	.13 max	.20 .60	.04	.035	.10 .40	.65 max	.40 .80		.20 .55	f						
E36W B3&B4	.19 max	.50 1.50	.04 max	.04 max	.50 max	.65 max	.40 .80	.30 max	.20 .55	.15 max						
Federal Republic of Germany: DAST 007 [DAST 1979]																
WT St- 37-2	.13 max	.20 .50	.05 max	.035 max	.10 .40		.50 .80		.30 .50						0.007	
WT St- 37-3	.13 max	.20 .50	.045 max	.035 max	.10 .40		.50 .80		.30 .50						0.009	g
WT St- 52-3	.13 max	.90 1.30	.045 max	.035 max	.10 .50		.50 .80		.30 .50	.02 .10					0.009	g
German Democratic Republic: TGL 28192 [DDR 1973]																
KT 45 2 & 3	.12 max	.30 .60	.05 .09	.04 max	.25 .50		.50 .80		.30 .50	.03 ^h min				0.02 ⁱ	0.008 0.016	
KT 52-3	.12 max	h	.05 .09	.04 max	.25 .50		.50 .80		.30 .50	h				h	h	
Czechoslovakia: [CSSR 1978]																
Atmofix 37	.12 max	.20 .55	.04 max	.04 max	.10 .35	.20 .40	.30 .60		.30 .50							
Atmofix 52B	.10 .17	.90 1.20	.04 max	.04 max	.20 .45	.30 .60	.40 .80		.30 .55	.02 .06		.02 .06		.01 min		
Atmofix 60	.16 max	1.10 1.50	.04 max	.04 max	.20 .40	.30 .50	.60 1.00		.30 .50			.03 .08		.01 min		
Japan: JIS G3114 (Non-painted application)^j [JIS 1981]																
SMA 41 A, B&C	.18 max	1.25 max	.035 max	.035 max	.15 .65	.05 .30	.45 .75		.30 .50							
SMA 50 A, B&C SMA 58	.18 max	1.40 max	.035 max	.035 max	.15 .65	.05 .30	.45 .75		.30 .50							
Japan: JIS G3114 (Painted application)^j [JIS 1981]																
SMA 41 A, B&C	.18 max	1.25 max	.035 max	.035 max	.55 max		.30 .55		.20 .35							
SMA 50 A, B&C SMA 58	.18 max	1.40 max	.035 max	.035 max	.55 max		.30 .55		.20 .35							

France [AFNOR 1979], Federal Republic of Germany [DAST 1979], German Democratic Republic [DDR 1973], Czechoslovakia [CSSR 1978], and Japan [JIS 1981]. This information is provided to show that (1) domestic and foreign steels are similar in composition and strength; and (2) any differences in corrosion resistance and fatigue behavior, examined in Chapters Three through Five, Seven, and Eight, are mainly a result of factors

other than type of steel. Only types/grades of weathering steels that appear to be used for bridges were included. Among the steels that were deleted are those with maximum phosphorus content greater than 0.10 percent and those supplied in plate thicknesses of $\frac{1}{2}$ in. (12 mm) or less. These types/grades, like ASTM A242 Type 1, are mainly used for architectural applications.

Table 11. Basic tensile requirements for foreign weathering steels for bridges.

Type or Grade	Structural Shapes	Plates & Bars Thickness mm (in.)	Tensile Strength MPa (ksi)	Yield Point MPa (ksi)	Max. Thickness Available ^a
Canada: G40.21 [CSA 1976]					
50A	All groups	t<102 t \geq (4)	480 - 660 (70 - 95)	345 (50)	102 (4)
60A	Groups 1&2	t<38 t \geq (1-1/2)	520 - 695 (75 - 100)	415 (60)	38 (1-1/2)
United Kingdom: BS 4360 [BS 1969]					
WR 50 B, B1, C&C1		t<40 t \geq (1-1/2)	480 (70)	345 (50)	50 (2)
France: A35-502 [AFNOR 1979]					
E24W 2, 3, & 4	t<30 t \geq (1-3/16)	3<t<30 (1/8<t<1-3/16)	360 - 460 (70 - 84)	235 (52)	110 (4-5/16)
E36W A2, A3, A4, B3, & B4	t<30 t \geq (1-3/16)	3<t<30 (1/8<t<1-3/16)	480 - 580 (70 - 84)	355 (52)	110 (4-5/16)
German Democratic Republic: TGL 28192 [DDR 1973]					
KT 45 2&3		t<16 t \geq (5/8)	440 - 590 (64 - 85)	295 (42)	40 (1-1/2)
KT 50-2		t<16 t \geq (5/8)	490 - 640 (70 - 92)	345 (50)	16 (5/8)
KT 50-3		t<16 t \geq (5/8)	510 - 610 (74 - 88)	355 (51)	40 (1-1/2)
Japan: JIS G3114 [JIS 1981]					
SMA 41 A, B, & C		t<16 (t \leq 5/8)	420 - 539 (58 - 78)	245 (36)	
		16<t<40 (5/8<t<1-9/16)	402 - 539 (58 - 78)	235 (34)	
		t>40 (t>1-9/16)	420 - 539 (58 - 78)	216 (31)	
SMA 50 A, B, & C		t<16 (t \leq 5/8)	490 - 608 (71 - 88)	363 (53)	
		16<t<40 (5/8<t<1-9/16)	490 - 608 (71 - 88)	353 (51)	
		t>40 (t>1-9/16)	490 - 608 (71 - 88)	333 (48)	
SMA 58		t<16 (t \leq 5/8)	569 - 716 (83 - 104)	416 (67)	
		16<t<40 (5/8<t<1-9/16)	569 - 716 (83 - 104)	451 (65)	
		t>40 (t>1-9/16)	569 - 716 (83 - 104)	431 (63)	

Note:

- (a) Tensile strength and yield point may be lower when the material thickness exceeds the value listed in the third column of the table.

The various type/grade designations identify steels with different levels of tensile requirements. Each country has a weathering steel with basic tensile strength and yield point, respectively, comparable to the values of 70 ksi (485 MPa) and 50 ksi (345 MPa) for ASTM A242 and A588 steels. Some have in addition higher and lower grades, with yield points ranging from 34 ksi (235 MPa) [AFNOR 1979] to 67 ksi (461 MPa) [JIS 1981]. Listed in Table 11 for reasons of brevity are only the tensile requirements for the smallest material thickness. The requirements are usually lower for thicker plates and sections.

The type/grade designation implies in several cases a minimum level of Charpy V-notch energy absorbed at specified service temperatures. These values are stated in the corresponding references.

Domestic and foreign weathering steels have similar contents of alloying elements added mainly for improving corrosion resistance. Most have, on average, 0.30 to 0.50 percent copper, 0.04 percent max. phosphorus, and greatly varying contents of nickel, chromium, and silicon. As will be shown in Chapter Three, a desired level of corrosion resistance can be achieved with many different combinations of these elements.

Like the ASTM standards, the Canadian standard specifies that the corrosion resistance of weathering steel is approximately four times that of carbon steel (0.02 percent Cu). The other foreign standards examined herein do not quantitatively specify the corrosion resistance.

EXPERIENCES

DOMESTIC USE

The extent to which weathering steel (unless noted otherwise, the term weathering steel should be interpreted as being in the unpainted condition) is used for bridges on state highway systems was determined from telephone interviews with officials in all 50 States and the District of Columbia. Table 12 summarizes the findings. In most cases the findings are the same as those obtained in separate surveys by Bethlehem Steel and the AASHTO Subcommittee on Maintenance [Bethlehem 1982, AASHTO 1983].

According to the present survey, there are about 1,800 weathering steel bridges on state highway systems, not including county, city, tollroad or mass transit bridges. Forty-three of 50 States and the District of Columbia have weathering steel bridges. Of those, 31 states and the District of Columbia foresee using bare weathering steel for new bridges, 2 states are hesitant, and 10 states have stopped using it. Indiana, Iowa, and Michigan have official policies banning the use of unpainted weathering steel for bridges on the state highway systems. There are several satisfied users among the 31 states and the District of Columbia who continue to build weathering steel bridges. Vermont, for example, specifies it almost exclusively for steel bridges. Many states carefully select the sites, avoiding industrial environments with severe atmospheric pollution, marine environments along the seashore and water ways, urban areas with high deicing salt use, and inland sites with high humidity and rainfall. Some states paint the steel girders up to 10 ft (3 m) on either side of joints. For aesthetic reasons Illinois, Missouri, and Wisconsin use weathering steel mainly for stream crossings; Wisconsin, also for grade separations in remote areas. The overriding reason why these States continue to specify weathering steel is the potential of saving the cost of maintenance painting. By virtue of having generally favorable environments or by carefully applying the material, they are able to make the concept of bare weathering steel construction work.

Of the 10 states who have stopped using weathering steel, Alabama, Indiana, Iowa, Michigan, Mississippi, and West Virginia have done so because of corrosion problems encountered in roadway deicing salt environments. Alaska discontinued the use because of the high cost of steel and a generally humid environment along its southeastern coast. The long life of painting systems in the dry climates of Montana makes painted steel construction more economical than bare weathering steel construction. In North Dakota concrete bridges are more economical than steel bridges.

That leaves 7 states who have never used bare weathering steel. Their main reasons are as follows. Arizona, Nevada, and New Mexico have dry climates in which painting systems last indefinitely and weathering steels would not for a long time develop an oxide coating of uniform appearance. Steel is expensive in Hawaii, relative to concrete. New Jersey and South

Dakota question the corrosion performance, particularly in salt environments, and dislike the rust staining of the concrete substructure. South Carolina takes the cautious approach of not adopting it as long as others are having problems.

Many steel users, including those who have rejected bare steel construction, have specified for economic reasons painted A588 steel either outright or as substitute for A572 in thicknesses greater than 2 m. (50 mm). This is done when the savings in weight is greater than the additional cost of high-strength low-alloy steel, as compared to A36 steel.

DOMESTIC EXPERIENCE

The majority of bare weathering steel bridges built in the United States are performing in a satisfactory manner. But the application of atmospheric corrosion resistant steels has not been without problems.

Michigan

During a lengthy evaluation period, which began with inspections of Detroit's Eight-Mile Road Bridge, Michigan DOT officials found that corrosion rates were not tapering off. As a result, the State of Michigan instituted in June 1979 a partial moratorium on the use of A588 steel in the unpainted condition for bridges in the following situations [VanKampen 1979]:

1. Depressed roadway sections where low underclearance (less than 20 ft (6.1 m)) and vertical retaining walls trap salt sprays and other atmospheric pollutants.
2. Urban and industrial areas where heavy roadway salting and automotive and industrial pollution create an aggressively corrosive environment.

Nine months later, in March 1980, the moratorium was extended to include all uses of unpainted A588 steel on the state highway system [VanKampen 1980]. This ban included rural bridges not affected by the earlier moratorium. The findings and conclusions that led to the moratoria were thoroughly documented and explained [Culp 1980, Arnold 1980, Allemeier 1981]. The following forms of corrosion were observed:

1. By far the most serious problem is salt contamination in the following manner: (a) runoff water drains onto the superstructure through expansion joints and leaky seals; and (b) spray kicked up by traffic passing under the bridge accumulates on the superstructure. Corrosion attack by leakage is more apparent because it is concentrated, and the steel area affected is a small percentage of the total area. It is prevalent below the leaks, but the salt water can advance for long distances along the bottom flange, both up and down grade from the source. Corrosion

Table 12. Summary of use of bare weathering steel bridges on federal highway systems.

State	Weathering Steel Bridges in Service				New Weathering Steel Bridges	
	Previously Used	No. of Bridges	Used Since	Comment	Future Use	Comment
1. Alabama	Yes	5	1980	--	No	Concerned about experiences by others.
2. Alaska	Yes	2	1973	--	No	Concrete is more economic; unsuitable for humid SE coastal area.
3. Arizona	No	--	--	Dry climate; paint systems last indefinitely; nonuniform weathering.	No	(Same)
4. Arkansas	Yes	50	1970	--	Yes	Used mainly where maintenance is difficult
5. California	Yes	1	1978	--	Yes	Careful site selection.
6. Colorado	Yes	25	1975	--	Yes	--
7. Connecticut	Yes	40	1970	--	Yes	Used mainly where maintenance is difficult.
8. Delaware	Yes	10	1975	--	Yes	--
9. District of Columbia	Yes	3	1978	--	Yes	--
10. Florida	Yes	2	1972	12 painted W/S bridges.	No	Unattractive.
11. Georgia	Yes	70	1976	Used in nonhostile environments.	Yes	--
12. Hawaii	No	--	--	Last steel bridge built in 1965.	No	Concrete is more economic.
13. Idaho	Yes	15	1970	Not used near smelters and processing plants.	Yes	(Same)
14. Illinois	Yes	31	1968	Mainly rural bridges over streams.	Yes	(Same)
15. Indiana	Yes	10	1970	--	No	Policy based on Michigan experience.
16. Iowa	Yes	5	1964	--	No	Policy.
17. Kansas	Yes	22	1971	Mainly painted hybrid girders in urban areas.	Yes	--
18. Kentucky	Yes	2	1971	--	Yes	--
19. Louisiana	Yes	16	1975	--	Yes	--
20. Maine	Yes	60	1968	--	Yes	--
21. Maryland	Yes	66	1973	--	Yes	--
22. Massachusetts	Yes	20	1972	Not used over stream crossings.	Yes	(Same)
23. Michigan	Yes	480	1965	--	No	Policy; severe corrosion effect of deicing salts.
24. Minnesota	Yes	60	1973	--	Yes	Painted in metropolitan, bare in rural areas.
25. Mississippi	Yes	16	1978	--	No	Salt usage in ice storms.
26. Missouri	Yes	20	1973	Mainly rural bridges over streams.	Yes	(Same)
27. Montana	Yes	2	1973	Initially nonuniform weathering.	No	Dry climate; paint systems last 30 years.
28. Nebraska	Yes	22	1969	--	Yes	--
29. Nevada	No	--	--	Dry climate; non-uniform oxide coating; esthetics.	No	(Same)
30. New Hampshire	Yes	100	1973	--	Yes	--

Table 12. Continued

State	Weathering Steel Bridges in Service				New Weathering Steel Bridges	
	Previously Used	No. of Bridges	Used Since	Comment	Future Use	Comment
31. New Jersey	No	--	--	Concern about loss of section due to corrosion.	No	(Same)
32. New Mexico	No	--	--	Dry climate; paint systems have long life.	No	(Same)
33. New York	Yes	70	1969	--	Yes	--
34. North Carolina	Yes	150	1968	--	Yes	--
35. North Dakota	Yes	3	1970	Semi-arid area; snow is mostly plowed.	No	Not economic compared with concrete.
36. Ohio	Yes	55	1971	--	Yes	--
37. Oklahoma	Yes	4	1973	Small jobs bare; large jobs painted.	Under evaluation	--
38. Oregon	Yes	3	1975	--	Yes	--
39. Pennsylvania	Yes	12	1968	--	Yes	Except in polluted and humid areas. Difficult to inspect.
40. Rhode Island	Yes	9	1975	--	Yes	--
41. South Carolina	No	--	--	Esthetics; problems in other states	No	(Same)
42. South Dakota	No	--	--	Esthetics; experience in other states with deicing salts.	No	(Same)
43. Tennessee	Yes	10	1974	--	Under evaluation	Esthetics
44. Texas	Yes	16	1974	--	Yes	--
45. Utah	Yes	14	1970	--	Yes	--
46. Vermont	Yes	100	1973	Used extensively	Yes	(Same)
47. Virginia	Yes	45	1972	--	Yes	--
48. Washington	Yes	15	1967	--	Yes	--
49. West Virginia	Yes	10	1972	--	No	Experience in other states
50. Wisconsin	Yes	25	1971	For stream crossings & grade separations in remote areas.	Yes	(Same)
51. Wyoming	Yes	20	1975	Dry climate	Yes	--

attack by spray can have just as great an effect as corrosion by leakage. It covers most steel surfaces. It is most severe over the traffic lanes and when the roadway is depressed and laterally confined by retaining walls.

2. Corrosion products, salt, dust, bird droppings, and other debris continuously accumulate, especially on top of sheltered bottom flanges. They trap moisture and create a corrosive poultice. Rain can only wash off debris on the outside surfaces of the fascia girders.

3. The capillaries in the oxide coating draw water and dissolved deicing salts several inches up the web, corroding the lower web strip along the flange-to-web junction more than the upper web surfaces.

4. The narrow gaps between loosely matched hanger plates, pin connections, and attachments foster crevice corrosion, an accelerated form of attack. The pressure of the corrosion products froze some hinges and failed pin nuts.

5. The surfaces of A588 steel were pitted, especially when

contaminated with salt. This raises a concern about the fatigue resistance which is known to be reduced by pitting.

6. The electrical potential difference between the mill scale and the base metal selectively corrodes the base metal adjacent to the mill scale. This galvanic corrosion caused widespread pitting on some bridges that were exposed to a rural environment, without heavy exposure to salt.

The protective oxide coating does not form where the steel is severely corroding. The flaky and laminar rust is periodically shed. Under these adverse conditions unpainted A588 steel is corroding at about the same rate as unpainted carbon steel. The Michigan DOT is now retrofitting the steel bridges that are in the worst condition. This includes replacement of joint seals and maintenance painting of girders. This rehabilitation program is expected to last many years since nearly one-fourth of all weathering steel bridges on the federal highway system are located in Michigan.

AISI Study

The Michigan ban focused general attention on the performance of weathering steel bridges. Under the auspices of the American Iron and Steel Institute, a task group of steel company specialists and state and federal highway officials inspected 49 weathering steel bridges in Illinois, Maryland, Michigan, New York, North Carolina, Wisconsin, and New Jersey Turnpike Authority [AISI 1982]. They found that:

1. Thirty percent of the bridges showed good performance in all areas.
2. Fifty-eight percent of the bridges showed good overall performance with moderate corrosion in some areas.
3. Twelve percent of the bridges showed good overall performance with heavy local corrosion in some areas.

The inspection reports indicated that the major portion of the steel had developed the expected oxide coating. One bridge on NY 8 with minimum clearance over the polluted Nine Mile Creek, near Syracuse, and several Michigan bridges did not develop the expected oxide coating. Other bridges exhibited moderate to heavy corrosion near leaky expansion joints in the bridge deck. The factors found to be responsible for the formation of the nonadherent flaky rust are the same as those described in the earlier Michigan report [Culp 1980].

Most bridges that were inspected did not need immediate attention or overall painting. Notable exceptions are most of the structures inspected in the Detroit, Michigan, area [AISI 1982].

The high rate of salt application in Wayne County and Detroit, the widespread use of cantilevered/suspended spans, and the large number of weathering steel bridges have resulted in a greater number of known corrosion incidents in Michigan than in other states.

However, the problems arising from unsuitable applications of weathering steel are not limited to Michigan alone. There are known cases of weathering steel bridges in Alaska, California, Indiana, Louisiana, and Ohio that have not performed to expectation. Some of them have remained practically unknown to the engineering profession. For example, the weathering steel bridge on County Road 37 over Little Still Water Creek, Tuscarawas County, Ohio, was built in 1973 and had to be painted in 1979, before the Michigan DOT reported on the condition of their weathering steel bridges [Smith 1983]. Information on the bridges in the aforementioned States are summarized in the following to illustrate the conditions unfavorable to the development of a protective oxide coating. These cases were not reported in Ref. AISI 1982.

Alaska

In 1972, four A588 Grade A steel bridges were built on the Big Salt Lake Road (FAS 929) east of Klawock, Prince of Wales Island, Alaska. They span across Little Salt Creek, Duke Creek, Black Bear Creek, and Steelhead River. They consist of two longitudinal plate girders with transverse floor beams resting on the top girder flanges. The floors are longitudinally laminated treated timber.

An inspection of the bridges in 1981 by Alaska and FHWA

officials found significant corrosion of the steel. It was most prominent on top of the bottom flange, on the exterior portion of the seaward girder. This part of the flange was generally covered with a $\frac{1}{4}$ to $\frac{3}{8}$ -in. (6 to 10-mm) thick layer of debris consisting of a mixture of dirt and rust scale. The rust-scale represented the major portion of the debris and had fallen off the lower 2 to 3 ft (0.6 to 0.9 m) of the web. The corrosion was most severe near the web-flange junction. At one location, a $4 \times 8 \times \frac{1}{16}$ in. (100 \times 200 \times 1.6 mm) thick rust sheet was removed intact from the flange. Rust pieces of similar thickness could also be removed from the surface of the web-flange fillet weld. The bridges were also corroding significantly at locations free of accumulated debris. All bridges are within $\frac{1}{4}$ mile (400 m) of salt water, and the area receives 150 to 180 in. (3.8 to 4.6 m) of annual rainfall.

The following findings were based on metallography, scanning electron microscopy, X-ray elemental analysis, and ultrasonic thickness measurements performed by a consultant [Kasza 1983]:

1. The composition of the steel was within the range specified by ASTM for A588 Grade A, with the exception of a slight reduction in silicon which is unlikely to affect the corrosion performance.
2. The scale was a corrosion product of the base metal, not mill scale as had been suggested.
3. The corrosion products were primarily crystalline iron oxide which is nonprotective. They contained only traces of chloride and sulfur. Absence of significant levels of those corrosive elements indicates that the prolonged time of wetness, without adequate drying time, was the main reason for the rapid corrosion.
4. The steel was corroding at an estimated rate of 2 mils (50 μ m) per year and per surface. That is an order of magnitude higher than the normal corrosion rate expected of weathering steels with protective oxide coatings. The total section loss could not be determined because the initial thickness of the steel had not been measured. After 10 years of exposure, the section loss below nominal design values was too small to affect the load carrying capacity (0.013-in. (0.33-mm) maximum loss in thickness of nominally 0.563-in. (14.3-mm) thick web).

The high annual rainfall and the proximity to bodies of water create a humid environment along the southeastern coast of Alaska. Wind-driven rain, fog, mist, and possibly water leaking through the deck frequently wet the steel. In addition, in humid environments steel surfaces condense water during the night when the temperature drops by, say, 15 F (8C). This greatly extends the time of wetness. The condensed water slowly runs down the web. Because the web dries from the top down, the lower web stays wet longer and corrodes more. The accumulated debris on the lower flange traps moisture and aggravates flange corrosion. Under these conditions, weathering steels corrode at a rate comparable to that of unpainted carbon steel. The four Alaska bridges are scheduled for remedial painting in 1984.

This case study is important because it is well documented [Kasza 1983]. It shows that weathering steel does not develop a protective oxide coating if the time of wetness is long. Salt contamination was not a significant factor as it had been in Michigan.

California

Five A588 Grades A and H steel bridges in Redwood National Park, California, have superstructures consisting of rolled beams and 2×8 in. (50×200 mm) or 2×10 in. (50×250 mm) nail-laminated timber decks. One was built in 1973 and four in 1975. The first (Prairie Creek Bridge) is located more in the open, appears to be more often dry, and is performing well. The other four are on Lost Man Creek, in a different drainage on the east side of the valley, which is much wetter and has a dense flora more like that of a rain forest. These environmental conditions are similar to those found in Southeast Alaska, except that the Redwood Park bridges are 10 to 15 miles (16 to 24 km) from the ocean, whereas those in Alaska were within sight of the ocean.

An inspection of the bridges by FHWA officials in 1981 revealed that the interior girders and the inside face of the exterior girders of two bridges on Lost Man Creek were severely corroding [Kasza 1983]. Large pieces of rust separated from the top flange and large rust flakes fell off the web. Debris was accumulating along the full length of the lower flange, about $\frac{1}{2}$ in. (12 mm) deep.

The exterior face of the girders could not be inspected because it is clad with slabs of redwood logs to make the structure look like a log stringer bridge. The humidity trapped in the logs is certain to foster corrosion of the underlying steel.

Metallurgical examinations similar to those performed on the Alaska bridges showed that: (1) the rust scale consisted of non-protective iron oxides of the base metal, not mill scale; (2) there was no evidence of any significant concentrations of chlorides and sulfides; and (3) the corrosion rate was about 2.5 mils (75 μ m) per year and per surface.

It appears that frequent rainfall, fog and high humidity are not letting the steel dry. The long time of wetness is fostering corrosion at a rate comparable to that of carbon steel.

Iowa

Four of the five weathering steel bridges on the state highway system were built between 1971 and 1982. Inspection of these bridges in 1983 by state highway officials showed that all four have tight rust of dark brown color and no evidence of unusual corrosion [Sheeler 1983]. The chloride content of rust samples taken from these four bridges varied from none to 0.11 percent.

The fifth and oldest bridge, built in 1964, is located on Iowa 28 over Racoon River, West Des Moines, in a rural/urban environment. It has been subjected to heavy salting. Most of the steel is in the normal light rust condition, except under expansion joints where the rust is flaking and the steel is pitting. An area of flaky rust also extends along the outside web, 2 to 3 in. (50 to 75 mm) above the bottom flange, and runs the full length of the outside stringers. There are minor areas of chloride drainage through the deck. Analysis of rust samples indicated 0.56 percent chloride content at an abutment, 0 percent on the web of an inside stringer, and 0.52 percent at the flange weld. Chloride contents of 0.20 percent are considered to be significant. The Racoon River Bridge is scheduled for remedial painting in 1984.

Louisiana

Sixteen weathering steel bridges were built in Louisiana since

1975. It was decided in 1976 to inspect these bridges on an annual basis, for a minimum of 10 years, to ensure that maintenance painting could be eliminated and that no additional maintenance problems, unknown at that time, would be inadvertently created. In 1983, 8 years later, it appears that 13 bridges were performing as expected, whereas 3 bridges were not. The Doullut Canal Bridge, located 40 miles (64 km) S-SE of New Orleans, was remedially painted in 1983. The Larose Bridge, 30 miles (48 km) S-SW of New Orleans, is showing signs of unstable corrosion. Sections of the newly erected Luling Bridge, 18 miles (29 km) W of New Orleans, were at first severely corroding because of sea-water contamination during shipping from Japan [Garrido 1983, Dunn 1983].

The Doullut Canal Bridge, on Louisiana Route 23 in Empire, is located in the lower Mississippi River delta several miles from the Gulf of Mexico and near bodies of brackish water. The five-span plate girder bridge, made of A588 Grade A steel, has a 55-ft (16.8-m) clearance over the Doullut Canal.

The boldly exposed surfaces of the northeast exterior girder exhibited a tight oxide of normal appearance. This surface is sheltered from southerly wind-blown fog and receives drying heat from the morning sun. All other steel surfaces were covered with flaky rust. The flakes ranged in size from $\frac{1}{8} \times \frac{1}{8}$ in. (3×3 mm) to $\frac{1}{4} \times \frac{1}{2}$ in. (6×12 mm). The steel surface beneath the rust was covered with broad, shallow pits. Wet poultices of rust flakes were accumulating on the top of the lower flanges and on horizontal gusset plates of lateral bracing members. At both locations, nuts partially covered by accumulated rust-flakes were scaling. The threads and the bolts ends protruding from some nuts had corroded away. The chloride content of rust flakes (0.09 and 0.22 percent) and poultice samples (0.11 percent) indicated significant chloride contamination.

The adherent oxide coating had not formed on the majority of the surfaces of the Doullut Canal Bridge because of location-related salt contamination of the steel, high incidence of fog, and high humidity. The bridge was sand blasted and painted (see Ch. Nine) in 1983 at a cost of \$195,400 for 17,075 U.S. tons (15,200 metric tons) of steel with 214,500 ft² (19,950 m²) surface area.

The Larose Bridge on Louisiana Route 657 over an Intra-coastal waterway is also located in the Mississippi River Delta, about 16 miles (24 km) from the Gulf. Some surfaces of this A588 Grade B steel bridge are developing a granular flaky rust layer that continuously scales off, leaving a pitted surface. However, the corrosion is less advanced than that at the Doullut Canal Bridge, likely because of its greater distance from the Gulf. Both are 8 years old.

The superstructure of the cable-stayed Luling Bridge was fabricated from A588 Grade A weathering steel. The sections were transported by ship from Japan and hosed with water on arrival in Louisiana, to remove the sea-salt residue that may have accumulated. There was no indication of severe corrosion when the steel arrived at the project site. The heavy flaking began to show up after erection had commenced. Not all sections were equally affected because some were transported in the holds of the ship, while others were on the deck and received more salt. After the corrosion problem became evident, the contact surfaces of the bolted joints of sections that had not yet been erected were sand blasted. Also sand blasted were all surfaces exposed to view. Before the epoxy wearing surface was applied, the steel deck was sand blasted to a near-white condition. It

was not always possible to remove all rust from the pits in the deck because the pits were smaller than the sand grains. There is no indication at the present time that the severe initial corrosion is reoccurring.

Ohio

There are known cases of weathering steel bridges that have performed poorly in Tuscarawas, Franklin and Butler Counties, Ohio.

The bridges on County Road 37 over Little Still Water Creek and on County Road 99 over Sugar Creek, Tuscarawas County, consist of 120-ft and 124-ft (36.6-m and 37.8-m) long trusses with wide flange members fabricated from A588 Grade A steel. They have about 12- to 15-ft (3.7 to 4.6 m) clearance above the creek. The County Road 37 Bridge, built in 1973, had corroded severely, with the worst locations being along the bottom chord, along the inside floor beams and at the truss joints. County officials stated that $\frac{1}{2}$ -in. (6.4-mm) thick rust pieces had come off the bottom chord and some sections had lost 15 percent of the steel. The corrosion was attributed mainly to prolonged time of wetness and insufficient drying of the steel. The bridge was remedially painted in 1979. The County Road 99 Bridge, built in 1979, was painted as a precautionary measure 2 months after it had been opened to service, to avoid the corrosion problems found in the County Road 37 Bridge [Smith 1983].

The three-span multigirder bridge on Brand Road over the North Fork of Indian Run, Franklin County, was fabricated from A588 Grade A steel. It was built in 1979. An inspection of the bridge in February 1983 revealed that the steel beams were rapidly corroding. Large sheets of rust were flaking off in several areas. Most severely affected were the lower one-third of the web and the lower flange in all three spans. The interior beams corroded more than the exterior beams. When the bridge was inspected at mid-morning, after the fog had dissipated, heavy condensation remained on the steel beams, and much moisture was found under the rust scale. Chemical analysis of rust scale samples removed from the beams revealed only traces of chlorides (0.01 percent) and sulfates (0.07 percent). The creek water was also found to be low in sulfates (0.18 ppm). Evidently, deicing salt and sulfates were not significant factors. The rapid rate of corrosion was caused by moisture present for prolonged periods of time. The low-lying bridge has only 8-ft (2.4-m) clearance over the stream, and the terrain limits sunlight and air circulation. The bridge was remedially painted in 1983 [Circle 1983].

Butler County, located north of Cincinnati, has remedially painted six of their ten weathering steel bridges in 1983. The Howard Road Bridge, built in 1971 over a stream with 10-ft (3-m) clearance, was badly corroding. The worst areas were along the lower part of the interior beams. After the steel had been blast cleaned it was found that corrosion had fully penetrated the 0.39-in. (9.9-mm) thick web of the W18 \times 55 beams at several locations. The largest rust hole was about 3 in. by 2 in. (75 mm long by 50 mm high). Parts of the beam webs were plated to avoid having to post a weight limit on the Howard Road Bridge [Brooks 1983]. The web penetration in 12 years corresponds to an average penetration of 16 mils (400 μ m) per year and per surface. This rate is about the same as the highest measured in Michigan. It may be an upper bound under adverse conditions of exposure.

Texas

A weathering steel bridge on State Route 124 over the Intracoastal Waterway in High Island, northeast of Galveston, is subjected to conditions similar to those previously described for the Louisiana bridges close to the Gulf of Mexico. The bridge is located 5 miles (8 km) from salt water in the prevailing wind direction and has an 85-ft (26-m) clearance. Materials and Tests Division officials of the State Department of Highways and Public Transportation, who have periodically inspected the bridges since 1979, reported loose rust flakes on the steel surfaces that range from $\frac{1}{8}$ to $\frac{1}{4}$ in. (1.6 to 3.2 mm) diameter. The steel surface under the flakes was covered with porous corrosion products instead of the dense oxide coating typical of good weathering steel performance. The steel surfaces are pitting. Observations of the corrosion process of various steels along the coast show that steel corrodes rapidly until a film of corrosion products builds up. Thereafter, the rate steadily decreases until the rust flakes. After flaking, the corrosion cycle starts again.

Test coupons were exposed 30 months on a rack mounted on the west main pier of the High Island Bridge. They were tensile tested in 1982. The loss of load carrying capacity, as compared to that of the nonweathered control coupons, was converted to equivalent loss in thickness. It was found that the coupons had corroded at a rate of 5.6 mils (142 μ m) per year and per surface [Raska 1983]. Some state officials believe that the bridge will have to be painted and that it would be more cost effective to paint it before pitting becomes severe.

Summary

The well-documented Michigan experience and the aforementioned corrosion problems with bridges in Alaska, California, Iowa, Louisiana, Ohio, and Texas indicate that weathering steel will not develop a protective oxide coating if the surfaces remain wet for extended periods of time or are subjected to salt contamination from any source. Under these conditions, weathering steel corrodes at a rate comparable to that of carbon steel, as is shown in Chapters Three through Five.

The aforementioned examples are intended to illustrate the conditions under which weathering steel has not performed as expected and should, therefore, not be used. It is well to remember that most weathering steel bridges built in favorable environments are performing well.

EUROPEAN EXPERIENCE

The experiences with bare steel construction in Europe are partly reflected in the weathering steel specifications that were issued by various countries. The following samples are indicative of the European provisions for the proper application of weathering steels.

France

The French specification for weathering steels cautions the user that the formation of the protective oxide coating depends not only on the chemical composition but also on environmental

factors (humidity, atmospheric pollution, and wet-dry cycles), overall and detail design, and exposure conditions [AFNOR 1979]. In applying weathering steel to structures, the user is directed to take all necessary precautions that will ensure the existence of conditions favorable to the formation of a protective coating. These measures must prevent ponding of water, trapping of humidity, and permanent humid conditions fostered by condensation or capillary action. Rain water must be able to run off, and humidity must be able to dry without hindrance. Particular attention should be given to low points of support and sheltered areas.

Federal Republic of Germany

The application of bare weathering steel in highway bridges on the federal system is only permitted with special approval of the Department of Transportation. This requirement virtually amounts to a ban. Since its inception in 1980, only two exceptions were granted, namely for composite girder bridges over electrified railways. This was done to avoid having to restrict railway traffic during maintenance painting, had the bridge been built of regular steel [DASt 1979, FRG 1982].

Weathering steel bridges built before 1980 are exhibiting large differences in corrosion performance. The relationship between macroclimate (rural, urban, industrial, and marine) and corrosion rate is found to be difficult to assess quantitatively. Poorly aerated and humid parts of the steel structure are performing very unsatisfactorily [FRG 1982]. The restrictions on the use of weathering steel were imposed because the corrosion rate of existing structures is not declining as anticipated. The steel periodically sheds the rust layer, depending on the atmospheric conditions at the bridge site.

The use of weathering steel has never been permitted for railway bridges because of concerns about its fatigue performance. For the same reason, exceptions are not granted for highway bridges with orthotropic weathering steel decks.

The weathering steel specifications set limits on atmospheric pollution [DASt 1979]. If these are exceeded, the steel must be painted. Corrosion protection is called for when the time of wetness is long or the steel is subjected to chloride contamination.

For purposes of designing load-carrying members, the thickness must be increased to account for net section loss. This allowance varies with service life and environment. It is at most $\frac{1}{16}$ in. (1.5 mm) per side for structures in industrial and marine environments with a 60-year service life (Fig. 47).

The corrosion performance of existing structures must be monitored with specimen prepared and installed at the site in accordance with set guidelines [DASt 1979].

German Democratic Republic

The conditions for the formation of a protective oxide coating are said to exist in the macroclimate of the German Democratic Republic, provided that: (1) the steel surface does not remain humid for extended periods of time; and (2) the atmospheres can be classified as rural and urban, as defined in Standard TGL 18704. For exposures in severe industrial and marine atmospheres and in other corrosive environments, the user must

determine in each case if the protective coating will form under the specific exposure conditions [DDR 1973].

Weathering steels can be used without restrictions if the corrosion penetration, as a function of time of exposure, does not exceed specified maximum values (Fig. 47). Guidelines for their application in rural, urban, and industrial environments are given as a function of the sulfur dioxide content of the atmosphere, and the degree of aeration and sheltering of the structure [DDR 1983]. Weathering steels may be used without corrosion protection in a 1.25-mile (2-km) coastal zone along the North Sea provided that even stricter limits on sulfur dioxide contents are met and that the steel is not exposed to salt water. The expected loss in section due to corrosion must be considered in determining the safe load-carrying capacity of a structure.

The guidelines also point out that the structural details influence the aeration of the structure, time of wetness, and corrosion performance.

JAPANESE EXPERIENCE

Atmospheric corrosion resisting steels, first introduced in Japan in the 1960's, are increasingly being used for highway and railroad bridges. The initial motivation was mainly to prolong the life expectancy of paint. Numerous atmospheric exposure tests of painted steel plates with scratches had indicated a significantly lower corrosion along the scribe marks when the underlying steel was of the corrosion resisting type. The prospect of longer paint life encouraged the use of painted weathering steels in relatively high humidity environments and in industrial areas.

Because repainting costs were increasing, bridge owners started, albeit cautiously, to use weathering steel in an unpainted condition in the 1970's. Many of these structures were treated once with a so-called "weather coat," a phosphorus-rich paint like surface coating. This sacrificial initial coating is believed to help the steel form a stable and protective oxide. It also has the advantage of hiding the early stage of rusting and preventing large amounts of iron oxide in the runoff water from staining concrete piers and abutments. Japanese designers consider the aesthetics of a bridge to be important, particularly when it is built in a heavily populated area.

According to a survey conducted by the Japanese Society of Civil Engineers, 115 weathering steel bridges had been built in Japan by March 1979. Of those, only a few were of nonpainted steel.

Since then, Japanese engineers examined numerous aged bridges, unpainted bridges, both domestic and foreign, and buildings to understand the possible problems of building with unpainted weathering steel. Convinced of its feasibility, bridge owners began to construct in the late 1970's and early 1980's unpainted highway and railroad bridges, but they carefully designed details in a manner that would help the steel form a protective oxide coating. These bridges are periodically monitored to obtain more information on their long-term behavior. Because weathering steels were first used for painted steel bridges, the chemical composition of the steel was mainly intended to provide better weldability rather than better weathering characteristics. Recent application to nonpainted construction led to the inclusion of two separate chemical compositions in the proposed modification of JIS G 3114, one for painted and one for nonpainted weathering steels.

Table 13. Nonpainted weathering steel highway bridges in Japan.

Bridge Name	Site	Weight [Ton]	Type of Structure	Environment	Completion	Name
Muranaka-Kobashi Br.	Komaki, Nagoya		Simple span	-	1967	Painted in 1982
Kawatetsu Chita No. 2 Br.	in the Chita Works of Kawatetsu Corp.	60	Simple supported composite girders	Sea-shore,	1967	Now, 1300m from sea-shore (8-years, 200m)
Daiichi Ryogoku Br.	in the Fukuyama Steel Works of NIPPON Kokan	860	Simple supported composite girders and others	Sea-shore, partially on the sea	1969	-
Kurogane Br.	in the Hirohata Works of NIPPON Steel Corp.	120	Simple supported composite girders	Sea-shore	1973	-
Shin-Ohashi B., Temporary	Tokyo	662	Simply supported girders	on the sea	1974	Removed now
Sanjin Br.	Hyogo	92	Simple supported curved box girders	Rural	1978	-
Mukogawa Br.	Hyogo	110	Simple supported composite girders	Rural	1980	-
Dejima Off-Ramp Br.	Osaka	120	3 spanned continuous un-composite girders	Industries	1980	Tentatively erected in the sea-shore for 2 years
Kokubgawa Br.	Kochi	291	Simply supported composite girders	Rural	1981	-

So far, only a few nonpainted weathering steel highway bridges have been built in Japan, as listed in Table 13. Some, in private use, are monitored periodically. The Kawatetsu Chita No. 2, Daiichi Ryogoku, and Kurogane bridges, located on the grounds of steel companies, have painted, weather coated, or nonpainted sections. This allows the companies to periodically monitor and compare the performance of the three systems. It should be noted that these early experimental bridges were constructed near the seashore, where the environmental conditions are not favorable for weathering steels. On the other hand, if all coastal areas were excluded, there would not be many sites left on which to build weathering steel bridges in Japan. The information obtained from these bridges is, therefore, considered to be extremely important. The performance of the four bridges listed at the top of Table 13 is summarized in the following.

The Muranaka-Kobashi Bridge is believed to be the first unpainted weathering steel bridge built in Japan for public use. The bridge is located in Komaki, a rural area near Nagoya. When inspected in 1976, 9 years after construction, the bridge was found to have a fairly good protective oxide coating on the webs and on most flanges [Yamada 1983a]. However, some top surfaces of the tension flange were flaking. Since these places were not washed by rain, the accumulated rust and debris had trapped moisture and kept the steel humid for extended periods of time. The bridge was painted in 1982.

The Kawatetsu Chita No. 2 Bridge was erected in 1967 in the Chita Works of Kawasaki Steel Co., near Nagoya. The site is located on reclaimed land, first about 660 ft (200 m) and later 4,260 ft (1300 m) from the sea because of further reclamation. The girders are divided into five sections that were left unpainted, weather-coated, and painted with three different types of paints. The exterior girders, running north-south, are dried by the morning and afternoon sun. They formed in general a protective oxide coating typical of weathering steel. The shel-

tered interior girders, however, exhibited a somewhat coarse and flaky oxide coating. Bridge engineers, who inspected the bridge over the years, felt that the concept of nonpainted weathering steel construction was feasible provided that rain water is drained, debris is cleaned up or prevented from accumulating, and splash water does not reach the steel.

The 1,547-ft (472-m) long Daiichi Ryogoku-bashi Bridge was built in 1969 in the Nippon Kokan Company's (NKK) Fukuyama Works, Hiroshima Prefecture. It crosses a channel, and a part is over the sea. The steel is Cupten 50. Two simple-span girder sections near the ends were left nonpainted and are inspected periodically. In 1979, 10 years after construction, the nonpainted sections were generally covered with a stable rust coating. The outsides of the exterior girders had a particularly good appearance typical of weathering steels. However, the expansion joints, supports, and details near the supports exhibited rapid and flaky corrosion. Excessive corrosion pits were also found. This unfavorable corrosion was believed to be caused by runoff water leaking through the expansion joints in the bridge deck and keeping the steel humid. It was, therefore, recommended that the structural details near the end of the bridge needed to be modified to prevent the runoff water from reaching the steel structure.

The Kurogane Bridge was erected in 1973 in the Hirohata Works of Nippon Steel Co., near Himeji, Hyogo Prefecture. This bridge is also privately owned. Sections were weather coated or left nonpainted to evaluate the long time performance of weathering steel under these conditions. The bridge consists of eight simple-span composite girders made of A588 Grade A rolled sections. It is located near the seashore. A survey of bridge engineers, who inspected the bridge in 1979, showed that most of them agreed that the exterior girders of the nonpainted spans had formed a stable oxide coating typical of weathering steels. However, about one-half of the engineers were not convinced

that corrosion of the interior girders, laterals, bracings, bearings, and shoes had stabilized. The nonpainted steel surfaces exhibited a nonuniform appearance because of differences in corrosion rates and water running over the steel, but 70 percent of the bridge engineers considered this to be negligible. On the other hand, the nonuniform appearance of the weather-coated surfaces led 80 percent of the engineers to favor nonpainted construction

over weather coating, as far as appearance was concerned.

The case studies of the three aforementioned bridges owned by Japanese steel companies and the deliberately cautious approach of Japanese bridge engineers convey a desire to make nonpainted weathering steel bridges perform as intended the first time around. Chapter Eleven gives examples of bridge details the Japanese use to help achieve this goal.

CHAPTER THREE

CORROSION MECHANISMS

BEHAVIOR OF STEEL IN THE ATMOSPHERE

The deterioration or corrosion of steel may simply be described as a consequence of exposure to the atmosphere. In a more precise sense, it is the reaction in air under moist conditions aided air pollutants, principally the sulfur oxides. The chemistry of the reaction is defined as being electrochemical in nature, because it involves ionization and transfer of electrons, as do most chemical reactions. The most frequently used metals (iron, aluminum, and zinc) are found in nature in the form of ore. A great deal of energy must be expended to release or reduce the ore to the metallic state. Obviously, these metals have a great deal of pent up energy to return to the mineral state. Aluminum and zinc react in the air to form thin protective films of aluminum oxide and basic zinc carbonate, respectively. Iron or steel does likewise, but the oxide film is unfortunately nonprotective.

At the present time the most interesting and least completely understood behavior is the controlling mechanism of the high-strength low-alloy steels, beginning with the simplest copper-bearing steels that contain as little as 0.04 percent copper. Here, for the first time, it has been observed that a self-limiting or stifling of corrosion is occurring in a mild carbon steel. The early trial and error compositions of Buck, Taylerson, and Larrabee, and the work by Larrabee and Coburn, have resulted in the ultimate development of the currently available proprietary weathering steels. The period of the late 1960's and early 1970's marked the beginning of a worldwide research interest into the nature of the corrosion products of various weathering steel compositions. Progress was expedited and theories evolved by virtue of the availability of much improved analytical instrumentation.

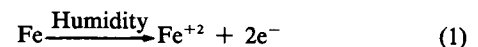
The familiar iron compounds are: $\text{Fe}(\text{OH})_2$, ferrous hydroxide; $\text{Fe}(\text{OH})_3$, ferric hydroxide; Fe_2O_3 , iron oxide or rust; and Fe_3O_4 , magnetite. However, numerous other species of iron compounds have been found and the simple representations are inadequate to explain what is already known. The most recent species isolated are the ferric oxyhydroxides known as α - FeOOH , β - FeOOH , γ - FeOOH , and δ - FeOOH . They have been examined by X-ray diffraction, infrared and far infrared. Each is different from the other. Knowledge of their existence is only about 20 years old.

As to the currently considered mechanisms for explaining the atmospheric corrosion process, three will be described.

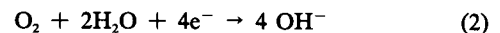
Humidity Film Reactions

When iron is attacked under humid conditions in a clean rural atmosphere, the first oxidation product is ferrous ion in the lowest oxidation state, namely, Fe^{+2} . Because of the presence of air dissolved in moisture, the ferrous ion can react with it and precipitate ferrous hydroxide, which can be quickly oxidized further to the ferric state, Fe^{+3} , to give the gelatinous precipitate of ferric hydroxide. This series of reactions can be expressed in a fashion suggested by Kunze and others as follows [Kunze 1974]:

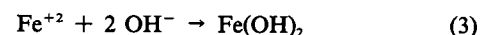
1. The anodic iron dissolves in the condensed moisture film.



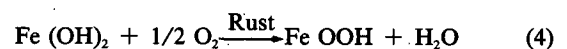
To counter balance this reaction, the cathode accepts the electrons and passes them on to oxygen which is converted to hydroxyl ion.



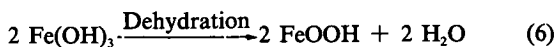
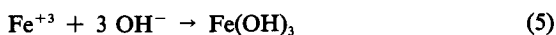
2. With free ferrous ions and free hydroxyl ions in solution, the following reaction occurs:



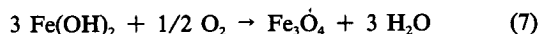
3. The fresh ferrous hydroxide is then oxidized by air to produce hydrated ferric oxide.



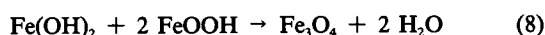
The following reactions may take place instead of those listed in steps 2 and 3. The final product in both cases is hydrated ferric oxide.



4. When the supply of oxygen is not sufficient, the reaction takes place according to Eq. 7 instead of Eq. 4:



Or, as suggested by Hiller [1966], according to Eq. 8.

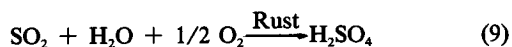


The aforementioned humidity film reactions are rather slow. Moist air, even if saturated, produces little rusting of iron in the absence of dust and pollutants [Evans 1972a]. Hence, one has to consider more active stimulants, such as the pollutants in an industrial environment, to explain the more rapid corrosion of iron observed in service.

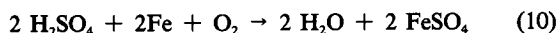
Acid Regeneration Cycle

The most common pollutants in an industrial environment are the sulfur oxides produced during the combustion of fossil fuels, such as oil and coal, each containing several percent of combined sulfur. Upon oxidation, sulfur dioxide is the predominant gas with the remainder, about 10 percent, being sulfur trioxide. The conversion of these gases to the mixed sulfurous and sulfuric acids becomes the source of the corrosive pollutant. This cycle, mentioned in Vernon's early work between 1943 and 1949, is called the acid regeneration cycle in the literature. It is a second mechanism for explaining the atmospheric corrosion of steels and consists of the following reactions:

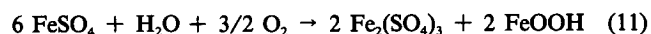
1. Sulfuric acid forms in the presence of rust, with rust acting as the catalyst.



2. The generated sulfuric acid chemically attacks the iron to form ferrous sulfate.



3. Ferrous sulfate is oxidized to ferric sulfate and hydrated ferric oxide (rust).



4. And, finally, ferric sulfate hydrolyses to additional rust.



Equation 12 indicates that all consumed acid is theoretically regenerated. Actually, this acid regeneration cycle is limited, as it has been reported that each molecule of SO_2 can lead to the formation of from 15 to 40 molecules of rust [Schickor 1963,

1964, 1967]. In addition, despite the reduction in supply of SO_2 , the corrosion can continue on an electrochemical basis instead of by acid attack [Evans 1972a]. Kunze [1974] confirmed this cycle of events.

Electrochemical Cycle

Several investigators proposed a two-layer model of the oxide during the advanced stable stages of rusting [Horton 1965, Evans 1972a, Bohni 1979, Misawa 1971-1974, Kunze 1974]. Photomicrographs of rusts on low-alloy steels support the idea of a two-layer oxide coating [Horton 1965].

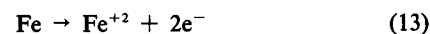
Kunze described in detail a two-layer model. As illustrated in Figure 1, the model consists of an inner layer bounded by the base metal and the oxidation-reduction front (red-ox front) inside the rust coating, and an outer layer bounded by the red-ox front and the atmosphere. Kunze based his model on the following findings. First, during the advanced, stable rusting stage, no cathodic diffusion current was measured in the outer layer. This indicates that electrochemical reactions, as explained later, cannot occur in the outer oxide layer. Second, the cathodic current density in the inner layer increases as the thickness of the rust layer increases. In other words, as the rust thickens, it provides a barrier to further corrosion, in effect slowing down the rate of corrosion attack. Third, the curve of current density versus potential in the inner layer is the same for an electrolyte free of oxygen. This indicates that the electrochemical reactions which are taking place in the inner layer do not directly involve oxygen. Oxygen is consumed only in the outer layer and at the interface with the base metal. Furthermore, because the corrosion process does not cease, either in water solutions or during natural and artificial weathering, and because the hydrated ferric oxide content of the oxide coating is not used up despite constant reduction, one must assume that the oxygen in the air reoxidizes the magnetite (Fe_3O_4) in the outer layer to hydrated ferric oxide. However, the reduction cannot take place electrochemically, because there is no cathodic diffusion current in the outer layer. It must take place by chemical reactions.

The conclusion is that the third mechanism of rusting involves an electrochemical cycle. The details are as follows:

1. Some rust forms on the steel surface during the acid regeneration cycle, by cathodic oxygen reduction. This step was already described.

2. An electrochemical cell is established. It involves oxidation, reduction, an anode (corroding metal), a cathode (existing rust), and an electrical current through the electrolyte (water solution).

3. At the metal-to-rust interface, the anodic region, iron liberates electrons



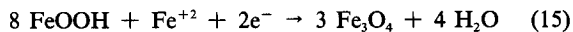
and, in the presence of sulfates in the rust, ferrous sulfate forms.



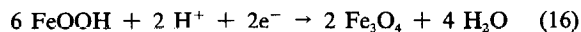
The electrons flow to the cathodic region near the red-ox front via the magnetite. As Schwarz [1972] pointed out, other iron

compounds containing Fe^{+2} and Fe^{+3} can also transmit the electrons.

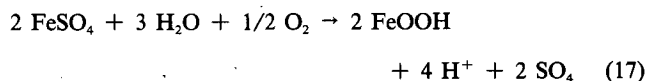
4. In the inner layer near the red-ox front, the cathodic region, rust is reduced to magnetite by either of the following reactions:



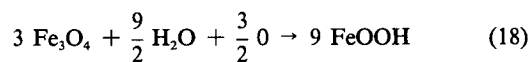
or



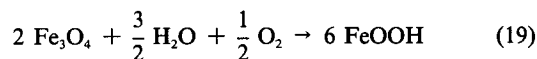
5. The first of the two reactions requires Fe^+ ions and the second H^+ ions. The latter come from hydrolysis of ferrous sulfate in the anodic region of the inner layer.



6. The magnetite that forms in the cathodic region reoxidizes again to rust.



or



These chemical reactions take place at the red-ox front, when

enough oxygen from the air penetrates the outer layer and reaches the red-ox front. Formation of a protective rust in the outer layer inhibits continuous supply of oxygen and water to this front. As a result, the corrosion attack may slow down considerably.

In summary, the red-ox front separates the rust coating into an inner layer, where rust is electrochemically reduced, and an outer layer, where magnetite is chemically oxidized to rust. The red-ox front is not stationary. It moves, to some degree, inwards and outwards depending on the humidity level and the oxygen supply.

The previously described corrosion mechanisms apply to steels in general. This is because marked differences between the atmospheric corrosion behavior of plain carbon steel and weathering steel do not start to show up until a fully developed rust-layer has formed [Horton 1965]. The reasons why weathering steel develops a protective rust layer are discussed in the next section.

WEATHERING STEEL

Structural carbon steels become atmospheric corrosion resistant when they are modified by the inclusion of 2 percent or less of such common alloying elements as copper, phosphorus, chromium, nickel, and silicon. To explain this behavior, the corrosion mechanisms of weathering steel reported in the literature will be reviewed. To begin with, one must accept that corrosion science has proven beyond doubt that the corrosion of metals, once initiated, proceeds by an electrochemical mechanism.

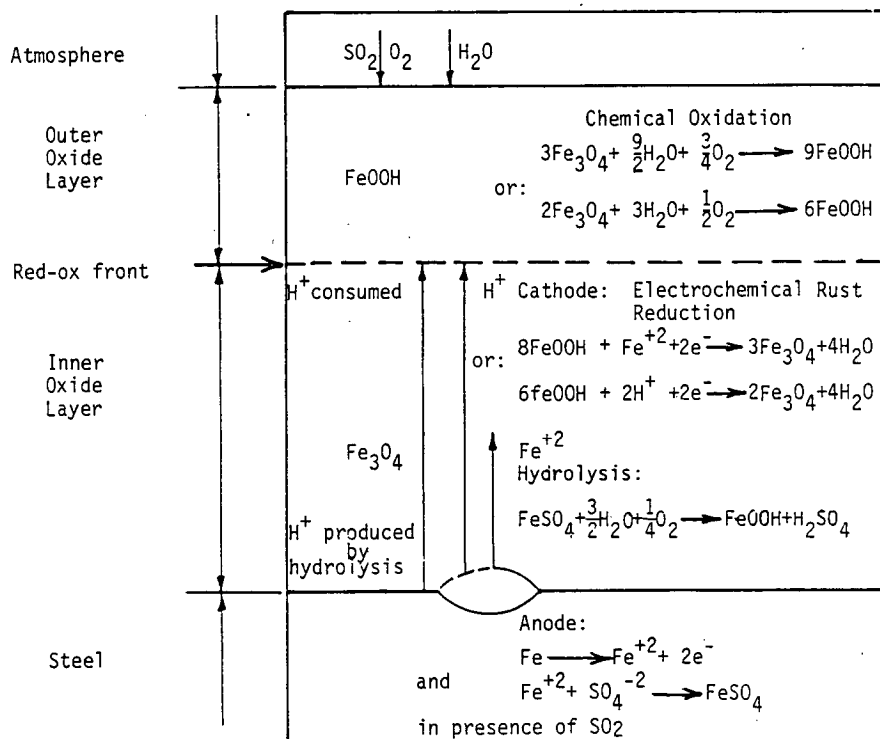


Figure 1. Schematic representation of two-layer oxide model. [Kunze 1974]

The existence of anodic and cathodic regions in the inner rust layer implies that cations diffuse outwards through the pores of the inner rust layer, which are filled with electrolyte when the steel is wet. Any means by which the pores are plugged retards the corrosion process. In all steels, the cations consist mainly of diffusible H^+ ions and Fe^{+2} ions. In weathering steels that are alloyed with copper, chromium, and nickel, the Cu^{+2} , Cr^{+3} , and Ni^{+2} ions strongly tend to form compounds that are left behind because of their large diameter and small diffusion velocity. For this reason, Cu^{+2} , Cr^{+3} , and Ni^{+2} concentrations are found in the inner regions of the rust coating of weathering steels containing these alloying elements [Bruno 1973, Kunze 1974].

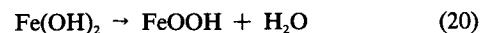
As was described earlier, the red-ox front separates the rust coating into an inner layer and an outer layer. Changing supplies of moisture and oxygen move the red-ox front inwards and outwards, as the steel undergoes wet-dry cycles during natural weathering.

During the wetting part of the cycle, the red-ox front moves outwards. Diffusion of cations increases the supply of hydrogen ions as well as copper, chromium, and nickel cations. The environment within the electrolyte becomes acidic and the alloying elements cannot precipitate.

During the drying part of the cycle, the red-ox front moves inwards. It loses hydrogen ions, the pH value increases, and compounds rich in copper, chromium, and nickel are precipitated. For example, the copper ions in the electrolyte precipitates as basic copper sulfate once a critical concentration of ions is reached after repeated wet-dry cycles. This process gradually plugs the pores and makes the layer denser, thus inhibiting the outward diffusion of Fe^{+2} ions and the penetration of moisture and oxygen. Hence, both the anodic and the cathodic reactions slow down, and the rate of corrosion diminishes. Other investigators have been unable to confirm the presence of basic copper sulfate [Suzuki 1980].

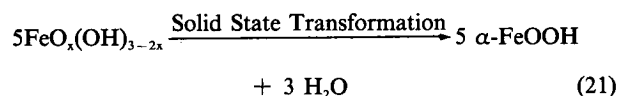
Additional theories about the formation of the protective oxide film on weathering steel have been suggested. Misawa et al. analyzed the rust coating with a wide variety of advanced analytical instruments [Misawa 1971, 1973, 1974a, 1974b]. They identified the chemical compounds whose physical features can account for the beneficial performance of copper-bearing steel and more particularly the weathering steels. Their work identifies the role of copper and phosphorus.

Misawa et al. concluded that rusting initiates by the aerial oxidation of ferrous ions, which are dissolved from the steel because of a slightly acidic thin water layer on the steel surface. For example, fresh rain dissolves atmospheric pollutants, mainly sulphur dioxide, and becomes acidic with a pH value of four. The low pH water layer covering the steel surface leads to the formation and precipitation of γ -FeOOH. The Greek alphabet prefix, γ , specifies the crystalline structure of the rust formed in this process. The γ -FeOOH dissolves in the slightly acidic rain water during the wetting periods and precipitates as amorphous ferric oxyhydroxide, $FeO_x(OH)_{3-2x}$, upon drying. Analysis of the rust indicated that $x = 0.4$ for the latter compound. This suggests that the chemical formula for the amorphous matter can be written as $Fe_5O_{13}H_{11}$. One can assume that $Fe_5O_{13}H_{11}$ is composed to two molecules of hydrated ferric oxide and three molecules of ferric hydroxide. The dehydration reaction is as follows:



In other words, each of the different corrosion products, γ -FeOOH, α -FeOOH, and $FeO_x(OH)_{3-2x}$, found on the surface of naturally weathered steel specimens is basically one form of partially dehydrated ferric hydroxide. The supply of oxygen and water, the pH value, and the presence of alloying elements govern the degree of dehydration and the crystal system.

The α -FeOOH mentioned above is obtained from solid state transformation of the amorphous ferric hydroxide when the rust dries and the amorphous ferric oxyhydroxide loses water.



Misawa reported remarkable differences in the rust coating that formed on plain carbon and low-alloy steels during natural weathering. The rust on low-alloy steel exhibited uniform thickness, and the rust-to-steel interface was even. In contrast, the rust on plain carbon steel was not uniform and the rust-to-steel interface was uneven. Misawa attributed the uniformity to the presence of alloying elements. Because copper, phosphorus, and chromium are enriched in the inner rust layer of low-alloy steel, the dissolution of steel, the formation of γ -FeOOH, and subsequent precipitation of the amorphous compound would also be uniform. Moreover, the amorphous ferric oxyhydroxide rust on low-alloy steels contained a considerable amount of bound water, as compared with that on carbon steel. Hence, one can assume that the rust on low-alloy steel dries more slowly. The rate of solid state transformation of the amorphous ferric oxyhydroxide, as shown by Eq. 21, is slower for low-alloy steel than for carbon steel. Horton's [1965] work had also indicated that the rust on low-alloy steels dried out more slowly than on carbon steel.

The uniformity of the rust and the slower rate of drying, both induced by the presence of alloying elements, help the low-alloy steel develop a compact rust coating with fewer cracks. Plain carbon steel, on the other hand, has a nonuniform, porous rust coating with many cracks that let water, oxygen, and other contaminants freely penetrate. Figures 2 and 3 show a schematic representation of the oxide coating on carbon steel and low-alloy steel, respectively.

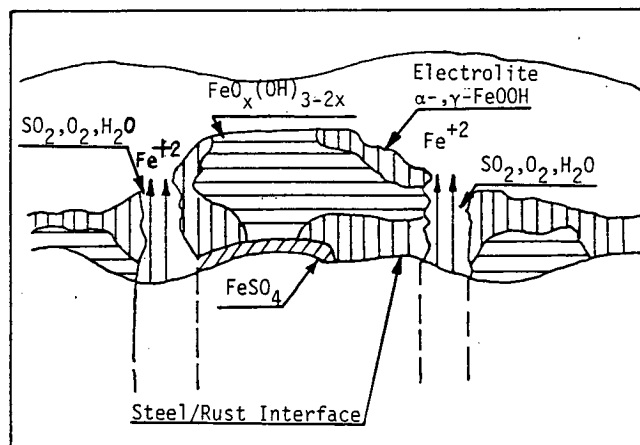


Figure 2. Schematic representation of the oxide coating on weathered plain carbon steel.

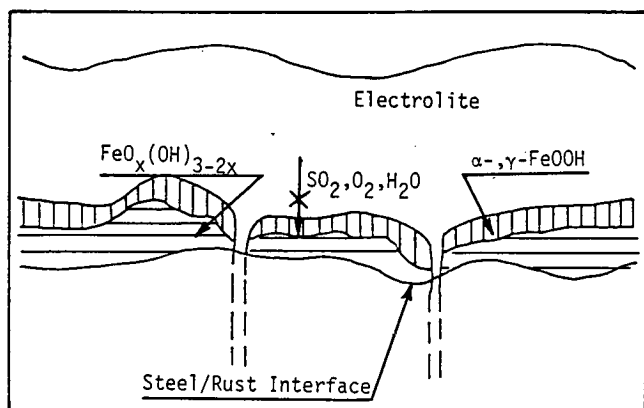


Figure 3. Schematic representation of the oxide coating on weathered low alloy steel.

In summary, there are two plausible mechanisms that give weathering steel its corrosion resistance. One suggests that the corrosion resistance is achieved by precipitation of basic compounds rich in alloying elements. The other presumes that the alloying elements catalyze the precipitation of amorphous ferric oxyhydroxide. The two mechanisms have several common features: two-layer structure of the rust coating, electrochemical corrosion cycle, enrichment of alloying elements in the inner rust layer, and basicity of the rust. The differences may be more a matter of viewpoint than substance.

For reference purposes, some other explanations are also listed in the following. An early attempt to explain the improved atmospheric performance of copper-bearing steel over that of carbon steel was made by Carius. He proposed that the resistance to atmospheric corrosion increased when a metallic copper film, or a copper-rich layer, form next to the steel [Carius 1930]. However, as Copson [1945] pointed out later, this film of copper has not been observed experimentally, nor have artificially applied films of copper been as effective in reducing corrosion as directly alloying the copper in the steel.

In 1945, Copson reported finding higher sulfate levels in the rust coating of copper bearing steels than in mild carbon steels [Copson 1945]. He attributed the beneficial influence of copper and nickel to the formation of basic copper and nickel sulfates, respectively. Vernon [1960] later characterized the basic copper sulfate as $[\text{Cu}(\text{OH})_2\text{Cu}_x]\text{SO}_4$. These basic sulfates were believed to plug the pores in the rust and to render it less permeable to oxygen and water.

In 1970, Fyfe et al. examined the corrosion behavior of eight pure iron alloys, one mild carbon steel, one copper-bearing steel, and one A588 Grade A steel exposed to rural, industrial, and marine atmospheres in England [Fyfe 1970]. They stated that the primary purpose of alloying with copper was to nullify the deleterious effect of sulfur, which can come from two sources: (1) present in the steel as ferrous sulfide, FeS , or manganese sulfide, MnS ; and (2) introduced from the atmosphere either directly as H_2S or from the cathodic reduction of SO_2 . The mechanism is somewhat speculative, because it is not clear how even small amounts of copper (as low as 0.05 percent) could nullify all of the available sulfur ions.

In 1980, Suzuki et al. examined natural and artificial rust layers that formed on a series of steels, from pure iron to weathering steel [Suzuki 1980]. They concluded that: (1) copper additions to iron accelerate the formation of amorphous substances and increase the mechanical strength of the rust coating; and (2) beneficial alloying elements decrease the formation of intermediate substances, which would be converted by cathodic reaction to crystalline magnetite (Fe_3O_4) and, thereby, decrease the electrical conductivity of the rust coating.

EFFECT OF AQUEOUS ENVIRONMENTS AND SALT

The formation of a protective rust film on weathering steel requires alternate wet-dry cycles of natural weathering and a salt-free atmosphere. Under prolonged periods of wetness and under severe exposure to deicing salts, weathering steel corrodes at the same rate as plain carbon steel [Larrabee 1958, Schmitt 1967, Zoccola 1976, Tinklenberg 1980, Reed 1982, Brockenbrough 1983].

If weathering steel remains wet for a long period of time, the necessary conditions are not met for the formation of the amorphous ferric oxyhydroxide, suggested by Misawa et al. [1974b], or the periodic inward/outward movement of the red-ox front resulting in precipitation of basic compounds rich in alloying elements, as described by Kunze [1974].

The deleterious effect of deicing salt on atmospheric corrosion resistance of steels results from several phenomena. First, deicing salt dissolved in runoff water creates an aggressive electrolyte. When the negatively charged chloride ions, Cl^- , come in contact with the steel, they shift the already negative potential of the steel to larger negative values. The ensuing increase in the galvanic potential accelerates metal corrosion [Uhlig 1973]. Other negatively charged ions, such as SO_4^{2-} and NO_3^- , which originate from industrial pollutants, have a similar though less pronounced effect on steel.

Secondly, when chloride ions are present, the rust coating contains large amounts of $\beta\text{-FeOOH}$. This form of hydrated ferric oxide is crystalline. In contrast to $\gamma\text{-FeOOH}$, it does not convert to the protective amorphous ferric oxyhydroxide.

Thirdly, the strong tendency of salts to absorb moisture from the atmosphere lowers the critical relative humidity at which steels begin to significantly corrode. In other words, the presence of salt raises the relative humidity in the immediate neighborhood of the steel, lengthens the wetting periods, and hinders the drying process. The chloride ions are reused; they do not need to be resupplied.

For the foregoing reasons, the superior corrosion resistance exhibited by the weathering steels, when properly utilized, is not usually observed in aqueous and saline environments. Therefore, under less than boldly exposed conditions, weathering steels cannot necessarily be assumed to have better corrosion resistance than carbon steel.

EFFECT OF PRIMARY ALLOYING ELEMENTS FROM LARRABEE-COBURN INVESTIGATION

Larrabee and Coburn [1961] performed the most extensive and systematic study of steel compositions alloyed with the elements copper, phosphorus, chromium, nickel, and silicon that

are known to contribute most to the atmospheric corrosion resistance of the weathering steels. They arranged 270 steels in a five-way factorial experiment with five levels of copper content (0.01 percent min. to 0.51 percent max.), three of phosphorus (0.01 percent min. to 0.12 percent max.), three of chromium (0.1 percent min. to 1.3 percent max.), two of nickel (0.05 percent min. to 1.1 percent max.), and three of silicon (0.1 percent min. to 0.64 percent max.). Of the nine grades of A588 steel, the following have chemical requirements for these elements that fall entirely within the range studied by Larrabee and Coburn: for copper, seven grades; for phosphorus and chromium, all nine grades; for nickel, eight grades; and for silicon, seven grades. Obviously, the Larrabee-Coburn data are the key to understanding the role of these five elements. All 270 steels had less than 0.1 percent carbon, 0.25 percent to 0.40 percent manganese, and less than 0.02 percent sulphur. Replicate coupons were mounted on exposure racks at a 30-deg angle from the horizontal facing south. They were exposed for 15.5 years at semirural South Bend, Pennsylvania, industrial Kearny, New Jersey, and marine Kure Beach, North Carolina, at a distance of 800 ft (240 m) from the shore.

The data are examined hereafter for the effect of each alloying element alone, in combination with a second element, and in combination with the remaining elements. Figures 4 through 18 (five elements times three environments) show the average corrosion penetration per side as a function of the element concentration and the environment. Each data point represents one of the 270 steels. The curves connect the data points for the steels with nearly equal concentrations of four of the five alloying elements. The concentrations of the fifth element vary along the abscissa. The chemical composition of the steels, corresponding to the number on the end of each curve, are identified in Table 14. The curves were cut off at 16-mils (400- μm) penetration. Some steels in Figures 5 and 7 are identified with an underlined individual composition number, written next to the corresponding data points. They correspond to the Larrabee-Coburn composition numbers given in Table 14.

For ease of comparing the A588 specification requirements for chemical composition with the Larrabee-Coburn data, the concentration ranges of copper, phosphorus, chromium, nickel, and silicon for each grade are shown at the top of Figures 4, 7, 10, 13, and 16.

Table 14 identifies for each curve the steel composition numbers assigned by Larrabee-Coburn and the average concentration of the alloying elements. Table 15 compares the "reduction in thickness per side" or the "corrosion penetration" of the steels containing one or two alloying elements.

Copper

Figures 4, 5, and 6, respectively, show the effect of copper content on the corrosion penetration in the rural, industrial, and marine environments. The curves 1 are for steels alloyed with copper alone. Their corrosion penetration in the three environments is given in the top five lines of Table 15(a). The left most points on these curves correspond to carbon steel (0.02 percent Cu max.) which exhibited a corrosion penetration of 12.3, 28.8, and 52.0 mils (312, 732, and 1,320 μm) in the rural, industrial, and marine environments, respectively. Moving along curves 1 to the right, it is evident that the initial 0.05 percent copper gave the greatest reduction in corrosion penetration in

Table 14. Identification of curves plotted in Figures 4 through 18 and chemical composition of steels. [Larrabee 1961]

Curve No.	Steel Identification No. ^a					Composition, Percent				
						Cu	P	Cr	Ni	Si
(a) Effect of copper (Figs. 4 to 6)										
1	1	2	3	4	5	-	-	-	-	-
2	6	7	8	9	10	-	-	-	1.0	-
3	11	12	13	14	15	-	0.64	-	-	-
4	21	22	23	24	25	-	1.30	-	-	-
5	31	32	33	34	35	-	-	-	-	0.20
6	61	62	63	64	65	-	-	-	-	0.52
7	91	92	93	94	95	0.06	-	-	-	-
8	131	132	133	134	135	0.06	0.62	-	-	0.22
9	136	137	138	139	140	0.06	0.62	1.0	-	0.18
10	181	182	183	184	185	0.10	-	-	-	-
11	261	262	263	264	265	0.10	1.20	-	-	0.58
12	266	267	268	269	270	0.10	1.20	1.0	-	0.53
(b) Effect of phosphorus (Figs. 7 to 9)										
1	1	91	181	-	-	-	-	-	-	-
2	4	94	184	0.21	-	-	-	-	-	-
3	5	95	185	0.47	-	-	-	-	-	-
4	6	96	186	-	-	-	1.0	-	-	-
5	11	101	191	-	-	0.66	-	-	-	-
6	21	111	201	-	-	1.30	-	-	-	-
7	31	121	211	-	-	-	-	-	0.24	-
8	41	131	221	-	-	0.62	-	-	0.22	-
9	45	135	225	0.44	-	0.65	-	-	0.21	-
10	46	136	226	-	-	0.64	1.0	-	0.24	-
11	50	140	230	0.45	-	0.64	1.0	-	0.18	-
12	61	151	241	-	-	-	-	-	0.55	-
13	81	171	261	-	-	1.30	-	-	0.59	-
14	85	175	265	0.44	-	1.20	-	-	0.52	-
15	86	176	266	-	-	1.20	1.0	-	0.49	-
16	90	180	270	0.45	-	1.30	1.0	-	0.50	-
(c) Effect of chromium (Figs. 10 to 12)										
1	1	11	21	-	-	-	-	-	-	-
2	4	14	24	0.23	-	-	-	-	-	-
3	5	15	25	0.46	-	-	-	-	-	-
4	6	16	26	-	-	-	1.0	-	-	-
5	31	41	51	-	-	-	-	-	0.27	-
6	61	71	81	-	-	-	-	-	0.59	-
7	91	101	111	-	0.06	-	-	-	-	-
8	121	131	141	-	0.06	-	-	-	0.27	-
9	125	135	145	0.45	0.06	-	-	-	0.19	-
10	126	136	146	-	0.06	-	1.0	-	0.23	-
11	130	140	150	0.45	0.06	-	1.0	-	0.17	-
12	181	191	201	-	0.09	-	-	-	-	-
13	241	251	261	-	0.11	-	-	-	0.55	-
14	245	255	265	0.46	0.10	-	-	-	0.57	-
15	246	256	266	-	0.11	-	1.0	-	0.49	-
16	250	260	270	0.45	0.10	-	1.0	-	0.60	-
(d) Effect of nickel (Figs. 13 to 15)										
1	1	6	-	0.045	-	-	-	-	-	-
2	2	7	-	0.045	-	-	-	-	-	-
3	4	9	-	0.22	-	-	-	-	-	-
4	5	10	-	0.46	-	-	-	-	-	-
5	11	16	-	-	-	0.61	-	-	-	-
6	21	26	-	-	-	1.30	-	-	-	-
7	31	36	-	-	-	-	-	-	0.23	-
8	61	66	-	-	-	-	-	-	0.59	-
9	91	96	-	-	0.06	-	-	-	-	-
10	131	136	-	-	0.065	0.61	-	-	0.24	-
11	134	139	-	0.20	0.06	0.60	-	-	0.17	-
12	135	140	-	0.45	0.055	0.65	-	-	0.20	-
13	181	186	-	-	0.09	-	-	-	0.55	-
14	261	266	-	-	0.115	1.20	-	-	0.53	-
15	264	269	-	0.23	0.095	1.25	-	-	0.53	-
16	265	270	-	0.45	0.10	1.25	-	-	0.60	-
(e) Effect of silicon (Figs. 16 to 18)										
1	1	31	61	-	-	-	-	-	-	-
2	4	34	64	0.23	-	-	-	-	-	-
3	5	35	65	0.48	-	-	-	-	-	-
4	6	36	66	-	-	-	1.0	-	-	-
5	11	41	71	-	-	0.64	-	-	-	-
6	21	51	81	-	-	1.30	-	-	-	-
7	91	121	151	-	0.06	-	-	-	-	-
8	101	131	161	-	0.06	0.61	-	-	-	-
9	105	135	165	0.45	0.06	0.61	-	-	-	-
10	106	136	166	-	0.07	0.57	1.0	-	-	-
11	110	140	170	0.45	0.06	0.66	1.0	-	-	-
12	181	211	241	-	0.10	-	-	-	-	-
13	201	231	261	-	0.10	1.20	-	-	-	-
14	205	235	265	0.46	0.09	1.30	-	-	-	-
15	206	236	266	-	0.11	1.20	1.0	-	-	-
16	210	240	270	0.44	0.08	1.30	1.0	-	-	-

a. The steel identification numbers are the same as in Ref. [Larrabee 1961].

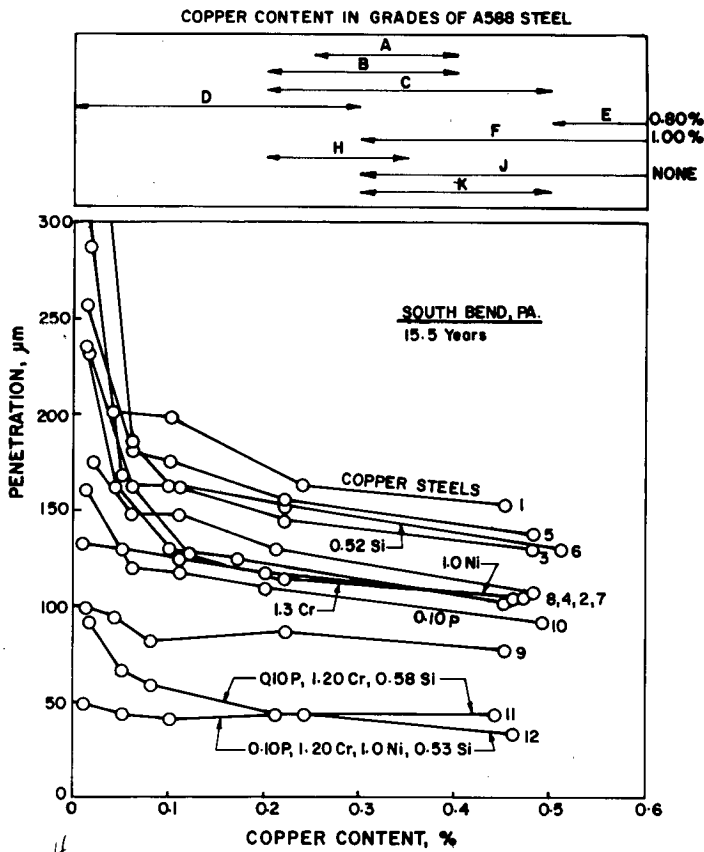


Figure 4. Effect of copper content on corrosion penetration at South Bend, Pa. rural site, 15.5-year exposure [Larrabee 1961]. Curves are identified in Table 14.

the three environments. Thereafter, the reduction tapered off, gradually approaching a plateau after 0.25 percent copper content.

The corrosion penetration of the steels containing copper plus a second alloying element decreased as the concentration of the second alloying element increased. This is evident from a comparison of the penetrations listed in Table 15(a) and from the corresponding curves in the copper plots. In the Kearny industrial environment, the addition of 1.3 percent chromium, 0.10 percent phosphorus, 1.0 percent nickel, or 0.52 percent silicon was most beneficial in the order listed (Fig. 5). The order changed in the other two environments, with 0.10 percent phosphorus being the most beneficial in semirural South Bend, and 1.0 percent nickel being the most beneficial in marine Kure Beach (Figs. 4 and 6). Adding 0.52 percent silicon to copper steels was marginally beneficial, except in the marine environment where it significantly decreased the penetration.

The corrosion penetration was lowest for the high-concentration multialloy combinations, as shown by curves 9, 11, and 12. In these cases, copper concentrations in excess of 0.10 percent conferred no additional benefits.

Given equal concentrations of phosphorus, chromium, and silicon, steels with 1.0 percent nickel provided the same corrosion resistance as steel with only 0.50 percent copper. For example, the underlined pairs of steels 5 and 6, 135 and 136,

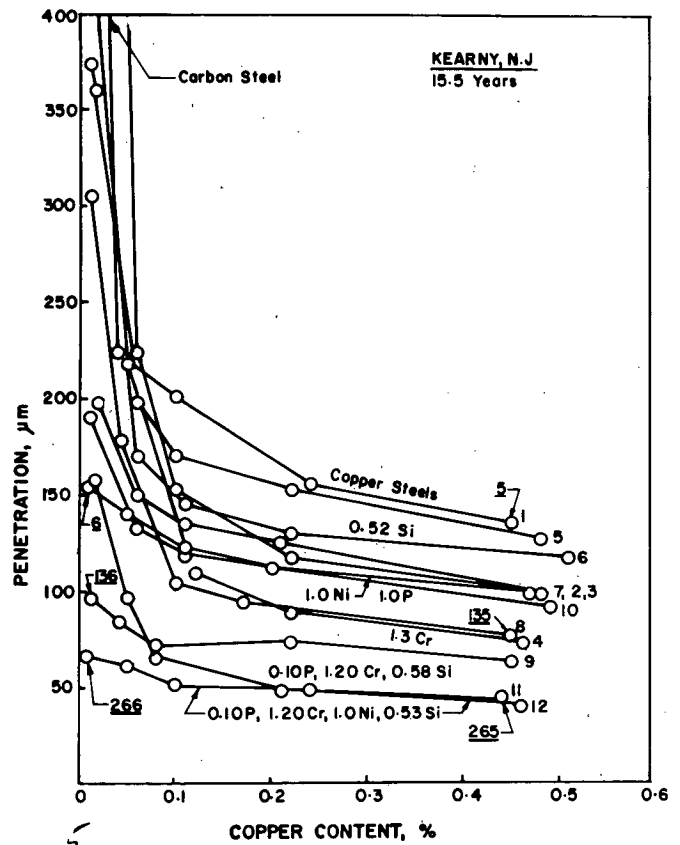


Figure 5. Effect of copper content on corrosion penetration at Kearny, N.J. industrial site, 15.5-year exposure. [Larrabee 1961]

and 265 and 266, shown in Figure 5 had comparable penetrations. Copper and nickel were interchangeable in these amounts, but not generally in other 1:2 proportions.

The aforementioned conclusions apply to all three environments. In general, the steels exposed in the industrial environment had about the same corrosion resistance as those exposed in the rural environment and a much higher resistance than those exposed in the marine environment. The corrosion penetration of the steels with multialloy combinations fell within a 2 to 4 mils (50 to 100 μm) scatterband for Mayari R weathering steels that represent a broad range of compositions and exposures in different environments [Bethlehem, undated]. See the high-copper points on curves 9, 10, 11, and 12.

Figure 4 shows the range of copper concentrations in the nine grades of the A588 steel. In five grades, the range varies from 0.2 percent to 0.5 percent copper. Grade D could have less copper, whereas Grades E, F, and J could have more copper. The low-copper and high-copper grades achieve a comparable corrosion resistance with the addition of correspondingly higher and lower contents of the other alloying elements.

Table 15. Effect of one and two-element compositions on corrosion penetration of steels. [Larrabee 1961]

Steel Identification No. ^a	Concentration of Elements, Percent		Corrosion Penetration After 15.5 Years, in μm		
	First Element	Second Element	South Bend, Pa. Rural	Kearny, N.J. Industrial	Kure Beach, N.C. Marine
(a) Effect of Copper (Figs. 4 to 6)					
1	0.01 Cu	--	312	732	1320
2	0.04 Cu	--	201	224	363
3	0.10 Cu	--	198	201	305
4	0.24 Cu	--	163	155	284
5	0.45 Cu	--	152	135	263
35	0.48 Cu	0.20 Si	127	127	226
65	0.51 Cu	0.52 Si	152	117	185
95	0.48 Cu	0.64 Cr	130	99	208
15	0.48 Cu	0.06 P	107	99	193
10	0.47 Cu	1.0 Ni	104	91	183
185	0.49 Cu	0.10 P	91	91	180
25	0.46 Cu	1.3 Cr	104	74	198
(b) Effect of Phosphorus (Figs. 7 to 9)					
1	--	--	312	732	1320
91	0.06 P	--	175	198	358
181	0.10 P	--	160	191	333
101	0.06 P	0.66 Cr	216	282	295
121	0.06 P	0.25 Si	175	241	295
151	0.06 P	0.53 Si	173	241	241
111	0.06 P	1.3 Cr	173	221	229
94	0.06 P	0.21 Cu	130	124	231
96	0.06 P	1.0 Ni	107	104	206
95	0.07 P	0.48 Cu	107	99	193
(c) Effect of chromium (Figs. 10 to 12)					
1	--	--	312	732	1320
11	0.61 Cr	--	419	1059	401
21	1.30 Cr	--	287	419	478
41	0.66 Cr	0.31 Si	351	485	381
71	0.64 Cr	0.59 Si	216	312	201
101	0.66 Cr	0.06 P	216	282	295
191	0.70 Cr	0.08 P	188	249	239
16	0.61 Cr	1.0 Ni	127	137	185
14	0.63 Cr	0.22 Cu	145	117	229
15	0.61 Cr	0.48 Cu	130	99	208
(d) Effect of nickel (Figs. 13 to 15)					
1	--	--	312	732	1320
6	1.0 Ni	--	132	155	293
11,16	0.5 Ni	0.60 Cr	273	598	313
21, 26	0.5 Ni	1.30 Cr	196	261	385
31, 36	0.5 Ni	0.22 Si	181	253	243
61, 66	0.5 Ni	0.59 Si	141	151	282
91, 96	0.5 Ni	0.06 P	125	147	258
181, 186	0.5 Ni	0.10 P	140	134	244
4, 9	0.5 Ni	0.22 Cu	135	117	223
5, 10	0.5 Ni	0.46 Cu	135	117	223
(e) Effect of silicon (Figs. 16 to 18)					
1	--	--	312	732	1320
31	0.22 Si	--	257	373	546
61	0.61 Si	--	231	361	333
71	0.59 Si	0.64 Cr	216	312	201
151	0.53 Si	0.06 P	173	241	241
81	0.54 Si	0.10 P	132	160	198
241	0.54 Si	0.10 P	132	160	198
64	0.47 Si	0.22 Cu	152	130	208
65	0.47 Si	0.51 Cu	130	117	185
66	0.57 Si	1.0 Ni	109	109	152

a. The steel identification numbers are the same as those in Ref. [Larrabee 1961]

Phosphorus

Figures 7, 8, and 9, respectively, show the effect of phosphorus content on the corrosion penetration in the rural, industrial, and marine environments.

The corrosion penetration of steels containing phosphorus as the single alloying element is summarized in Table 15(b) and plotted as curves 1 in the three phosphorus plots. In all three environments, adding up to 0.06 percent phosphorus to these steels significantly reduced corrosion penetration. Adding phosphorus in excess of 0.06 percent was only of marginal additional benefit. Phosphorus alone was more effective in reducing cor-

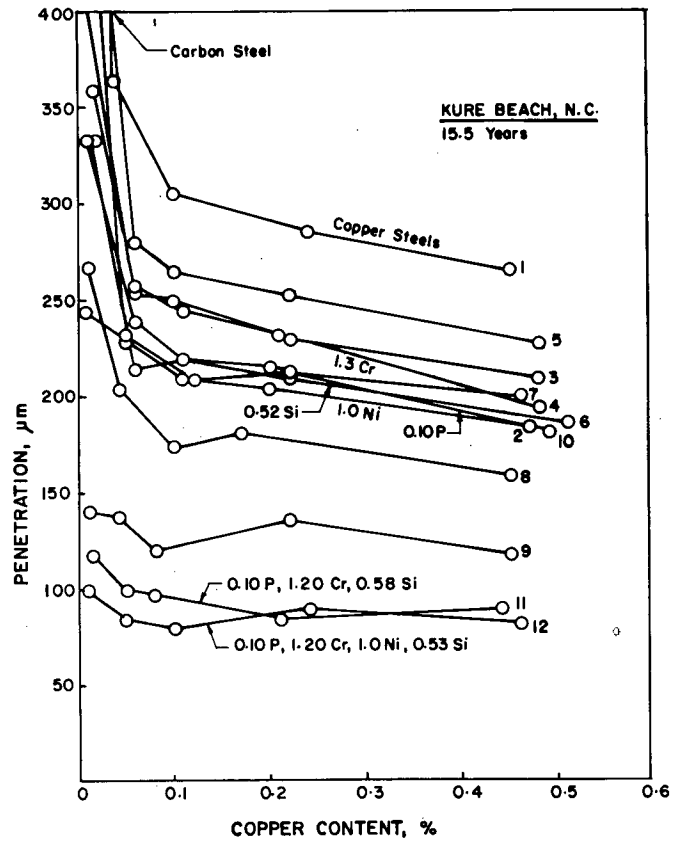


Figure 6. Effect of copper content on corrosion penetration at Kure Beach, N.C. marine site, 15.5-year exposure. [Larrabee 1961]

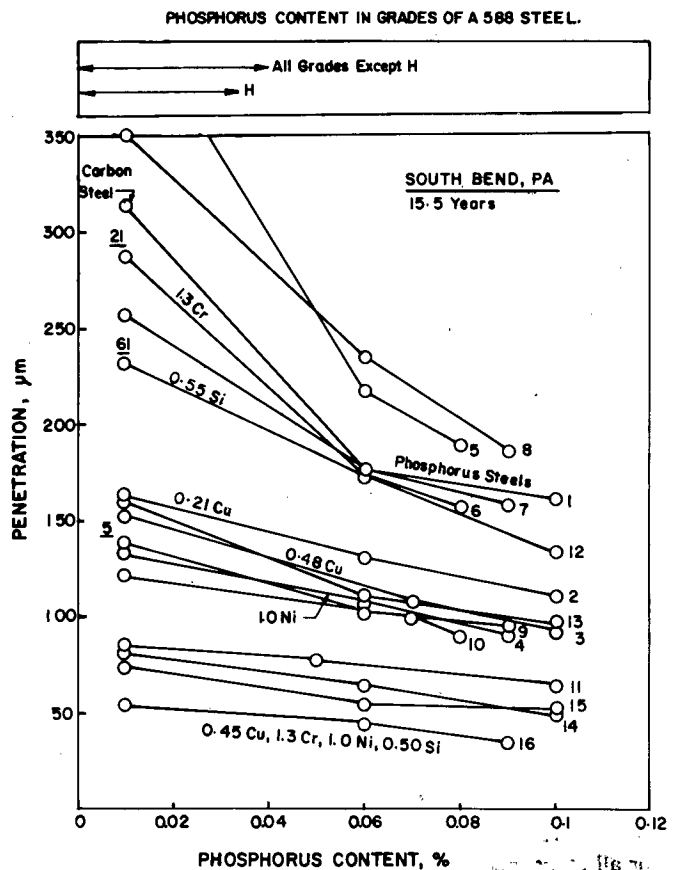


Figure 7. Effect of phosphorus content on corrosion penetration at South Bend, Pa. rural site, 15.5-year exposure. [Larrabee 1961]

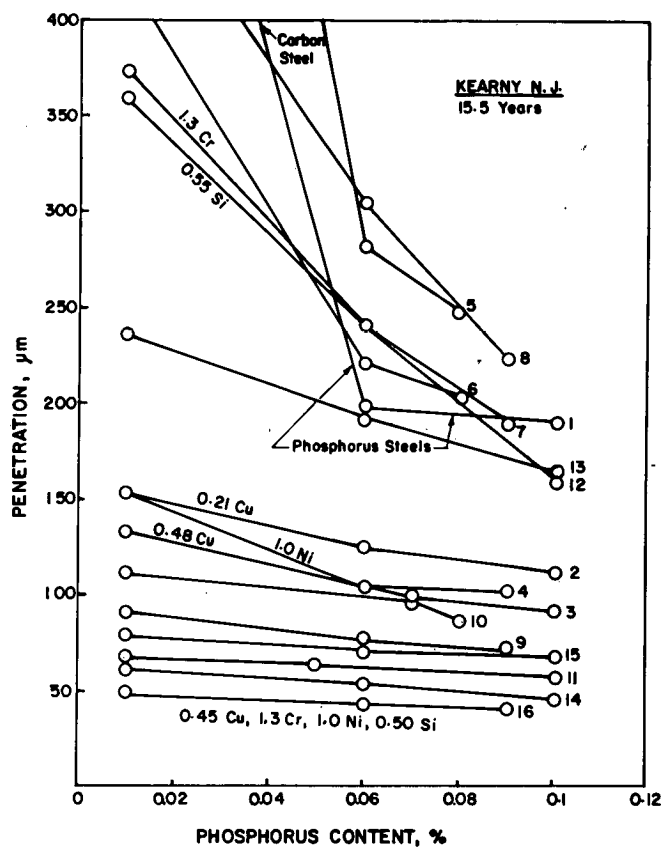


Figure 8. Effect of phosphorus content on corrosion penetration at Kearny, N.J. industrial site, 15.5-year exposure. [Larrabee 1961]

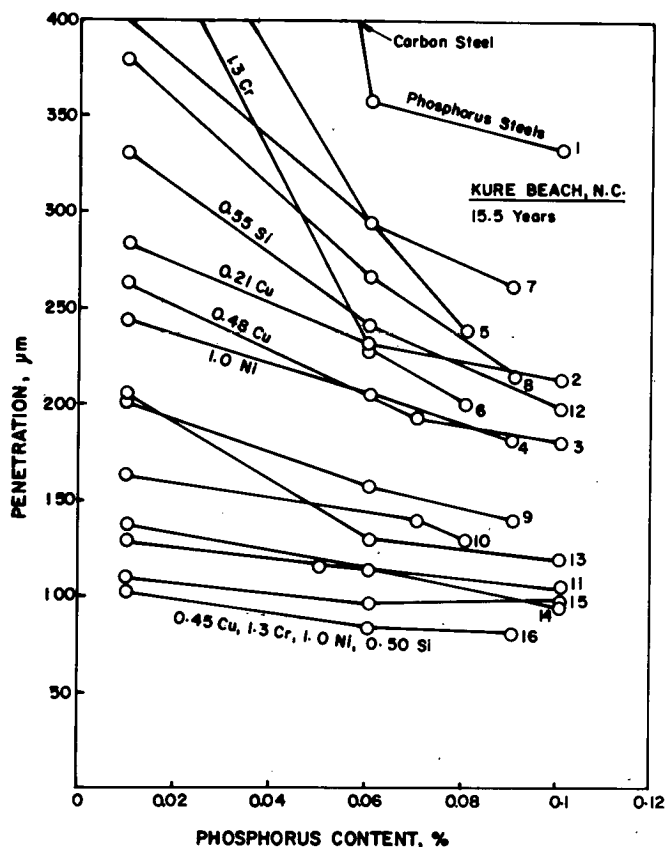


Figure 9. Effect of phosphorus content on corrosion penetration at Kure Beach, N.C. marine site, 15.5-year exposure. [Larrabee 1961]

rosion penetration in the marine environment than in the other environments.

The reduction in corrosion penetration of steels with about 0.06 percent phosphorus and a second alloying element are given in Table 15(b), again in increasing order of corrosion resistance in the industrial environment. Adding 0.48 percent copper, 1.0 percent nickel, or 0.21 percent copper reduced penetration most, in that order. However, adding chromium or silicon made the 0.06 percent phosphorus steels more susceptible to corrosion in the rural and industrial environments, but was beneficial in the marine environment.

In fact, the upper set of curves in Figure 8 are for steels that contained neither copper nor nickel. Adding either one, or both, dropped the steels to the lower set of curves, in which case up to 0.10 percent phosphorus only moderately increased the corrosion resistance. The four and five-alloy combinations in the lower set of curves (9-11, 14-16) exhibited the highest corrosion resistance.

All A588 steel grades contain copper, and all but grade D contain nickel. Therefore, phosphorus plays only a moderate role in increasing corrosion resistance. In fact, it is used mainly for its strengthening effect (see Ch. Six, section under "Strengthening Mechanisms"). The A588 specification limits the phosphorus level to 0.035 percent for Grade H and to 0.04 percent for all other grades, as noted at the top of Figure 7.

Chromium

The effect of chromium content on the corrosion penetration of steels in the three environments is illustrated in Figures 10, 11, and 12.

The corrosion penetration of steels with chromium as the single alloying element is summarized in Table 15(c). A portion of the chromium steel data is shown as curve 1 in Figure 10 for the rural environment, but not in Figures 11 and 12 because the corrosion penetration in the other environments exceeded 16 mils (400 μm). The penetration in the rural environment increased from 12.3 mils (312 μm) at less than 0.1 percent chromium to 16.5 mils (419 μm) at 0.61 percent. Thereafter, it decreased to 11.3 mils (287 μm) at 1.3 percent chromium, a value comparable to the penetration with only residual levels. The same detrimental effect of chromium alone, at intermediate concentrations, was also observed in the industrial environment, though not in the marine environment.

The two-element and three-element combinations involving chromium, phosphorus, and/or silicon also exhibited a peak penetration at about 0.60 percent chromium when the steels were exposed in the rural and industrial environments. See curves 5 to 8, 12 and 13 in Figures 10 and 11. Again, this phenomenon was not observed in the marine environment. The peaks disappeared and the corrosion resistance significantly in-

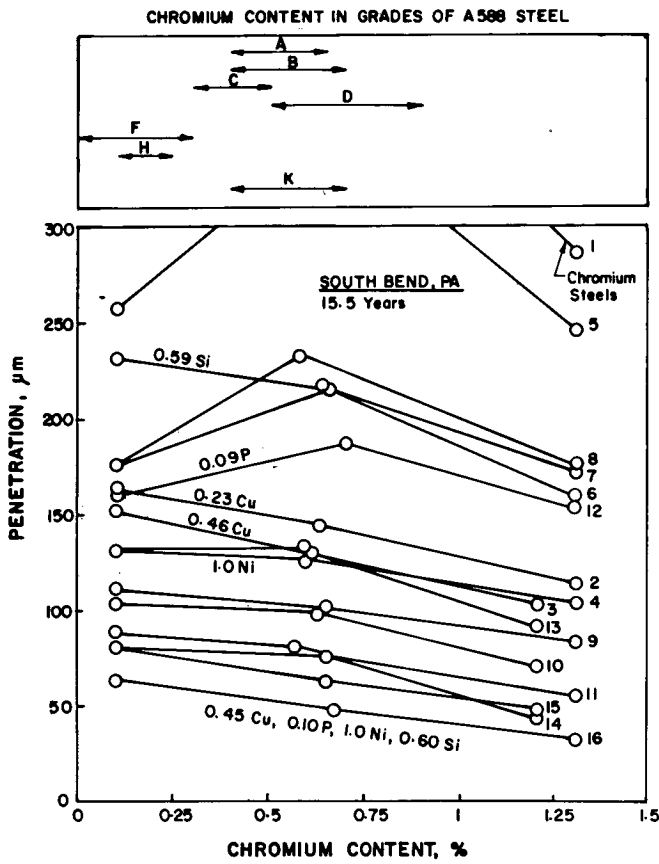


Figure 10. Effect of chromium content on corrosion penetration at South Bend, Pa. rural site, 15.5-year exposure. [Larrabee 1961]

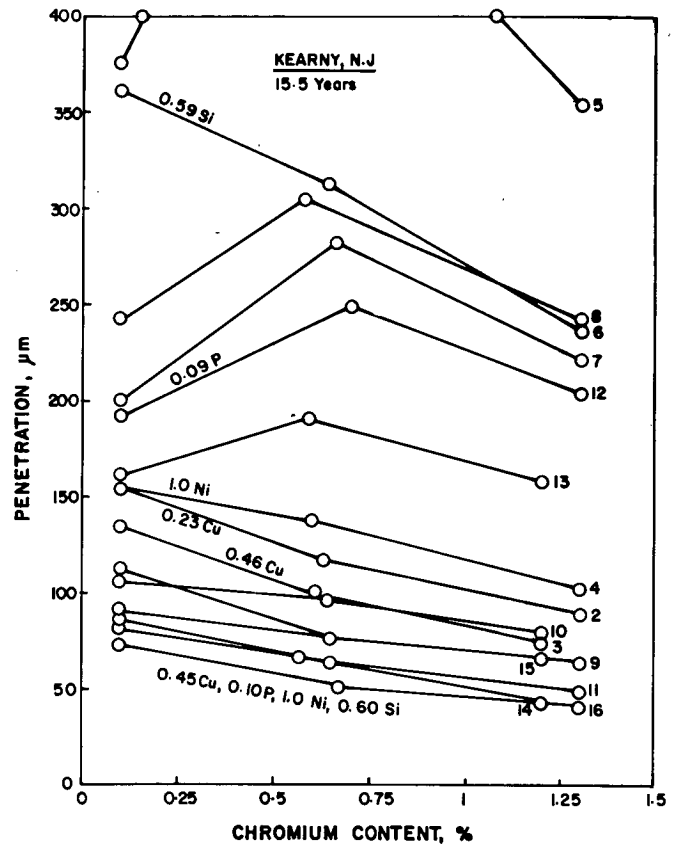


Figure 11. Effect of chromium content on corrosion penetration at Kearny, N.J. industrial site, 15.5-year exposure. [Larrabee 1961]

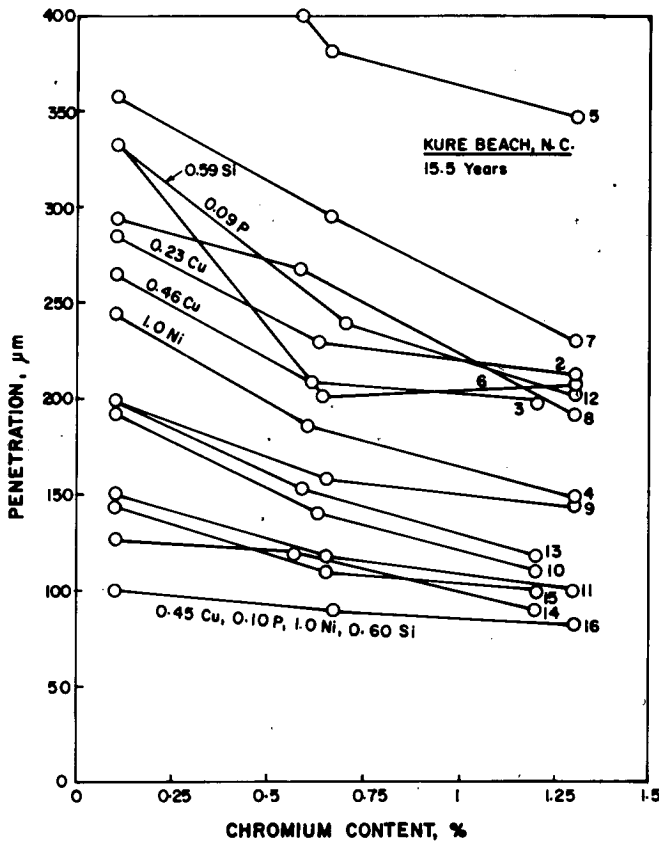


Figure 12. Effect of chromium content on corrosion penetration at Kure Beach, N.C. marine site, 15.5-year exposure. [Larrabee 1961]

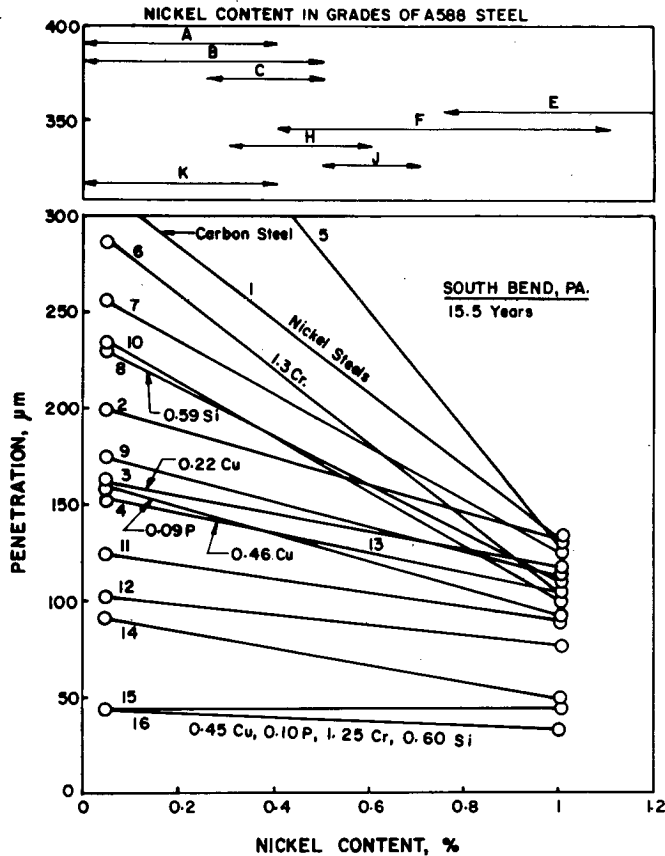


Figure 13 Effect of nickel content on corrosion penetration at South Bend, Pa. rural site, 15.5-year exposure. [Larrabee 1961]

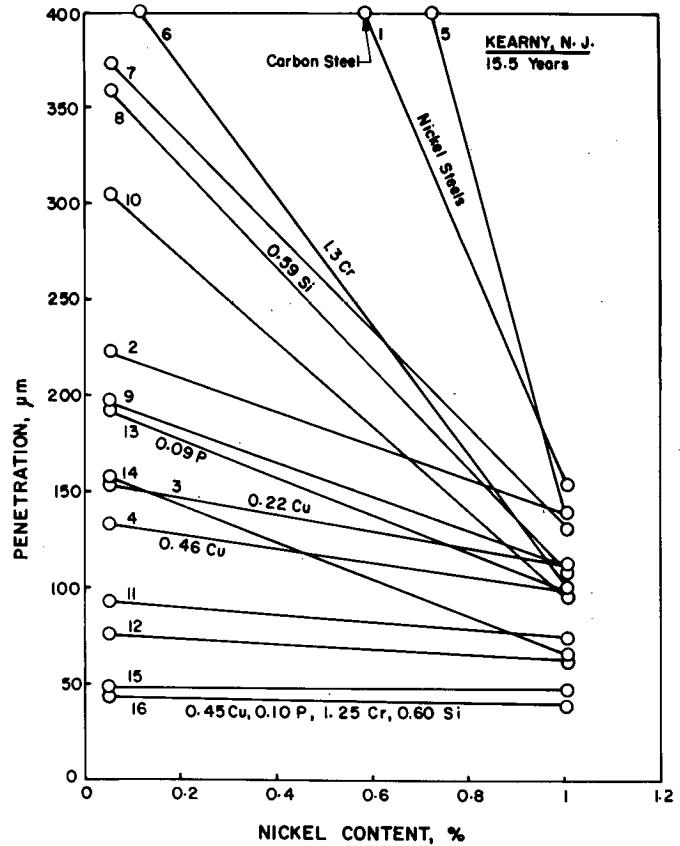


Figure 14 Effect of nickel content on corrosion penetration at Kearny, N.J. industrial site, 15.5-year exposure. [Larrabee 1961]

creased when either copper or nickel was added, as shown by the lower set of curves 2-4, 9-11, and 14-16.

The penetration was lowest for the high-concentration multielement combination (curve 16). In this instance, and for the alloy combinations involving copper and/or nickel, the benefit derived from chromium increased with the content. See, for example, the curves without peaks in Figure 11. Chromium was relatively most beneficial when the steels were exposed in the marine environment.

The chromium content of the A588 steels varies considerably. The high-copper steels are either low in chromium (Grades F and H) or they have no chromium (Grades E and J). The low-copper Grade D composition has the highest chromium range.

Nickel

Figures 13, 14, and 15, respectively, show the effect of nickel content on corrosion penetration in the rural, industrial, and marine environments. Previous unpublished work had shown that the improvement in atmospheric corrosion resistance of a steel was nearly linear with increasing nickel content. In other words, steels made with 0.5 percent nickel would have penetration losses about half way between those given in the three figures for steel with 1.0 percent nickel and those with residual

nickel. For this reason nickel was added only at the 1.0 percent level [Larrabee 1961].

The corrosion penetration of steels with nickel as the single alloying element is summarized in Table 15(d) and plotted as curve 1 in the three nickel plots. The steep slopes show the effectiveness of nickel in decreasing corrosion penetration. In the industrial environment, for example, the penetration decreased from 28.8 mils (732 μm) for the carbon steel to 6.1 mils (155 μm) for the 1.0 percent nickel steel. The equivalency between 1.0 percent nickel and 0.5 percent copper in achieving the same corrosion resistance was discussed earlier.

The corrosion penetration of steels with 0.5 percent nickel and a second alloying element is given in Table 15(d), in increasing order of corrosion resistance in the industrial environment. The 0.5 percent nickel level is of interest because it corresponds well with that found in many grades of A588 steel. The tabulated penetrations are the average of those for steels with 1.0 percent nickel and those with residual nickel, all other element concentrations being the same. The addition of copper as the second element was most beneficial. Phosphorus, silicon, and chromium followed copper, in that order. But none of the three was as effective as doubling the nickel content from 0.5 percent to 1.0 percent.

The five-element alloy combinations resulted in the lowest penetrations, as shown in curves 11-12 and 15-16. In these combinations, increasing the nickel content to 1.0 percent had

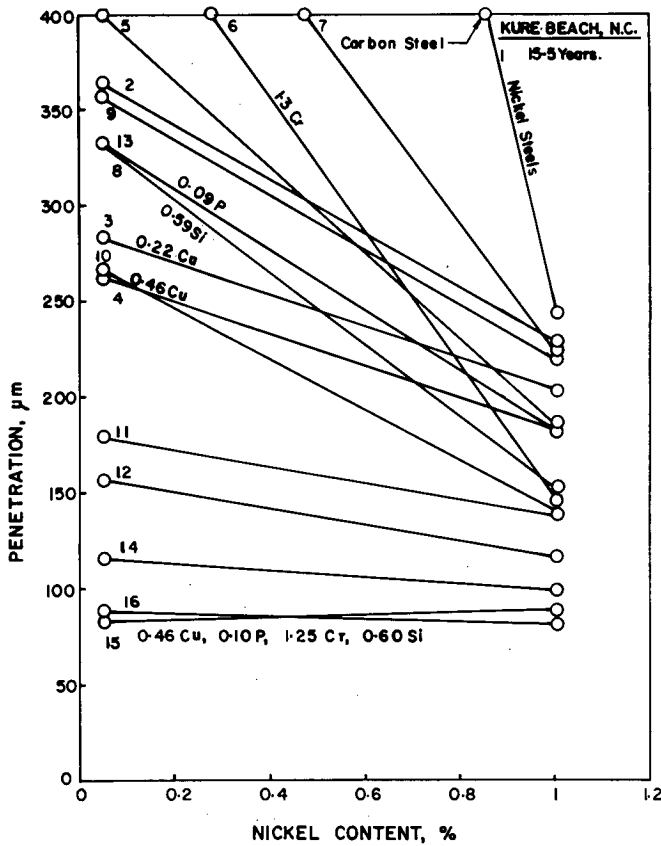


Figure 15. Effect of nickel content on corrosion penetration at Kure Beach, N.C. marine site, 15.5-year exposure. [Larrabee 1961]

no significant beneficial effect, as indicated by the negligible slope of the curves. In fact, this is the reason why some steel suppliers do not publish a minimum nickel content only a maximum. When copper was omitted (curve 14, Fig. 14), nickel assumed copper's role and reduced the penetration from 6.2 mils (157 µm) at residual nickel to 2.6 mils (66 µm) at 1.0 percent nickel. This indicates, again, that copper and nickel are interchangeable from the standpoint of conferring corrosion resistance.

The nickel content of A588 steels also varies greatly. The high-copper low-chromium grades tend to be high in nickel. The medium-copper medium-chromium grades have a lower nickel content.

Silicon

Figures 16,17, and 18, respectively, show the effect of silicon content on the corrosion penetration of steels exposed in the rural, industrial, and marine environments.

The corrosion penetration of steels alloyed with silicon only is summarized in Table 15(e) and plotted as curves 1 in the three silicon plots. Relative to the carbon steel, silicon was most beneficial in the marine environment in which the addition of 0.61 percent silicon reduced the penetration by a factor of four.

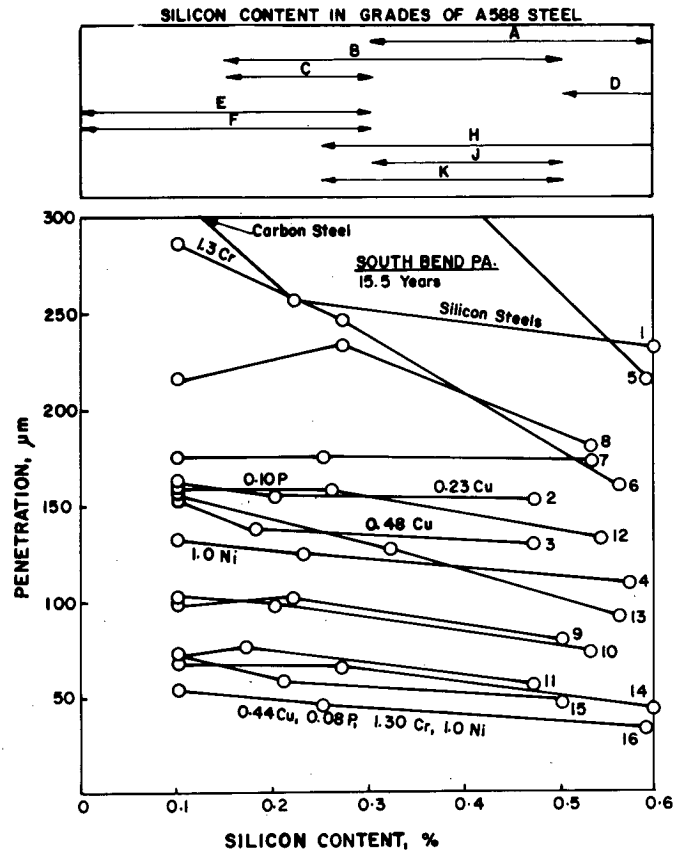


Figure 16. Effect of silicon content on corrosion penetration at South Bend, Pa. rural site, 15.5-year exposure. [Larrabee 1961]

But in no environment was 0.61 percent silicon as effective as, for example, 0.04 percent copper (steels 2 and 61 in Tables 15(a) and (e)).

The corrosion penetration of steels with about 0.55 percent silicon and a second element is given in Table 15(e). The greatest additional benefit came from adding nickel or copper, followed by phosphorus and chromium. Generally, silicon was most effective in the marine environment, as indicated by the steeper downward slopes of the two-element alloy combinations with increasing silicon content. In contrast, increasing the silicon content did not significantly reduce the corrosion penetration in the rural and industrial environments.

The multielement alloy combinations were the most effective, particularly those that included copper and nickel, as indicated by curves 9-11 and 14-16.

The silicon content of A588 steels is plotted at the top of Figure 16.

Summary

Based on their weight loss study of 270 steels with systematic variations of copper, nickel, chromium, silicon, and phosphorus after 15.5-year exposures in rural, industrial, and marine atmospheres, Larrabee and Coburn [1961] concluded that:

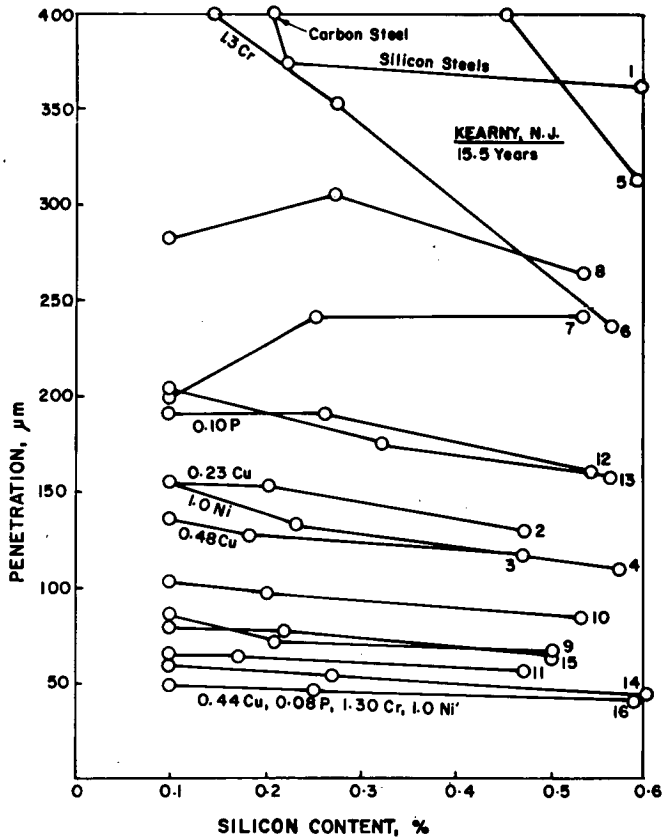


Figure 17. Effect of silicon content on corrosion penetration at Kearny, N.J. industrial site, 15.5-year exposure. [Larrabee 1961]

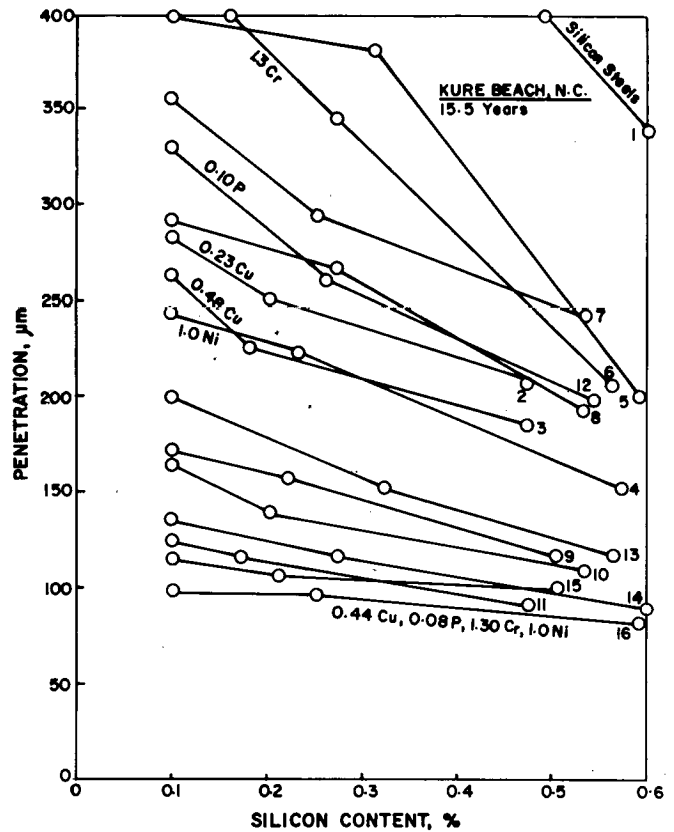


Figure 18. Effect of silicon content on corrosion penetration at Kure Beach, N.C. marine site, 15.5-year exposure. [Larrabee 1961]

1. Increasing the copper content of a steel from 0.01 percent to 0.04 percent increased the corrosion resistance more than did the addition of a similar amount of any single element investigated.

2. Further increases in copper were beneficial, but not to as great an extent as shown by the initial addition.

3. Relatively small single additions of nickel, silicon, or phosphorus improved the corrosion resistance of carbon steel, but the largest improvements were obtained by the addition of specific combinations of these alloying elements.

4. The beneficial effects of several alloying elements were not additive. Therefore, data from long-term exposure tests of many steel compositions were needed to determine the potential corrosion resistance of steel compositions with particular combinations of alloying elements.

It is apparent that various combinations of the five alloying elements discussed herein can provide a desired corrosion resistance in a given environment. Within the bounds set forth in the ASTM specification for each grade of A588 steel, the content of copper, phosphorus, chromium, nickel, and silicon chosen by a supplier depends on: (1) the desired corrosion resistance; (2) the desired tensile and toughness properties; (3) the element content naturally present in the iron ore; and (4) the availability and cost of the respective elements.

Chapter Six discusses the effect of the five aforementioned elements on mechanical properties.

EFFECT OF ALLOYING ELEMENTS FROM OTHER INVESTIGATIONS

Next to the Larrabee-Coburn study discussed in the previous section, Copson's [1960] work on atmospheric corrosion of steels is the other important source of data on the effect of alloying elements. Copson exposed 76 steel compositions in the industrial environment of Bayonne, N.J., and in the marine environments of Kure Beach, N.C., and Block Island, R.I. The steels were arranged according to composition in 13 groups. Some were nearly free of alloying elements; others were alloyed upwards to 5 percent. The primary alloying elements were copper, phosphorus, nickel, and chromium. Some of the steels also contained molybdenum and higher than normal amounts of manganese and silicon. Summarized herein are the data for the compositions identified in Table 16, in which the content of a single element was varied while the others remained unchanged.

Additional data of interest were reported by Horton [1965] for a steel composition with an alloy content resembling that of A242 Type 1 steels. All steels were exposed for 17 years in industrial Pittsburgh, Pa.

The applicable Copson and Horton data are plotted in Figures 19 to 22 to examine the respective effects of copper, chromium, nickel, and manganese on corrosion resistance. Additional information on the effect of manganese, sulfur, molybdenum, and carbon was obtained primarily from two sources: (1) Chandler and Kilcullen's review of the corrosion resistance of low-alloy

Table 16. Chemical composition of steels corresponding to curves plotted in Figures 19 through 22.

Figure No.	Reference	Steel Identification No. ^a						Composition, %					
		Cu	P	Cr	Ni	Si	Mn						
19	Copson 1960	26	11	2	12	16	3	varies	.09	.04	.05	.03	0.38
20	Copson 1960	49	59	14	13	-	-	.41	.10	varies	.30	.32	0.41
	Horton 1965	-	-	-	-	-	-	0.60	0.10	varies	0.40	0.28	0.70
21	Copson 1960	32	27	-	-	-	-	0.08	.01	.03	varies	.19	.63
	Horton 1965	-	-	-	-	-	-	0.60	0.10	0.60	varies	0.28	0.70
22	Copson 1960	55	40	64	56	66	17	0.28	0.10	0.3	.06	.07	varies
	Horton 1965	-	-	-	-	-	-	0.60	0.10	0.6	0.4	0.28	varies

a. The steel identification Nos. are the same as those in Ref. [Copson 1960].

steels in the generally more corrosive British environments [Chandler 1970]; and (2) Boyd's review of corrosion of metals in the atmosphere, including the weathering steels [Boyd 1974].

The effect of copper, chromium, nickel, manganese, sulfur, molybdenum, and carbon is summarized in the following.

Copper

Copson's data for two marine environments (curves 1 to 4) and one industrial (curves 5 and 6) environment, plotted in Figure 19, show a rapid decrease in corrosion penetration when the copper content is increased to 0.2 percent. Thereafter, the gain in corrosion resistance is minimal. These steels had, in addition to copper, 0.09 percent phosphorus and negligible amounts of other elements (Table 16). The comparable steel compositions evaluated by Larrabee and Coburn had 0.10 percent phosphorus (Table 14(a), curve 10). The data from both studies correlated well. Despite some differences in exposure conditions, both the shape and ordinates of curve 10 in Figure 5 for industrial Kearny, N.J., are quite similar to those of curve 5 in Figure 19 for industrial Bayonne, N.J. Similarly, curve 10 in Figure 6 for marine Kure Beach, N.C., compares well with curve 1 in Figure 19 at the same site. The corrosion penetration of the copper-phosphorus steels exposed in the marine environment was about a factor of two greater than in the industrial environment.

Chromium

The beneficial effect on corrosion resistance of increasing the chromium content is shown by Copson's data for the same three sites, plotted in Figure 20. The steels contained 0.41 percent Cu, 0.10 percent P, 0.30 percent Ni, and 0.32 percent Si and varying amounts of chromium. The most closely related steels in the Larrabee-Coburn series contained 0.46 percent Cu, 0.10 percent P, less Ni (0 percent), and more Si (0.57 percent) (Table 14(c), curve 14). The findings from both studies again were quite similar as is evident by comparing curves 1 and 5 in Figure 20 with curve 14 in Figures 11 and 12, respectively.

Horton's data for A242 Type 1 steel further confirm the beneficial effect of chromium [Horton 1965]. Curve 7 in Figure 20 for the industrial Pittsburgh, Pa., environment shows the same trend as the other curves.

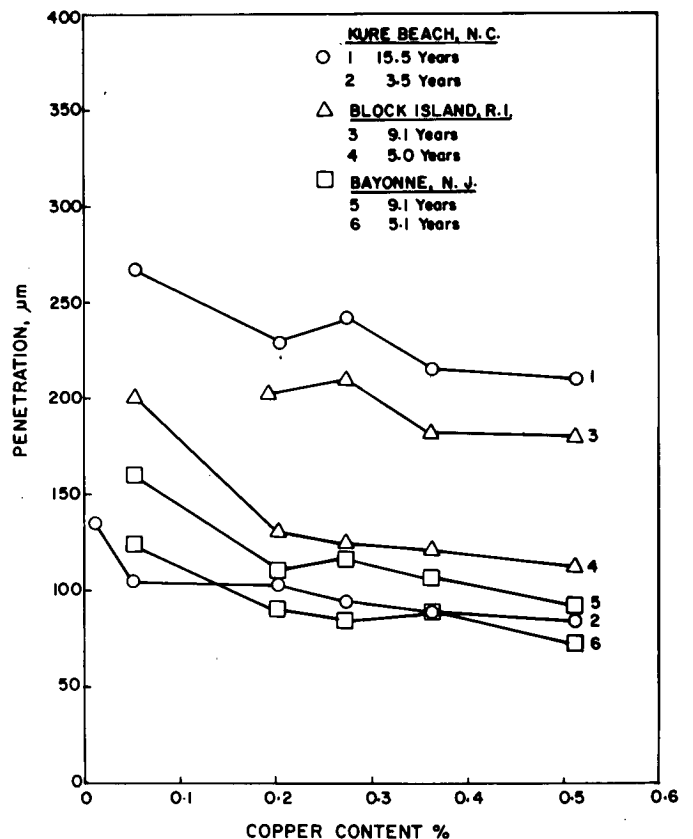


Figure 19. Effect of copper content on corrosion penetration of low-alloy steels. [Copson 1960]

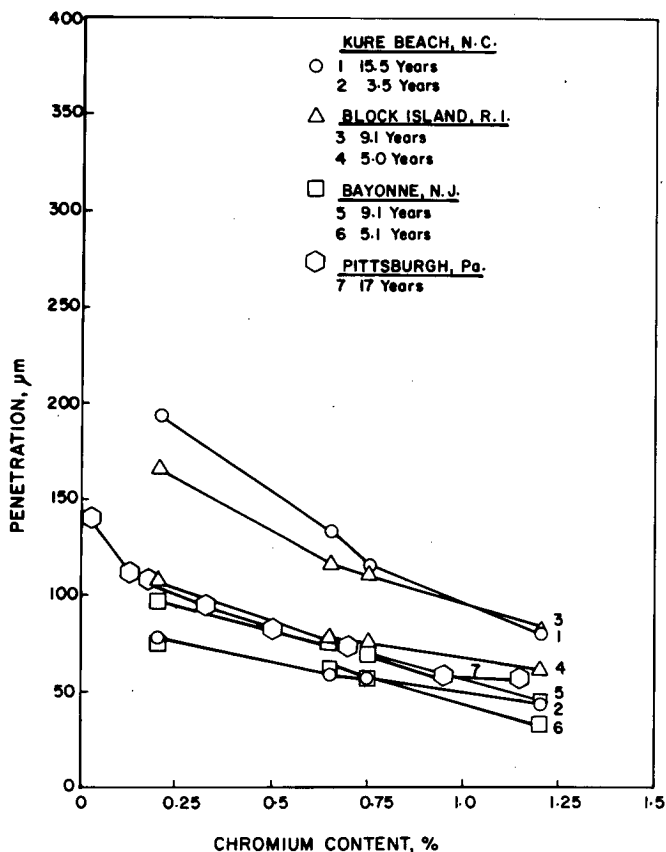


Figure 20. Effect of chromium content on corrosion penetration of low-alloy steels. [Copson 1960, Horton 1965]

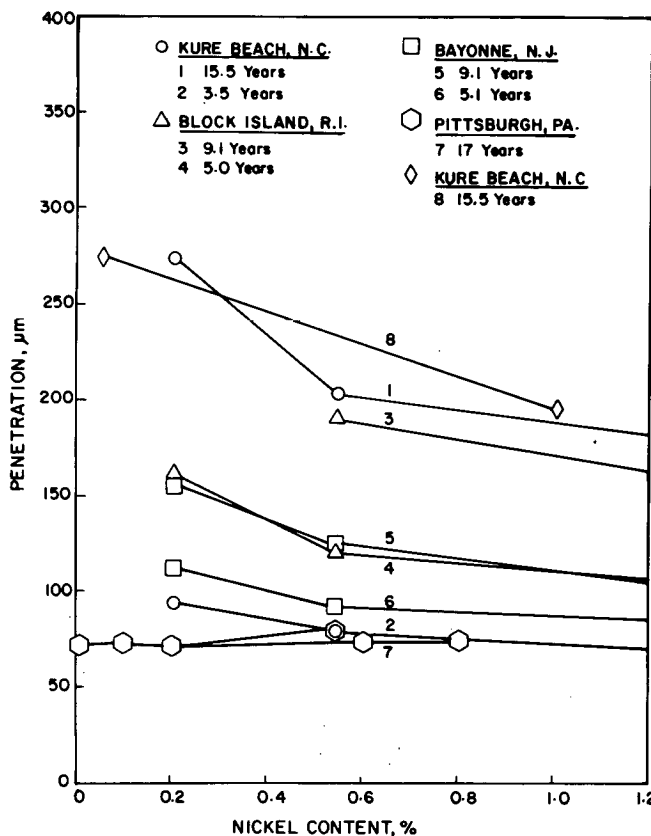


Figure 21. Effect of nickel content on corrosion penetration of low-alloy steels. [Copson 1960, Horton 1865]

Nickel

Copson's data on the effect of nickel, plotted as curves 1 to 6 in Figure 21, show the same trend as seen in the Larrabee-Coburn data plotted in Figures 13 to 15. That is, adding nickel has little effect when the corrosion penetration is already low.

The addition of nickel, however, is increasingly more beneficial at higher corrosion penetrations. The data from both sources correlated well for the 15.5-year exposures at Kure Beach, N.C., given by curves 1 and 8 in Figure 21. The former is for steels containing 0.08 percent Cu and 0.19 percent Si; the latter for steels with 0.08 percent Cu and 0.22 percent Si (mean of steels No. 32/33 and 37/38). All compositions contained varying amounts of nickel. The noticeable difference between the nickel plots of the Copson and Larrabee-Coburn data is the shape of the curves. Based on previous unpublished data, Larrabee and Coburn had assumed that the improvement in corrosion resistance of a steel was nearly linear with increasing nickel content. For this reason, they tested at residual and 1 percent nickel contents only. Copson's nickel curves have a slight nonlinear shape.

Horton's data for A242 Type 1 steel confirm Larrabee and Coburn's finding that nickel has no significant beneficial effect when it is added to steels that already are alloyed with Cu, P, Cr, and Si. This is evident from curve 7 in Figure 21 and curve 12 in Figure 14, both being for steels of comparable compositions.

Manganese

Copson's curves 1 to 6, plotted in Figure 22, are for steels containing 0.28 percent Cu, 0.10 percent P, 0.30 percent Cr, varying amounts of manganese and residual quantities of other alloying elements (Table 16). They show a significant reduction in corrosion penetration of medium copper steels exposed for many years in marine environments, when the manganese content is increased to 0.60 percent. Beyond this level the gain in corrosion resistance is not significant. Manganese had practically no effect when the copper-bearing steel was exposed in the Bayonne, N.J., industrial environment.

According to Horton, the corrosion resistance of A242 Type 1 weathering steel is not affected by the manganese content (curve 7, Fig. 22). Evidently, in high-strength low-alloy steels, any beneficial effect of manganese is masked by the more potent alloying elements. Boyd reported that the amount of sulfur in the steel influences the effect of manganese [Boyd 1974]. If the ratio of manganese to sulfur is greater than four, manganese sulfide forms. Its low electrical conductivity, he concluded, improves corrosion resistance. If insufficient manganese is present, iron sulfides of good electrical conductivity predominate and promote the formation of local corrosion cells. Although all grades of A588 have a manganese-to-sulfur ratio of 10, the other elements mask any beneficial effect of manganese, as Horton had shown.

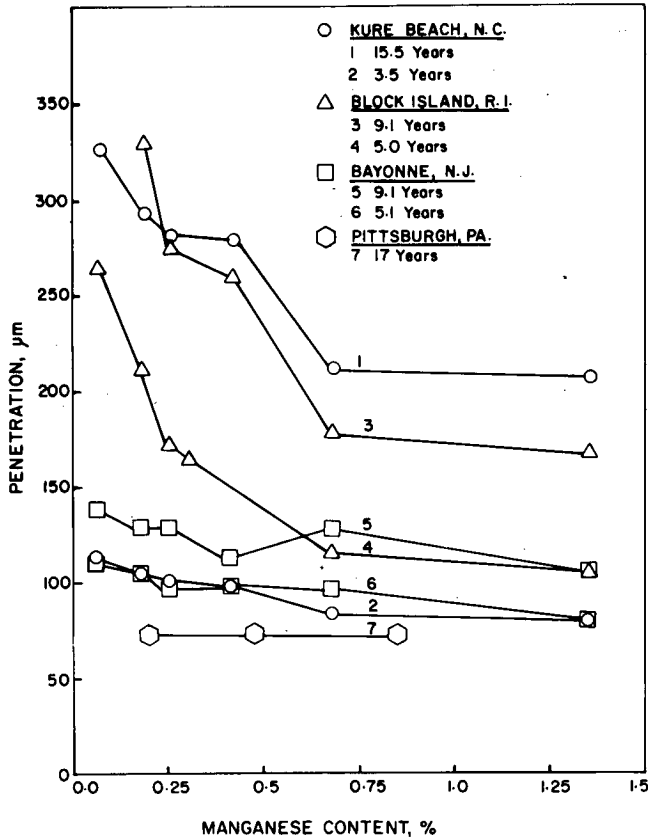


Figure 22. Effect of manganese content on corrosion penetration of low-alloy steels. [Copson 1960, Horton 1965]

Molybdenum

Chandler [1970] concluded from an analysis of data reported by others that molybdenum may be a particularly effective alloying element in British environments. For example, Edwards [1966] had shown molybdenum to be as effective as copper over a 9-year exposure period in British rural, industrial, and marine sites. On adding 0.40 percent molybdenum, the corrosion rate was cut in half at each site. Adding 0.50 percent to 0.40 percent molybdenum reduced the corrosion rate further in the industrial atmosphere, but had little effect in the rural and marine atmospheres. In other tests in British atmospheres, 0.5 percent molybdenum reduced the corrosion rate of mild steel from 5.6 to 3.5 mil (140 to 90 μm) per year [Hudson 1955]. Finally, Horikawa [1966] showed that molybdenum was more effective than either chromium or nickel in copper-phosphorus steels.

Only four grades of A588 (E, F, H, and K) contain molybdenum in amounts up to 0.25 percent. The remaining grades (A, B, C, D, and J) do not list molybdenum as an alloying element.

Sulfur

All the evidence indicates that sulfur is harmful [Chandler 1970]. For example, when the sulfur content of A242 Type 1

steel was increased from a residual amount to 0.1 percent, the 17-year corrosion penetration of the steel in industrial Pittsburgh, Pa., increased from 2.8 to 3.3 mils (71 to 84 μm) [Horton 1965].

Other Elements

The other elements found in A588 weathering steels are carbon, vanadium, zirconium, columbium, and titanium. Carbon does not appear to affect the corrosion resistance, except for possibly being harmful in high-chromium steels [Hudson 1955]. Titanium was reported to have about the same effect as nickel [Otake 1962]. In fact, copper-phosphorus-titanium steels exhibited corrosion resistance superior to that of copper-phosphorus steels and copper-phosphorus-chromium steels [Muta 1965]. Vanadium (0.25 percent) steels without copper and columbium were not effective [Hudson 1955]. Within the range of alloy contents specified in ASTM A588, none of these other elements contribute to the corrosion resistance of the weathering steels.

PREDICTION OF CORROSION PENETRATION

Legault and Leckie derived equations that allow one to estimate the 15.5-year corrosion penetration of a weathering steel with a given chemical composition [Legault 1974]. Because the equations are based on the Larrabee-Coburn data, they apply, in a strict sense, only to environments comparable to those in semirural South Bend, Pa., industrial Kearny, N.J., and marine Kure Beach, N.C. Quadratic curves were fitted to the data with stepwise multiple-regression analysis techniques. The model initially consisted of five main effect terms (one for each element), ten binary-interaction terms, and five squared main effect terms. The statistically insignificant terms were eliminated.

The regression equations listed below were obtained for the average corrosion penetration, C , at the three sites after 15.5 years exposure. The element concentration in percent must be substituted for the terms given in parentheses to obtain the corrosion penetration in mils. No attempt was made to extrapolate the results beyond the compositional levels of the Larrabee-Coburn data set.

South Bend, Pa., rural atmosphere:

$$C \text{ (mils)} = 8.50 - 13.39 (\% \text{ Cu}) - 3.03 (\% \text{ Ni}) - 21.27 (\% \text{ P}) + 3.48 (\% \text{ Cu } \% \text{ Ni}) + 2.41 (\% \text{ Cu } \% \text{ Si}) + 9.55 (\% \text{ Ni } \% \text{ P}) - 1.11 (\% \text{ Cr } \% \text{ Si}) + 15.31 (\% \text{ Cu})^2 - 0.65 (\% \text{ Cr})^2 - 2.82 (\% \text{ Si})^2$$

Kearny, N.J., industrial atmosphere:

$$C \text{ (mils)} = 10.00 - 26.01 (\% \text{ Cu}) - 3.88 (\% \text{ Ni}) - 1.20 (\% \text{ Cr}) - 1.49 (\% \text{ Si}) - 17.28 (\% \text{ P}) + 7.29 (\% \text{ Cu } \% \text{ Ni}) + 9.10 (\% \text{ Ni } \% \text{ P}) + 33.39 (\% \text{ Cu})^2$$

Kure Beach, N.C., marine atmosphere:

$$C \text{ (mils)} = 15.49 - 16.30 (\% \text{ Cu}) - 4.34 (\% \text{ Ni}) - 4.79 (\% \text{ Cr}) - 12.41 (\% \text{ Si}) - 32.01 (\% \text{ P}) + 2.93 (\% \text{ Cu } \% \text{ Ni}) + 2.46 (\% \text{ Cu } \% \text{ Cr}) + 4.36 (\% \text{ Cu } \% \text{ Si}) + 2.74 (\% \text{ Ni } \% \text{ Si}) + 12.82 (\% \text{ Ni } \% \text{ P}) + 1.75 (\% \text{ Si } \% \text{ P}) + 16.60 (\% \text{ Cu})^2 + 1.20 (\% \text{ Cr})^2 + 4.25 (\% \text{ Si})^2$$

Optimum additive levels of each element, within the compositional ranges tested by Larrabee-Coburn, were determined by taking partial derivatives of the predictive equations with respect to each of the five additives. The optimum levels of each element are given in Table 17 for each exposure site. The letter "a" indicates that the element concentration is the maximum tested by Larrabee and Coburn. For the South Bend, Pa., and Kearny, N.J., exposure sites, only the copper concentration was optimum at a level other than the highest represented in the data set. For the Kure Beach, N.C., exposure site, silicon joined copper in showing an optimum concentration at an intermediate level. All other elements reached their optimum level at the maximum concentration in the data set.

The three Legault-Leckie equations were used to predict the corrosion penetration of the nine grades of A588 steel, were they to be exposed for 15.5 years in South Bend, Pa., Kearny, N.J., and Kure Beach, N.C. The range of alloy content in each grade does not always fall entirely within the corresponding concentration range of the data set. To avoid extrapolation of the equations, the maximum and minimum alloy levels in any grade of A588 steel were, therefore, not allowed to exceed the maximum and minimum concentrations in the data set. The results of the predictions are summarized in Table 18. As can be seen, all grades of steel exhibit comparable penetrations in a given environment. Thus, a desired corrosion resistance can be achieved, indeed, with several combinations of the five alloying elements: copper, phosphorus, chromium, nickel, and silicon.

The Legault-Leckie equations supply a useful tool for comparing the relative corrosion resistance of steels with different chemical compositions. They show also that, even if the concentration of each element in a grade of weathering steel were

kept at the lowest level, which is unlikely, these steels should still exhibit a 15.5-year corrosion penetration of not more than 6.4 mils (163 μm) in the rural, 7.1 mils (180 μm) in the industrial, and 9.3 mils (236 μm) in the marine environments tested.

The penetrations reported for some weathering steel bridges that are exhibiting severe corrosion exceed these values after 1 year. The excessive penetration cannot be explained in terms of a lean composition. It is caused primarily by a misapplication of the steel in unsuitable environments.

Table 17. Optimum concentration of alloying elements. [Legault 1974]

Alloying Element	Optimum Alloy Concentration, %		
	South Bend, Pa.	Kearny, N.J.	Kure Beach, N.C.
Copper	0.17	0.18	0.25
Phosphorus	0.12 ^a	0.12 ^a	0.12 ^a
Chromium	1.30 ^a	1.30 ^a	1.30 ^a
Nickel	1.10 ^a	1.10 ^a	1.10 ^a
Silicon	0.64 ^a	0.64 ^a	0.42

Note:

a. Maximum concentration in data set.

Table 18. Predicted 15.5-year range of corrosion penetration for A588 steels.

Grade of A588 Steel	Minimum and Maximum Concentration, %					Predicted 15.5-Year Corrosion Penetration, Mil		
	Cu	P	Cr	Ni	Si	South Bend, Pa.	Kearny N.J.	Kure Beach, N.C.
	A	0.25 0.40	0.01 0.04	0.40 0.65	0.05 ^b 0.40	0.30 0.65	5.5 2.9	4.4 2.3
B	0.20 0.40	0.01 0.04	0.40 0.70	0.05 ^b 0.50	0.15 0.50	5.9 2.9	5.1 2.3	9.3 2.9
C	0.20 0.50	0.01 0.04	0.30 0.50	0.25 0.50	0.15 0.30	5.6 4.1	4.8 3.7	9.0 5.3
D	0.05 0.30	0.01 0.04	0.50 0.90	0.50 ^b 0.05 ^b	0.05 0.60	6.4 3.2	7.1 2.5	7.2 3.0
E	0.50 0.50 ^a	0.01 0.04	0.10 ^b 0.10 ^b	0.75 1.10 ^a	0.10 ^b 0.30	4.6 3.9	4.8 4.2	8.0 5.5
F	0.30 0.50 ^a	0.01 0.04	0.10 ^b 0.30	0.40 1.10	0.10 ^b 0.30	4.9 3.7	4.1 4.0	9.1 4.9
H	0.20 0.35	0.01 0.035	0.10 0.25	0.30 0.60	0.25 0.60 ^a	5.5 3.3	4.8 2.6	8.7 4.1
J	0.30 0.50 ^a	0.01 0.04	0.10 ^b 0.10	0.50 0.70	0.30 0.50	4.6 4.0	3.7 3.9	7.2 5.2
K	0.30 0.50	0.01 0.04	0.40 0.70	0.05 ^b 0.40	0.25 0.50	5.3 3.6	4.1 3.1	7.7 3.9

Notes:

- Concentration limited by maximum value in data set. Maximum concentration in A588 specification is higher.
- Concentration limited by minimum value in data set. Minimum concentration in A588 specification is lower.

ATMOSPHERIC CORROSION

Chapter Four examines the atmospheric corrosion penetration data for weathering steels ideally and boldly exposed in rural, industrial, and marine environments. Corrosion performance under conditions that deviate from bold exposure are covered in Chapter Five under "Service Corrosion."

All values of corrosion penetration are per exposed side (surface) of the specimen.

TEST METHOD

The Standard Recommended Practice for Conducting Atmospheric Corrosion Tests on Metals, ASTM Designation G50, sets forth the general procedures that should be followed in conducting any atmospheric exposure test of metals and alloys. These tests are commonly used to evaluate the corrosion resistance of weathering steels.

Multiyear exposure periods should be considered to minimize the influence of the variability and complexity of weather effects and the industrial and natural factors influencing the atmospheric corrosivity of a test site. Recognizing that the corrosivity may vary at a site from season to season, the Standard recommends that specimen exposures be made either at the same time of the year, to minimize variability, or that these differences should be established by multiple exposures.

Control specimens of a composition having established weathering characteristics should also be exposed along with weathering steel specimens. The common control specimens are made of steels containing 0.02 percent max. copper and those containing 0.20 to 0.22 percent copper. They are hereafter referred to as carbon and copper (copper-bearing) steels, respectively.

The Standard's recommendations ensure uniformity of test practice and eliminate extraneous variables. The resulting exposure conditions are, therefore, somewhat reproducible but not typical of bridge service conditions. Summarized below are the principal recommendations for test site location, exposure rack fabrication, test specimen size, specimen preparation, and calculation of corrosion penetration. Suggestions for special types of exposure are also included.

Test sites should be chosen at a number of locations representative of the environment where the steel is likely to be used. For general testing of a new steel composition's corrosion resistance, the selection should include sites representative of rural, industrial, and marine environments.

Exposure racks should be located in cleared, well-drained areas such that the exposed specimens will be subjected to the full effects of the atmosphere at the location of the test site. Shadows of trees, buildings, or structures should not fall on the specimens. Local contaminants of the atmosphere should be avoided, unless the specific influences of such conditions are intended to be assessed. The test racks should be designed to

give exposure to as large an area of the underside of the specimens as possible. Maximum exposure to the sun can be obtained by exposing specimens facing south (for the northern hemisphere) at an angle equal to the latitude of the test site. Exposure at this angle will yield the lowest corrosion rate. For this reason, most American data on atmospheric corrosion of weathering steels are based on an exposure angle of 30 deg from the horizontal, facing south. Most continental European data are based on 45 deg, and most British data on 90 deg (vertical) exposure angles. However, in special instances, it may be desirable to orient the racks in the direction of a specific corrodent source, such as an industrial plant or the ocean, or at a specific exposure angle. Where it is important to obtain corrosion rates involving a microenvironment, such as bridge, specimens should be mounted directly on the structure.

The ground beneath the exposure racks should be kept free of weeds, bushes, and debris. Organic herbicides, defoliants, or pesticides should not be used for this purpose.

The steel test specimens should not be smaller than 4 by 6 in. (100 by 150 mm), nor thinner than 0.030 in. (0.75 mm). For reliable results, triplicate or more specimens should be used for multiple removals at each exposure period. A suggested removal schedule is 1, 2, 4, 8, and 16 years.

ASTM Standard Practice G1 recommends procedures for Preparing, Cleaning and Evaluating Corrosion Test Specimens. According to this standard, a suitable procedure for preparing the specimens for testing might include: (1) degreasing in an organic solvent or hot alkaline cleaner; (2) pickling in an appropriate solution if oxides or tarnish are present; (3) abrasion of the steel surfaces with a slurry of an appropriate abrasive, or with an abrasive paper; (4) removal of edge burrs; and (5) rinsing and drying.

Any mill scale or rust should be removed from all ferrous specimens unless it is specifically desired to perform the test with the mill scale intact. Pickling with inhibited acid as well as blasting with sand or grit are acceptable descaling methods.

At the end of an exposure interval, the corrosion products are removed from the surfaces of the specimens either by electrolytic cleaning or by chemical cleaning. Electrolytic cleaning may result in re-deposition of metal, such as copper from reducible corrosion products and, thus, lower the apparent mass loss.

The average corrosion penetration per side is calculated from the mass lost during the time of exposure as follows:

$$C = K \frac{W}{AD} \quad (22)$$

where:

K = conversion constant;

C = average corrosion penetration per side, in μm ;
 W = mass loss, in grams;
 A = total surface area (all sides), in cm^2 ; and
 D = density, in g/cm^3 .

The constant is $K = 10^4$ when the other variables are substituted in the indicated units. The density of low-alloy steels is $D = 7.85 \text{ g}/\text{cm}^3$.

TYPES OF ENVIRONMENTS

This chapter examines the data reported in the literature for test panels exposed to the atmosphere in macroenvironments broadly classified by human activity as being rural, urban, industrial, or marine. Unless noted otherwise, the data reported come from specimens exposed at an angle of 30 deg from the horizontal, facing south.

Rural Environments

Rural environments are usually free of aggressive agents. Their principal corrosives consist of moisture, oxygen, relatively small amounts of sulfur oxides (SO_x), and carbon dioxide (CO_2) from various combustion products. Ammonia (NH_3), resulting from the decomposition of farm fertilizers and animal excrement may also be present. Steels rust significantly when the relative humidity exceeds a certain value. For clean air this value is about 70 percent. However, moisture-saturated air produces little rusting when dust and pollution are minimal. Some forms of particulate, such as silica particles, cause no rusting, whereas those of ammonium sulfate are very aggressive. Rural environments generally are not aggressive toward steel.

Urban Environments

Urban environments contain sulfur oxides and nitrogen oxides from the combustion of fossil fuels and from automobile exhaust gases. Both contaminants promote the corrosion of steels. Because they are found also, to a larger degree, in industrial environments, many studies combine urban and industrial environments into one. This approach is taken herein.

Industrial Environments

The most potent causes of corrosion in industrial environments are the sulfur oxides (SO_x) and nitrogen oxides (NO_x) produced burning automotive fuels and fossil fuels in power stations [Evans 1972a, Boyd 1974, Fletcher 1979]. The sulfur oxides dissolve in films of condensed moisture on steel surfaces where they can be further oxidized to form sulfuric acid that attacks the steel. Sulfur oxide also can be windborne great distances before droplets condense out on a steel surface. Such transport is termed "acid rain." Atmospheric corrosion is most severe where the sulfur oxide levels are high and nightly temperature drops are sufficient to result in condensation. Steels rust more rapidly during the heating season when the production of sulfur oxides reach a maximum. However, the sulfur oxide emission levels are decreasing since the advent of the regulations by the

Environmental Protection Agency (EPA) which require scrubbing of the effluent gases from such combustion. The critical relative humidity, above which steels are attacked, drops to between 50 and 60 percent when the air is contaminated with hygroscopic pollutants that deposit on the steel where they attract and retain moisture. In the absence of moisture and condensing conditions, sulfur oxides are not necessarily aggressive toward steel.

Marine Environments

The losses from corrosion are greatest in a marine environment. Airborne salt spray droplets, salt fog, and nightly temperature drops can keep the steel damp for long periods in certain environments. On drying, salt crystals hygroscopically attract moisture at lower relative humidities than the 80 percent level found by Vernon. In fact the critical humidity is that approaching the vapor pressure of a saturated solution of the salt [Bukowiecki 1966]. Thus, visible condensate in the form of droplets is not necessary for corrosion to proceed. The aggressiveness of a marine environment is dependent on the nature of wave action at the surf line, prevailing wind direction, shoreline topography, and relative humidity. Corrosivity from marine salts decreases rapidly with increasing distance from the shore [LaQue 1951, Hein 1977, ASTM 1968]. Severe storms can carry salt spray inland as much as 10 miles [Ambler 1953]. Insofar as the weathering steels are concerned, the presence of chlorides inhibits the role of sulfur oxides in forming a protective oxide film. Information gained from the performance of the weathering steels in marine exposure tests, and from structures built in coastal areas, aid in determining their behavior when contaminated with roadway deicing salts.

Corrosiveness of Atmospheric Test Sites

During the periods 1951–1956 and 1960–1962, ASTM Committee G-1 annually exposed carbon steel and zinc specimens at a number of atmospheric test sites around the world, although primarily in the United States, to determine their relative corrosiveness toward these specific metals and the effect of periodic variation in weather conditions on the corrosion rate [ASTM 1968]. In the first study there were eight sites in the United States and eight in Canada. The second study was expanded to include an additional 19 sites in the United States, five in England, four in Panama, and one each in the Phillipine Islands and Australia. Several sites are of particular interest because they are the same as those for which weathering steel data are examined in this chapter. They are: Pittsburgh and Bethlehem, Pa.; the Battersea district in London, England; Newark and Bayonne, N.J.; Kure Beach, N.C.; and Point Reyes, Calif.

Table 19 summarizes the average corrosion penetration per side for the carbon steel specimens after 2-year exposure periods during both studies. Also given is the rating of each site relative to the corrosion penetration measured at State College, Pa., a rural site where the data were most nearly reproducible over many years. It is apparent that a wide diversity in atmospheric corrosiveness exists between the various sites as well as weather-related significant yearly variations at a given site. Two extreme examples among the American sites illustrate these points. First, at the 80-ft (24-m) lot in Kure Beach, N.C., the severe marine

Table 19. Relative corrosiveness of atmospheres. [ASTM 1968]

Ranking	Location	Type of Environment	1-Year Exposure		2-Year Exposure	
			Penetration (μm)	Rating	Penetration (μm)	Rating
1	Norman Wells, N.W.T., Canada	Rural	.45	.02	3.0	.07
2	Phoenix, Az.	Rural	6.60	.26	9.2	.20
3	Saskatoon, Sask., Canada	Rural	6.17	.25	11.4	.25
4	Esquimalt, Vancouver Island, B.C., Canada	Rural Marine	17.3	.69	26.7	.53
5	Detroit, Mich.	Industrial	23.0	.93	28.9	.63
6	Fort Amador Pier, Panama, C.Z.	Marine	14.8	.69	29.2	.65
7	Morenci, Mich.	Rural	26.7	1.07	39.2	.85
8	Ottawa, Ont., Canada	Urban	25.1	1.00	39.5	.86
9	Potter County, Pa.	Rural	21.8	.88	41.1	.89
10	Waterbury, Conn.	Industrial	--	--	45.2	.95
11	State College, Pa.	Rural	25.1	1.00	45.9	1.00
12	Montreal, Quebec, Canada	Urban	32.5	1.30	47.1	1.02
13	Melbourne, Australia	Industrial	34.1	1.37	52.2	1.14
14	Halifax (York Redoubt), N.S.	Urban	31.7	1.27	53.4	1.16
15	Durham, N.H.	Rural	35.4	1.42	54.7	1.19
16	Middletown, Ohio	Semi-Industrial	36.2	1.44	57.6	1.25
17	Pittsburgh, Pa.	Industrial	42.8	1.70	61.3	1.33
18	Columbus, Ohio	Urban	41.1	1.65	65.8	1.43
19	South Bend, Pa.	Semi-rural	39.9	1.60	66.6	1.45
20	Trail, B.C., Canada	Industrial	48.5	1.95	69.5	1.51
21	Bethlehem, Pa.	Industrial	55.1	2.21	75.3	1.64
22	Cleveland, Ohio	Industrial	53.1	2.13	78.2	1.70
23	Miraflores, Panama, C.Z.	Marine	55.9	2.29	86.0	1.87
24	London (Battersea), England	Industrial	57.2	2.25	94.6	2.06
25	Monroeville, Pa.	Semi-Industrial	57.2	2.29	97.9	2.13
26	Newark, N.J.	Industrial	72.4	2.90	102	2.21
27	Manila, Philippines	Marine	66.6	2.67	108	2.34
28	Limon Bay, Panama, C.Z.	Marine	51.0	2.10	125	2.70
29	Bayonne, N.J.	Industrial	127	5.08	155	3.37
30	East Chicago, Ind.	Industrial	111	4.43	169	3.68
31	Cape Kennedy, 0.8 km from the ocean	Marine	41.1	1.65	173	3.75
32	Brazos River, Tex.	Industrial-Marine	107	4.30	187	4.06
33	Pilsey Island, England	Industrial-Marine	89.3	3.60	206	4.48
34	London (Stratford), England	Industrial	156	6.25	223	4.85
35	Halifax (Federal Building), N.S.	Industrial	137	5.50	227	4.95
36	Cape Kennedy, 54 m from the ocean, 18 m elevation	Marine	61.3	2.46	263	5.72
37	Kure Beach, N.C., 240 m lot	Marine	85.1	3.41	292	6.35
38	Cape Kennedy, 54 m from the ocean, 9 m elevation	Marine	70.8	2.84	330	7.17
39	Daytona Beach, Fla.	Marine	209	8.36	592	12.9
40	Widness, England	Industrial	398	15.95	716	15.5
41	Cape Kennedy, 54 m from the ocean, ground level	Marine	191	7.66	884	19.2
42	Dungeness, England	Marine	302	12.2	979	21.3
43	Point Reyes, Ca.	Marine	315	12.6	1004	21.8
44	Kure Beach, N.C., 24 m lot	Marine	712	28.5	1070	23.3
45	Galeta Point Beach, Panama, C.Z.	Marine	652	26.2	1382	30.2
46	Cape Kennedy, Beach		1057	42.5	--	--

atmosphere was 117 times more aggressive than the desert atmosphere in Phoenix, Ariz. Secondly, the wide variations in rainfall, temperature, and fog that characterize Point Reyes on the Pacific Ocean in northern California made this site 11.5 times more corrosive during a 2-year period in 1960-1962 than during a 2-year period a decade earlier. Variations at other sites were much lower.

The ASTM study suggested that rural atmospheres are less corrosive than industrial atmospheres and much less than marine atmospheres. But, this broad classification by macroenvironment is not necessarily an accurate description of the aggressiveness of the atmosphere towards steel.

Examining the industrial sites, one finds that Detroit, Mich., Middletown, Ohio., and Pittsburgh and Bethlehem, Pa., are known for heavy industrial activity. Nevertheless, the effects of local weather conditions and the location of the test sites made the atmospheres only mildly corrosive by comparison with the Battersea and Stratford districts in London, and Pilsey Island and Widness, England. The heavy concentration of industry in Newark and Bayonne, N.J., and East Chicago, Ill., is reflected by the relatively high corrosiveness of their atmospheres.

The observed aggressiveness of the marine sites greatly varied with the direction of prevailing winds, rainfall, fog, temperature, and the yearly variation of these factors. With the exception of the seaside site at Galeta Point Beach, Panama, tropical locations are not as aggressive toward the steel specimens. There are explanations to account for the severity of several of the American marine sites. For example: Brazos River, Tex., is near a chemical complex; Point Reyes, Calif., has variable rainfall and frequent salt fog; and Cape Kennedy, Fla., and the 80-ft (24-m) lot in Kure Beach, N.C., are subjected to salt spray.

Local industrial activity, traffic density, yearly changes in weather patterns, and the exact location of the exposure rack can result in a misleading measure of the actual aggressiveness of the atmosphere at a particular geographic location. The following examples are cited in the study. At Daytona Beach, Fla., installation of new racks shielded some specimens on existing racks and reduced their corrosion rate. At Halifax, Canada, the effluent gases emitted by the smoke stack atop the Federal Building suggested the area to be unduly aggressive. Indeed, the measured corrosion penetrations were 2.5 to 4 times higher than at another Halifax (York Redoubt) exposure site. At Es-

quimalt, Vancouver Island, prevailing winds kept the sea spray away from the specimens, suggesting that the marine area was relatively rural despite being located a few hundred feet from the ocean. In Cape Kennedy, Fla., rip-rap rock that stabilizes the beach caused the waves to disperse into large airborne water droplets which do not travel far inland, yet severely corrode specimens exposed near the rocks.

In summary, in rural environments zinc and carbon steel specimens performed consistently from year to year. In industrial environments their performance was affected by the variations in corrosiveness of the microclimates. In some marine environments the degree of corrosion penetration depended greatly on initial time of exposure, prevailing wind direction, storm activity, and distance from the shore.

The corrosiveness of any exposure site depends mainly on: (1) the time of wetness of the specimens, determined by the length of time during which the relative humidity exceeds a critical value; (2) airborne sulfur oxide pollution; and (3) airborne chloride pollution. Several other aggressive factors exist but they are less important than the three cited herein.

Unfortunately, the atmospheric variables were not recorded during exposure testing of the weathering steels in most American studies and some European studies. Methods of measuring the time of wetness of steel surfaces have only recently been developed [ASTM 1982]. The lack of climatic data prevents the quantitative correlation of the corrosiveness of various sites. Therefore, one can only infer the corrosion resistance of a steel at a new site from its known behavior at other sites. One can, however, present the available data on the atmospheric corrosion rates of weathering steels at various sites that are broadly classified by macroclimate as being rural, industrial, or marine. This is done in the following sections. When using the data to estimate the corrosion performance of weathering steel highway bridges, one should carefully consider the effects of microenvironment associated with local topography, climate, and service conditions, as illustrated by the data in Chapter Five.

Recent Studies

The more recent studies have increasingly focused on quantitative and predictive methods of assessing corrosion resistance. This includes the classification and calibration of atmospheric corrosion test sites [Knotkova 1982b, Baker 1982, Townsend 1982], the evaluation of the effects of microclimate on corrosion [Haynie 1982], the prediction of long-term atmospheric corrosion from short-term experimental data [Bragard 1982], and the measurement of time-of-wetness of the specimen [Sereda 1982]. Several of these studies attempted to correlate the degree of corrosion with the relevant climatological data. Application of the predictive models to the large data base from previous exposure tests, with at least partial reconstruction of the climate from weather records, may in the future greatly enhance the ability to quantitatively predict the corrosion resistance of weathering steels.

RURAL ENVIRONMENTS

Figure 23 shows data on the average penetration per side following 1 to 8 years exposure in Saylorsburg, Pa., at Bethlehem Steel's rural test site in the Pocono Mountains located about 31

miles (50 km) north of Bethlehem, Pa. [Cosaboom 1979, Townsend 1982]. Three steels were examined: (1) Bethlehem's A242 Type 1, commercially available as Mayari R steel; (2) Bethlehem's A588 Grade B, commercially available as Mayari R-50 steel; (3) a 0.21 percent copper (copper-bearing) steel; and (4) a carbon steel (0.021 percent Cu) similar to that used for purposes of determining the corrosiveness of various atmospheric test sites in an ASTM study [ASTM 1968]. As expected, the weathering steels had a higher corrosion resistance than the copper and carbon steels, with A242 performing better than A588.

Figure 24 shows corrosion penetration data after 1 to 5 years exposure at three rural sites in England: (1) Loudwater, Bucks, a rural area adjacent to Loudwater viaduct M40 with nearby light industry; (2) Silverdale, Lanes, a rural area about 3.1 miles (5 km) inland from Morecombe Bay; and (3) Brixham (McKenzie 1978, Kilcullen 1979). The test specimens, exposed vertically in the open facing the prevailing wind direction, were: (1) A588 Grade A, commercially available as Cor-Ten B Steel; (2) A242 Type 1, commercially available as Cor-Ten A Steel; and (3) a proprietary steel containing 0.22 percent Cu, 0.27 percent Ni, and 0.12 percent Cr plus other alloying elements. The latter was labeled "copper" steel in Figure 24 and is not the equivalent of the copper steels used in American studies. The weathering steels corroded at a rate generally lower than the "copper" steel at the three rural sites. The degree of corrosion penetration appeared to correlate with the atmospheric sulfur content, but not with the other environmental factors. Indeed, Table 20 shows that the atmosphere at Loudwater contained more sulfur oxides than at Silverdale. It was also more corrosive to both "copper" and weathering steels.

Burgmann and Grimme reported the results of 8-year tests of nine steel compositions exposed at various sites in the Federal Republic of Germany [Burgmann 1981]. Of these steels, three satisfied the chemical requirements of A588 Grade A and Grade B steels for copper, chromium, nickel, and silicon, the primary alloying elements required for enhanced atmospheric corrosion resistance. One composition was a carbon steel control with 0.02 percent copper. The data for the Olpe rural site, plotted in Figure 25, show about the same corrosion penetration as was observed for the weathering steels exposed in Saylorsburg, Pa., Figure 23.

Bonnarens and Bragard summarized the results of atmospheric corrosion tests that were performed at 12 sites in Belgium, France, Germany, Holland, Italy, and United Kingdom [Bonnarens 1981]. The test specimens, exposed at an angle of 45 deg facing south, were made of: (1) Fe 37 carbon steel; (2) Fe 52 HSLA steel; and (3) two weathering steels, respectively, with 0.38 percent and 0.54 percent copper, 0.108 percent and 0.119 percent phosphorus, 0.48 percent and 0.49 percent nickel, 0.69 percent and 0.0 percent chromium, and 0.28 percent and 0.03 percent silicon. Figure 26 shows the corrosion penetration band for these steels exposed 4 years at the following three rural sites: (1) Bari, Italy; (2) Eupen, Belgium; and (3) Saint Germain, France. All steels exhibited lower corrosion rates in Bari than in Eupen and Saint Germain. In general, the corrosion penetration data correlated with the level of sulfur oxides in the atmosphere and with the latitude. Bari's latitude is about equal to the exposure angle, a condition that maximizes exposure to the sun. The other two sites have higher latitudes and generally shorter drying times than those at the southern European sites.

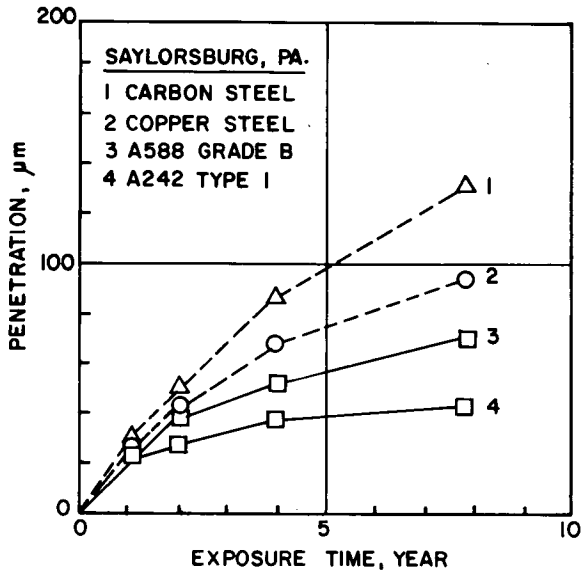


Figure 23. Corrosion penetration of carbon, copper-bearing, and weathering steels exposed to the Saylorburg, Pa., rural environment. [Townsend 1982, Cosaboom 1979]

- 1 "COPPER" STEEL, LOUD WATER
- 2 A588 GRADE A, LOUD WATER
- 3 "COPPER" STEEL, SILVERDALE
- 4 A588 GRADE A, SILVERDALE
- 5 "COPPER" STEEL, BRIXHAM
- 6 A588 GRADE A, BRIXHAM
- 7 A242 TYPE I, BRIXHAM

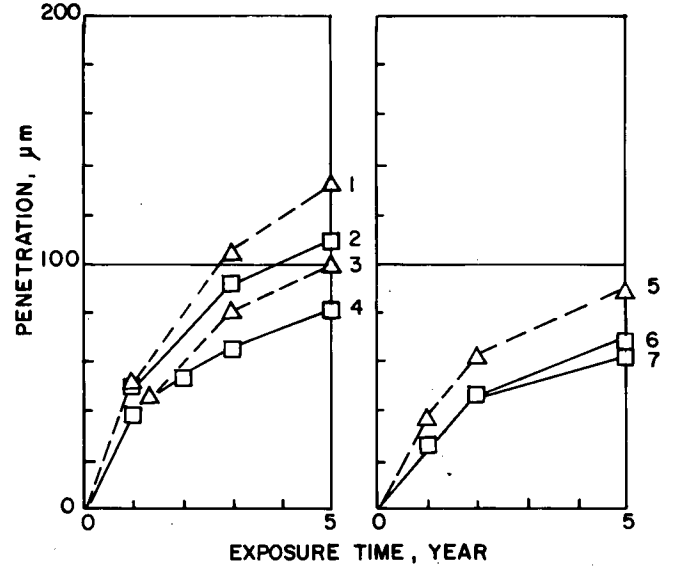


Figure 24. Corrosion penetration of copper-bearing and weathering steels exposed to British environments. [McKenzie 1978, Kilcullen 1979]

Figure 25. Corrosion penetration of carbon and weathering steels exposed to the Olpe, Federal Republic of Germany, rural environment. [Burgmann 1981]

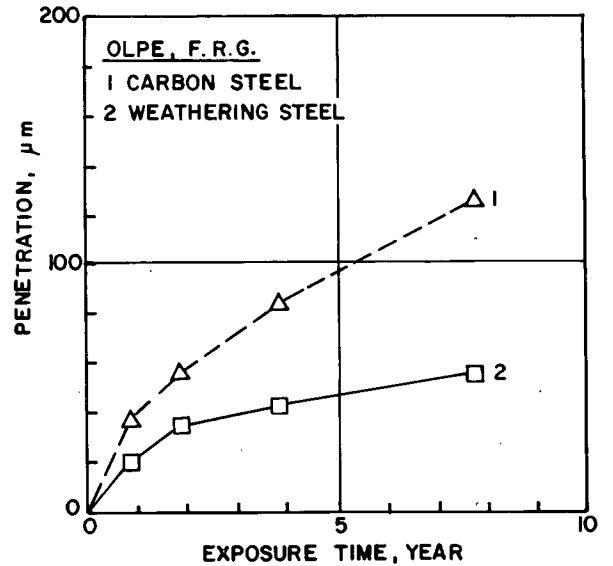


Table 20. Environmental conditions during first year of testing and relative rating of A588 Grade A steels at British sites. [McKenzie 1978]

Site	Average Atmospheric Sulfur Compounds (mg SO ₃ /dm ² /day)	Average Atmospheric Chlorides (mg Cl ⁻ /dm ² /day)	Yearly Rainfall (mm)	% of Year that Relative Humidity Was		Average Temperature (°C)	Relative Rating
				>70% (%)	>80% (%)		
Silverdale (rural)	0.30	0.03	1050	85.2	68.0	8.7	1.00
Portishead (industrial)	0.41	0.04	834	85.0	62.4	9.6	1.20
Eastney (marine)	0.49	0.15	481	76.4	53.2	10.5	1.20
Loudwater (rural)	0.48	0.03	575	82.4	64.8	8.8	1.34
Middlesbrough (industrial)	1.10	0.05	653	85.1	61.5	9.0	1.76
Tinsley (industrial)	2.05	0.07	687	74.6	47.7	10.1	2.68

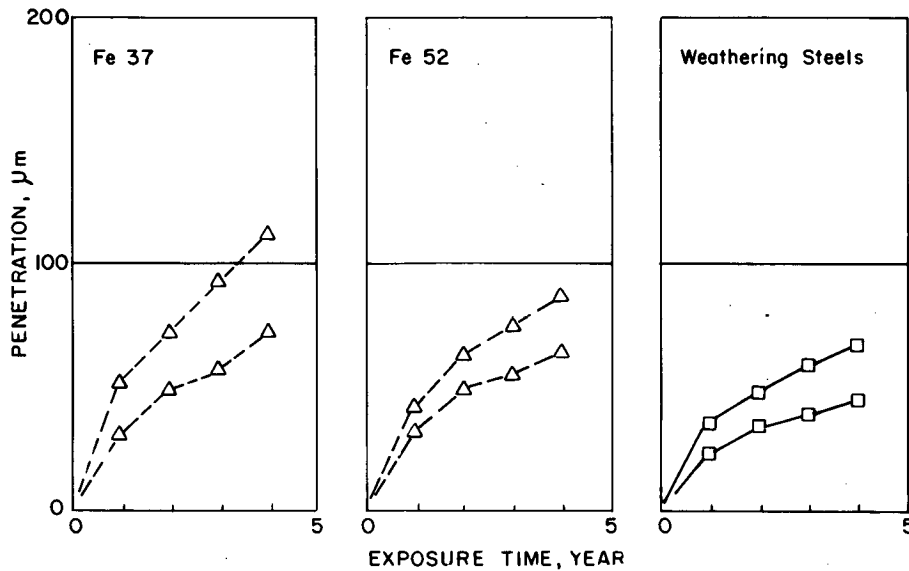


Figure 26. Corrosion penetration bands for Fe 37 carbon, Fe 52 HSLA, and weathering steels exposed to rural environments in Bari, Italy; Eupen, Belgium; and Saint Germain, France. [Bonnarens 1981]

Figure 27 summarizes the corrosion penetration data for weathering steels exposed to several rural environments in the United States (curves 5 and 7), Federal Republic of Germany (curve 6), and England (curves 1 to 4). These curves were replotted from Figures 23 to 25. Type of steel and exposure site are identified in Table 21.

The corrosion penetration data for the three British sites had a factor of two scatter and fell higher than the Saylorsburg, Pa., data. The gradual reduction in corrosion rate indicates that the same corrosion mechanism that forms a protective rust film is at work in both countries, albeit at different rates. The higher corrosiveness of the British sites may be due to the longer time of wetness. Indeed, McKenzie reported that the length of time during which the relative humidity exceeded 70 percent (critical humidity for clean air) was 82.4 percent of the exposure time at Loudwater and 85.2 percent in Silverdale. In other words, during more than 80 percent of the time, the air was sufficiently humid to ensure that a condensed film of moisture on the steel surface would persist [Evans 1972a]. Given that the relative humidity was comparable at both British sites, the atmosphere with the higher sulfur oxide content was the more corrosive. The severity of attack did not correlate with yearly rainfall or temperature.

The seven weathering steel curves in Figure 27 were compared with the scatterband of 2 mils to 4 mils (50 µm to 100 µm) corrosion penetration of Mayari-R steels with a broad range of chemical compositions that were exposed for 18 years in different environments (Bethlehem undated). Extrapolation of the rural curves to 18 years suggests that the Saylorsburg and Olpe curves for weathering steel would remain inside the 18-year scatterband, but those for the British sites would rise above the band. The corrosion penetration band for Bari, Eupen, and Saint Germain (Fig. 26) was not plotted in Figure 27. But, extrapolation of the data to 18 years, with the best-fit equations reported in Bonnarens [1981], indicates that the corrosion penetration at Bari would fall inside Bethlehem's 18-year scatterband. Those for Eupen and Saint Germain would exceed the 18-year band.

The corrosion resistance of the weathering steels, in comparison with the the copper and carbon steels, is discussed under "Corrosion Resistance," later in this chapter.

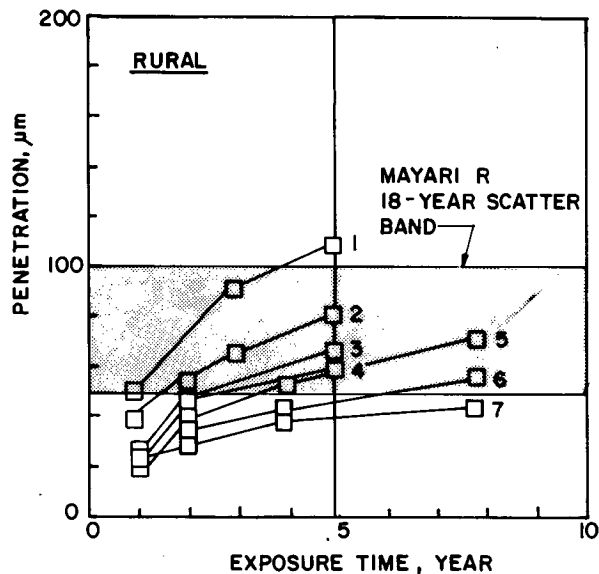


Figure 27. Corrosion penetration of weathering steels exposed in rural environments. [Townsend 1982, Cosaboom 1979, Kilcullen 1979, McKenzie 1978, Burgmann 1981]

Table 21. Description of corrosion penetration curves for weathering steel exposed in rural environments (see Fig. 27).

Curve No.	Type of Steel	Exposure Site
1	A588 Grade A	Loudwater, U.K.
2	A588 Grade A	Silverdale, U.K.
3	A588 Grade A	Brixham, U.K.
4	A242 Type 1	Brixham, U.K.
5	A588 Grade B	Saylorsburg, Pa.
6	Weathering Steel	Olpe, F.R.G.
7	A242 Type 1	Saylorsburg, Pa.

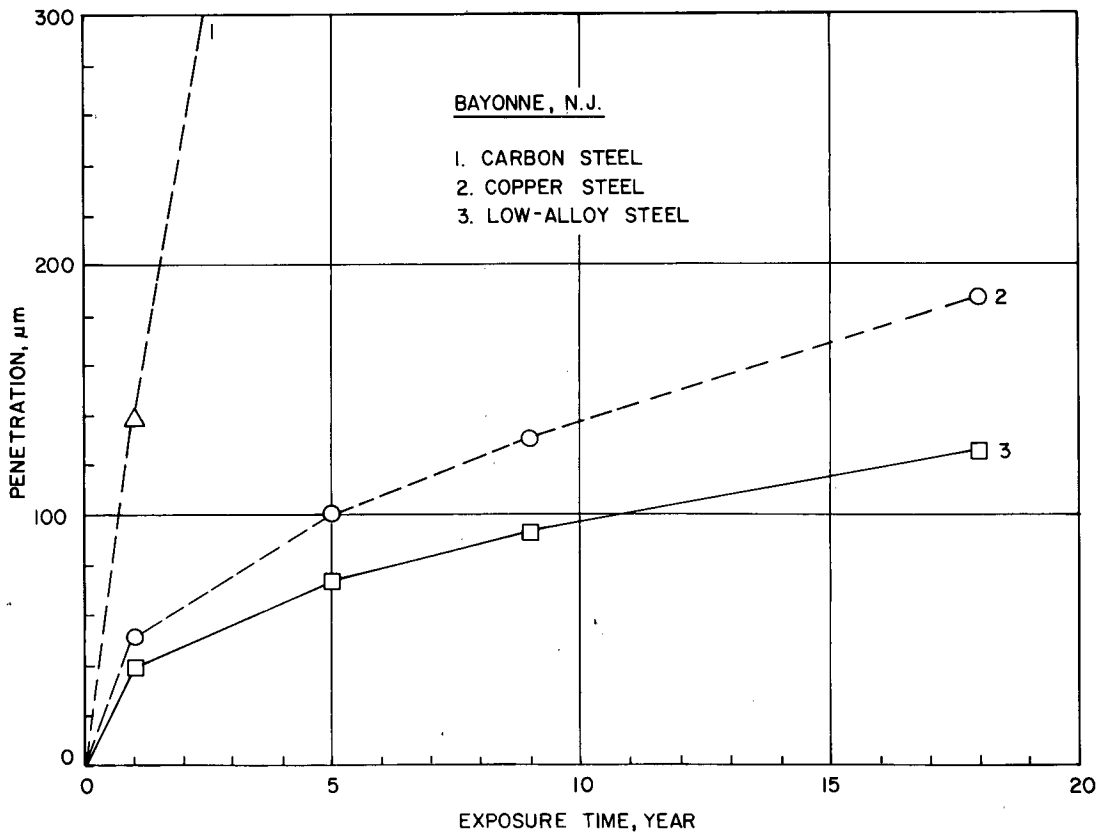


Figure 28. Average corrosion penetration of several carbon, copper, and low-alloy steels exposed to the Bayonne, N.J., industrial environment. [Copson 1960]

INDUSTRIAL ENVIRONMENTS

Copson reported the results of atmospheric exposure tests of 76 steels exposed in the industrial environments of Bayonne, N.J., and in the marine environments of Kure Beach, N.C., and Block Island, R.I. [Copson 1960]. The steels were arranged according to composition in 13 groups. Some were nearly free of alloying elements, in others the alloy content ranged upwards to 5 percent. The principal alloying elements were copper, phosphorus, nickel, and chromium; though some of the steels also contained molybdenum and higher than the usual amounts of manganese and silicon. Presented herein are the results for the steels that had chemical compositions typical of carbon steel (No. 1, 10, 44, and 45), 0.22 percent to 0.24 percent copper steels (No. 41 and 43), and the steels that contained copper, phosphorus, nickel, and chromium levels comparable to those in the weathering steels (No. 3, 13, 42, 48, 53, 66, 119 and 121). Figure 28 shows the average penetrations for the three aforementioned classes of steels that were exposed for 18 years (1941–1959) at the industrial site, a laboratory yard in Bayonne, N.J. Corrosion was rapid during the initial stage of exposure. Thereafter, the corrosion rate of the low-alloy and copper steels decreased with time, but that of the carbon steels continued unabated.

Figure 29 shows Horton's data for two A242 Type 1 steels, a 0.22 percent copper steel and a carbon steel (0.02 percent Cu) [Horton 1965]. They were exposed in Rankin, a severe industrial atmosphere in the Pittsburgh, Pa., area. The weathering steels again developed a protective rust film, whereas the carbon and copper steels continued to corrode at appreciable rates.

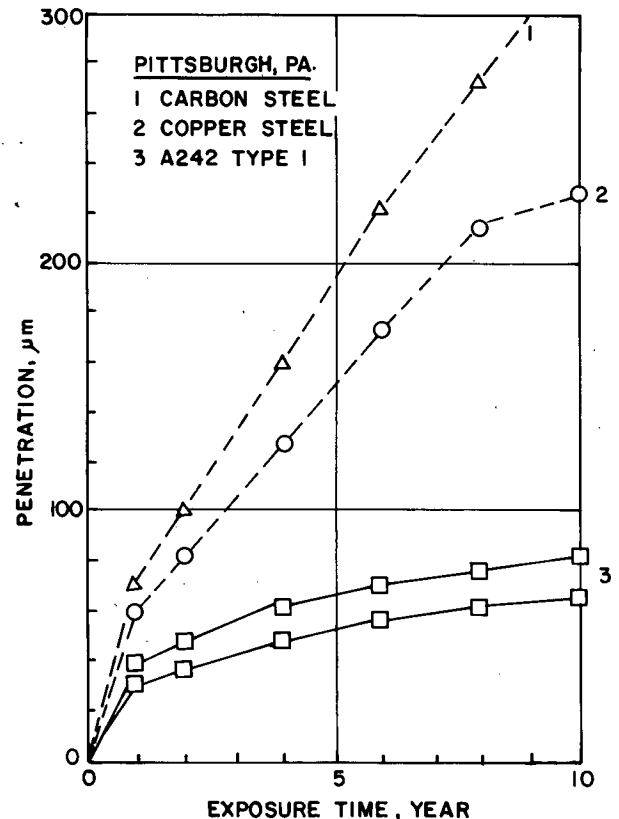


Figure 29. Corrosion penetration of carbon, copper, and weathering steels exposed to the Pittsburgh, Pa., industrial environment. [Horton 1965]

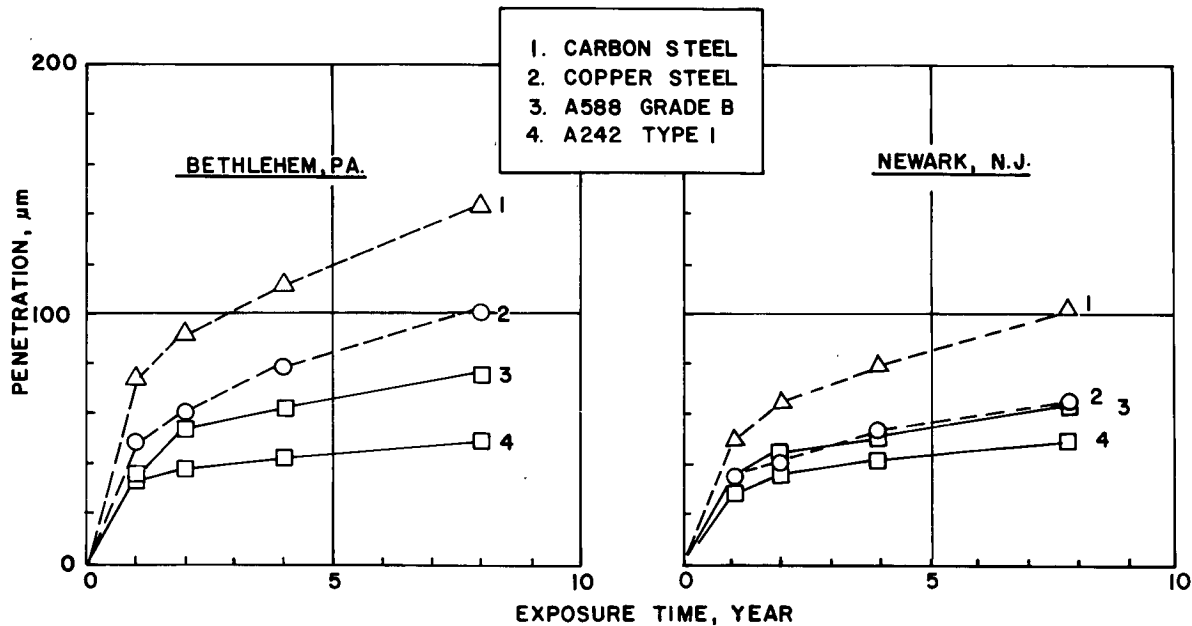


Figure 30. Corrosion penetration of carbon, copper, and weathering steels exposed to the Bethlehem, Pa., and Newark, N.J., industrial environments. [Cosaboom 1979, Townsend 1982]

A third set of data was obtained from exposures at the following industrial sites: (1) Bethlehem, Pa., 1.9 miles (3 km) from Bethlehem's integrated steel making facility; and (2) Newark, N.J., atop a two-story building occupied by the New Jersey Department of Transportation, near heavily traveled expressways [Cosaboom 1979, Townsend, 1982]. The three steels were the same as those exposed in Saylorsburg, Pa. (Fig. 23). The data from these two industrial sites are shown in Figure 30. The weathering steels showed only minor variations in corrosion penetration at the two sites. By contrast, the copper steel and particularly the carbon steel showed marked variations. The carbon steel specimen exposed in Newark, N.J., exhibited a corrosion rate much lower than in the ASTM tests (Table 19). The low corrosion rates of the carbon and copper steels suggest a possible error of mistaken steel identification. The carbon steel behavior was also atypical of those exposed in Bayonne, N.J., and Pittsburgh, Pa. (Figs. 28 and 29), although their chemical compositions were quite similar. This can be explained, in part, by the fact that Bayonne was a very severe environment. Likewise, the Rankin environment near Pittsburgh was a locally severe environment by comparison with that of Newark.

The three sets of industrial exposure data raise questions about the adequacy of the ASTM specification requirements which state that weathering steels have enhanced atmospheric corrosion resistance of approximately two times that of carbon structural steels with copper and four times that of carbon structural steel without copper (0.02 percent max. Cu). If the corrosion resistance is based on the ratio of average penetration at the end of the exposure time, the weathering steels satisfied the requirements in Pittsburgh, but not in Bethlehem and Newark. In Newark, where the corrosion penetration was lowest, the A588 Grade B steel had ironically less than two times the penetration of carbon steel. The corrosion resistance of weath-

ering steel, relative to that of carbon steel, is discussed under "Corrosion Resistance."

McKenzie exposed the same copper and A588 Grade A steels, cited in conjunction with Figure 24 for rural environments, also to the following three urban/industrial environments in England: (1) Tinsley, Yorks, a highly industrial area of Sheffield adjacent to Tinsley viaduct M1; (2) Middlesbrough, Cleveland, an industrial area near the Billingham ICI works; and (3) Portishead, Somerset, an urban area with some light industry, about 1.9 miles (3 km) from the coast [McKenzie 1978]. The specimens were mounted vertically and boldly exposed to the prevailing wind direction. The corrosion penetration data, plotted in Figure 31(a), show the weathering steel corroding at a lower rate than the copper steel at the three sites. However, the 5-year corrosion penetration of the weathering steel varied greatly from 100 μm at Portishead to 220 μm at Tinsley. McKenzie's statistical evaluation of the relationship between corrosion rate and environmental conditions suggested that the level of atmospheric sulfur compounds was the main influence causing the variations in penetration. Indeed, the atmospheric sulfur level (Table 20) correlated well with the industrial data plotted in Figure 31(a), as well as with the rural data plotted in Figure 24. Atmospheric chlorides, at the levels of 0.03-0.07 $\text{mg Cl}^-/\text{dm}^2/\text{day}$, measured at the sites, did not have a significant effect. Although there were signs of reduction in corrosion rate with time, McKenzie concluded that at none of the sites could the corrosion rate be considered negligible, as the British Steel Corporation had suggested in a brochure entitled "Weathering Steel in Bridge Work."

An additional set of British data comes from a joint study by the British Steel Corporation and the British government's Transport and Road Research Laboratory [Kilcullen 1979]. In this study, specimens of Cor-Ten A Steel (A242 Type 1) and

Cor-Ten B steel (A588 Grade A) were exposed in: (1) Battersea, an industrial site; and (2) Teesside, a severe industrial site. The environmental conditions, angle of exposure, and chemical composition of the two weathering steels and of the control steel were not reported. Presumably, the latter steel was the same as in Figure 31(a). The corrosion penetration data, plotted in Figure 31(b), were similar to those for the Middlesbrough and Portishead sites. But, for unexplained reasons, the control steel corroded much faster at Teesside than at any of the four other sites. Kilcullen and McKenzie concluded that the protective properties of the rust film took quite a long time to develop, in some cases many years, and the extent of development of the rust depended very much on the atmospheric conditions of exposure. Consequently, they further concluded, it is difficult to predict the long-term performance of these steels even on the basis of tests carried out over a number of years.

There is some support for this conclusion when one considers a high-strength low-alloy steel composition, USS Mariner Steel, used in the form of sheet piling in sea water. This product contains 0.5 percent Cu, 0.5 percent Ni, and 0.1 percent P and exhibits enhanced atmospheric corrosion resistance in the splash and atmospheric zones. Experience indicates that it requires about 7 years before the corrosion rate deviates sharply from that of carbon steel.

Burgmann exposed the same weathering and carbon steels, cited in conjunction with Figure 25 for the Olpe rural environment, also (1) atop a 3-story building in urban Duisburg-Huckingen; and (2) atop a building on the grounds of a steel rolling plant in industrial Muellheim/Ruhr [Burgmann 1981]. The times of exposure were 16 and 8 years, respectively. The data, plotted in Figure 32, show that the corrosion penetration of the carbon steel was comparable to that of the carbon steel exposed in Pittsburgh, Pa. (Fig. 29), whereas the penetration of the weathering steels was 2 to 3 times higher than in Pittsburgh. At the German sites, the corrosion penetration of the weathering steels after 8 years was 1.6 to 2.3 times greater than that of the carbon steel. The investigators concluded that: (1) the corrosion penetration during the first 8 years declines exponentially with time; (2) in severe industrial environments, the penetration increases as a linear function of time after 4 years, meaning that it does not level off; (3) changes in atmospheric pollution can significantly change the microenvironment and the corrosion rate; (4) the classification of the severity of environments as rural, urban/industrial, and marine is not satisfactory; and (5) careful attention must be given to industrial environments in coastal cities where the simultaneous effect of sulfates and chlorides may accelerate corrosion. It is a fact, however, that rooftop exposures introduce variables not present at ground levels, such as exposure to building vents, etc.

The band No. 4, shown in Figure 32, envelops the corrosion penetration data from an ongoing German study [Hein 1981]. Control specimens of two weathering steels, Resista HRL and A588 Grade A, were exposed at six sites at some distance from freeways. The specimens were mounted at 45 deg and 90 deg from the horizontal, facing south. Hein's data were showing comparable scatter and somewhat smaller penetration after 3 years exposure than Burgmann's data. As all steels had similar alloy contents, the difference in corrosion penetration from site to site is more likely caused by variations in prevailing atmospheric conditions rather than chemical composition. Table 22 rates the two weathering steels at the six sites relative to the

- | | |
|----------------------------------|------------------------------|
| 1. "COPPER" STEEL, TINSLEY | 7. "COPPER" STEEL, TEESSIDE |
| 2. A588 GRADE A, TINSLEY | 8. "COPPER" STEEL, BATTERSEA |
| 3. "COPPER" STEEL, MIDDLESBROUGH | 9. A588 GRADE A, TEESSIDE |
| 4. A588 GRADE A MIDDLESBROUGH | 10. A588 GRADE A, BATTERSEA |
| 5. "COPPER" STEEL, PORTISHEAD | 11. A242 TYPE I, TEESSIDE |
| 6. A588 GRADE A, PORTISHEAD | 12. A242 TYPE I, BATTERSEA |

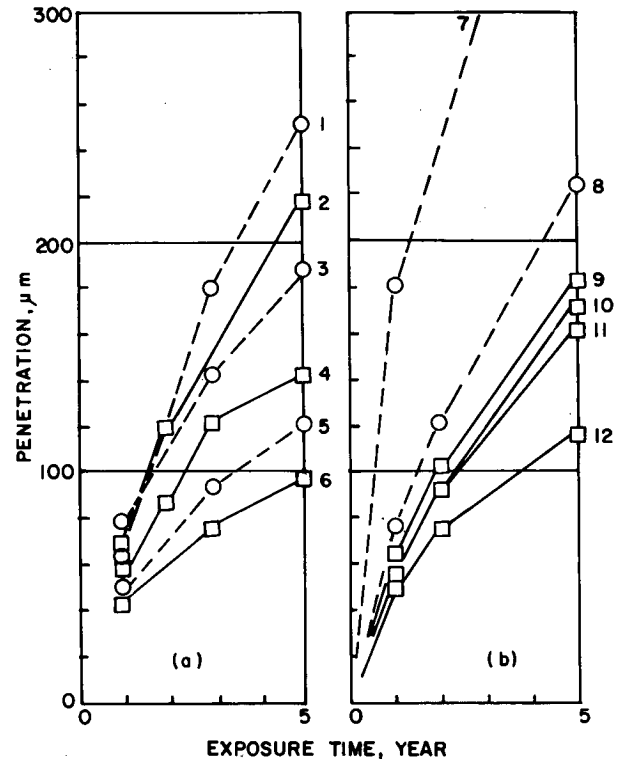


Figure 31. Corrosion penetration of copper and weathering steels exposed to industrial atmospheres in U.K. [McKenzie 1978, Kilcullen 1978]

smallest 3-year penetration, that of the A588 Grade A steel in Merklingen. The following can be concluded: (1) the A588 Grade A steel performed slightly better than the HRL 37 steel; (2) the order of rating of both steels was about the same at the six sites; and (3) the ratings did not correlate with the sulfur content in the rust, nor with the type of climatological data recorded at nearby 1.2, 3.1, and 12.5 miles (2, 5, and 20 km) weather stations. The atmospheric sulfur content, which was found to correlate with penetration at the English sites, was not recorded at the German sites. Surprisingly, both weathering steels rated only average at the Trockau site, where the 53.2 percent mean relative humidity was much lower than at any other site. The corrosion penetration did not correlate with yearly rainfall, sunshine, or temperature. It must be recognized from all of the other exposure data that 3 years is not a sufficiently long period for coating A588 steel to develop a protective oxide coating.

There are two interesting sets of exposure data for urban/industrial environments in California [Reed 1982]. Instead of being made of standard thin-gage sheet, the specimens consisted of butt-welded plates, intermittently welded angles, all-welded angles, and mechanically attached angles. They were fabricated

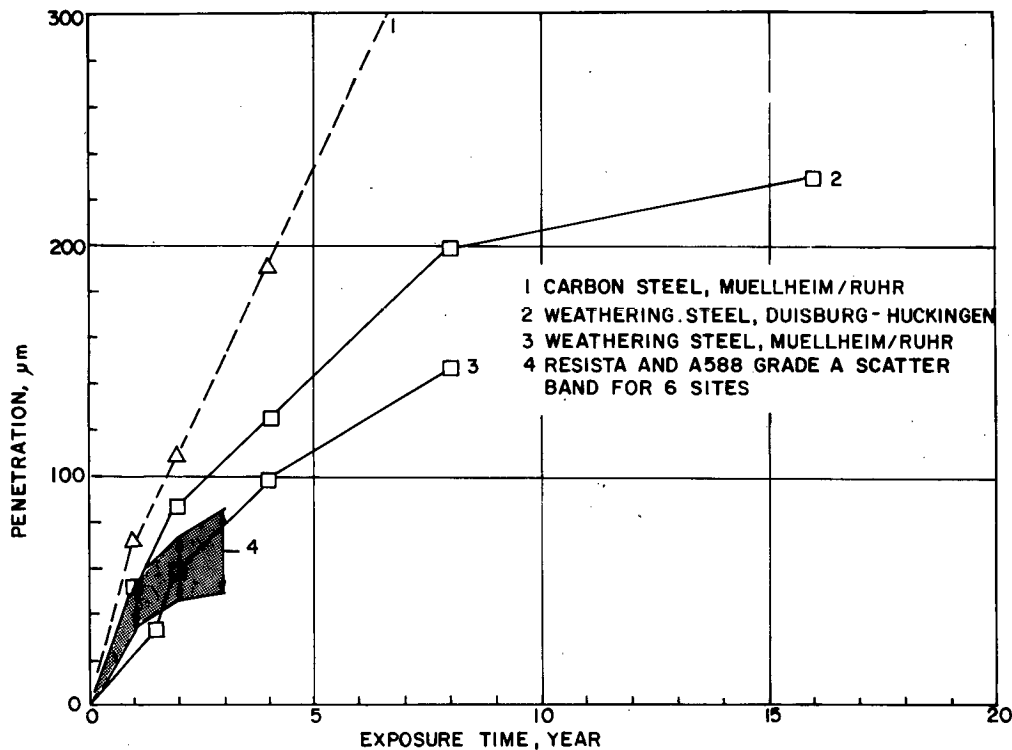


Figure 32. Corrosion penetration of carbon and weathering steels exposed to industrial atmospheres in the Federal Republic of Germany. [Burgmann 1981, Hein 1981]

Table 22. Environmental conditions and relative rating of A588 Grade A and HRL 37 steels during third year of testing at German sites. [Hein 1981]

Site	Distance from Weather Station (km)	Sulfur Content in Rust of A588 Grade A (%)	Yearly Rainfall (mm)	Mean Relative Humidity (%)	Yearly Sunshine (hrs.)	Mean Temperature (°C)	Relative Rating	
							A588 Grade A	HRL 37
Merklingen	20	1.41	1158	81.4	1535	6.6	1.00	1.18
Karlsruhe	5	2.20	696	75.8	1714	10.2	1.16	1.25
Rendsburg	5	1.46	856	83.5	1471	7.2	1.23	1.38
Trockau	2	0.96	751	53.2	1451	6.2	1.24	1.38
Cologne	5	1.56	884	73.2	1352	9.7	1.32	1.54
Duisburg	5	2.03	889	77.4	1289	10.0	1.36	1.43

from one A7 carbon steel and from the following weathering steels: A242 Type 1, and A588 Grades A, G, and H (Mayari R, Cor-Ten A and B, Hi Strength A, and Kaisalloy 50 CR). The specimens were exposed for 13 years: (1) on the grounds of the Caltrans Laboratory, California Department of Transportation, on the southwestern outskirts of Sacramento; and (2) in Commerce, an industrial area 6 miles southeast of the Los Angeles Civic Center. Both sites have low rainfall.

The average sulfur content in the corrosion products of the weathering steels exposed in Commerce was slightly higher than in Sacramento (0.90 percent versus 0.80 percent). The chloride content was tenfold higher (0.55 percent versus 0.05 percent). The average corrosion penetration was, accordingly, higher in Commerce 2.05 mils + 0.11 mils ($52 \mu\text{m} \pm 2.8 \mu\text{m}$) standard

deviation) than in Sacramento ($1.19 \text{ mils} \pm 0.14 \text{ mils}$ ($30 \mu\text{m} \pm 3.6 \mu\text{m}$)). The Commerce and Sacramento data were plotted in Figure 34 and labeled No. 18 and 19, respectively. The low corrosion penetration at both sites is a result of the dry climate which prevents corrosion from occurring despite the presence of sulfur and chloride contaminants.

The same steels, cited in conjunction with Figure 26, were also exposed at the following five European industrial sites: (1) Milan, Italy; (2) Liege, Belgium; (3) Stratford, U.K.; (4) Dueseldorf, Germany; and (5) Delft, Holland [Bonnarens 1981]. The corrosion penetration bands for the three steels, plotted in Figure 33, show that corrosion penetrations of all steels exposed in the industrial environments were consistently higher than those of the same steels exposed in the rural environments,

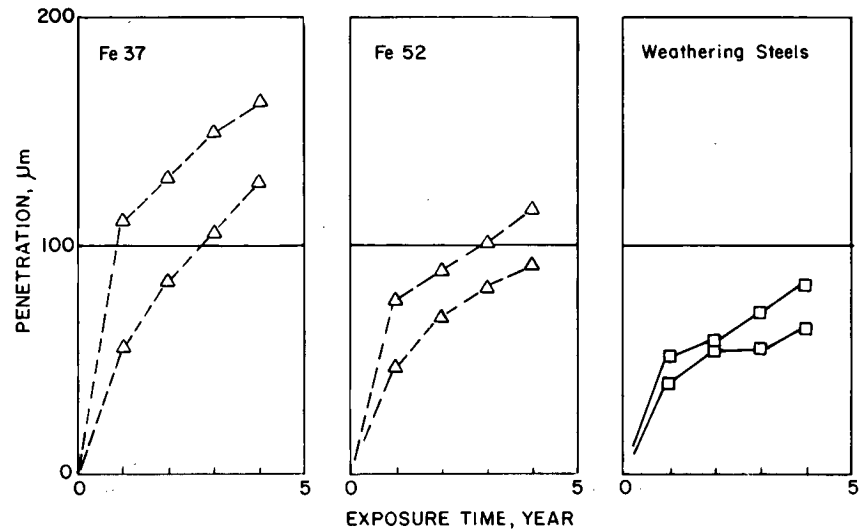


Figure 33. Corrosion penetration bands for Fe 37, carbon Fe 52 HSLA, and weathering steels exposed to industrial environments in Delft, Holland; Düsseldorf, F.R.G.; Liege, Belgium; Milan, Italy; and Stratford, U.K. [Bonnarens 1981]

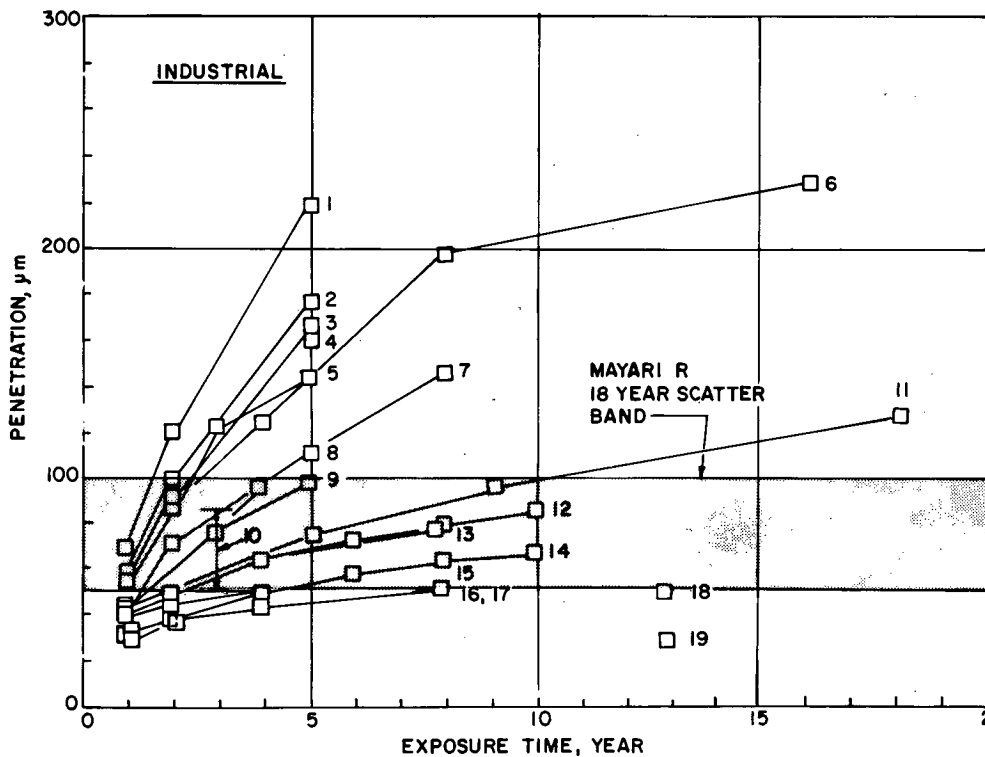


Figure 34. Corrosion penetration of weathering steels exposed to industrial environments. [Copson 1960, Horton 1965, Cosaboom 1979, Townsend, 1982, Kilcullen 1979, McKenzie 1978, Reed 1982]. See Table 23 for identification of curves.

Figure 26. In comparison with the other four industrial sites, the Fe 37 and Fe 52 steels exposed in Milan corroded most, whereas the weathering steels corroded least. The atmosphere in Milan had by far the highest sulfur oxide content. It appears that the combination of high sulfur oxide and low moisture content in the Milan atmosphere was less corrosive to weathering steel than the combinations of lower sulfur oxide and higher moisture content at the other sites.

All weathering steel data from Figures 28 to 32 are compared in Figure 34. The type of steel and exposure site corresponding to each curve are summarized for convenience in Table 23. Also shown is the 18-year scatterband for Mayari-R weathering steels. The steels are either of the A242 and A588 type, or have com-

parable chemical compositions. Curves 1 to 10 are for exposure sites in England and Germany. Curves 11 to 19 are for exposures sites in the United States. The data band for the other five European sites (Fig. 33) were not plotted so as not to overcrowd Figure 34. The band falls between curves 9 and 12. As the steels used were about the same, the wide differences in corrosion penetration must be attributed to environmental variables of which relative humidity, diurnal temperature ranges, and atmospheric contamination with sulfur oxides are the most important. It is not possible to conclusively correlate the corrosion penetration with the environmental conditions because weather and pollution data for most exposure sites are lacking. Only McKenzie, Hein, and Reed have reported such data. Hein also

Table 23. Description of corrosion penetration curves for weathering steel exposed to industrial environments (see Fig. 34).

Curve No.	Type of Steel	Exposure Site
1	A588 Grade A	Tinsley, U.K.
2	A588 Grade A	Teesside, U.K.
3	A588 Grade A	Battersea, U.K.
4	A242 Type 1	Teesside, U.K.
5	A588 Grade A	Middlesbrough, U.K.
6	Weathering Steel	Duisburg-Huckingen, F.R.G.
7	Weathering Steel	Muelheim/Ruhr, F.R.G.
8	A242 Type 1	Battersea, U.K.
9	A588 Grade A	Portishead, U.K.
10	Weathering Steel and A588 Grade A	Six sites in F.R.G.
11	Low-alloy	Bayonne, N.J.
12	A242 Type 1	Pittsburgh, Pa.
13	A588 Grade B	Bethlehem, Pa.
14	A242 Type 1	Pittsburgh, Pa.
15	A588 Grade B	Newark, N.J.
16	A242 Type 1	Bethlehem, Pa.
17	A242 Type 1	Newark, N.J.
18	A242 Type 1 and A588 Grades A, G, and H	Sacramento, Ca.
19	A242 Type 1 and A588 Grades A, G, and H	Commerce, Ca.

reported the concentrations of ammonia, nitrate, and chloride in the runoff water collected from his specimens. However, other than yearly rainfall and mean temperature, two secondary variables, the types of data collected do not match. The following conclusions can be drawn:

1. The corrosion penetration was much higher at the European sites than at the American sites.
2. The primary reason for the higher penetrations at the English sites appeared to be the combination of sulfur pollution and the longer period during which the relative humidity and, therefore, the time of wetness exceeded 70 percent.
3. The degree of corrosion increased with the atmospheric sulfur oxide level.
4. Despite the relatively high chloride and sulfur oxide concentrations found in the rust in Commerce, Calif., the dry climate greatly limited the corrosion penetration.
5. The differences in corrosion behavior at the German and American sites are difficult to explain precisely, because atmospheric pollution data lack for the German sites, and climatological and pollution data lack for the American sites.
6. Since the specimens exposed at 10 of 19 sites were fabricated from A588 Grade A steel, one can assume that composition was not a significant variable affecting the large observed variations in corrosion resistance.
7. The extrapolated curves 12 to 17 would fall within Bethlehem's 18-year scatterband. The curves for the European sites, 1 to 10, exceed the 18-year upperbound curve after 1.5 to 6 years. Curve 11 for the Bayonne, N.J., site exceeds it after 10 years.
8. Extrapolation of the corrosion curves for the five European

sites (Fig. 33) shows that the Milan curve alone would fall within Bethlehem's 18-year scatterband; the other four would exceed it after 6 to 13 years exposure.

9. The A242 steels exhibited a higher corrosion resistance than A588 steels exposed at the same site.

MARINE ENVIRONMENTS

Copson exposed at two marine sites the same steel compositions given in conjunction with Figure 28 [Copson 1960]. One site was in Block Island, R.I., on a bluff 15 miles (24 km) from the mainland overlooking the open ocean; the other in Kure Beach, N.C., on low ground about 800 ft (240 m) from the ocean. The corrosion penetration data, plotted in Figure 35, indicated that the more highly alloyed steels had better corrosion resistance than the copper and carbon steels. The rate of corrosion decreased somewhat over the first four years of exposure. Thereafter, it remained constant or increased. There were no signs that the rates for the copper and more highly alloyed steels were tapering off, even after 17 years of exposure. The Block Island site was more corrosive than the Kure Beach site. Both marine sites were more corrosive to the copper and weathering steels than the Bayonne, N.J., industrial site, but they were less corrosive toward the carbon steel (Figs. 28 and 35). Evidently, the performance of carbon steel is not necessarily indicative of the performance of low-alloy steels at the same site.

Additional corrosion penetration data for specimens exposed at the 800-ft (240-m) lot, Kure Beach, N.C., were gathered by Cosaboom, Townsend, and Schmitt [Cosaboom 1979, Townsend 1982, Schmitt 1969]. The former tested the same steels that had been exposed to the Bethlehem, Pa., and Newark, N.J., industrial environments (Fig. 30). As shown in Figure 36, all four steels exhibited linear or accelerated corrosion rates after the initial years of exposure, a behavior similar to that reported by Copson for the Block Island, R.I., and Kure Beach, N.C., sites. Schmitt identified his steels as being structural carbon, copper, and A242 Type 1 weathering steel, but the chemical compositions could not be established. Each of Schmitt's steels exhibited low corrosion rates, which were decreasing with time, as shown by the data plotted in Figure 37.

Comparing the Kure Beach, N.C., data reported by the three aforementioned investigators, one can see that the corrosion penetration of the copper and weathering steels can vary by a factor of about two. By contrast, the carbon steels show much larger variations than the weathering steels, and appeared to be extremely sensitive to the environment and to yearly variations in climate at the same site.

Figure 38 shows corrosion penetration data for two British sites: one in Eastney, Hants, on a narrow promontory in Langstone Harbour; the other in Rye [McKenzie 1978, Kilcullen 1979]. The steels were the same as those that were exposed in the industrial environments shown in Figure 31. It seems surprising, at first, that the corrosion penetration of the weathering steels at the two marine sites were about equal to the lowest penetrations for the same steels exposed at the industrial sites. Compare for example the Portishead curve 6 in Figure 31 with the Eastney curve 2 in Figure 38. But the environmental conditions, listed in Table 20, suggest that both sites may indeed be equally corrosive. Both sites had comparable atmospheric

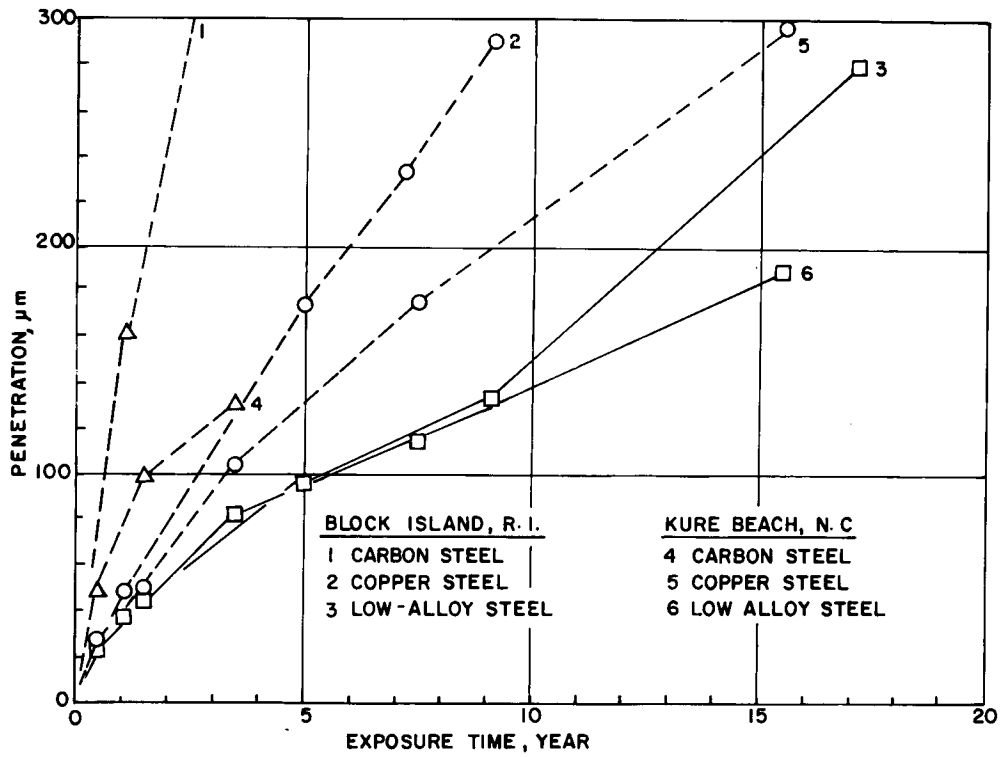


Figure 35. Corrosion penetration of carbon, copper-bearing, and weathering steels exposed to the Block Island, R.I., and Kure Beach, N.C., marine environments. [Copson 1960]

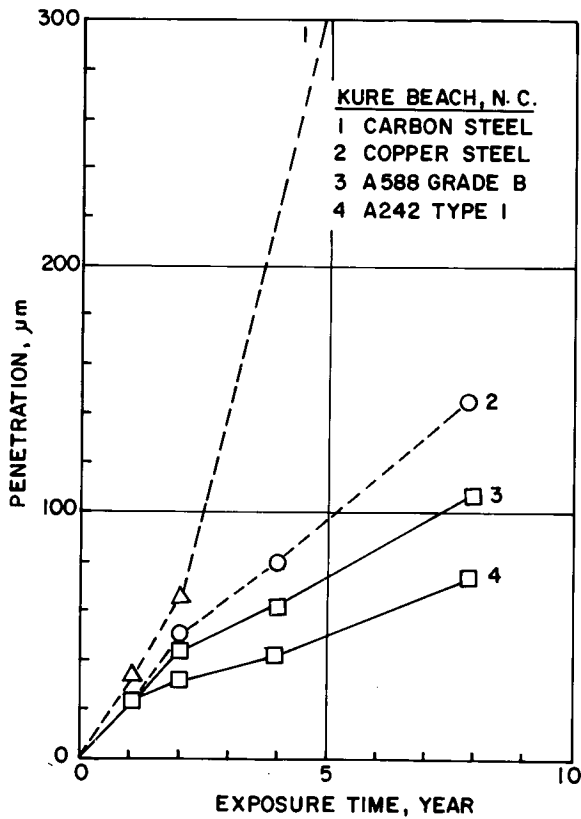


Figure 36. Corrosion penetration of carbon, copper, and weathering steels exposed to the Kure Beach 800-ft (240-m) lot, N.C., marine environment. [Cosboom 1979, Townsend 1982]

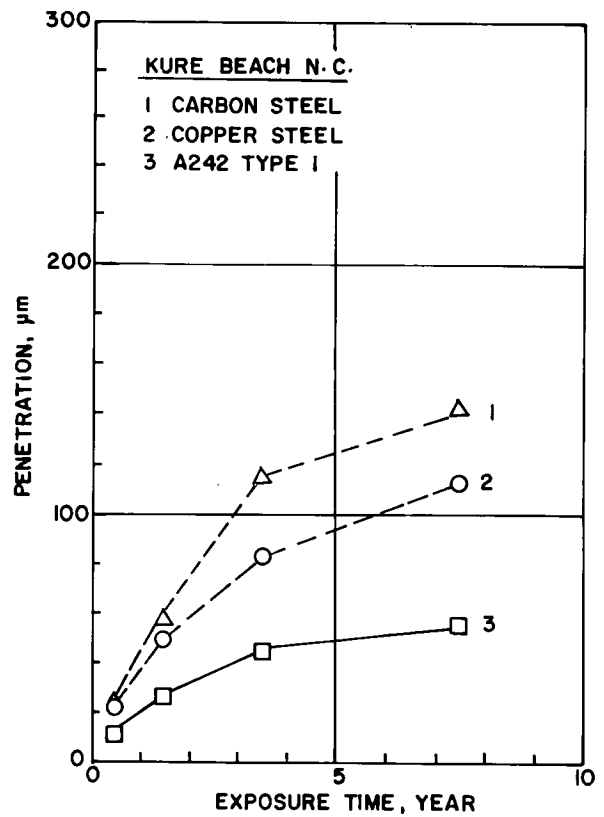


Figure 37. Corrosion penetration of carbon steel, copper-bearing steel, and weathering steel exposed to the Kure Beach, N.C., marine environment. [Schmitt 1969]

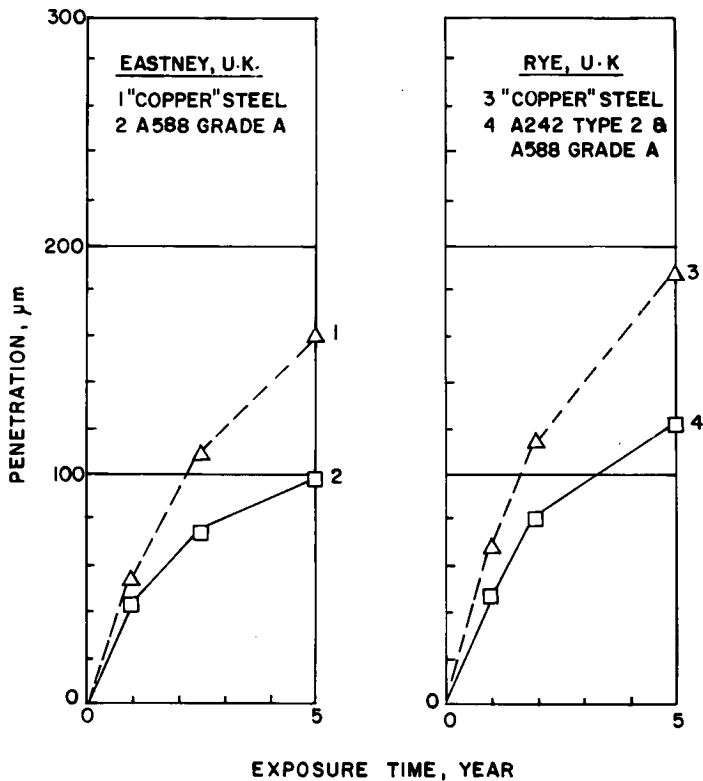


Figure 38. Corrosion penetration of copper-bearing and weathering steels exposed to the Eastney, U.K., and Rye, U.K., marine environments. [McKenzie 1978, Kilcullen 1979]

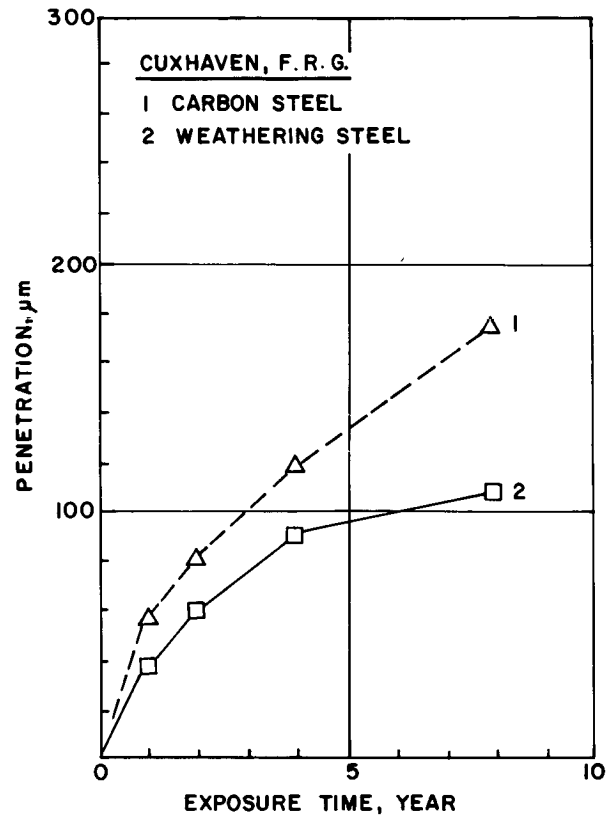


Figure 39. Corrosion penetration of carbon steel and weathering steel exposed to the Cuxhaven, F.R.G., marine environment. [Burgmann 1981]

sulfur levels. Eastney had three times the atmospheric chloride level, but 10 percent lower relative humidity.

The same carbon and weathering steels that were exposed to the Muellheim/Ruhr industrial environment (curves 1 and 3 in Fig. 32) were also exposed at the Cuxhaven, F.R.G., marine site, 500 ft (150 m) from the seashore [Burgmann 1981]. The data were plotted in Figure 39. The aforementioned comments concerning the British data also apply to the German data; i.e., for the same steels, the corrosion penetration in the marine environment was less than that found in the industrial environment. This is evident by comparing curves 1 and 2 in Figure 39 with curves 1 and 3 in Figure 32. Relative to the industrial environment, the corrosion penetration of the carbon steel in the marine environment decreased much more than that of the weathering steel.

The same steels cited in conjunction with Figures 26 and 33 for exposures in rural and urban environments were also exposed at the following four European marine sites: (1) Biarritz, France; (2) Rye, U.K.; (3) Cuxhaven, Germany; and (4) Den Helder, Holland [Bonnarens 1981]. The corrosion penetration bands for the three steels are plotted in Figure 40. The corrosion penetrations of all steels, after 4 years of exposure in the marine environments, were about the same as those of the same steels exposed in the industrial environments (Fig. 33), but the corrosion rates were not leveling off with time. No correlation was found between the corrosion penetrations and the amount of atmospheric chlorides and sulfur oxides.

The susceptibility of steels to corrosion in marine environments decreases with increasing distance from the shoreline, as the salt spray and chloride contents of the air drop. This finding is valid for steels in general. For example, Figure 41 shows the corrosion penetration of carbon and copper steels that were mounted on racks located 80 ft (24 m) and 800 ft (240 m) from the shoreline in Kure Beach, N.C. [Herzog 1960, ASTM 1968]. At 240 m from the shore, the corrosion penetration had dropped by an order of magnitude. However, after 4 years of exposure, there was no indication that the corrosion rate of the copper steel was leveling off. Songa [1975] also reported a rapid decrease in corrosion penetration of specimens exposed 250 m from the Belgian sea coast as compared to those exposed at the shoreline.

ASTM's calibration study of the atmospheric corrosiveness of test sites showed the effect of distance from the ocean and elevation aboveground on the corrosion of carbon steel in Cape Kennedy, Fla. [ASTM 1968]. As summarized in Table 24, the test sites were: (1) 2,700 ft (800 m) from the ocean at ground level; (2) 180 ft (54 m) from the ocean at 60 ft and 30 ft (18 m and 9 m) aboveground, and at ground level; and (3) on the riprap rock used to stabilize the beach. The corrosion penetration was modest after 1 year exposure and about 4.5 times higher after 2 years exposure. The most severe corrosion occurred on the beach, where constant exposure to spray at low and high tides consumed the carbon steel specimens in less than 2 years. At 180 ft (54 m) from the ocean, the penetration had decreased several-fold and was about three times larger at ground level

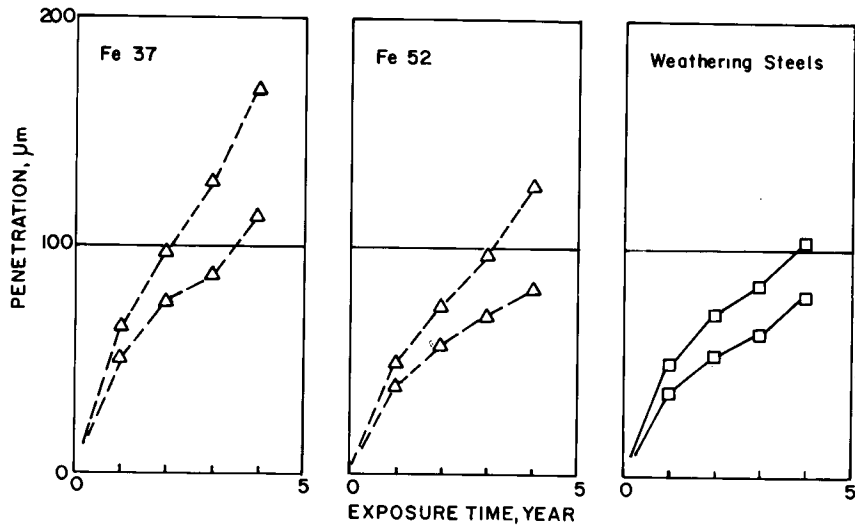


Figure 40. Corrosion penetration bands for Fe 37 carbon, Fe 52 HSLA, and weathering steels exposed to marine environments in Biarritz, France; Cuxhaven, F.R.G.; Den Helder, Holland; and Rye, U.K. [Bonnarens 1981]

Table 24. Corrosion penetration per side of carbon steel exposed in Cape Kennedy, Fla., at various distances and elevations from the ocean. [ASTM G1 1968]

Distance from ocean (m)	Elevation (m)	1-Year Exposure		2-Year Exposure	
		Penetration (μm)	Rating ^a	Penetration (μm)	Rating ^a
800	ground	41	1.0	173	1.0
54	18	61	1.5	263	1.5
54	9	71	1.7	330	1.9
54	ground	191	4.7	884	5.1
beach	ground	1056	25.8	---	---

Note:

a. Based on corrosion penetration at 800-m site.

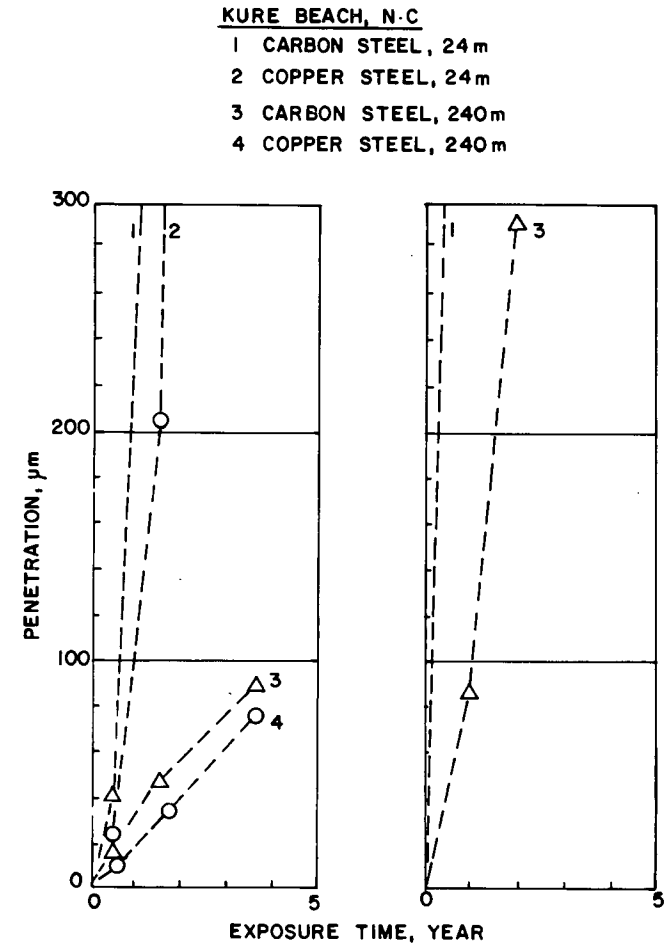


Figure 41. Corrosion penetration of carbon and copper-bearing steels exposed to the marine atmosphere at Kure Beach, N.C. [Herzog 1960, ASTM G-1 1968]

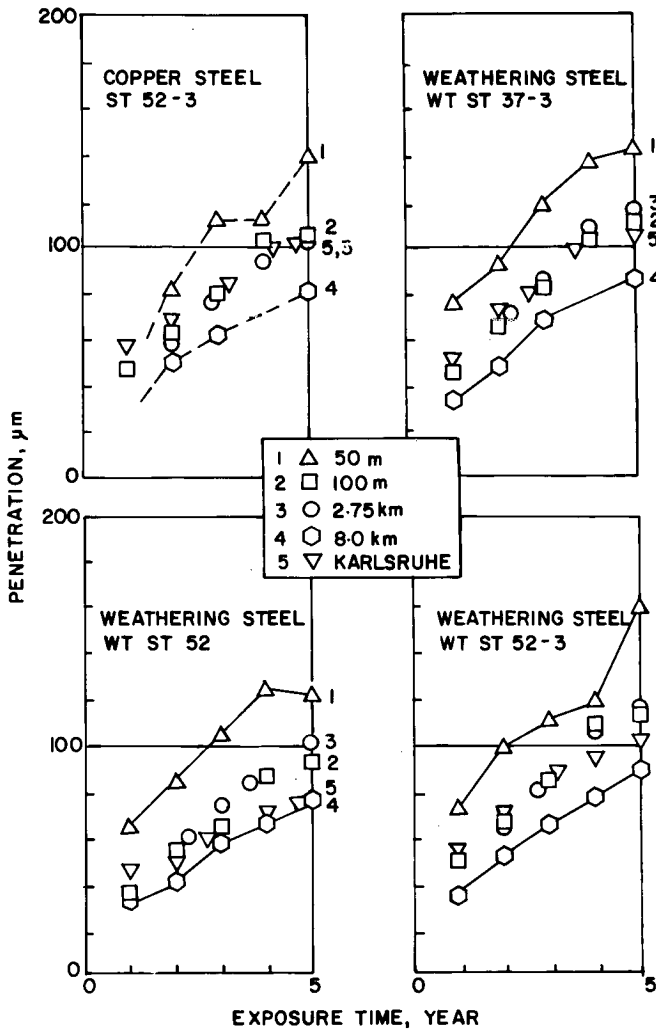


Figure 42. Corrosion penetration of copper-bearing and weathering steels exposed to marine environments at different distances from the shoreline. [Hein 1977]

than at 30-ft and 60-ft (9-m and 18-m) elevation. Presumably, the winds carried the large airborne droplet, caused by waves breaking on the riprap rock only to low elevations. At 0.5 mile (800 m) from the ocean, the corrosion level was comparable to that of a severe industrial site.

Hein investigated the effect of atmospheric chlorides on the corrosion resistance and attempted to verify the requirement of the 1970 German specification for weathering steels that had set a minimum 3,300-ft (1-km) distance from the shoreline for building bare steel structures (Hein 1977). He exposed specimens of copper steel (0.23 percent Cu) and three low-alloy steels containing Cr, Cu, Si, and Ni at 170 ft, 330 ft, 1.7 miles, and 5.0 miles (50 m, 100 m, 2.75 km and 8.0 km) from the shore of the North Sea, Federal Republic of Germany. The first three sites were located on Sylt Island, off the North Sea coast, the fourth site in Niebuell. In addition, control specimens were exposed in the city of Karlsruhe, atop a building subjected to the exhaust gases emitted from the chimney of a heating plant.

The data, plotted in Figures 42 and 43, show a sharp drop in corrosion penetration between the 170-ft and 330-ft (50-m and 100-m) marine sites, followed by a gradual drop to the 8 km urban-industrial site. The three marine sites [≤ 2.75 km (1.7 miles)] were more aggressive than the Karlsruhe industrial site. The corrosion rates do not appear to be leveling off after 5 years exposure. The decreasing corrosion rates at the three marine sites correlated with the decreasing amount of chlorides found in the collected rain water that had run off the specimen surfaces (Fig. 44). The chloride contents for the first year of exposure were 170 ft, 330 ft, 1.7 miles (64.6, 24.3, and 12.9 mg/l at the 50-m, 100-m, and 8.0-km) sites, respectively. Heavy storms pounded the coast during the third year of exposure. This tripled the chloride content. As a result, the 33.1 mg/l third-year chloride content of the runoff water at the 5.0 mile (8.0-km) site was higher than the first-year content at the 330-ft (100-m) site and about one-half of the first-year content at the 170-ft (50-m) site. Evidently, strong winds carried salt spray far inland. Water from the specimens at the Karlsruhe site had only a 6.5-mg/l chloride content, but up to 15 times greater

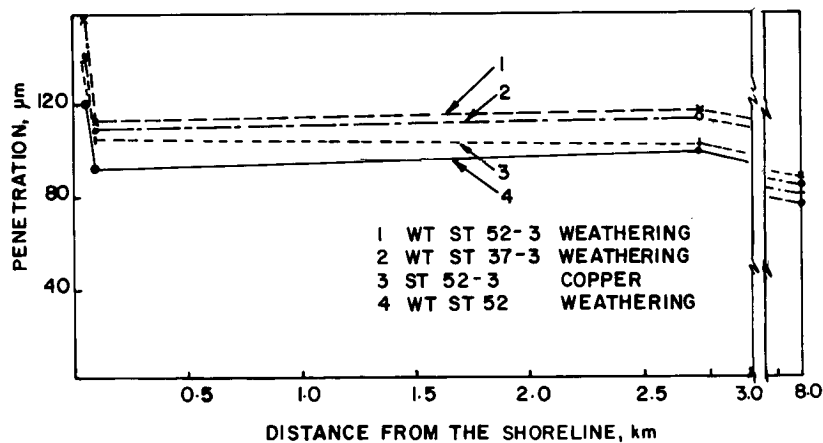


Figure 43. Five-year corrosion penetration of steels as a function of the distance from the shoreline. [Hein 1977]

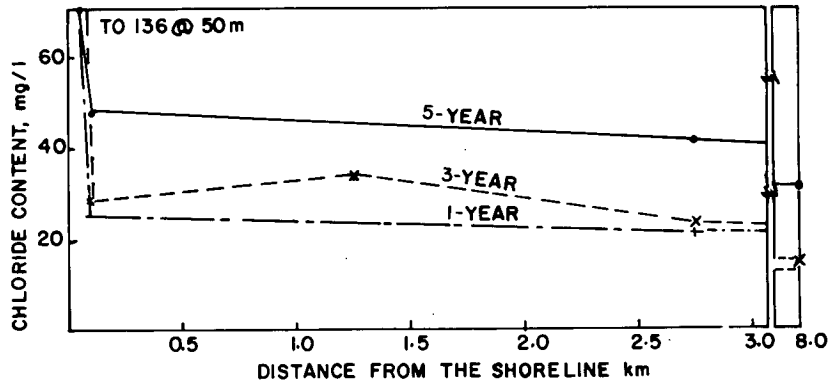


Figure 44. Chloride content of runoff water from specimens as a function of distance from the shoreline. [Hein 1977]

sulfate content. The high-sulfate, low-chloride industrial environment was about as corrosive as the moderate marine environments at 1.7 and 5.0 miles (2.75 km and 8.0 km) from the seashore.

Hein concluded, as did many other investigators before him, that corrosion near the seashore is a function of the chloride content of the atmosphere.

Corrosion decreases with increasing distance from the seashore. The distance from the shore, beyond which weathering steels would corrode at a rate comparable to nonmarine sites, depends both on the weather conditions and the topography of the area.

Finally, Reed exposed in Point Reyes, Calif., the same types

of specimen configurations and steel compositions he had exposed in Sacramento and Commerce [Reed 1982]. The marine test site, also used by ASTM, is located 1,300 ft (390 m) from the ocean on the Point Reyes peninsula, 36 miles (56 km) northwest of San Francisco. It has the highest incidence of fog of any area in the State. The scatterband of data, after 13 years exposure, are labeled 11 in Figure 45. The corrosion penetration in Point Reyes 4.39 mils \pm 0.55 mils (112 μ m mean \pm 14 μ m) standard deviation was about twice as high as in Commerce, although the rust product of the specimens exposed in Commerce retained a higher sulfur content (0.90 percent versus 0.19 percent) and a higher chloride content (0.55 percent versus 0.19 percent) than the rust on the Point Reyes specimens. The reason

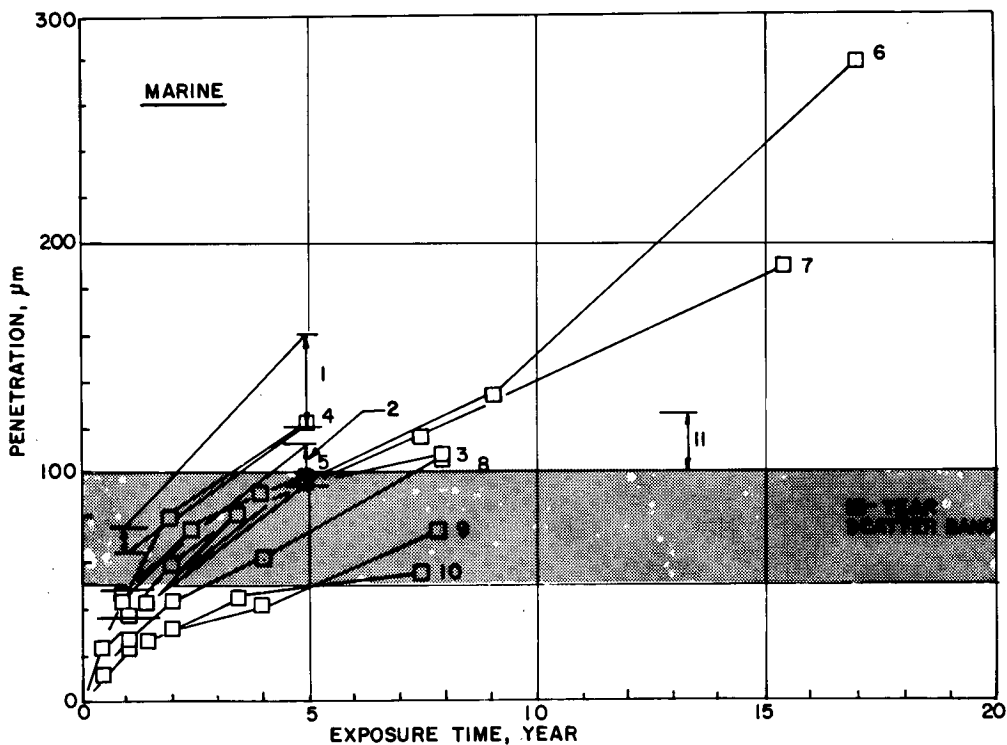


Figure 45. Corrosion penetration of weathering steels exposed in marine environments. [Copson 1960, Hein 1977, McKenzie 1978, Townsend 1982, Cosaboom 1979, Kilcullen 1979, Burgmann 1981, Reed 1982]. See Table 25 for identification of curves.

for this difference was attributed to the longer time of wetness in foggy Point Reyes than in dry Commerce.

All weathering steel data from Figures 35 to 39 and Figure 42 are compared in Figure 45. The data band from Figure 40 falls between curves 4 and 6. The type of steel and the exposure site corresponding to each curve are summarized in Table 25. Also shown is the 18-year scatterband for Mayari-R weathering steels exposed in nonmarine environments. The following general conclusions can be drawn:

1. As was already noted for industrial environments, the data for marine environments are stratified, with the corrosion rates being higher at the European sites (curves 1 to 5) than at the American sites (curves 6 to 11).

2. The corrosion rates at several marine sites did not begin to level off as they do in the industrial environments. They remained at about a constant rate established after 3 to 4 years exposure. In these environments, the protective oxide film was forming at a relatively slow rate.

3. The variability in corrosion penetration cannot be attributed to the chemical compositions of the steels because they were similar for all specimens. The variations were caused by differences in airborne chloride content and time of wetness of the specimens. Rainfall also plays a significant role in marine locations.

4. At the British and German test sites, the corrosion rates for the weathering steels appeared to be comparable for both the marine and industrial environments.

5. All but curve 10 exceed the upperbound of Bethlehem's 18-year scatterband after 2 to 11 years exposure.

On the basis of the reported data, it appears that the persistent presence of salt mist and the lack of rainfall, together with a prevailing high relative humidity for lengthy periods, combine to slow the formation of the protective oxide film on bare weathering steel structures. The marine locations in the United States most susceptible to these conditions can be found mainly in the foggy areas in northern California and in portions of Oregon. Similar foggy and humid conditions exist in certain areas along the Texas gulf coast. They are less severe from New Orleans to the west coast of Florida, and from Cape Canaveral to beyond Boston. In these areas, rainfall is evenly distributed throughout the year in adequate amounts to wash boldly exposed steel surfaces. Furthermore, fog and high relative humidity occur during a few weeks to a couple of months, not year round.

Although the geographic locations which might pose a serious corrosion problem are relatively few in number, no generalized rule should or can be made as to recommending or limiting the application of weathering steel in coastal areas. These must be determined on a case by case basis. Inspection of other steel structures in the vicinity of a proposed bridge site can assist in evaluating how weathering steel may perform. These include existing bridges and their maintenance history, utility structures, thin-gage steel structures such as barns, mail boxes, guardrails, fencing, etc. The performance of galvanized steel structures that have lost some zinc at cut edges also provides clues. These are some of the factors engineers from the steel suppliers use in their investigations. Chloride candle detectors can monitor the site for chlorides.

To explain some of the differences in corrosion penetration of specimens exposed in Europe and the United States, it should

Table 25. Description of corrosion penetration curves for weathering steel exposed to marine environments (see Fig. 45).

Curve No.	Type of Steel	Exposure Site
1	WT St 37 & 52	50-m site, Sylt Island, F.R.G.
2	WT St 37 & 52	100-m site, Sylt Island, F.R.G.
3	Weathering	Cuxhaven, F.R.G.
4	A588 Grade A	Eastney, U.K.
5	A242 Type 1 & A588 Grade A	Rye, U.K.
6	Low-alloy	Block Island, R.I.
7	Low-alloy	Kure Beach, N.C.
8	A588 Grade B	Kure Beach, N.C.
9	A242 Type 1	Kure Beach, N.C.
10	A242 Type 1	Kure Beach, N.C.
11	A242 Type 1 & A588 Grades A, G & H	Point Reyes, Ca.

be appreciated that this country is situated between latitudes 35 deg on the south to 49 deg along the Canadian border. Germany, in the center of Europe, lies between 48 deg and 54 deg latitude, and England even further north between 52 deg and 58 deg latitude. Because of the lower angle of the sun, the time-of-wetness at the German and English exposure sites was longer and the intensity of the drying cycle was less than at the American sites. Thus, the northern European coastal areas are generally more aggressive. In conclusion, it should be noted that the Japanese use weathering steel despite having extensive marine exposures with high relative humidity and much rain. They try to overcome the problem by careful design and selective use of a porous protective coating applied to vulnerable areas (Chapters 9 and 11).

Careful consideration should be given, however, to those locations where winter conditions are of such severity that large amounts of deicing salts are used. In such instances the application of protective coatings to vulnerable areas, based on past experience, should be considered.

CORROSION RESISTANCE

The ASTM specifications A242, A588, and A709 state that the atmospheric corrosion resistance of weathering steels is approximately equal to two times that of carbon structural steel with copper, which is equivalent to four times that of carbon structural steel without copper (Cu 0.02 percent max.). This requirement is vague in three respects. First, it does not specify how the corrosion resistance of the weathering and reference steels should be measured. Secondly, it ties the performance of the weathering steel to that of the carbon steel, whose performance varies with the nature of the environment. Thirdly, it does not specify the chemical composition of the carbon steel other than to limit the maximum copper content to 0.02 percent.

There are three possible ways of quantifying the relative corrosion resistance of various compositions of weathering steels. They are illustrated in Figure 46 and consist of calculating the:

1. Ratio of corrosion penetration of the reference steel to that of the weathering steel, C_r/C_w , after a given period of exposure t ;

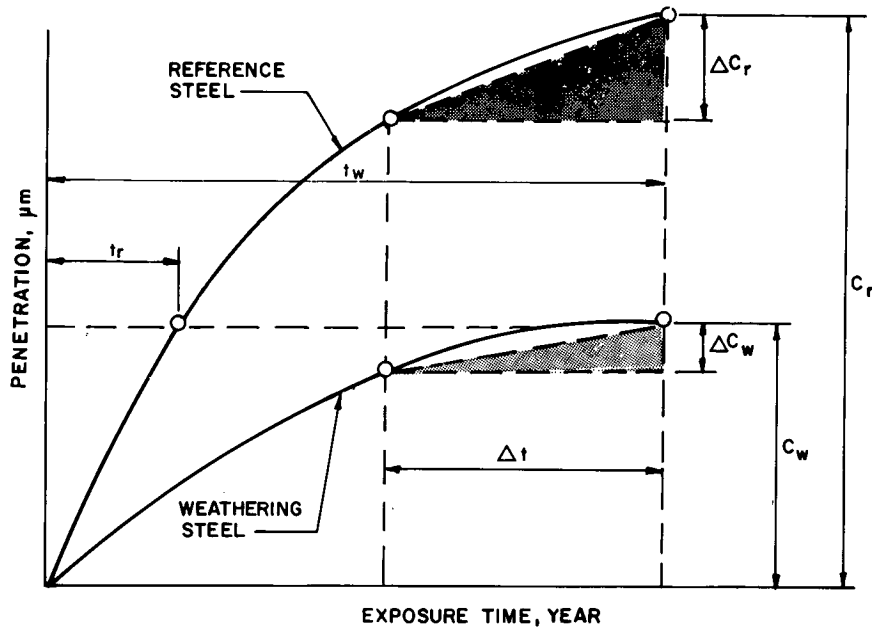


Figure 46. Basis for evaluating corrosion resistance of weathering steel relative to reference steel.

2. Ratio of corrosion rates, $\Delta C_r / \Delta C_w$, over a given increment of exposure time Δt ; and
3. Ratio of periods of exposure, t_w / t_r , at a given corrosion penetration C .

The task of evaluating the corrosion resistance by the three methods is simplified by fitting time-corrosion penetration curves to the data. The curve that best fits the time-penetration data is a power function [Bohnenkamp 1973, Townsend 1982]

$$C = A t^B \quad (23)$$

which, in its logarithmic form, becomes a straight-line relationship.

$$\log C = \log A + B \log t \quad (24)$$

In this study, the following definitions and units are used:

- C = average corrosion penetration determined from weight loss;
 t = exposure time, in years;
 A = regression coefficient numerically equal to the penetration after 1-year exposure; and
 B = regression coefficient numerically equal to the slope of Eq. 24 in a log-log plot.

For any pair of reference and weathering steels, one can calculate from Eq. 23 the ratio of corrosion penetrations at time t :

$$\frac{C_r}{C_w} = \frac{A_r}{A_w} t^{(B_r - B_w)} \quad (25)$$

the ratio of corrosion rates, dC/dt at time t :

$$\frac{dC_r/dt}{dC_w/dt} = \frac{A_r B_r}{A_w B_w} t^{(B_r - B_w)} \quad (26)$$

and the ratio of exposure times at penetration C :

$$\frac{t_w}{t_r} = \frac{A_r^{(1/B_r)}}{A_w^{(1/B_w)}} C^{(1/B_w - 1/B_r)} \quad (27)$$

The regression coefficients A and B for all sets of rural, industrial, and marine data are given in Tables 26 to 28. As indicated by the high values of the correlation coefficient ($r = 1$ means perfect correlation), Eq. 23 represents well the relationship between corrosion penetration and exposure time.

The ratios of penetrations, rates, and times are summarized in Tables 29 to 31. The first two values were calculated, using Eq. 23, at the maximum time of exposure for both the weathering and the reference steels. The ratio of times was calculated at the largest measured penetration of the weathering steel. In this way, extrapolation of the data was avoided (Fig. 46). The values of corrosion resistance of weathering steels which exceeded four times those of carbon steel and two times those of copper steel were underlined. The results suggest the following conclusions:

1. *Rural environments (Table 29)*—One A242 steel exposed in Saylorsburg met the ASTM criteria in five or six comparisons. The other weathering steels failed all criteria. Turning to Figure 27, one observes the paradoxical situation where, despite the good corrosion resistance of several weathering steels, only the one corresponding to curve 7 would be acceptable according to the ASTM criteria.

2. *Industrial environments (Table 27)*—Ten of 17 weathering steels, for which enough data were available to calculate cor-

Table 26. Regression analysis of corrosion penetration data for steels exposed to rural environments.

Curve No.	Type of Steel	Regression Coefficients		Correlation Coefficient	Max. Time of Exposure (years)
		A	B		
Saylorsburg, Pa. (Fig. 23)					
1	Carbon	31.9	.697	1.00	8
2	Copper	28.8	.602	0.99	8
3	A588 Grade B	27.1	.481	0.99	8
4	A242 Type 1	23.3	.316	0.99	8
Loudwater, U.K. (Fig. 24a)					
1	Copper	51.7	0.600	1.00	5
2	A588 Grade A	50.7	0.494	0.99	5
Silverdale, U.K. (Fig. 24a)					
3	Copper	38.7	0.601	1.00	5
4	A588 Grade A	38.4	0.471	1.00	5
Brixham, U.K. (Fig. 24b)					
5	Copper	39.1	0.530	0.98	5
6	A588 Grade A	28.6	0.574	0.98	5
7	A242 Type 1	29.1	0.505	0.96	5
Olpe, F.R.G. (Fig. 25)					
1	Carbon	36.1	0.602	1.00	8
2	Weathering	21.8	0.468	0.98	8

Table 28. Regression analysis of corrosion penetration data for steels exposed to marine environments.

Curve No.	Type of Steel	Regression Coefficients		Correlation Coefficient	Max. Time of Exposure (years)
		A	B		
Block Island, R.I. (Fig. 35)					
1	Carbon	149.8	0.755	1.00	3.3
2	Copper	44.3	0.848	1.00	9.1
3	Low-Alloy	33.1	0.706	0.96	17.1
Kure Beach, N.C. (Fig. 35)					
4	Carbon	71.9	0.522	0.99	3.5
5	Copper	41.7	0.711	1.00	15.5
6	Low-Alloy	35.3	0.612	1.00	15.5
Kure Beach, N.C. (Fig. 36)					
1	Carbon	31.7	1.459	1.00	8
2	Copper	29.2	0.770	1.00	8
3	A588 Grade B	28.0	0.621	1.00	8
4	A242-Type 1	22.1	0.545	0.99	8
Kure Beach, N.C. (Fig. 37)					
1	Carbon	43.5	0.656	0.99	7.5
2	Copper	36.1	0.614	0.99	7.5
3	A242 Type 1	19.7	0.569	0.99	7.5
Eastney, U.K. (Fig. 38)					
1	Copper	53.6	0.669	1.00	5
2	A588 Grade A	42.9	0.511	1.00	5
Rye, U.K. (Fig. 38)					
3	Copper	70.3	0.623	1.00	5
4	A242 Type 1 & A588 Grade B	49.3	0.585	0.99	5
Cuxhaven, F.R.G. (Fig. 39)					
1	Carbon	56.2	0.547	1.00	8
2	Weathering	40.4	0.512	0.98	8
Point Reyes, Cal. (Fig. 45)					
--	Carbon				13
11	A588 Grades A, G, and H A242 Type 1				13

Table 27. Regression analysis of corrosion penetration data for steels exposed to industrial environments.

Curve No.	Type of Steel	Regression Coefficients		Correlation Coefficient	Max. Time of Exposure (years)
		A	B		
Bayonne, N.J. (Fig. 28)					
1	Carbon	139.0	0.869	1.00	3.0
2	Copper	51.2	0.439	1.00	18.1
3	Low Alloy	39.8	0.394	1.00	18.1
Pittsburgh, Pa. (Fig. 29)					
1	Carbon	68.5	0.665	1.00	10
2	Copper	58.8	0.601	1.00	10
3	A242 Test F	40.8	0.314	1.00	10
	A242 Test H	31.3	0.337	1.00	10
Bethlehem, Pa. (Fig. 30a)					
1	Carbon	74.8	0.339	1.00	8
2	Copper	47.0	0.403	1.00	8
3	A588 Grade B	47.2	0.258	0.96	8
4	A242 Type 1	34.1	0.184	1.00	8
Newark, N.J. (Fig. 30b)					
1	Carbon	50.4	0.346	1.00	8
2	Copper	35.0	0.310	1.00	8
3	A588 Grade B	36.1	0.273	0.97	8
4	A242 Type 1	28.7	0.273	0.99	8
Tinsley, U.K. (Fig. 31a)					
1	Copper	65.4	0.872	1.00	5
2	A588 Grade A	71.2	0.709	1.00	5
Middlesbrough, U.K. (Fig. 31a)					
3	Copper	80.0	.534	1.00	5
4	A588 Grade A	59.1	.585	0.99	5
Portishead, U.K. (Fig. 31a)					
5	Copper	49.4	.562	1.00	5
6	A588 Grade A	42.0	.527	1.00	5
Teesside, U.K. (Fig. 31b)					
7	Copper	182.0	0.469	1.00	5
9	A588 Grade A	65.3	0.646	1.00	5
11	A242 Type 1	56.6	0.666	1.00	5
Battersea, U.K. (Fig. 31b)					
8	Copper	76.9	0.669	1.00	5
10	A588 Grade A	57.2	0.693	1.00	5
12	A242 Type 1	49.2	0.550	1.00	5
Mullheim/Ruhr, F.R.G. (Fig. 32)					
1	Carbon	68.4	0.748	1.00	--
3	Weathering	27.2	0.848	0.97	8
Commerce, Cal. (Fig. 34)					
--	Carbon	--	--	--	13
19	A588 Grades A, G, and H A242 Type 1	--	--	--	13
Sacramento, Cal. (Fig. 34)					
--	Carbon	--	--	--	13
18	A588 Grades A, G and H A242 Type 1	--	--	--	13

rosion resistance, passed the ASTM criteria at least once. Only six steels passed it in terms of the ratio of penetrations. But paradoxically, 7 of 8 weathering steels with the best corrosion behavior, curves 12 to 19 in Figure 34, had less than four times the corrosion penetration of their carbon steel counterparts. The weathering steels fared better in terms of the ratio of times, with 10 of 15 passing the ASTM criteria. But, those passing included weathering steels with good (curves 12 to 17) and poor corrosion resistance (curves 2 and 4).

3. *Marine environments (Table 28)*—The distribution of weathering steels passing and failing the ASTM criteria is quite

Table 29. Corrosion resistance of weathering steels in rural environments (see Fig. 27).

Curve No.	Type of Steel	Exposure Site	Corrosion Resistance Based on Ratio of					
			Penetrations		Rates		Times	
			C	Cu	C	Cu	C	Cu
1	A588 Grade A	Loudwater, U.K.	...	1.2	...	1.5	...	1.4
2	A588 Grade A	Silverdale, U.K.	...	1.2	...	1.6	...	1.4
3	A588 Grade A	Brixham, U.K.	...	1.3	...	1.2	...	1.6
4	A242 Type 1	Brixham, U.K.	...	1.4	...	1.5	...	1.9
5	A588 Grade B	Saylorsburg, Pa.	1.8	1.4	2.7	1.7	2.4	1.7
6	Weathering Steel	Olpe, F.R.G.	2.2	...	2.8	...	3.7	...
7	A242 Type 1	Saylorsburg, Pa.	3.0	<u>2.2</u>	<u>6.7</u>	<u>4.3</u>	<u>4.9</u>	<u>3.8</u>

Table 30. Corrosion resistance of weathering steels in industrial environments (see Fig. 34).

Curve No.	Type of Steel	Exposure Site	Corrosion Resistance Based on Ratio of					
			Penetrations		Rates		Times	
			C	Cu	C	Cu	C	Cu
1	A588 Grade A	Tinsley, U.K.	...	1.2	...	1.5	...	1.2
2	A588 Grade A	Teesside, U.K.	...	<u>2.1</u>	...	1.5	...	<u>4.8</u>
3	A588 Grade A	Battersea, U.K.	...	<u>1.3</u>	...	1.2	...	<u>1.5</u>
4	A242 Type 1	Teesside, U.K.	...	<u>2.3</u>	...	1.6	...	<u>6.1</u>
5	A588 Grade A	Middlesbrough, U.K.	...	<u>1.2</u>	...	1.1	...	<u>1.5</u>
6	Weathering Steel	Duisburg, F.R.G.
7	Weathering Steel	Muellheim/Ruhr, F.R.G.	2.0	...	1.8	...	2.6	...
8	A242 Type 1	Battersea, U.K.	...	1.9	...	<u>2.3</u>	...	<u>2.6</u>
9	A588 Grade A	Portishead, U.K.	...	1.2	...	<u>1.3</u>	...	<u>1.5</u>
10	Weathering Steel & A588 Grade A	Six Sites in F.R.G.
11	Low-alloy	Bayonne, N.J.	<u>5.9</u>	1.5	<u>13.0</u>	1.6	<u>20.5</u>	<u>2.4</u>
12	A242 Type 1	Pittsburgh, Pa.	<u>3.8</u>	<u>2.8</u>	<u>8.0</u>	<u>5.3</u>	<u>7.3</u>	<u>5.5</u>
13	A588 Grade B	Bethlehem, Pa.	1.9	<u>1.3</u>	<u>2.5</u>	<u>2.1</u>	<u>6.4</u>	<u>2.1</u>
14	A242 Type 1	Pittsburgh, Pa.	<u>4.7</u>	<u>3.4</u>	<u>9.2</u>	<u>6.2</u>	<u>10.1</u>	<u>7.9</u>
15	A588 Grade B	Newark, N.J.	<u>1.6</u>	<u>1.0</u>	<u>2.1</u>	<u>1.2</u>	<u>4.1</u>	<u>1.2</u>
16	A242 Type 1	Bethlehem, Pa.	3.0	<u>2.2</u>	<u>5.6</u>	<u>4.8</u>	<u>26.3</u>	<u>6.9</u>
17	A242 Type 1	Newark, N.J.	2.0	<u>1.3</u>	<u>2.6</u>	<u>1.5</u>	<u>7.9</u>	<u>2.4</u>
18	A242 Type 1 & A588 Grades A, G & H	Sacramento, Ca.	1.1
19	A242 Type 1 & A588 Grades A, G & H	Commerce, Ca.	1.5

Table 31. Corrosion resistance of weathering steels in marine environments (see Fig. 45).

Curve No.	Type of Steel	Exposure Site	Corrosion Resistance Based on Ratio of					
			Penetrations		Rates		Times	
			C	Cu	C	Cu	C	Cu
1	WT St 37 & 52	Sylt Island, F.R.G.
2	WT ST 37 & 52	50 and 100-m sites
3	Weathering	Cuxhaven, F.R.G.	1.5	...	1.6	...	2.1	...
4	A588 Grade A	Eastney, U.K.	...	1.6	...	<u>2.1</u>	...	<u>2.0</u>
5	A242 Type 1 & A588 Grade A	Rye, U.K.	...	1.5	...	1.6	...	1.9
6	Low-alloy	Block Island, R.I.	<u>4.8</u>	1.8	<u>5.1</u>	<u>2.2</u>	<u>8.9</u>	<u>2.3</u>
7	Low-alloy	Kure Beach, N.C.	<u>1.8</u>	1.5	<u>1.6</u>	<u>1.8</u>	<u>2.4</u>	<u>1.9</u>
8	A588 Grade B	Kure Beach, N.C.	<u>6.5</u>	1.4	<u>15.2</u>	1.8	3.6	1.6
9	A242 Type 1	Kure Beach, N.C.	<u>9.6</u>	<u>2.1</u>	<u>25.7</u>	<u>3.0</u>	<u>4.7</u>	<u>2.6</u>
10	A242 Type 1	Kure Beach, N.C.	<u>2.6</u>	<u>2.0</u>	<u>3.0</u>	<u>2.2</u>	<u>4.4</u>	<u>3.1</u>
11	A242 Type 1 & A588 Grades A, G & H	Point Reyes, Ca.	1.9

similar to that cited for the industrial environments, with the inconsistencies repeating themselves. For example, those passing on the basis of the ratio of times again included weathering steels with good (curve 10 in Fig. 45) and poor corrosion resistance (curve 4).

The clean air standards imposed by the Environmental Protection Agency have gradually reduced air pollution. The general improvement in air quality appears to have benefited the carbon steels more than the weathering steels. For example, comparing the pre-1965 data in Figure 29 with the pre-1982 data in Figure 30, one notes a large drop in the carbon steel curve and a small drop in the A242 Type 1 steel curve. The improved air quality and the manner in which the ASTM standards specify corrosion resistance have combined to create an anomaly. That is, the advantages of the more corrosion resistant steels tend to be understated in the less corrosive environments. Furthermore, the ASTM approach does not consistently discriminate between good and poor corrosion resistance. Townsend and Zoccola calculated the corrosion resistance as a ratio of times at what they considered to be a design allowable average corrosion penetration per side of 0.010 in. (250 μm) [Townsend 1982]. Applying this method to the data for the mildly corrosive Newark, N.J., industrial environment (see Fig. 30) raises indeed the corrosion resistance of the weathering steel to 2.1 and 11.8 times those of the copper and carbon steels. But it also requires extrapolation of the weathering steel data, beyond the maximum 8-year exposure time in Newark, to 1,260 years in order to calculate the ratio of times to 0.010 in. (250 μm) penetration [Townsend 1982]. The data for the highly corrosive Teesside, U.K., industrial environment (see Fig. 31) gives an extreme example on the other side of the spectrum. Applying Townsend's method to this case, gives the weathering steel more than five times the corrosion resistance of the copper steel, although it

reaches the 0.010-in. (250- μm) penetration after only 8-years exposure.

The two curves for the Newark, N.J., and Teesside, U.K., environments, labeled 15 and 2 in Figure 34, are examples of very good and very poor corrosion behavior of weathering steels. Bare weathering steel could be used in Newark, but definitely not in Teesside. But Townsend's method of calculating corrosion resistance, as the ratio of time to 0.010-in. (250- μm) penetration, makes the steel satisfy the ASTM requirements at both sites.

The question does not seem to be whether one should use the ratio of penetrations, rates, or times to calculate corrosion resistance. The observed anomalies come from tying the behavior of weathering steel to that of copper and carbon steels. The decision to build a bridge from bare weathering steel should be based on the anticipated corrosion behavior of the weathering steel at the site under consideration. Data for copper and carbon steels help to assess the corrosiveness of the environment, but the behavior of these reference steels should not be part of the specification requirements for corrosion resistance of weathering steels. Bridges would not be built from bare copper and carbon steels anyway.

From an engineer's viewpoint, design specifications should state the requirements for corrosion resistance in terms of the penetration per side of the weathering steel. Two approaches seem possible. In one, the projected corrosion penetration at the end of the service life could be limited to a value small enough so that a plate thickness increase would not be needed to compensate for section loss. Figure 2 of Bethlehem's Mayari R Booklet implies this approach. It shows a scatterband of penetration data for weathering steels representing a broad range of chemistries and exposures in different environments [Bethlehem undated]. The upper-bound curve of the scatterband reaches 4-mils (100- μm) penetration at 14 years and continues to rise at 0.05 mils (1.27 μm) per year. Weathering steel bridges

designed on the basis of this information would be expected to have corrosion penetration at 50 years of service of $C = 100 \mu\text{m} + (50-14) \text{ years} \times 1.27 \mu\text{m}/\text{year} = 150 \mu\text{m}$ (6 mils) per side. Townsend states that in practical applications of structural steels, where members are generally $\frac{1}{2}$ in. (13 mm) or greater, a 10-mil (250- μm) penetration per side results in a loss of load-bearing cross section of 4 percent or less [Townsend 1982]. This loss is said to be insignificant and to fall within the design allowances. The upper-bound curve on Bethlehem's data and Townsend's design allowance are plotted in Figure 47.

Taking another approach, the allowable limit on corrosion penetration could be raised provided that member thicknesses are increased, above the values required for carrying load, to offset the loss due to corrosion. The West-German and East-German weathering steel specifications follow this approach.

The West-German specifications give an envelope for weathering steels exposed to severe industrial environments [DASt 1979]. They require the following increase in minimum thickness above that needed for carrying load;

SERVICE LIFE (years)	REQUIRED THICKNESS INCREASED PER SIDE, μm (mils)		
	LIGHT CORROSION	MEDIUM CORROSION	SEVERE CORROSION
30	—	800 (32)	1000 (40)
60	800 (32)	1200 (48)	1500 (60)

The following examples of corrosiveness are given: severe—industrial and marine atmospheres; medium—urban atmospheres; light—rural atmospheres. The envelope and the required thickness increase per side are plotted in Figure 47.

The East German specifications set the following upper limits on corrosion penetration per side of their KT weathering steels [DDR 1983].

SERVICE LIFE (years)	ALLOWABLE CORROSION PENETRATION PER SIDE, μm (mils)
1	75 (3)
5	150 (6)
> 5	150 (6) + 20 $\mu\text{m}/\text{year}$ (0.8 mils/year)

These limits are drawn in Figure 47 as a continuous curve. The specifications also require that the designer check the load-carrying capacity of the net section after corrosion losses.

To compare the observed performance of weathering steels against the aforementioned design guidelines, Figures 48, 49, and 50 for rural, industrial, and marine environments, respectively, were constructed. In these log-log plots, the exponential curves fitted through the data shown in Figures 27, 34, and 45 become straight lines. The lines were drawn solidly for the length of exposure time and extrapolated with dashed lines to 50 years, the design life of highway bridges in the United States. Comparing the guidelines for allowable corrosion penetration (Fig. 47) with the three plots leads to the following observations regarding the corrosion performance at the end of the 50-year design life:

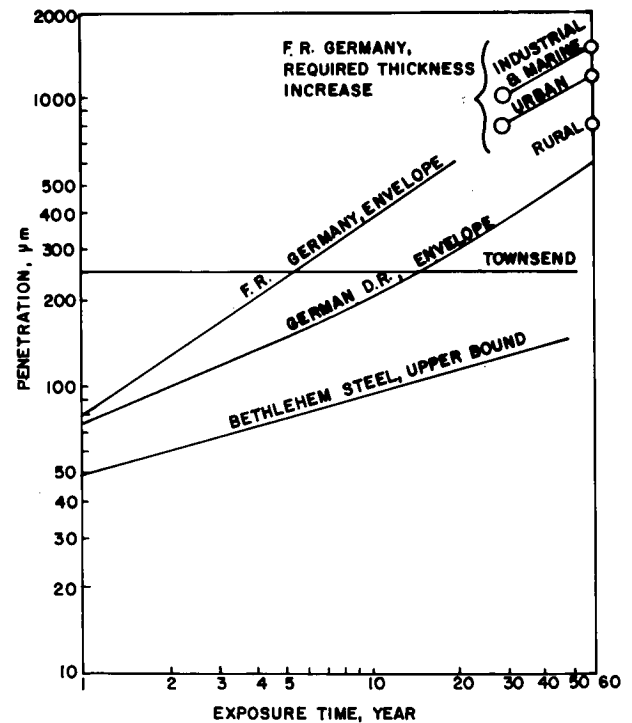


Figure 47. Guidelines for corrosion penetration of weathering steels.

1. Bethlehem Steel's upper-bound envelops the data for American rural and industrial sites, but not for the marine sites.
2. Townsend's limit envelops the data for all American rural and industrial sites. It also envelops the A242 Type 1 curves for the Kure Beach, N.C., marine site. But this grade of weathering steel is no longer used for bridges, because the high phosphorus content reduces impact strength.
3. The West German envelope covers all domestic and foreign data for rural, industrial, and marine environments.
4. The East German envelope covers all domestic and foreign data for rural and marine sites, but not the data for most European industrial sites.

Two additional factors should be considered in choosing guidelines for corrosion design. First, corrosion penetration under service conditions can be significantly higher than under ideal conditions of bold exposure examined in this chapter. Secondly, corrosion pits, which run two to three times greater than the average penetration, reduce the fatigue life of some details [Copson 1960; Hiam 1978]. These factors are examined in Chapters Five and Seven, respectively.

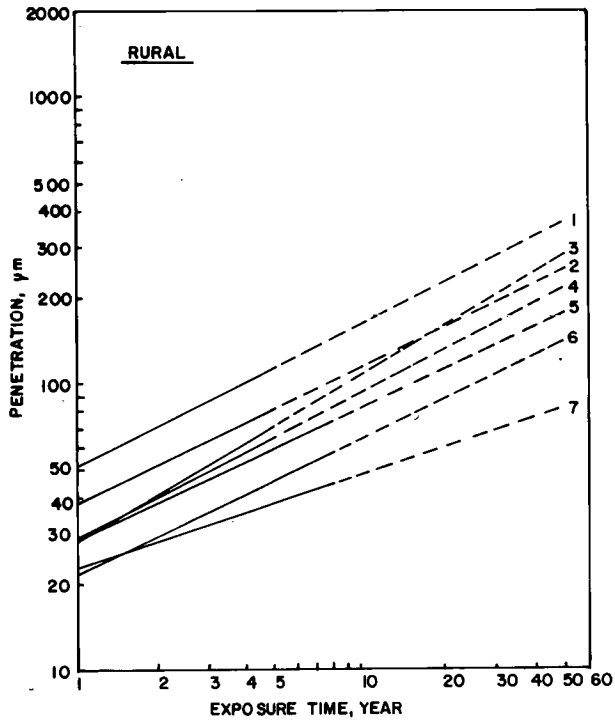


Figure 48. Projected corrosion penetration of weathering steels in rural environments (see Table 21 for identification of curves).

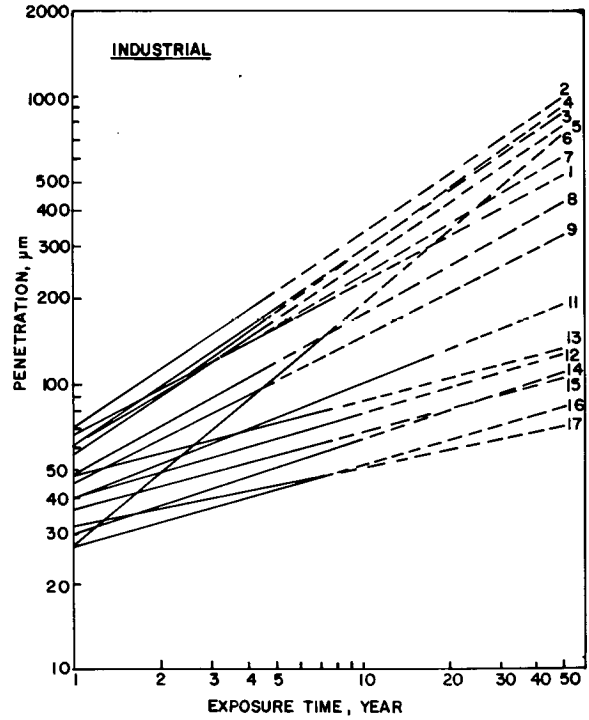


Figure 49. Projected corrosion penetration of weathering steels in industrial environments (see Table 23 for identification of curves).

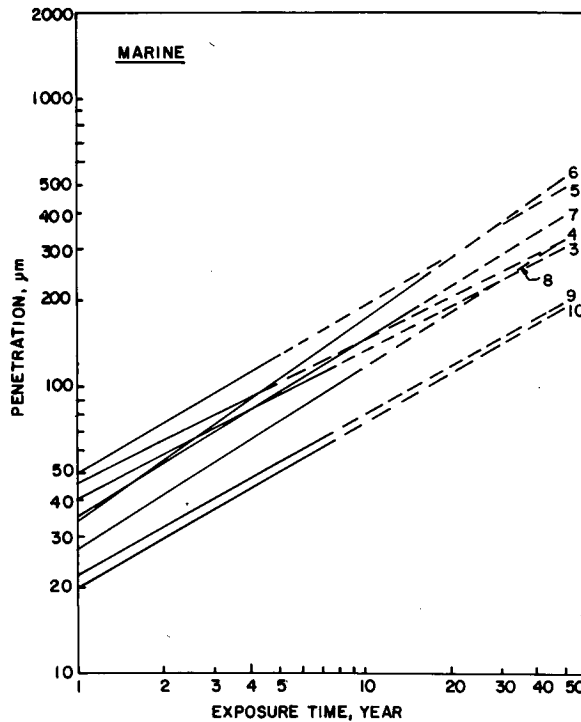


Figure 50. Projected corrosion penetration of weathering steels in marine environments (see Table 25 for identification of curves).

SERVICE CORROSION

Weathering steels develop a protective oxide coating that effectively shields the underlying steel base from the corrosive environment when the following conditions are met:

1. Exposure to the atmosphere wherein intermittent cycles of wetting and drying occur without prolonged periods of dampness.
2. Washing of the exposed surface by rainwater to remove contaminants.
3. Absence of heavy concentrations of corrosive pollutants, especially salt spray from any source.
4. Absence of detail geometries that trap moisture, dirt or debris and, hence, foster a corrosive environment.

The service environments in which weathering steel bridges are exposed do not necessarily fulfill each of the aforementioned conditions. In many cases a bridge will still exhibit adequate corrosion resistance, whereas under severe conditions it may not.

This chapter examines the corrosion behavior of weathering steels under localized or microenvironment conditions more typical of the service conditions encountered at bridge sites. The main effects to be examined are initial climate, sheltering, orientation, angle of exposure, time of wetness, atmospheric pollutants, salt, and debris. In addition, depending on service conditions and type of detail, the steel may be subjected to the following forms of corrosion: poulitce, galvanic coupling, pitting, and crevice corrosion. The sections of this chapter summarize relevant data available for each case.

All values of corrosion penetration are per exposed side (surface) of the specimen.

INITIAL CLIMATIC CONDITION

The corrosion rate of steel specimens is influenced by the climatic condition at the beginning of the exposure period. Schramm and Taylerson reported major differences in the corrosion rate of steel specimens that were exposed at the same site at successive 2-month intervals [Schramm 1934].

Larrabee [1966] reported the corrosion penetration of specimens that were exposed at various times to the Kearny, N.J., industrial environment. The specimens were fabricated from a low-carbon steel with 0.05 percent Cu and a high-strength low-alloy steel containing 0.80 percent Si, 0.15 percent P, 0.43 percent Cu, and 1.1 percent Cr. The original exposure was in the spring or fall of the same year, or it was in the same season of different years. As can be seen from the data plotted in Figure 51, those specimens initially exposed in the spring, regardless of the type of steel, corroded less than those exposed in the fall. The curves also are markedly different for specimens initially

exposed in the same season of different years. These differences are greater for carbon steel than for weathering steel, meaning that the more corrosion resistant steels are less affected by the initial climatic conditions. The increase in heating fuel use with the onset of winter, and the corresponding emission of combustion gases, was one reason why higher corrosion penetrations were observed in fall exposures. A second reason may be attributed to the fact that a semiprotective oxide film forms during the spring and summer months.

Larrabee [1966] also cited data for carbon steel (0.04 percent Cu) exposed to the Kure Beach, N.C., marine environment, beginning in 1944 and 1949. The specimens faced north, east, south, and west. They were sheltered to various degrees. In all

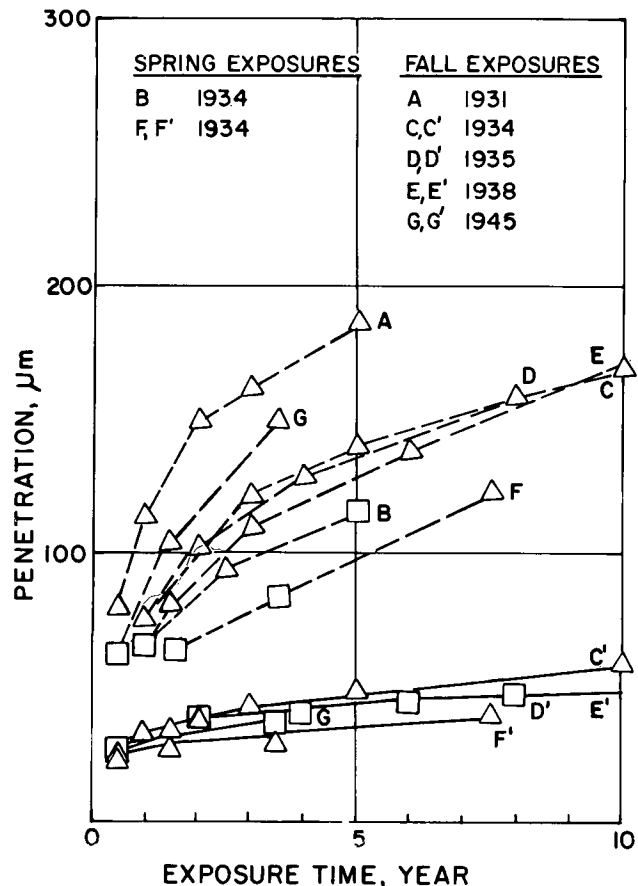


Figure 51. Effect of climatic condition on corrosion penetration of carbon and HSLA steels exposed to the Kearny, N.J., industrial atmosphere. [Larrabee 1966]

cases, the 2-year corrosion penetration of the specimens exposed in 1944–1946 (Fig. 52(a)) was two to three times greater than those exposed in 1949–1951 (Fig. 52(c)). These results suggest the possibility of more frequent bad weather, such as storms and hurricanes, during the first exposure period.

During the periods 1948–1955 and 1960–1962, a Task Group of ASTM Committee G-1 calibrated the corrosiveness of the atmosphere at many domestic and foreign sites [ASTM 1968]. For this purpose, new low-carbon steel (0.09 percent C) specimens were exposed for 1 or 2 years, beginning in each year of the test periods. Figure 53 shows the 2-year corrosion penetration data for the following sites: (1) Kure Beach, N.C., 800 ft (240 m) from the ocean; (2) semirural South Bend, Pa.; (3) semi-industrial Middletown, Ohio; and (4) rural State College, Pa. The figure shows that the atmosphere at the rural, semirural and industrial sites have not changed significantly during the two test periods. The Kure Beach data illustrate the large differences that can be caused by changes in climatic conditions, even at the same test site. The committee speculated that the combination of salt spray and little rain, or widely dispersed periods of rain, considerably increased corrosion.

In summary, the climatic conditions at initial exposure can produce large differences in corrosion penetration. These differences are much larger for carbon steel than for weathering steel. For neither type of steel, however, does the difference disappear with increasing exposure time. Evidently, the initial reactivity of the steel with its environment affects the long-term corrosion behavior. For this reason, initially contaminated weathering steel should be cleaned after installation. This was done in the case of the Luling Bridge, La., many of whose

sections had been exposed to salt spray during transport from Japan.

SHELTER AND ORIENTATION

The effect of shelter on the corrosion resistance of steels can be examined in two types of test. In one type, the corrosion penetration of the skyward surface versus that of the groundward surface is determined by painting one surface of the specimen and exposing the nonpainted surface either skyward or groundward. In the other type of test, nonpainted specimens are either boldly exposed or sheltered from wind, rain, and sunshine. The first test gives the corrosion penetration of only one surface, whereas the second test yields the mean for both surfaces.

To determine the relative corrosion penetration that occurs on each surface, Larrabee [1941] performed the first type of test by exposing carbon, copper, and low-alloy steel specimens painted on one side in semirural South Bend, Pa., and industrial Kearny, New Jersey. The chemical composition of the steels were not given. The angle of exposure was 30 deg facing south. The data, plotted in Figure 54, reveal a much smaller corrosion penetration of the skyward surfaces than of the groundward surfaces. The slopes of the respective curves indicate that the rust on the skyward surface became increasingly more protective with time of exposure. After 3.5 years, the ratio of corrosion penetration of the skyward and groundward surfaces was about 38 percent to 62 percent for the three steels at both South Bend, Pa., and Kearny, New Jersey.

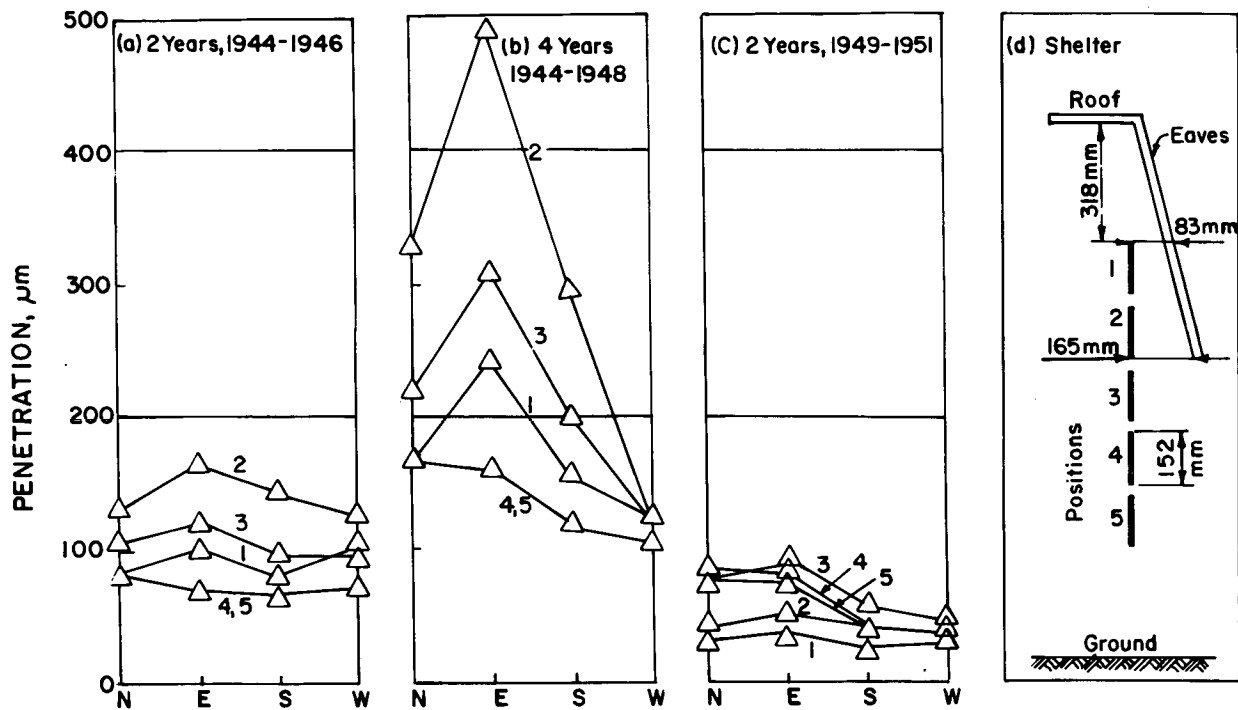


Figure 52. Effect of orientation and sheltering on corrosion of carbon steel exposed to the Kure Beach, N.C., marine atmosphere. [LaQue 1951; Larrabee 1966]

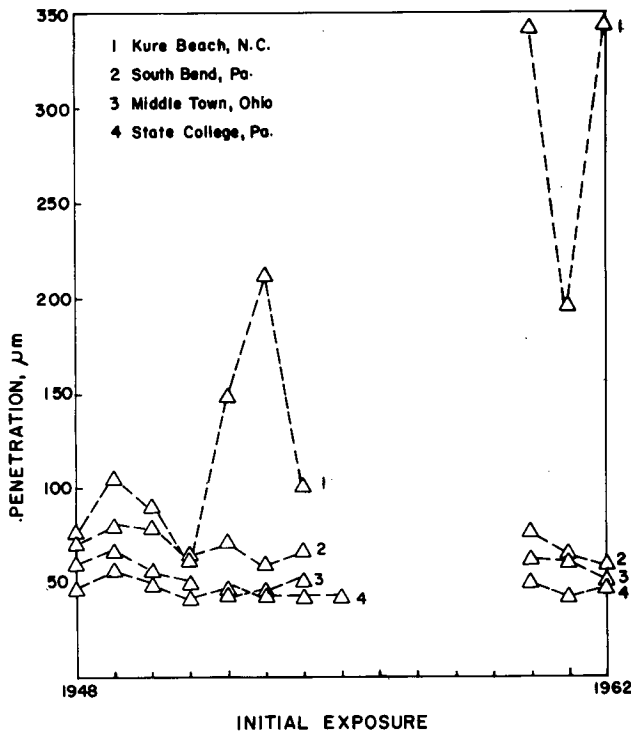


Figure 53. Effect of initial year of exposure on 2-year corrosion penetration of carbon steel. [ASTM G-1 1968]

According to Larrabee, the most likely explanation for one side corroding less than the other was the washing of the skyward surface by rain. As a result, the sulfur compounds, which are believed to be largely responsible for the severity of atmospheric corrosion of inland sites, were washed away to a greater extent on the skyward than on the groundward surface. The latter became wet by dew, but were not washed by rain. A second reason for the difference was the more rapid drying of the skyward than the groundward surface. Larrabee also observed more severe corrosion along the cut edges than on the flat faces.

Additional information on the effect of shelter was obtained from specimens exposed vertically in groups of five each, as shown in Figure 52(d) [Larrabee 1966]. Specimens 1 and 2 were completely sheltered and specimen 3 was partially sheltered. Specimens 4 and 5 were boldly exposed to wind, rain, and sunshine. The specimens faced north, east, south, and west. The sheltered test racks were located in semirural South Bend, Pa., and in marine Kure Beach, N.C. 800 ft (240 m) from the ocean. The types of steel exposed were carbon, copper, and a HSLA steel of the Cu-P-Cr-Si type.

The 4-year data for the semirural test site (1944–1948) showed that the differences in the amount of shelter and direction of exposure caused only small variations in penetration for the three steels. The effect of the amount of shelter in the four directions was as follows [Larrabee 1966]:

- *East*—Practically no effect of shelter.
- *North*—The boldly exposed specimens 4 and 5 were about 20 percent more corroded than the sheltered specimen 1, 2, and 3.
- *South or West*—The boldly exposed specimens 4 and 5

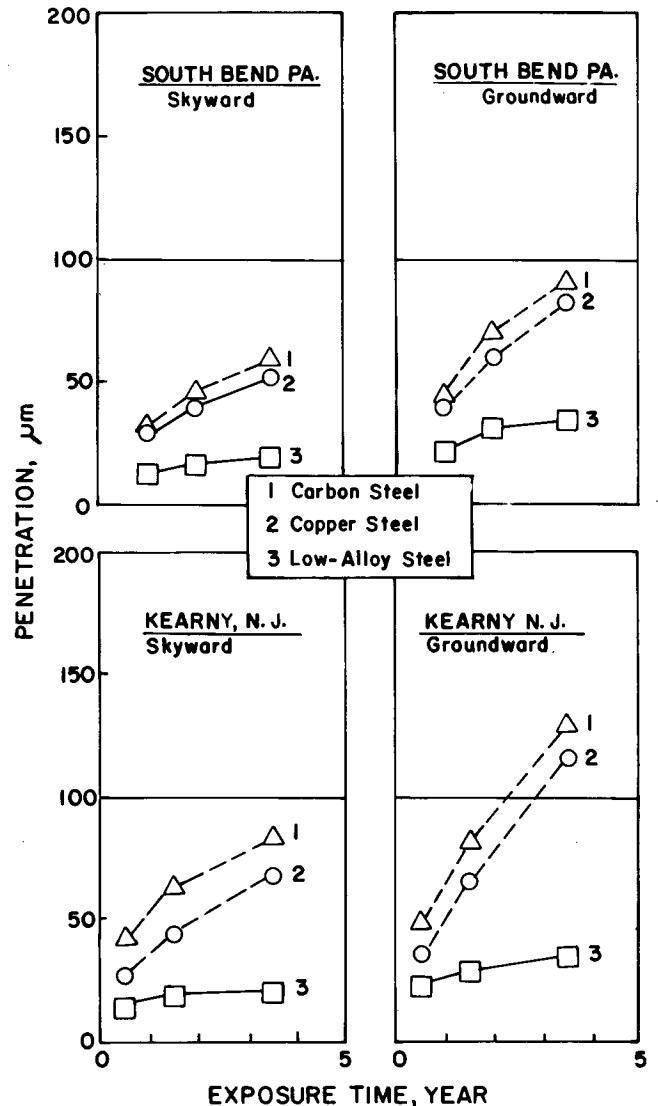


Figure 54. Effect of skyward versus groundward exposure on corrosion of steels in the rural atmosphere of South Bend, Pa., and the industrial atmosphere of Kearny, N.J. [Larrabee 1944]

were about 20 percent less corroded than the sheltered specimens.

The greater amount of sunlight received by the two bottom specimens facing south and west reduced the time of wetness. This helped to form a more protective rust film than that found on the specimens facing north. But even the bottom specimens facing south in this test had 50 to 100 percent higher corrosion losses than the duplicate control specimens facing south at a 30-deg angle from the horizontal.

The results were quite different for the marine atmosphere at Kure Beach, N.C. The 4-year data for carbon steel, plotted in Figure 52(b), show the following [Larrabee 1966]:

- *East*—The sheltered specimen 2 lost three times as much weight as the boldly exposed specimens 4 and 5. The partially sheltered specimen 3 lost twice as much weight.

- *North and South*—The sheltered specimen 2 lost 2–2.4 times as much weight as the boldly exposed specimens 4 and 5.
- *West*—The losses at all positions, sheltered and exposed, were within 20 percent.

These extreme variations in corrosion loss occurred because during the first few months of exposure hurricane-force winds deposited more seawater spray on the specimens facing east (toward the ocean) than on those facing north and south. The specimens facing west received least seawater spray. The losses of the boldly exposed specimens 4 and 5 were lowest because driving rains washed off the salt.

Figures 52(a) and (c) compare the 2-year data from the 1944 tests with data from a similar test begun in 1949 at the same site. Two important differences are apparent. The specimens in the 1949 test lost about one-half as much weight as those in the 1944 test. Furthermore, in the 1949 test, the two sheltered specimens 1 and 2 lost less weight than the boldly exposed specimens 4 and 5. Evidently, lower velocity winds during the 1949 test failed to drive seawater spray under the protective eaves. The specimens in all five positions lost less weight than the control specimens facing south at a 30-deg angle from the horizontal.

Knotkova, et al., examined the effect of orientation on carbon steel and Czechoslovak weathering steel, designation Atmofix 52A (Table 10) [Knotkova 1982a]. The specimens were exposed for 3 years at a 45-deg angle from the horizontal, facing east, west, and south. The results of the weight loss measurements, given in Table 32, showed that the eastward exposure was the most aggressive. The reason for this behavior was attributed to the slower drying time of the specimens facing east. Southerly exposure was the most favorable orientation for both steels. It also resulted in the greatest corrosion resistance for the weathering steel relative to that of the carbon steel.

In Chapter Four, section under "Industrial Environments," corrosion penetration data were reported for carbon and A242 Type 1 steels boldly exposed in industrial Newark, N.J., at a 30-deg angle from the horizontal [Cosaboom 1979]. These same steels also were used to examine the effect of shelter on corrosion.

The specimens were mounted vertically and horizontally at three locations: (1) on an interior girder of Bridge No. 9 on Interstate I-78, Section 5V; (2) on an exterior girder of the same bridge; and (3) on the roof of a nearby two-story building occupied by the New Jersey Department of Transportation. The I-78 bridge is part of a ramp. It was opened to traffic 6.5 years after construction and exposure of the weathering steel specimens. During that time there was no traffic on or below the bridge.

The corrosion penetration data are plotted in Figures 55(a) and (b) for the vertically and horizontally mounted specimens, respectively. In both cases, the bridge-sheltered specimens (curves 1 and 2) corroded up to 65 percent more than those boldly exposed on the building roof (curve 3). The vertically mounted specimens, which simulated a web, corroded more on the interior girders than on the exterior girders. The horizontally mounted specimens, which simulated a flange, corroded equally on interior and exterior girders. The results were comparable for both carbon and A242 Type 1 steels.

There was no difference in corrosion penetration of specimens mounted on an interior girder at midspan or near the abutment.

Table 32. Effect of orientation of 3-year corrosion penetration of carbon steel and Czechoslovak weathering steel Atmofix 52A [Knotkova 1982a].

Orientation	Angle	Corrosion Penetration, μm	
		Carbon Steel	Weathering Steel
East	45°	104	73
West	45°	61	61
South	45°	86	40
South	horizontal	83	53

This finding seems reasonable, as the bridge was not opened to traffic for the initial 6.5 years of testing. Hence, no salt-contaminated water could have leaked through the joints, and little, if any, debris would have accumulated near the abutments.

Zoccola performed a corrosion study of carbon and A242 Type 1 steels in Detroit, Michigan, that was similar to the previously cited Newark, N.J., study [Zoccola 1976]. The specimens were vertically and horizontally mounted on: (1) an interior girder of Eight-Mile Road over U.S. 10; and (2) the roof of the National Guard Armory roof, nearby the bridge. The bridge was opened to traffic in 1964, the freeway below in 1965. The horizontally mounted specimens had been painted on one side and were exposed with the nonpainted surface facing either skyward or groundward. In this way, corrosion penetration data were obtained for surfaces that simulated the top and bottom faces of a girder flange.

The data, plotted in Figures 56(a) and (b) for the vertically and horizontally mounted specimens, respectively, show the effect of shelter in a corrosive bridge environment. The bridge-sheltered A242 specimens (solid curve 1) corroded up to 6.4 times more than those boldly exposed on the roof of the Armory building (solid curve 2). In fact, when sheltered by the bridge, the A242 specimen corroded much like carbon steel. The poor performance of the A242 steel was due to the accumulated deposits of road salts, debris, and corrosion products that kept the surfaces wet, trapped chlorides and sulfates, and caused accelerated poulitice-type attack [Zoccola 1976]. The data in Figure 56 also show that the corrosion rate of the bridge-sheltered A242 steel specimens was increasing with time.

The horizontal exposure data (Fig. 56(b)) suggest two conclusions. First, the corrosion penetration of A242 steel on the skyward surface was lower (46 to 54) under bold exposure, but higher (65 to 44) under sheltered exposure. This behavior resulted from beneficial washing of the boldly exposed skyward surfaces by the rain, and from detrimental debris accumulation on sheltered skyward surfaces, respectively. Secondly, for bold and sheltered exposures, the average corrosion penetration per side was greater for the horizontal than for the vertical surfaces.

The last example on the effect of shelter presented herein comes from British data on carbon and A588 Grade A steels [McKenzie 1978]. The test sites were located in rural Iden, rural Loudwater, industrial Tinsley, and marine Shoreham-by-Sea. At each test site, sets of specimens of each steel were exposed vertically: (1) under the deck of a bridge; (2) under a 12 × 12 in. (300 × 300 mm) wide-flange beam, with the web placed horizontally so that the lower halves of the flanges would shelter the specimens and with the upper half of the beam thermally

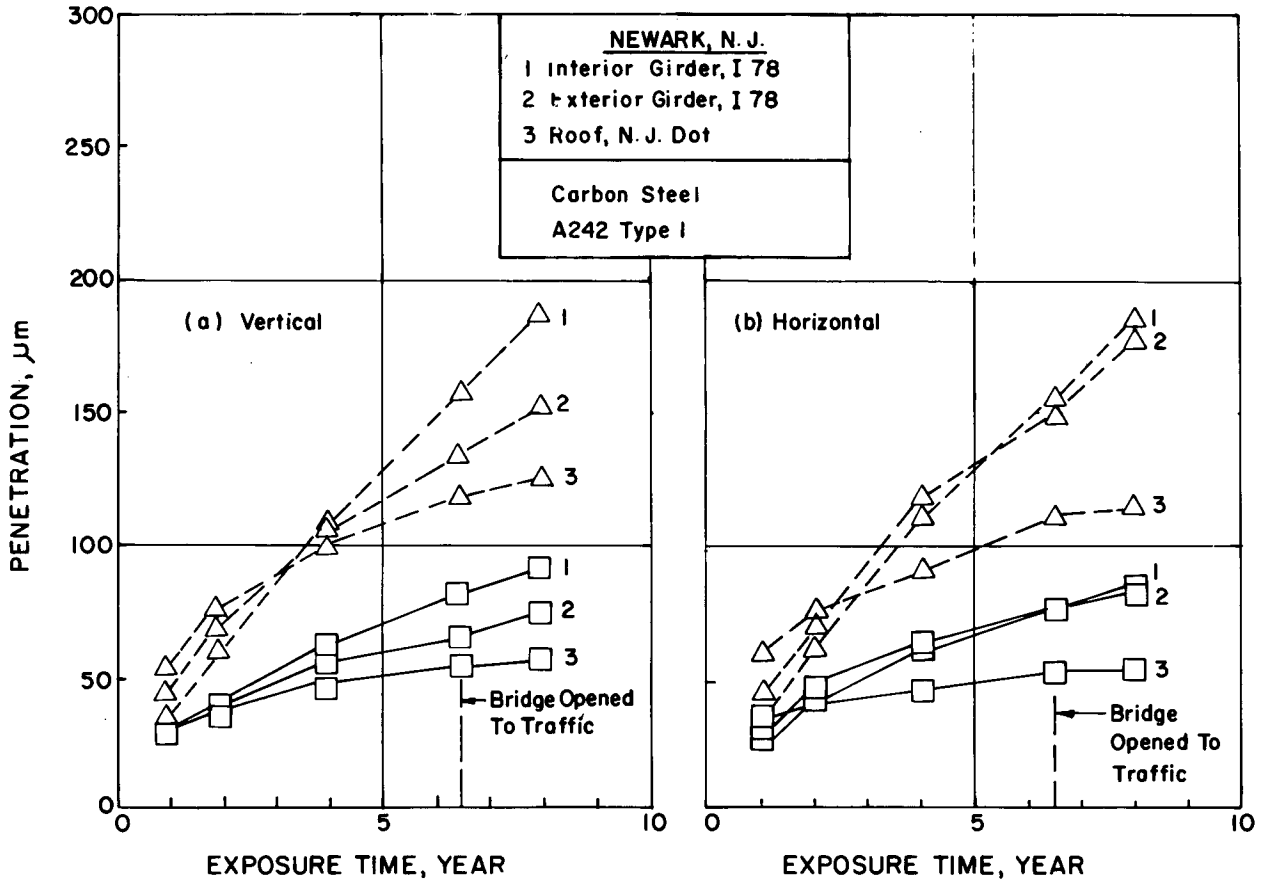


Figure 55. Effect of sheltering on corrosion of steels exposed to the Newark, N.J., industrial atmosphere. [Cosaboom 1979]

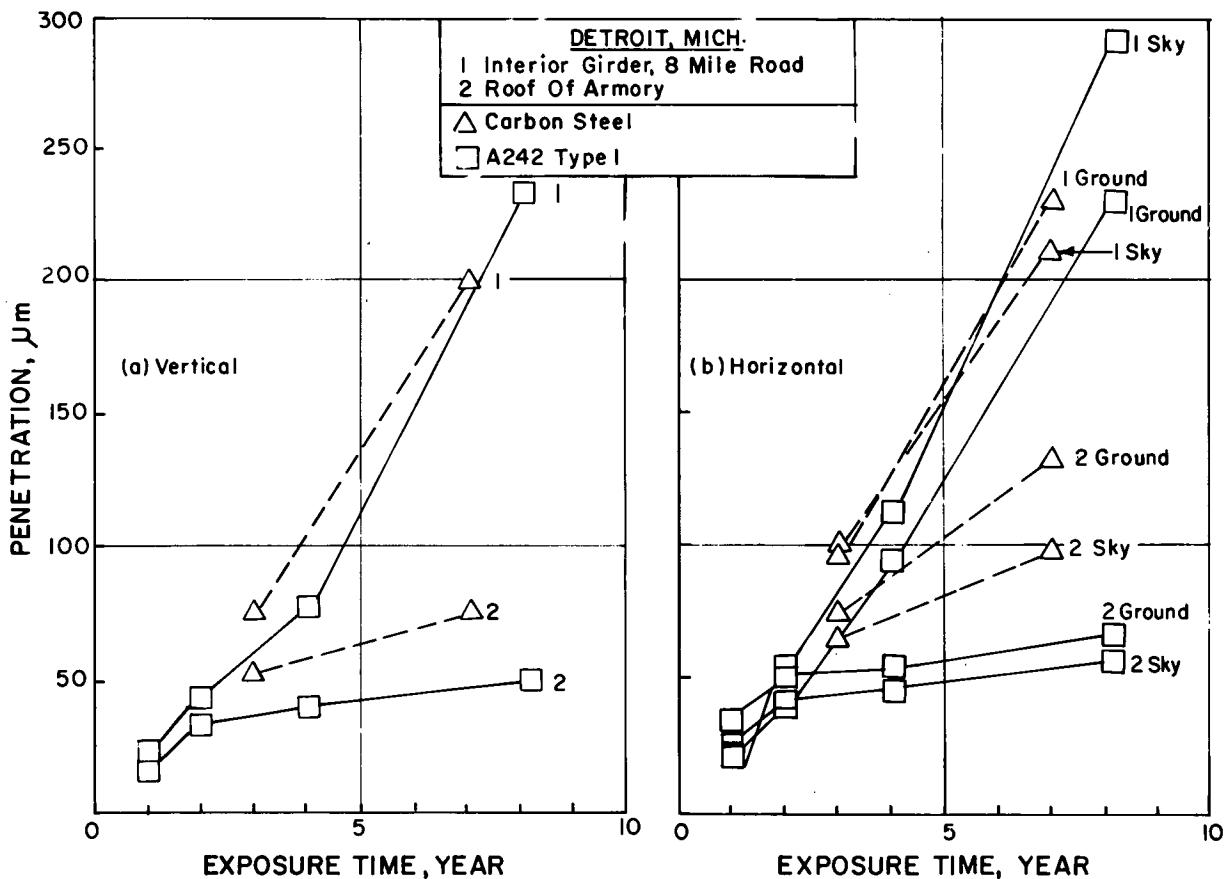


Figure 56. Effect of sheltering on corrosion of steels exposed to the Detroit, Mich., industrial atmosphere. [Zoccola 1976]

insulated; and (3) in the open facing the prevailing wind direction.

McKenzie concluded the following from the data plotted in Figure 57. The corrosion rates measured beneath a bridge (curve 1) can differ considerably from those measured in open exposure (curve 3). At Loudwater and Tinsley, bridge-sheltered corrosion rates for both steels were much lower than in open exposure. At the marine site, however, the opposite was found, indicating the adverse effect of chlorides under sheltered conditions. At Iden, corrosion rates under open and bridge-sheltered exposures were similar. The shapes of the penetration versus time curves differed for open and bridge-sheltered exposures. In particular, at rural Loudwater and marine Shoreham-by-Sea, the corrosion rates did not decrease with time under sheltering so that, for A588 Grade A, the rate over the last 2 years of the test was higher than in open exposure.

McKenzie further concluded that the beam shelter did not adequately represent the bridge shelter. The corrosion penetration under these exposures was similar at Loudwater, but not at any other site. The penetration measured for specimens beneath the beam depended on the position of the specimen. It was found in some later tests that specimens placed vertically beneath the beam had $1\frac{1}{2}$ times the penetration of those placed near the sides. This suggests variations in the microenvironment and, therefore, in corrosion penetration with type of shelter. The most noticeable difference in beam sheltered versus open

exposure was the increase in penetration at Shoreham-by-Sea, the marine site, again pointing to the severe corrosive effect of chlorides under sheltered conditions.

Generally, in open and sheltered exposure at British sites, weathering steel did not perform significantly better than carbon steel.

ANGLE OF EXPOSURE

Exposure of a specimen facing south (for the northern hemisphere) at an angle equal to the latitude of the test site yields the lowest corrosion rate for most materials [ASTM G-50 1976] according to the ASTM G50 Standard Recommended Practice for conducting Atmospheric Corrosion Tests on Metals. In the United States, most published data on atmospheric corrosion of metals are based on an exposure angle of 30 deg from the horizontal. In comparison, central United States falls between the latitudes of 30 deg in New Orleans, La., and 49 deg at the North Dakota border with Canada. The British normally expose specimens vertically, with the United Kingdom reaching from about 50 deg to 60 deg latitude. The Germans and Czechoslovaks mostly use 45 deg, their latitude being about 48 deg to 55 deg.

Taylorson reported that vertically exposed specimens lost weight somewhat faster than those exposed at 30 deg [Taylorson

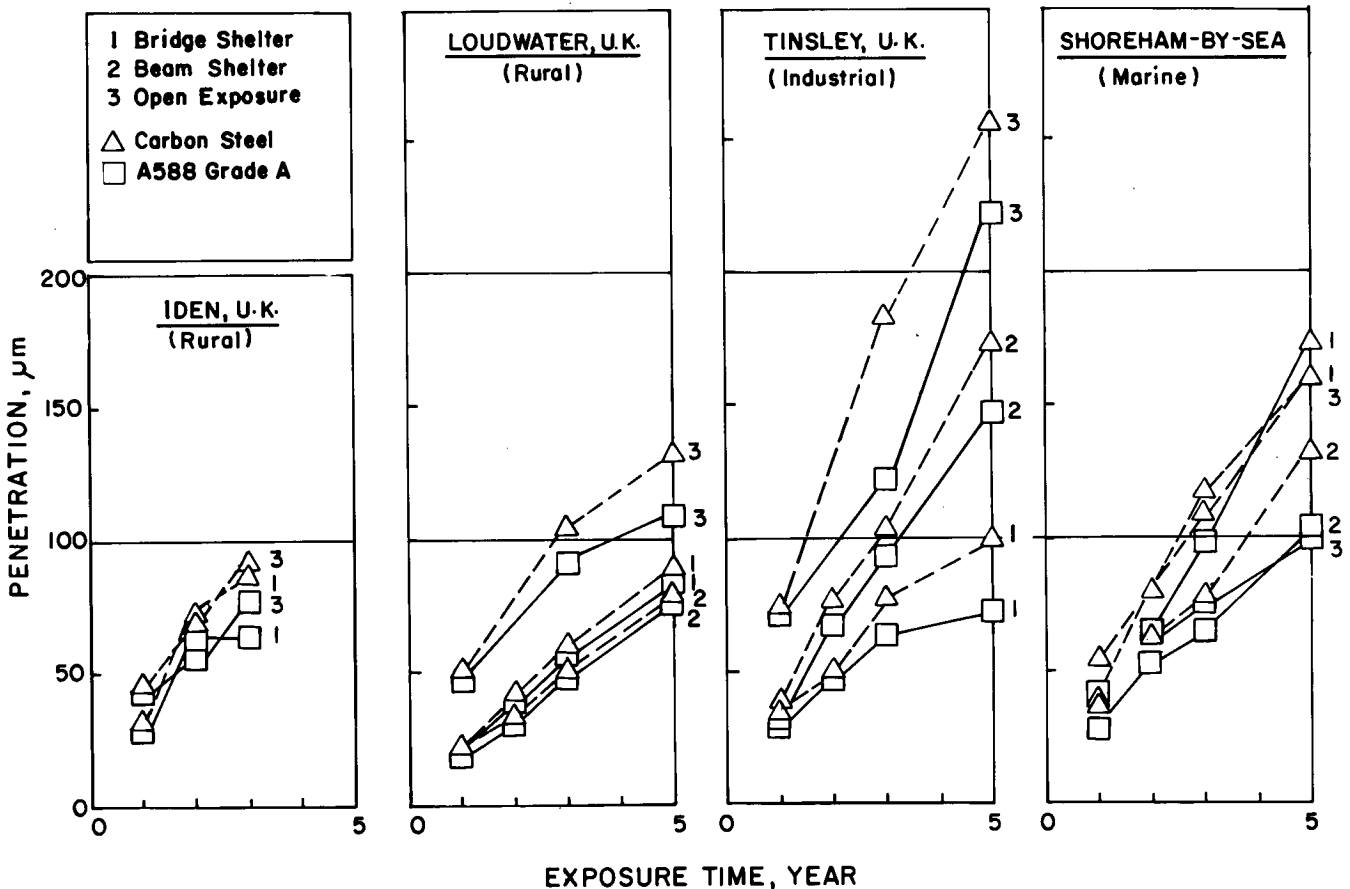


Figure 57. Effect of sheltering corrosion on steels exposed at four British sites. [McKenzie 1978]

1941]. Larrabee quotes LaQue as stating that the reproducibility of results was poorer when the specimens were exposed vertically than when they were exposed at 30 deg [Larrabee 1966]. He attributed this to a slight variation in the position of the specimen from the vertical which results in a slight sheltering of the specimen and reduces the amount of rain washing that surface.

The following three examples illustrate the effect of angle of exposure on corrosion of carbon and A242 Type 1 steels. Cosboom [1979] measured the corrosion penetration of specimens mounted vertically, horizontally, and at a 30-deg angle. The test sites were on the roof of the New Jersey D.O.T. building and on Bridge No. 9 on I-78. The data from Figures 30 and 55 are replotted in Figure 58(a) and (b), respectively, to show the effect of angle of exposure. The roof specimens corroded more in vertical exposure than in horizontal and 30-deg exposures, as one would expect when specimens are rain washed and sun dried. Both steels exhibited this behavior.

The fully sheltered specimens on interior girders corroded more in a vertical than in a horizontal position. For the partially sheltered specimens on exterior girders, however, just the opposite was found. Both observations are contrary to the following expectations: (1) in fully sheltered exposure, vertical specimens trap less debris and should corrode less than horizontal speci-

mens and (2) in partially sheltered exposures, vertical specimens are washed less and should corrode more than horizontal specimens. Since the specimens were mounted on the I-78 bridge 6.5 years after it was opened, traffic could not have affected the results. The differences cannot be explained with the available information.

Zoccola [1976] in a similar series of tests, exposed specimens vertically and horizontally on the roof of the National Guard Armory and on an interior girder of Eight-Mile Road over U.S. 10 in Detroit, Michigan. The data for carbon and A242 Type 1 steels from Figure 56 are replotted in Figure 59 to show the effect of angle of exposure. (The curves for skyward and groundward surfaces were averaged). On the roof of the Armory building, the vertical specimens corroded less than the horizontal specimens, contrary to what one would expect under bold exposure. They also corroded less when hung on an interior girder of the Eight-Mile Road bridge. But in this case the observed behavior was expected, because less debris and salt spray kicked up by the traffic below the bridge accumulated on the vertical than on the horizontal specimens, owing to drainage.

The third example comes from German data for A588 Grade A steel specimens that were exposed at six sites, both vertically and at 45 deg from the horizontal [Hein 1981]. The 45-deg exposure data were replotted from Figure 32. The vertical ex-

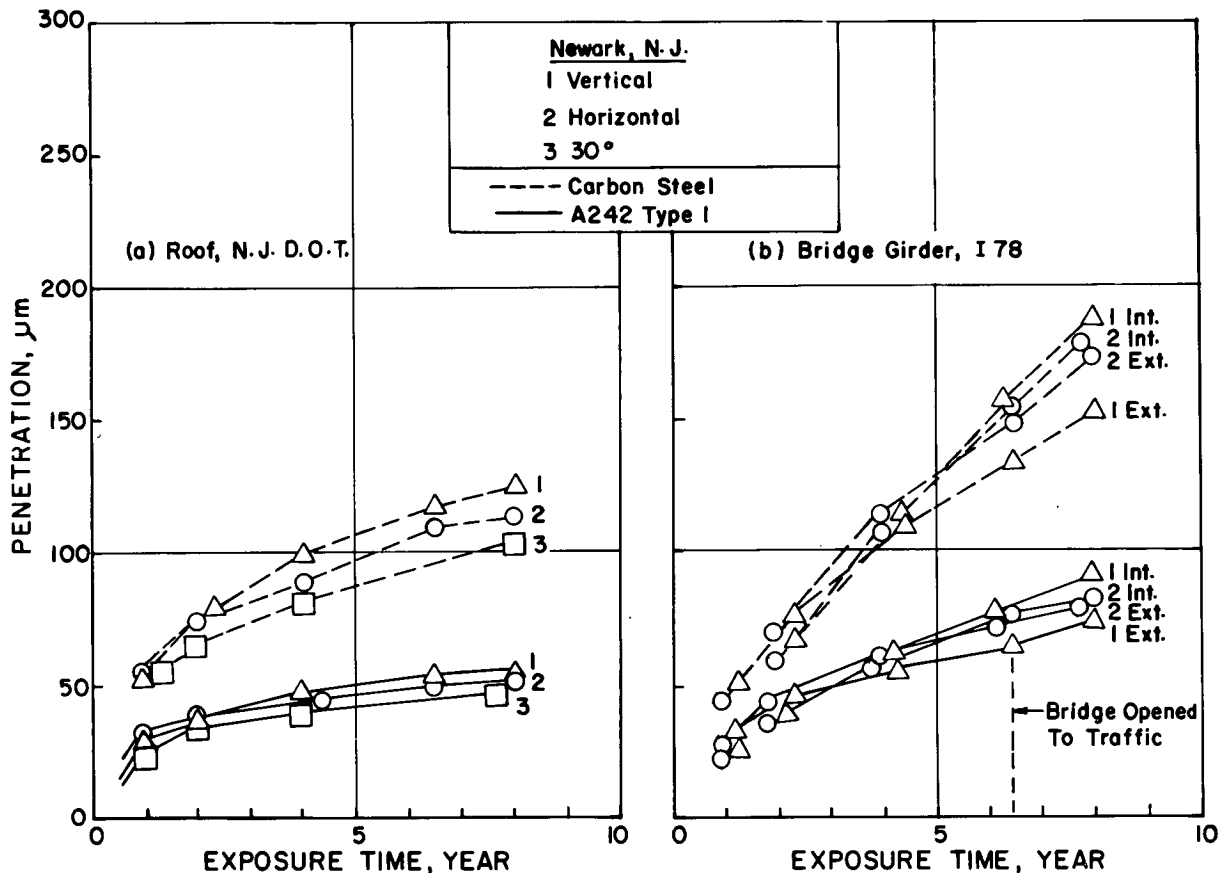


Figure 58. Effect of exposure angle on corrosion of steels in Newark, N.J., industrial atmosphere. [Cosboom 1979]

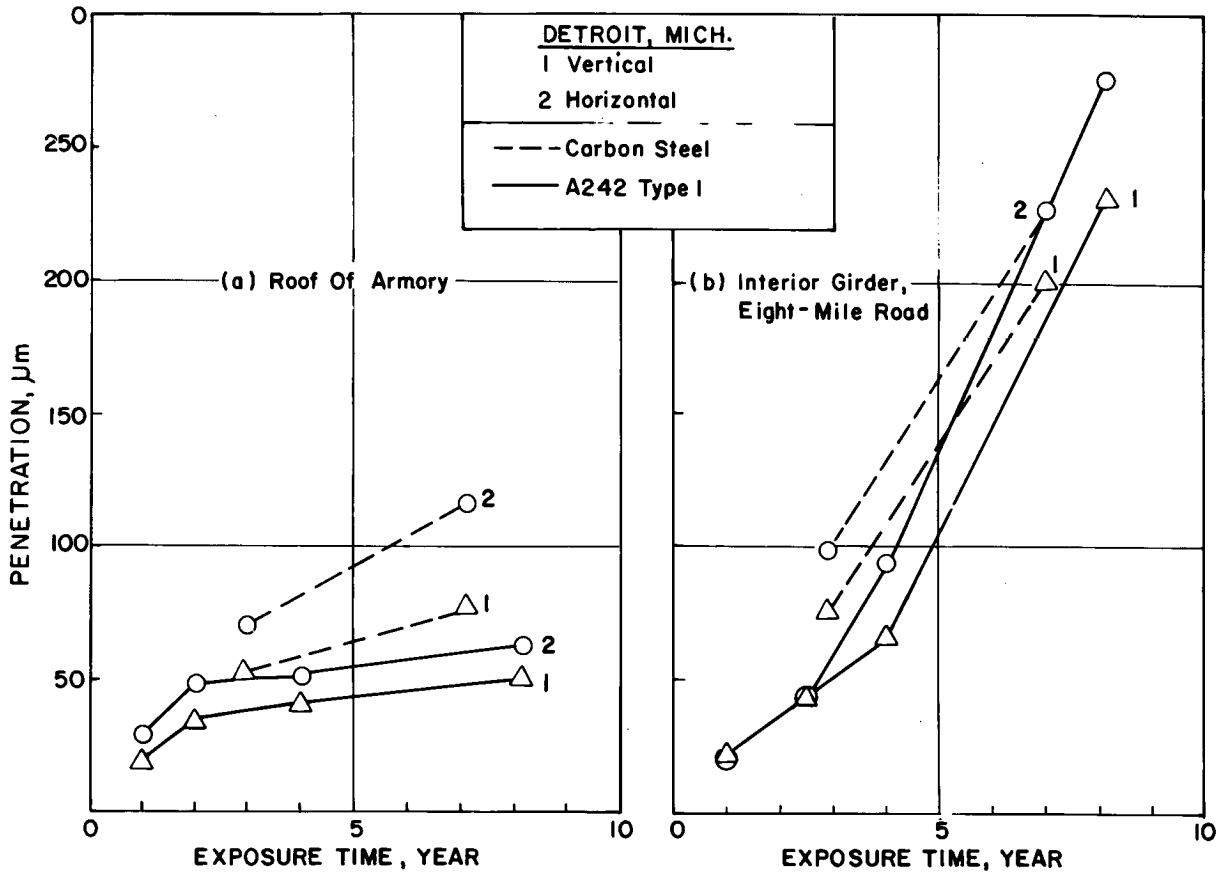


Figure 59. Effect of exposure angle on corrosion of steels in Detroit, Mich. [Zoccola 1976]

posure data were added in this section. Figure 60 shows that at five of six sites vertical specimens corroded more than those exposed at 45 deg, as one would expect for bold exposure.

Horizontal exposure was also found to be more corrosive than 45-deg exposure in 3-year tests of Czechoslovak weathering steel, Atmofix 52A [Knotkova 1982a].

In summary, under bold exposure, vertical and horizontal specimens corroded more than specimens exposed at an angle close to the latitude of the site. This has been known for a long time. Vertical specimens were expected to corrode more than horizontal specimens in bold exposure, but not in sheltered exposure. Some data confirm those expectations, other data do not. The perceived contradictions could result from differences in microclimate that were not fully understood.

CONTINUOUSLY MOIST CONDITIONS

Alternate cycles of wetting and drying are one of the four necessary conditions under which the weathering steels develop a protective oxide coating. Under continuously moist conditions, especially with water of low pH, weathering steels do not exhibit superior corrosion resistance [Larrabee 1966].

For example, Figure 61 shows corrosion penetration data for a low carbon steel (0.024 percent Cu) and an HSLA steel (0.36 percent Cu, 0.13 percent P, 0.08 percent Ni, 1.0 percent Cr,

and 0.66 percent Si) [Larrabee 1966]. They were exposed in: (1) a railroad tunnel with continuously wet walls; (2) a second tunnel with dry walls; and (3) on the roof of an office building in industrial East Pittsburgh, Pa. The HSLA steel specimens exposed on the building roof (curve 3 in Fig. 61) was more corrosion resistant than the weathering steels in the industrial environments corresponding to curves 1 to 15 in Figure 34. Its 4-year penetration was 4.4 times smaller than that of carbon steel at the same site. In the two tunnels, the HSLA steel specimens corroded like their carbon steel counterparts. The atmosphere in the wet tunnel was so severely corrosive that after one year of exposure the penetration of the HSLA steel was 11 times that of the same steel exposed on the roof.

Direct precipitation of rain, snow, or condensation is not needed for a steel surface to remain continuously wet. Moisture can be deposited by capillary action of the porous oxide coating, and adsorption by corrosion products or salt deposits on the surface. Pores in the oxide coating, cracks and crevices, small pits, and the space between the steel surface and a settled speck of dust all foster capillary action. Forces of attraction between water molecules and a solid surface increase the possibility of adsorption condensation of water vapor on the surface of a solid [Tomashov 1966].

Such moist atmospheric corrosion, as opposed to wet atmospheric corrosion, takes place when a thin, invisible film of electrolyte forms on the surface at a relative humidity well below 100 percent. The critical relative humidity, above which the

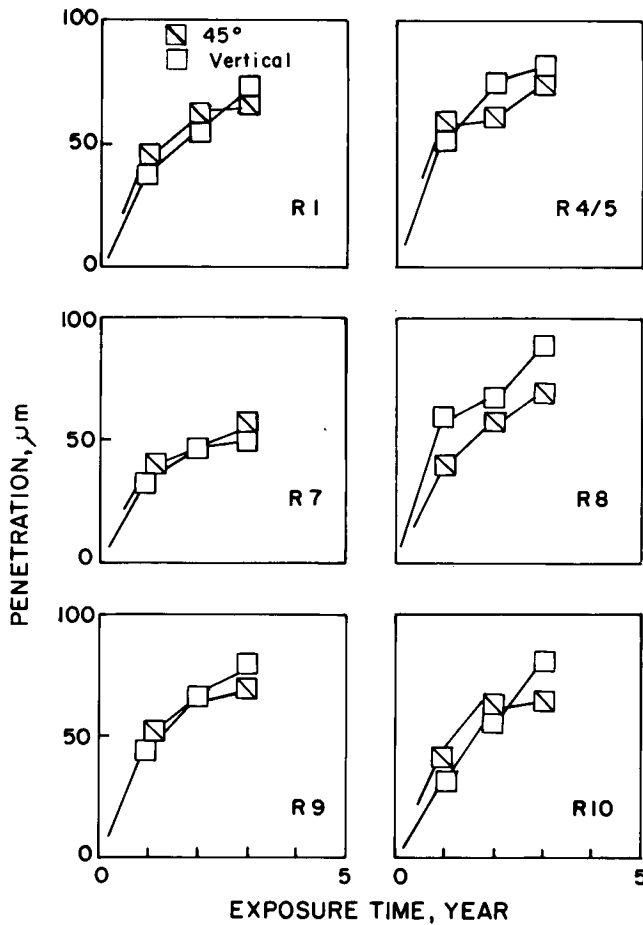


Figure 60. Effect of angle of exposure on corrosion of A588 Grade A steel at six German reference sites. [Hein 1981]

atmospheric corrosion rate increases sharply, depends on the type of metal, surface contaminants, atmospheric pollutants, and the nature of the corrosion product. In the following, data on critical relative humidity for several combinations of steel surface conditions and pollutants are examined.

Vernon [1931, 1933, 1935] first showed the effect of sulfur dioxide, in conjunction with relative humidity, on the corrosion of iron. He corroded carbon steel specimens in bell-jar atmospheres with controlled humidity, SO_2 content and temperature. In the absence of SO_2 , even saturated humid air (100 percent relative humidity) did not cause appreciable rusting. When the SO_2 content was raised to 0.01 percent, the critical relative humidity dropped to 60 percent. At 0.01 percent SO_2 content and relative humidities of 70, 75, and 99 percent, iron corroded 7, 8, and 12 times as much as in 100 percent humid air free of SO_2 . The presence of 0.03 percent carbon dioxide, the normal atmospheric content, greatly reduced corrosion.

Vernon also found that silica particles (dust) neither appreciably increased corrosion nor lowered the critical relative humidity. Ammonium sulfate, however, lowered the critical relative humidity to 80 percent.

Table 33 gives relative humidity values of air in an enclosed space located above saturated solutions of various salts [Tomashov 1966]. These values correspond to the critical relative humidity at which the respective salts begin to wet by

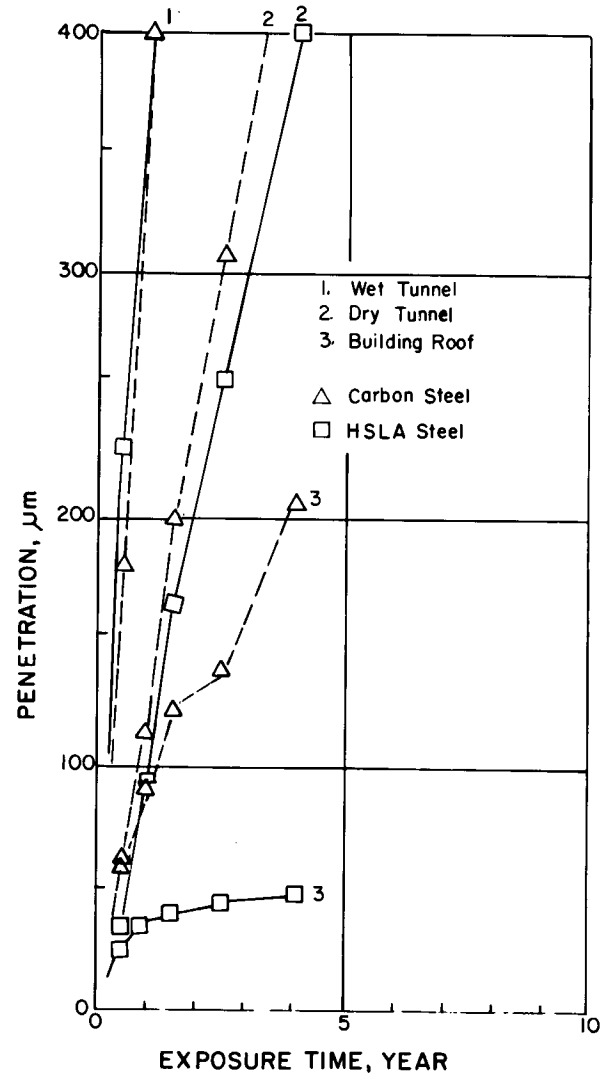


Figure 61. Effect of continuously moist conditions on corrosion of steels. [Larrabee 1966]

Table 33. Water vapor pressure above saturated aqueous solution of salts at 20°C (68°F) [Tomashov 1966].

Salt	ZnCl	CaCl	NHNo	NaCl	NHCl	Na SO	(NH) SO	KCl	ZnSO	K So
Relative Humidity	10	35	67	78	79	81	81	86	91	99

dissolution in moisture adsorbed from the air, forming an electrolyte. Most notably, the results indicate that sodium chloride, a widely used deicing salt, becomes moist at 78 percent relative humidity and 20 C (68 F).

Table 34 shows the effect of steel surface condition on the critical relative humidity. The corrosion rate of iron, with a prior oxide coating that formed in water or in 3 percent sodium chloride solution, increases sharply when the relative humidity of the air rises above 65 and 55 percent, respectively. At these

Table 34. Critical value of relative humidity at which the corrosion rate of iron sharply increases [Tomashov 1966].

Surface Condition	Critical R.H. (%)
Clean surface in pure air	about 100
Clean surface, corroding in air + 0.01% SO ₂	70
Prior slight corrosion in water	65
Prior corrosion in 3% NaCl solution	55

critical humidities, a continuous film of moisture forms by capillary condensation or by the hydration of salts, corrosion products, and other films that might be present on the surface. These results indicate that weathering steels with a dense rust coating (smaller capillaries) in a salt-free environment would likely corrode beginning at relative humidities somewhat higher than 65 percent, while those with a loose rust coating (larger capillaries) in a salt-laden environment would corrode at 55 percent.

Structural steel members sheltered from the sun and wind generally dry at a slower rate than exposed members. As a result of the high heat conductance of steel, its surface temperature can fall below the ambient and dew point temperatures. Both factors tend to lengthen the time of wetness. However, because of the massiveness of a structure, there could be a time delay before condensation occurs.

Forty-eight years have passed since Vernon found that corrosion of iron accelerates only beyond a critical relative humidity. The significance of this fact was not fully appreciated until Sereda developed a method for measuring the percentage of time when this critical humidity is exceeded [Sereda 1960]. This period, called "time of wetness," was subsequently shown to be the most important factor promoting the atmospheric corrosion of metals.

The time of wetness can be measured with a moisture-sensing element consisting of alternate electrodes of copper and gold, or zinc and gold, spaced from 4 to 8 mils (100 to 200 μm) apart [ASTM 1982]. The recorded output potential gives a measure of the duration of wetness on the sensing element. Experience has shown that the sensing element reacts to factors causing wetness in a similar manner as the surface on which it is mounted.

This method does not relate the time of wetness to levels of ambient relative humidity. However, studies showed that the annual time of wetness of panels exposed under standard conditions is equivalent to the cumulative time during which the relative humidity exceeded a certain value. The time of wetness of bridges at different locations can be greatly influenced by local atmospheric and service conditions. There is at present no reliable method of estimating the time of wetness for various surface conditions from meteorological records.

INDUSTRIAL POLLUTANTS

An indication of how weathering steels might perform in industrial environments around chemical plants can be gained from the results of short exposure tests of carbon (0.01 percent Cu), A242 Type 2, and A588 Grade A steels [Schmitt 1967].

Table 35 lists the chemical plants and the atmospheric constituents. The data for specimens exposed 2 years are plotted in Figure 62. Those exposed only 6 months and one year were not plotted because reliable assessment of corrosion behavior is not possible after such a short time. The curves are identified in Table 35.

The data show that the atmosphere at the sulfur plant (curve 1) was much more corrosive than at the other plants. The atmosphere at the chlor-alkali plant (curve 2) was the next most corrosive, but it was appreciably less corrosive than at the sulfur plant. The atmospheres at the other plants (curves 3 to 7) were somewhat more corrosive than the industrial atmospheres at American industrial sites for which data were reported in Figure 34. The A588 Grade A steel was generally superior to carbon steel, but slightly inferior to the more richly alloyed A242 Type 2 steel.

Knotkova, et al., tested carbon steel and Czechoslovak weathering steel, Atmofix 52A, in the localized climate of chemical plants, agricultural areas, metallurgical and textile production plants, as well as in automotive and street car environments [Knotkova, 1982a]. The results of these 5-year exposures are given in Table 36. They show that weathering steels are generally not suited for such applications; most definitely not for atmospheres containing chlorides. In localized climates, where the major pollutant is sulfur dioxide (SO₂), weathering steel is significantly more resistant than carbon steel.

Knotkova, et al., also investigated the corrosion behavior of low-carbon steel and Atmofix 52A weathering steels as a function of the level of SO₂ pollution. The results of the 5-year exposure tests at a number of Czechoslovak sites were grouped as follows: (a) rural and urban atmospheres with average annual SO₂ pollution levels of 40 mg/m²/day or less; (b) urban and industrial atmospheres where the SO₂ pollution level reached 90 mg/m²/day; and (c) heavily polluted industrial atmospheres where the SO₂ level reached 120 to 150 mg/m²/day. The data for the three groups, plotted in Figure 63, show a progressive increase in corrosion penetration with the SO₂ content in the air. The rust coating on the weathering steel had a dark brown to violet color. Its structure in the industrial atmospheres was coarser than in the rural atmospheres. For heavily polluted atmospheres (Fig. 63(c)), the corrosion level was higher than is generally seen on weathering steels. In this case, the rust coating was spalling and regrowing, both regularly and intensively.

Knotkova, et al., recommended a maximum limit of 90 mg/m²/day on annual average SO₂ pollution above which a weathering steel structure should be protected against corrosion. Above this limit the structure should be designed for loss of metal and load-bearing capacity during the service life. The periodic shedding and regrowth of the rust coating becomes significant, and the rust coating ceases to be protective as intended. However, in view of the EPA standards, the installation of SO₂ scrubbers in fossil fuel plants and the use of low-sulfur coal, excessive level of SO₂ are unlikely to occur in most American industrial sites.

The effect of sulfur dioxide on corrosion was also investigated by Haynie, et al., who tested A242 Type 1 specimens in environmental chambers at controlled levels of sulfur dioxide, nitrogen dioxide, ozone, relative humidity and temperature [Haynie 1978]. During the 1,000-hour exposure time, weathering was simulated with programmed dew/light cycles of 40-min

Table 35. Corrosion penetration of carbon and weathering steels exposed to various chemical plant atmospheres (see Fig. 62). [Schmitt 1967]

Curve No.	Type of Plant	Atmospheric Constituents	Corrosion Penetration, μm			Exposure Time, Years
			Carbon Steel	A242 Type 2	A588 Grade A	
1	Sulfur	Chlorides, sulfur and sulfur compounds	1100	518	823	2
2	Chlor-alkali	Moisture, chlorides and lime	478	145	188	2
3	Chlorinated hydrocarbons	Chloride compounds	272	56	56	1
4	Chlor-alkali	Moisture, lime and soda ash	211	53	48	2
5	Sulfuric acid	Sulfuric acid fumes	114	53	56	1
6	Petrochemical	Chlorides, hydrogen sulfide and sulfur dioxide	86	30	48	2
7	Petrochemical	Ammonia and ammonium acetate fumes	58	33	48	1

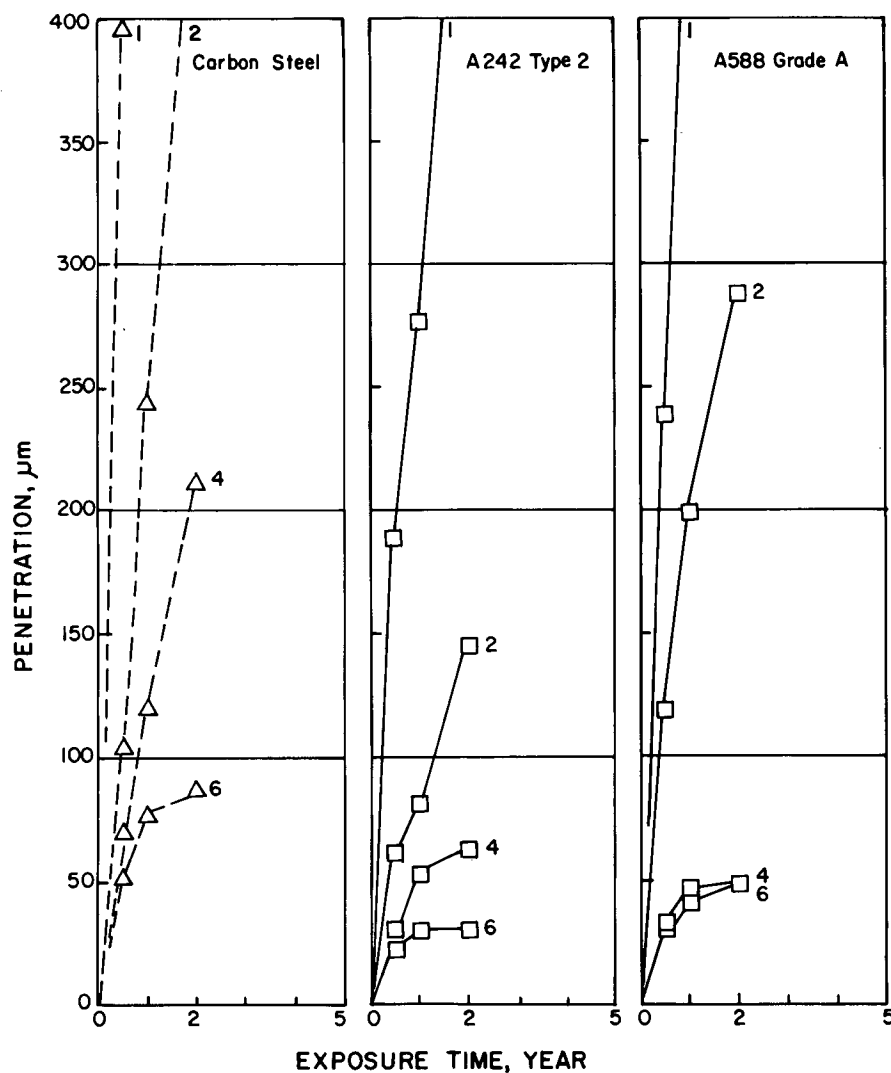


Figure 62. Corrosion penetration of carbon and weathering steels exposed to various chemical plant atmospheres (see Table 35). [Schmitt 1967]

duration. The corrosion penetration of the specimens was essentially a linear function of time, contrary to the declining power function of time normally observed in naturally weathered A242 steel. Evidently, a protective oxide coating did not form. The results of the study are of interest in the sense that they showed concentration of SO_2 , relative humidity, and temperature to be the important factors controlling corrosion.

The following empirical equation accounted for 91 percent of the variability in the experimental data.

$$C = \left[5.64 \text{SO}_2 + e \left(55.44 - \frac{31150}{RT} \right) \right] t_w \quad (28)$$

in which:

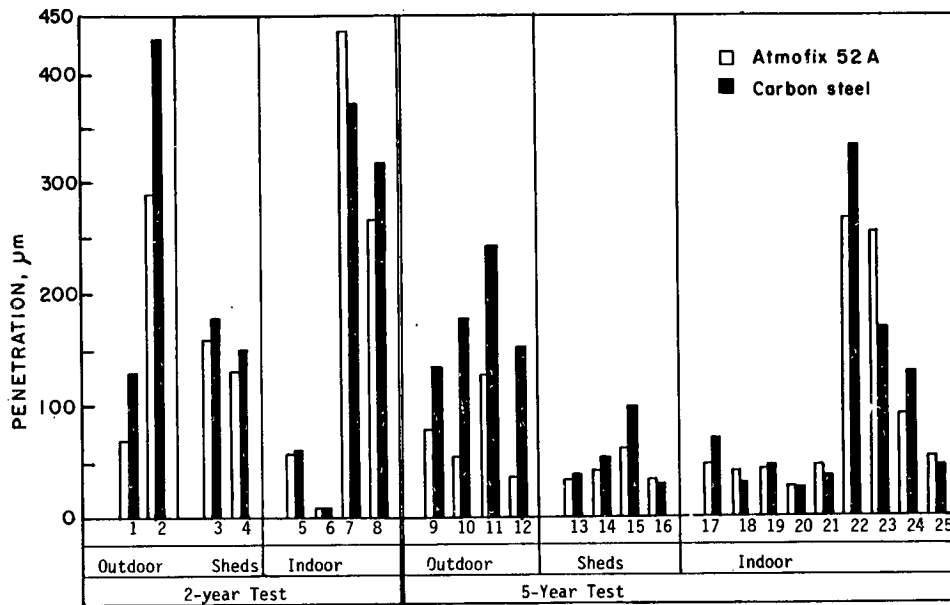


Table 36. Corrosion behavior of carbon and Atmofix 52 A steels in manufacturing and other specific microclimates: (1) production of Glauber salt; (2) production of H₂SO₄; (3) NaCl transporter; (4) Cl₂ bottling platform; (5) NaCl storehouse; (6) compressor room, NH₃; (7) electrolysis of NaCl; (8) liquefying of Cl₂; (9) atmospheric test station Letnany; (10) open-cut mining excavator; (11) overburden dumping machine; (12) separator of combustion products; (13) road bridge; (14) stadium; (15) storehouse of wood; (16) winter stadium; (17) greenhouse; (18) underground coal mine; (19) cowhouse-skylight; (20) cowhouse, loft; (21) little bull house and loft; (22) lead-coating workshop; (23) synthetic fertilizer storehouse; (24) textile dyeworks; and (25) textile washing works.

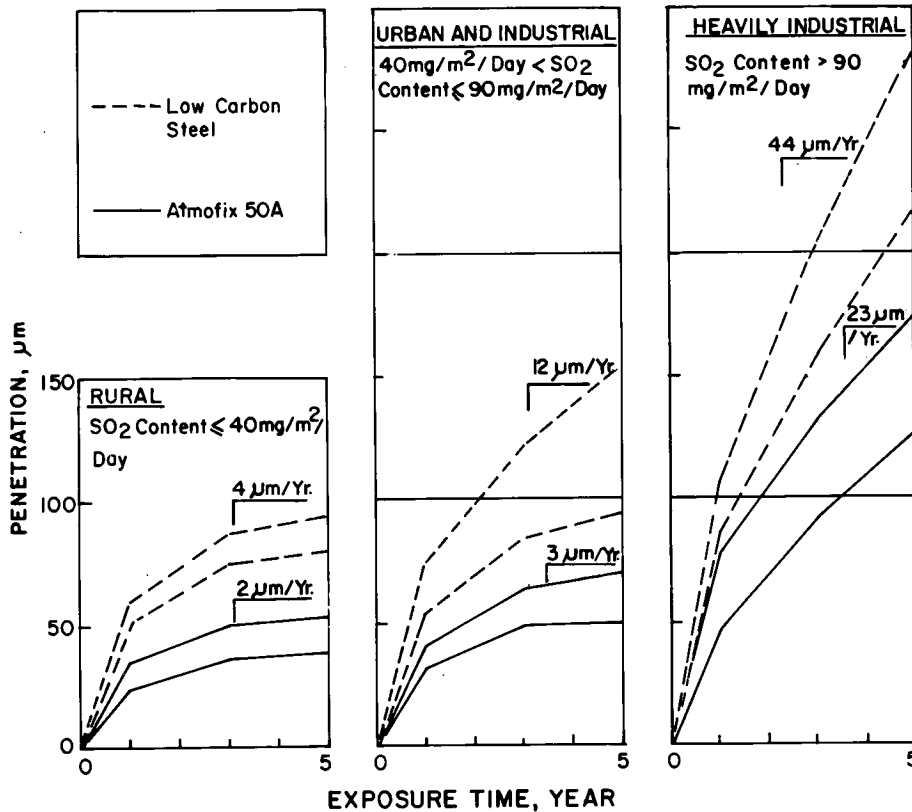


Figure 63. Effect of sulfur dioxide content on corrosion of low carbon and weathering steels exposed to various atmospheres in Czechoslovakia. [Knotkova 1982(a)]

- C = corrosion penetration, μm ;
 t_w = time of panel wetness, calculated from an empirical equation, year;
 SO_2 = concentration of SO_2 , g/m^3 ;
 R = gas constant, $1.9872 \text{ cal}/\text{g}\cdot\text{mol}$; and
 T = geometric mean panel temperature when wet, $^\circ\text{K}$.

Although the predicted corrosion penetrations agreed poorly with actual values measured in cities, the results of the chamber study give valuable insight into the effect of SO_2 , relative humidity, and temperature.

DEICING SALTS

Deicing salts spread on highways during the winter can create a more corrosive environment for the bridge than that encountered at marine sites (Chapter Four "Marine Environments"). On heavily salted highways in the snow-belt region of the United States, the salt contaminates the steel structure in the following ways: (1) salt water leaks through the bridge deck, mainly at expansion and construction joints, and drains longitudinally along sloped bottom flanges; (2) a mist of saltwater runoff is kicked up in the wake of trucks passing beneath the bridge and settles on the steel structure; (3) dust containing salt particles from dry roadways is blown against the steel structure; and (4) salt water lying on the upper surface of the bottom flange wicks up the web by capillary action as much as 10 in. (250 mm).

Salt accumulates on sheltered surfaces that are not washed by rain. Quantitative data are available from the analysis of debris samples that were removed from 16 weathering steel bridges in Michigan and analyzed for sodium chloride [Culp 1980]. Salt concentrations as high as 8 percent were found in the rust removed from flanges beneath leaking expansion joints. Salt concentrations as high as 3 percent were found where roadway spraying and dusting were the only sources of contamination. The saltwater runoff from Michigan's highways can have a sodium chloride concentration higher than the 3.5 percent concentration in seawater.

To determine the section loss of weathering steel bridges due to corrosion, McCrum [1983] measured the thickness of webs and flanges of 52 bridges located in lower Michigan. Twenty-five bridges were located in rural areas and 27 in urban areas. Most were less than 8 years old. Comparing the thickness measured with an ultrasonic gage with the nominal thickness yielded information on corrosion rates and locations of most severe corrosion. The findings were as follows:

1. Salt, regardless of source, is the major cause of accelerated corrosion of weathering steel bridges.
2. Salt spray from traffic passing under a bridge is as corrosive as salt water leaking through expansion joints. The former affects large surface areas of the steel beams; the latter is usually limited to areas near joints.
3. The first three or four beams facing the on-coming traffic corrode almost twice as much as those following later in the traffic spray path.
4. On urban bridges older than 7 years, steel beams nearest to the on-coming traffic beneath the bridge are corroding at a rate of 3 to 4 mils (75 to 100 μm)/year/surface on the top of the lower flange nearest the web and on the lower $\frac{1}{4}$ of the web.

Some beams are corroding at a rate of 5 to 6 mils (125 to 150 μm)/year/surface.

5. The cross-section loss of the upper and the lower flanges of beams not exposed to salt spray appears to be about the same, even though the upper flange is only exposed on one side.

6. The upper and lower portions of the web corroded at about the same rate, and slightly faster than the middle portion.

7. The portions of a beam near the abutments had lower corrosion rates than those directly over the roadway.

The results of the exposure tests in Newark, N.J., and Detroit, Mich., cited under "Shelter and Orientation," were replotted in Figures 64 and 65 in a manner that shows the effect of salt contamination on corrosion of weathering steel bridges [Cosa-boom 1979; Zoccola 1976]. The specimens exposed on building roofs (curve 3) indicate that the atmospheres at both locations were comparable, with Newark being marginally more corrosive. The corrosion penetration of the weathering steel specimens mounted on the interior girders of the I-78 bridge in Newark (curves 1 in Fig. 64) was 1.6 times that of their roof counterparts. This increase was caused by sheltering, not salt, because the bridge was not opened to traffic during the first 6.5 years. The corrosion penetration of the weathering steel specimens mounted on the interior girders of the Eight-Mile Road Bridge in Detroit (curves 1 in Fig. 65) was, on average, 4.4 times that of the specimens on the Armory roof. The difference between the two ratios, 1.6 for I-78 and 4.4 for Eight-Mile Road, shows the effect of salt that existed at the latter but not at the former bridge. In fact, at the Eight-Mile Road Bridge, the weathering steels specimens corroded like the carbon steel specimens.

Raska [1983] investigated the corrosiveness of the atmosphere along the Texas Gulf Coast by exposing coupons of A588 Grade B steel at bridge sites located in High Island, Corpus Christi, and Port Isabel. The exposure racks at High Island and Port Isabel were placed on the bridge piers for the main span about 80 ft (24 m) above ground; the rack at Corpus Christi at the south end of the US 81 Nueces Bay Causeway about 8 ft (2.4 m) above water. The High Island Bridge is made of weathering steel; the others are not.

The coupons were tensile tested after weathering. The measured loss of load-carrying capacity, as compared to that of the nonweathered specimens, was converted into an equivalent uniform corrosion penetration. The results are plotted in Figure 66. The corrosion penetration at all three sites exceeded, after less than one year, the 4 mils (100 μm) upper bound of Bethlehem's 18-year scatterband. Salt fog caused the extreme corrosion penetration of these weathering steel specimens that were exposed several miles inland from the coast. Since the specimens were exposed beneath the bridge, on top of the main span pier, it appears that the weathering steel superstructure of the High Island Bridge is also corroding at an excessive rate.

Hein [1981] vertically mounted Resista HRL 37 and A588 Grade A weathering steel specimens on guardrails along West-German highways in Rendsburg, Duisburg, Merklingen, and Trockau. He also exposed control specimens at reference sites near each bridge. The specimens at the highway sites were subjected to deicing salt spray from the roadway, whereas those at the reference sites were not. The data are plotted in Figure 67. The corrosion penetration of the weathering steel specimens at the four highway sites (curves 1 and 2) were, on average, 1.9, 1.3, 2.1 and 1.9 times that of their counterparts at the reference sites (curves 3 and 4).

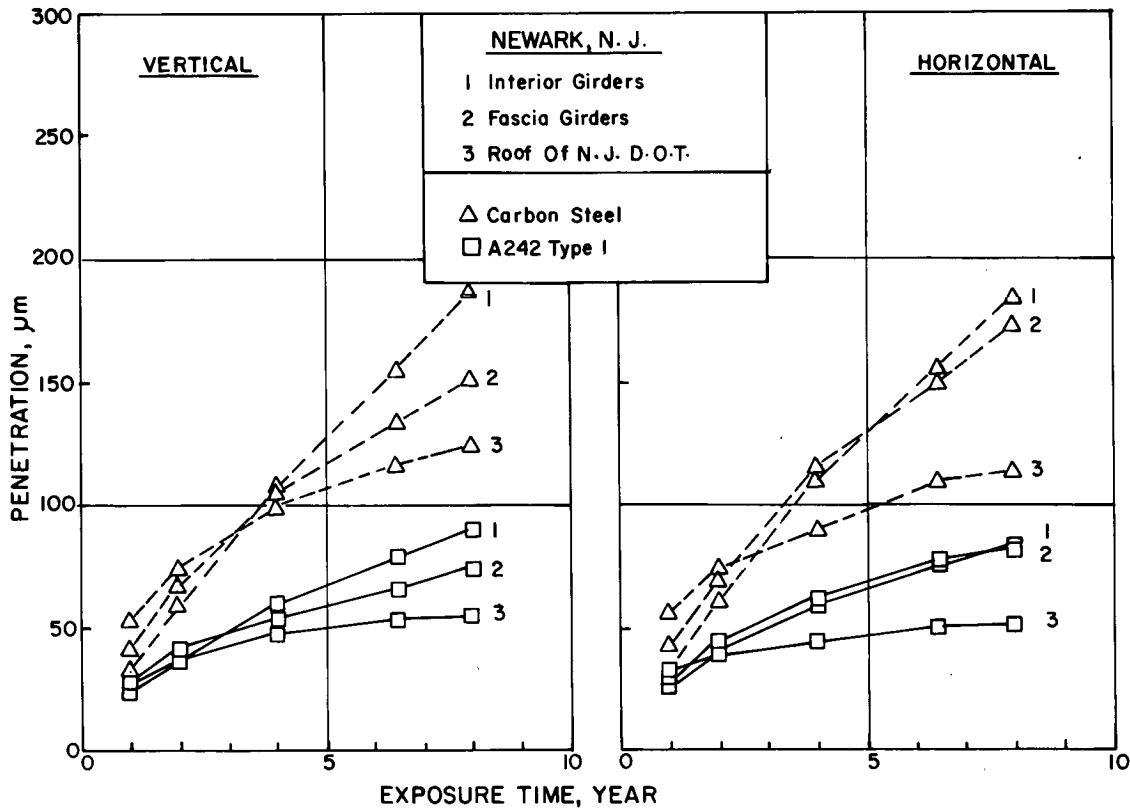


Figure 64. Effect of salt contamination on corrosion of carbon and weathering steels exposed vertically and horizontally to industrial atmosphere of Newark, N.J. [Cosaboom 1979]

The aforementioned Michigan, Texas, and West-German data collected at bridge sites show that the corrosion penetrations exceeded Bethlehem Steel's 18-year scatterband of 2 to 4 mils (50 to 100 μm) for Mayari R steels after 1 to 3 years exposure. The corrosion rates did not level off with exposure time. The weathering steels were unable to develop a protective oxide coating and corroded at the same rate as the carbon steels.

Additional information on the detrimental effect of salt, presented in other sections, is summarized in the following:

1. When the chloride ions present in salt-bearing water come in contact with the steel, they shift the negative potential of the rusted steel to higher negative values. The ensuing increase in the galvanic potential accelerates metal corrosion (Chapter Three, "Effect of Aqueous Environments and Salt").

2. The oxide coating on salt-contaminated steel surface contain large amounts of crystalline nonprotective rust of the form $\beta\text{-FeOOH}$ (Chapter Three, "Effect of Aqueous Environments and Salt").

3. The strong moisture adsorption tendency of salt deposits

lengthens the time of wetness (Chapter Three, "Effect of Aqueous Environments and Salt").

4. The corrosion penetration at marine sites decreases as the distance from the ocean increases and, correspondingly, as the airborne salt content decreases (Chapter Five, "Corrosion Resistance").

5. Discrete salt deposits can cause pitting (Chapter Five, "Pitting").

6. When a differential aeration cell is established in crevices, the chloride ions present in the crevice permit the electrochemical corrosion reaction to continue (Chapter Five, "Crevices").

Deicing salt deposits that remain on weathering steel bridge components for extended periods both inhibit the formation of a protective oxide coating and break down the passive condition that such protective coatings confer. As a result, the corrosion rate is not reduced. Knowing that in the absence of deicing salts weathering steels perform effectively, and in the presence of deicing salts the contaminated areas on a bridge structure perform unsatisfactorily, the following steps should be taken to

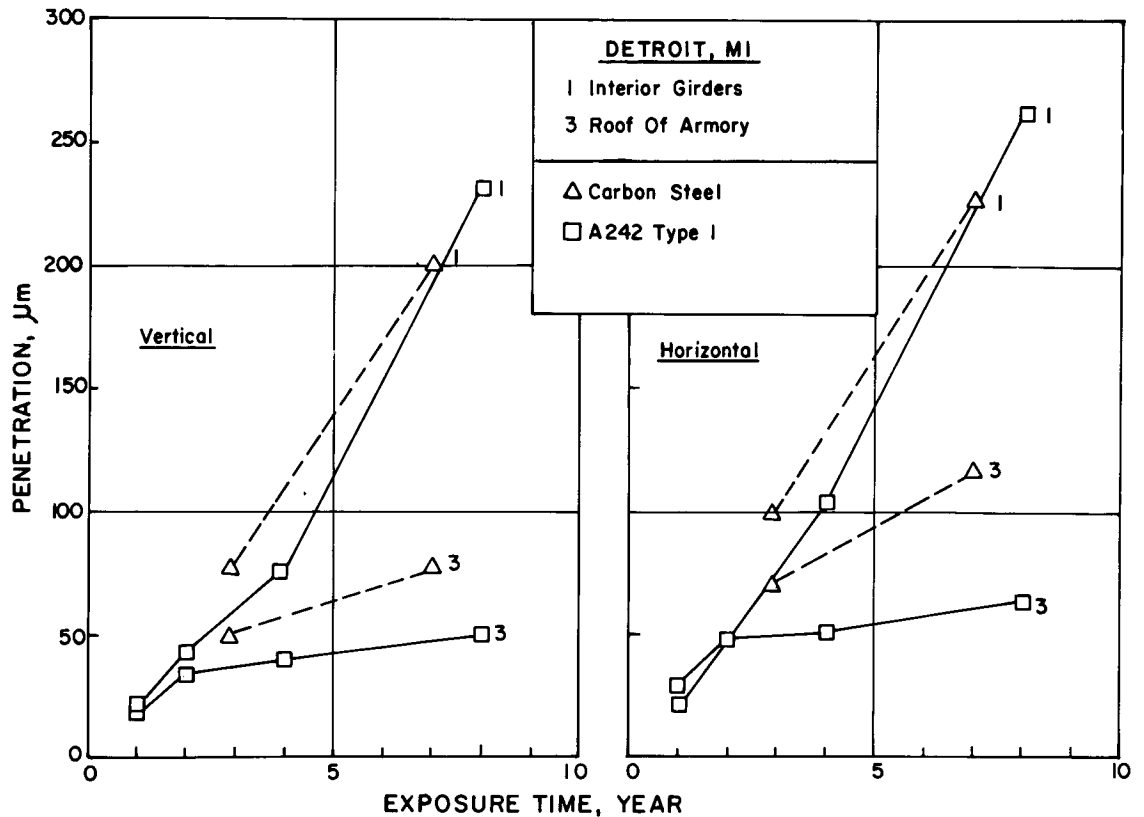


Figure 65. Effect of salt contamination on corrosion of carbon and weathering steels exposed vertically and horizontally to industrial atmosphere of Detroit, Mich. [Zocolla 1976]

ensure adequate corrosion performance of A242 and A588 weathering steels:

1. Salt-contaminated surfaces should be steam-detergent cleaned at the end of the winter season.
2. It is prudent to spot paint areas which, on the basis of experience, are most seriously attacked by corrosion.

By taking these two forms of maintenance action, one can derive the benefits from the use of weathering steels in those portions of the country where unusually hazardous environments influence adversely the performance of a weathering steel bridge.

The environmental hazards are confined to regions of the country where relatively large amounts of deicing salts are used and locations where salt fogs and nightly condensation occur. Where deicing salt is used, it is prudent to paint potentially affected areas. Where salt fogs prevail, it may be prudent to paint areas that are not washed by rain. Coastal areas in the northwest and along the Texas Gulf Coast are especially sus-

ceptible to salt fog. The area where bridge construction is likely should be carefully examined for the performance of steel structures before a decision is made.

DEBRIS

The debris that accumulates on the horizontal surfaces of girder elements consists of road dirt, salt, leaves, rust flakes spalling from the webs, and occasionally bird nests and droppings. Long retaining walls along the shoulders and low clearance over the highway create tunnel-like conditions in an underpass. These conditions intensify the air blast caused by truck traffic and carry more road dirt and salt spray to the steel surfaces of the overhead bridge. Debris accumulates more often towards the supports and builds up against the web at corners formed with stiffeners. By retaining moisture, debris deposits keep chlorides and sulfates in close contact with the steel, greatly prolong the time of wetness, thus inducing, poultice corrosion.

Poultice corrosion feeds on debris from accumulated dirt and

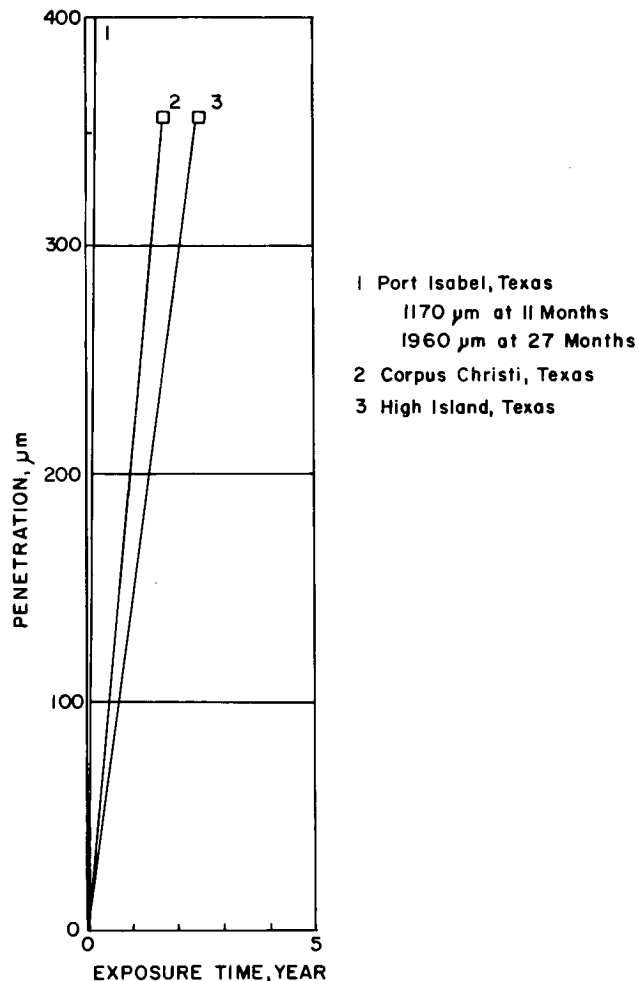


Figure 66. Effect of salt contamination on corrosion of weathering steels exposed to the Texas Gulf Coast atmosphere. [Raska 1983]

corrosion products. For this reason, the debris should be periodically removed from all steel surfaces, preferably once a year. New designs should avoid confined areas (Chapter Eleven).

GALVANIC CORROSION

When two dissimilar metals are in contact in a moist or immersed environment, the difference in solution potential between the two induces a current flow through the solution (electrolyte) from the less noble (anodic) to the more noble (cathodic) metal and back through the metallic contact. Such a galvanic couple causes the anodic metal to corrode more than it would when exposed alone in the same environment, and the cathodic metal to corrode less than when exposed alone.

The degree of corrosion depends partly on the relative position of the two metals to one another in a galvanic series. One such galvanic series is given in Figure 68. It should not be confused with the similar electromotive force series that shows exact potentials based on highly standardized exposure conditions which rarely exist in nature. The galvanic series of Figure 68 is a list of common metals and alloys arranged according to

their tendency to corrode galvanically in seawater flowing at 7.9–13.1 ft/sec (2.4–4.0 m/sec) for 5–15 days at 41–86 F (5–30 C). The farther apart any two metals are in the table, the stronger the metal to the right will corrode. Mild steel (–0.6 to –0.7 V) and low-alloy steel (–0.6 V) have low nobility relative to the other metals and alloys listed in Figure 68. It is possible for certain metals, such as the stainless steels, to reverse their positions in some environments, but the relative positions as given will generally hold in natural waters and in the atmosphere.

Berger [1980] reported that, in practice, mill scale is more noble than steel and is found to be one of the more common causes of pitting. Few data quantitatively document the relative nobility of mill scale. Potential measurements for mill scale in seawater at a velocity of 2 ft/sec (0.6 m/sec) over a period of 135 days showed a fluctuating value of 0 to –0.1 V against a saturated calomel reference electrode [LaQue 1955]. Comparing this value with the potentials listed in Figure 68 indicates that mill scale is 0.6 V more noble than mild steel and 0.55 V more noble than low-alloy steel. Thus, in saltwater immersion, mill scale can cause galvanic corrosion and pitting at breaks in the mill scale [Coburn 1974]. Under conditions of alternate wetting and drying, the mill scale slowly disappears due to undercutting of the scale in the cracked areas. To avoid deep pitting in locations that are continuously damp, the mill scale should be removed before the particular component of a structure is placed in service.

The area ratio of two galvanically coupled metals under immersed conditions affects the degree of corrosion of both metals. Phelps and Schmitt measured the relative corrosion rates of carbon, A242, and stainless steels when tested alone or coupled to each other [Phelps 1970]. The couples were immersed in standing seawater for 6 months. The area ratios of the specimens were 1:8, 1:1, and 8:1. The corrosion penetration of A242 steel alone was 4.1 mils (105 µm), and of carbon steel alone 5.0 mils (128 µm), as shown in Figure 69. By coupling the steels, the corrosion penetration decreased for the cathodic A242 steel and increased for the anodic carbon steel. Most notably, the corrosion of carbon steel tripled when its area was one-eighth that of the A242 steel. Low carbon-to-A242 steel area ratios can occur in practice when bolts and welding rods are less noble than the high-strength low-alloy steel. Under arid conditions, no harm will occur. However, under persistently damp conditions, the less noble member of the couple will corrode at an accelerated rate in the contact areas.

Bolts joining weathering steel members should have a composition similar to the weathering steel, because carbon steel bolts would corrode at a more rapid rate in an aggressive environment. An example of such behavior was found in a bolted A242 Type 1 steel angle specimen that was weathered for 13 years in marine Point Reyes, California [Reed 1982]. In this test, 60 percent of a low-carbon steel nut corroded away.

Similarly, to avoid galvanic corrosion in welds, the rod compositions should be selected on the basis of their being noble to the base steel. Single pass welds may absorb enough alloying elements from the weathering steel base to preclude galvanic corrosion. But the external or final passes of multipass welds should have a composition similar to the steel base, when the thickness exceeds 0.25 in.

In the Phelps-Schmitt experiments, when coupled to Type 410 stainless steel, both carbon steel and A242 steel became the

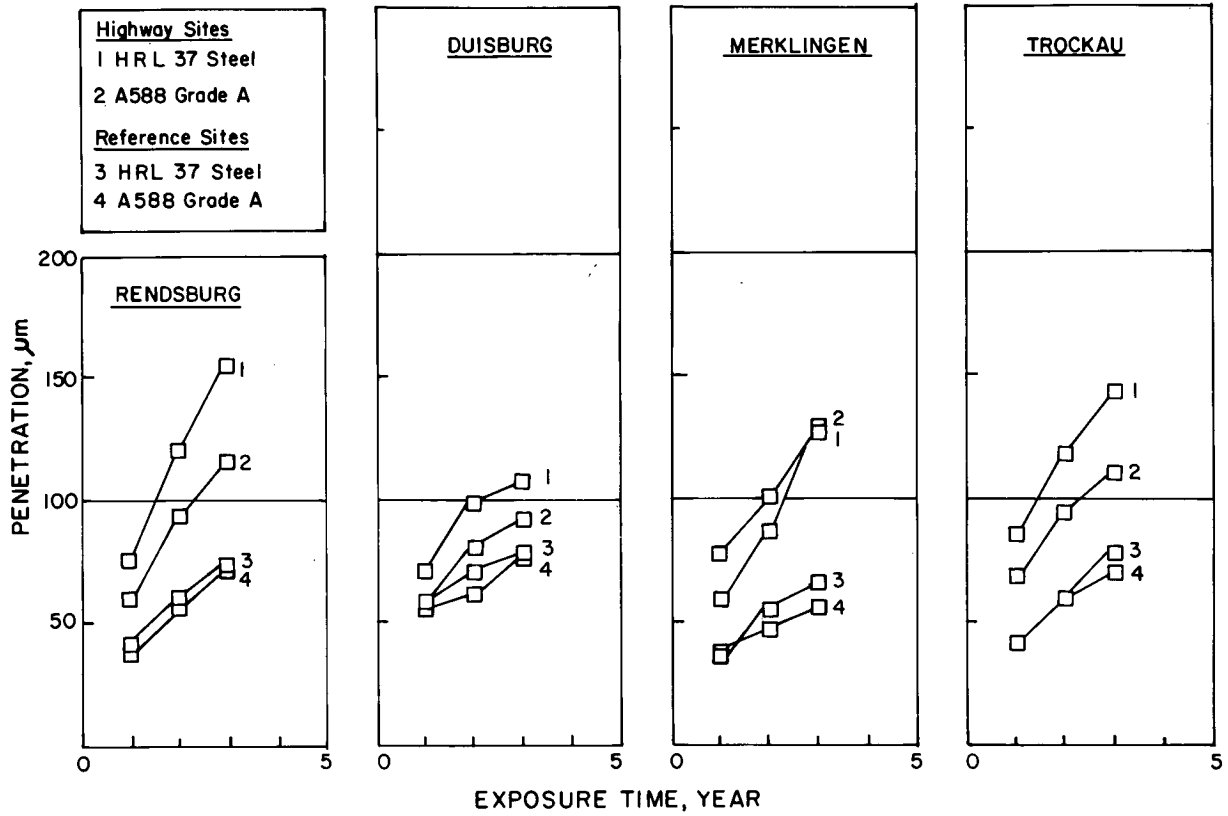
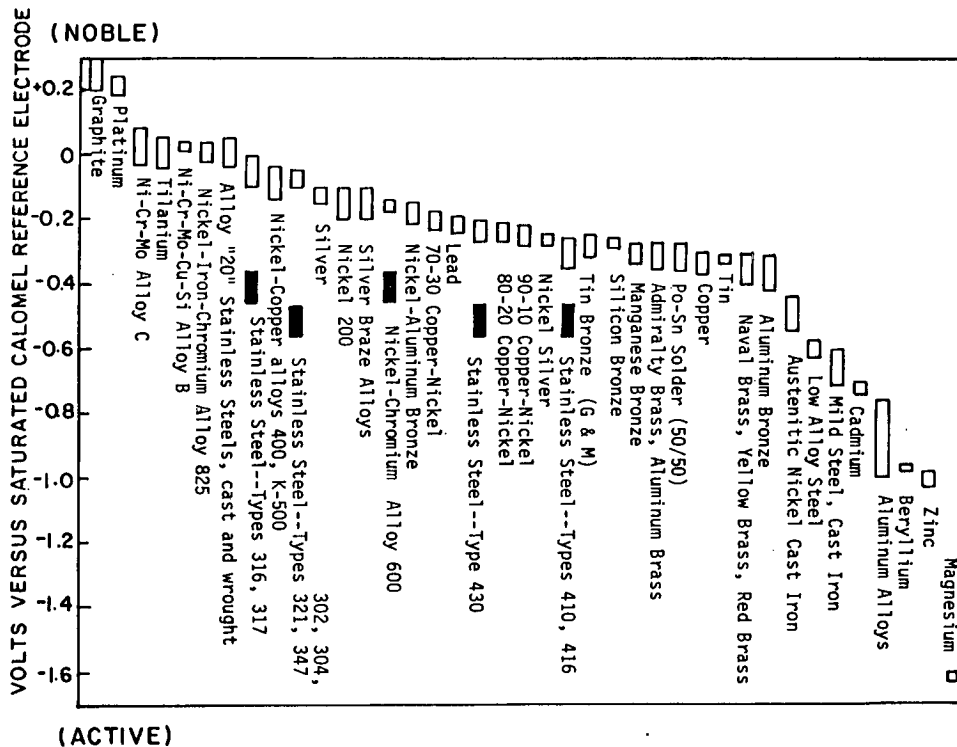
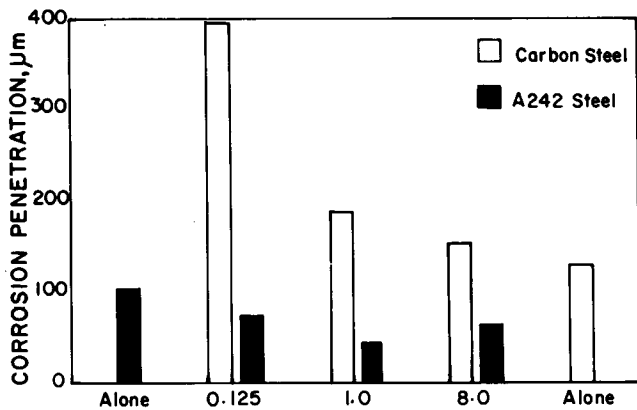


Figure 67. Effect of salt contamination on corrosion of weathering steel specimens exposed vertically to various atmospheres in F.R.G. [Hein 1981]





CARBON / A242 STEELS AREA RATIO

Figure 69. Corrosion penetration of uncoupled and coupled steels after 6 months in seawater. [Phelps 1969]

sacrificially corroding (anodic) member of their respective couples, regardless of area ratio.

The galvanic couple created by the bronze washers in the expansion joints of some Michigan bridges contributed to the corrosion of the web and link plates [Culp 1980]. The main factor, as discussed under "Crevice," was crevice corrosion. The severe corrosion, including as much as 0.25 in. (6 mm) deep pitting, restricted the movement of the expansion joint and required replacement of the link plates and pins.

The phenomenon of galvanic coupling is also relevant to crevice corrosion, as discussed under "Crevice," later in this chapter.

PITTING

Pitting is a complex corrosion phenomenon. It is the initial form in which all forms of corrosion manifest themselves. If one visualizes the surface of a carbon steel structure to be in the form of a grid, such as a sheet of graph paper with an infinite number of small containers filled with a saltwater electrolyte, then corrosion begins simultaneously in random locations in numerous but not all of the minute containers. The steel in the adjacent containers will not corrode for some time because they are galvanically protected by being coupled to the adjacent corroding areas. The result is called general corrosion. The identical action occurs with the weathering steels. It is stimulated by the physical and chemical heterogeneities present on the surface.

The pitting is generally of a shallow nature, with the weathering steels showing the least signs of deep pitting. Mill scale tends to generate somewhat deeper pits along cracks in the scale, because the relatively more noble potential of the scale, compared to the base steel, forms a strong galvanic cell. This condition is not too serious since it provokes additional fracturing and eventual spalling of the mill scale. As an example of another metal, pits form in stainless steels much less frequently. However, the individual pit can perforate the steel in a relatively short time.

In all instances, the chloride ions from deicing salt or seawater spray are the primary stimulants that accelerate pitting. Differential aeration cells, resulting from scattered deposits of damp dirt, leaves, twigs and oil, likewise stimulate pitting of the surface of all steels, from carbon through stainless. In view of the foregoing, it is evident that:

1. Bridges covered with mill scale may develop numerous pits at cracks in the scale when these cracks are covered with dirt, leaves, etc. and are damp for relatively long periods of time.
2. Bridges that are free of mill scale which, because of their geometry, trap and retain drainage, debris, windblown dust, dirt, deicing salts and maintain a damp condition can develop deep pits beneath the deposits.

As part of the corrosion study cited in Chapter Five, under "Industrial Environments" and "Marine Environments," Copson measured the average depth of the four deepest pits on the skyward and groundward surfaces of many steel specimens [Copson 1960]. Figure 70 shows the pitting and uniform corrosion of steels with chemical compositions typical of copper-bearing and weathering steels, as a function of time of exposure in industrial Bayonne, N.J., and marine Block Island, R.I. Pitting and uniform corrosion at the marine site were more severe than at the industrial site. For all steels tested by Copson, pit depths after 17 years were two to three times the average penetration calculated from weight loss. The pitting factor (ratio of pit depths to average corrosion penetration) for the low-alloy steels plotted in Figure 70 was about three. Pit depths tended to increase with time in a manner similar to that of average corrosion penetration.

The Copson data and the experience with the Michigan bridges and the Doullut Canal Bridge on the Mississippi Delta, Louisiana, indicate that salt contamination stimulates conditions responsible for both pitting and general corrosion. [Copson 1960, Culp 1980, Arnold 1981, Gallagher 1982].

The effect of pitting on fatigue life is discussed in Chapter Eight, under "Application to Current Designs."

CREVICES

Crevice or joint corrosion is a form of localized attack of a metal surface at, or immediately adjacent to, an area shielded from bold exposure to the environment. Bolted connections, cantilever expansion joints, and lap joints not welded all around are examples of details susceptible to crevice corrosion.

The mechanism of crevice corrosion involves electrochemical reactions that take place between the surfaces within the crevice, or between the surfaces of the crevice and those freely exposed to the environment outside the crevice [Ellis 1951]. Crevice corrosion of weathering steels occurs in two stages. At the initial stage, when the protective oxide coating has not yet formed on freely exposed surfaces, a differential aeration cell is established between the steel within the crevice, which is shielded from access to the dissolved oxygen in the electrolyte, and the steel outside the crevice, which is freely exposed to the oxygen-bearing electrolyte. Lacking a supply of oxygen, the steel inside the crevice becomes anodic and corrodes.

After several years exposure, the protective oxide film covers

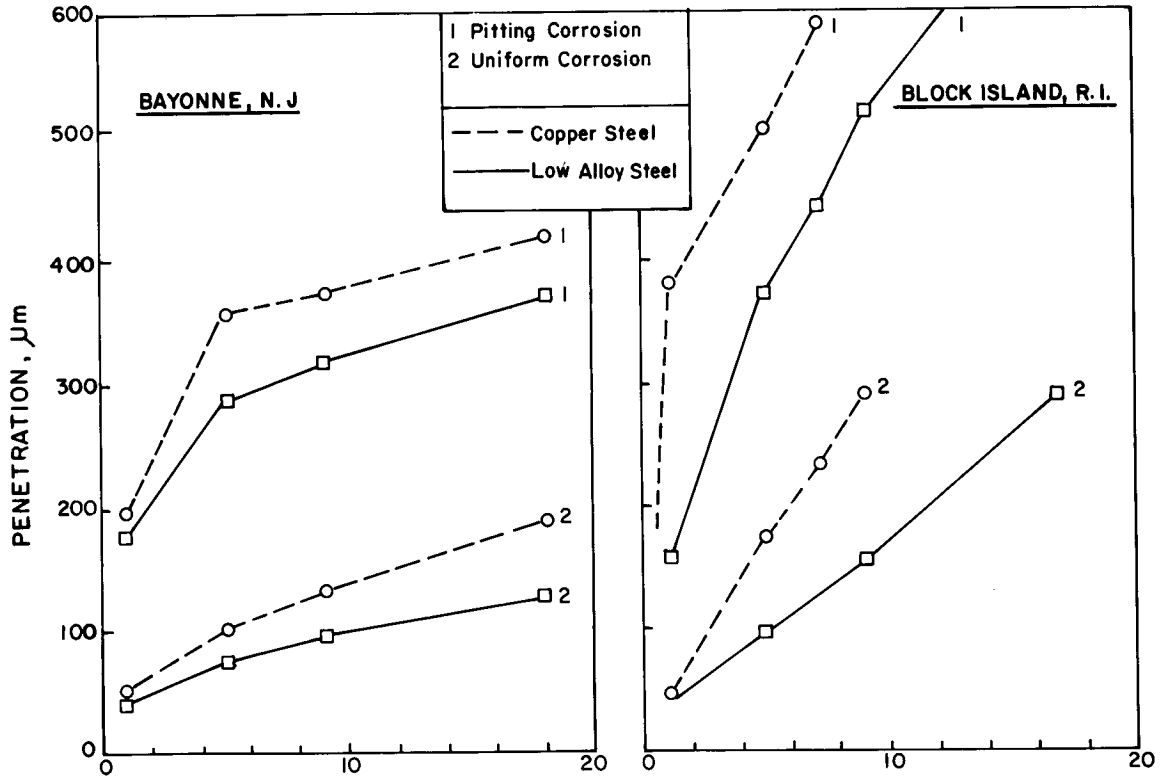


Figure 70. Comparison of pitting and uniform corrosion of copper steel and low-alloy steel exposed in industrial Bayonne, N.J., and marine Block Island, R.I. [Copson 1960]

the freely exposed surfaces, but not the steel within the crevice. At this stage, in addition to the differential aeration cell, a galvanic cell is established between the active steel within the crevice and the passive steel outside the crevice. The difference in potential between the active and passive states of the steel becomes an additional driving force for galvanic corrosion of the steel within the crevice.

In the presence of salts and to a lesser degree various corrosion products, the differential aeration cell electrochemically transports the salts' anions, particularly chlorides, into the crevice. Simultaneously, anodically dissolved iron ions and salt anions form acidic salts. This, in turn, lowers the pH value of the electrolyte within the crevice. The combination of high concentrations of anions, low pH, and low oxygen concentration causes high corrosion rates and promotes pitting.

The following two cases illustrate the deleterious effect of deicing salts on corrosion of steel within crevices. Visual inspection of a disassembled link plate in an expansion joint removed from a weathering steel bridge in Detroit, Michigan, revealed severe crevice corrosion between the link plate and the web. It resulted in tight rust packing of the gap between the two elements [Culp 1980], prevented movement of the expansion

joint and caused other structural damage to the bridge. After sandblasting, it became apparent that the web behind the link plate was more severely corroded in an area around the bronze washer outline than in contact with the washer. This points to crevice corrosion as the dominant corrosion mechanism [Fig. 13, Culp 1980].

Among the oldest weathering steel structures in Michigan is a test stretch of guardrail installed in February 1963 along a highway in Lansing, Michigan [McCrum 1980]. An examination of the guardrail in 1978 revealed bulging lapped joints expanding outward from the internal pressure of the corrosion products. This condition has also been termed "packout." Ultrasonic thickness measurements indicated a 40 percent section loss of the guardrail in the lapped areas. The average effective corrosion rate was 1.4 mils/year/surface (35 µm/year/surface). That is almost three times the corrosion rate of the freely exposed guardrail surfaces. Some areas of several square inches exhibited six to seven times the corrosion rate of the freely exposed surfaces, or about 3 mils/year/side (75 µm/year/side).

Adequately tightened and stiff high-strength bolted joints should not normally be prone to crevice corrosion, as is discussed in Chapter Ten.

STRENGTHENING MECHANISMS AND TOUGHNESS

INTRODUCTION

Weathering steels for bridges are supplied to ASTM Specifications A242, A588, and A709 which set minimum requirements for tensile properties, that is yield point, tensile strength, and elongation.

Different manufacturers have developed their own compositions for meeting the specification requirements, and this is reflected in the different grades that are available in A588 and in the different compositions actually used to meet A242 and A709. The weathering HSLA steels A242 and A588 generally have a minimum yield point of about 50 ksi (345 MPa) and are usually supplied in the as-hot-rolled condition. The weathering alloy steels A709 Grade 100W have a minimum yield strength of 100 ksi (690 MPa). They are quenched and tempered alloy steels, essentially the same as the A514 steel with its numerous grades.

All of the above-named steels are of structural quality. They are frequently not supplied to any notch toughness requirements. On Federal-aid highway programs in the United States, however, Charpy V-notch toughness requirements have been adopted for primary tension-loaded members of bridges, as listed in specification A709. Thus, when agreed on by the manufacturer and the purchaser, CVN impact tests may be included for bridge steels as a supplementary requirement, with the tests conducted in accordance with the requirements of specification A673.

The imposition of notch toughness requirements may well necessitate making some of the steels with slightly different compositions and deoxidation practices as well as the addition of heat-treatment.

To assess the performance of weathering steel, the designer should have some knowledge of the strengthening mechanisms that the suppliers use. Fletcher and Porter, in reviewing the evolution and physical metallurgy of high-strength low-alloy steels, discussed the various strengthening mechanisms commonly used in practice [Fletcher 1979; Porter 1982]. The mechanisms that apply to weathering steels are summarized in the following sections.

STRENGTHENING MECHANISMS

High strength in bridge steels can be achieved with several strengthening mechanisms. Steel manufacturers may choose a combination of these mechanisms to produce a weathering steel with desirable yield and tensile strength as well as toughness.

Grain Refinement. One way by which steels can be strengthened is to decrease the grain size; in other words, to increase grain-boundary area per unit volume. Indeed, the lower yield

point is a linear function of $d^{-1/2}$, where d is the mean grain diameter [Petch 1959].

Ferritic-pearlitic low-carbon steels, such as the A588 steels, can be grain refined by: (1) rolling at reduced temperature, or controlled rolling; (2) alloying; and (3) normalizing.

Controlled rolling involves large thickness reductions at the lower hot-rolling temperatures and may involve accelerated cooling rates. This produces fine austenite grains that do not have sufficient time to recrystallize at the low temperatures before austenite transforms to ferrite upon cooling. The resulting ferrite grains are then relatively fine. Grain size is increasingly refined as rolling temperature is lowered and reduction at the lower temperatures is increased while still staying above the transformation temperature. Only light-gage product is usually control-rolled.

Certain alloying elements help to refine grain size. Their main function is to form precipitating compounds in austenite (aluminum), and to retard the growth of recrystallized austenite grains during further hot rolling (aluminum, manganese, chromium, molybdenum and nickel). Maintaining a fine austenite grain size results in fine-grain ferrite on subsequent transformation.

Normalizing is a process in which an alloy steel is heated to a suitable temperature above the transformation temperature range and subsequently cooled in still air at room temperature. The presence of aluminum, columbium, or vanadium inhibits coarsening of the austenite grains of the reheated steel and results in fine pearlite formation and grain refinement. This, in turn, promotes toughness and lowers the impact transition temperature. Normalizing only refines the grain if the proper grain-refining elements are present.

The A588 steels are all made by fine-grain practice but are usually supplied as rolled. Even on fine-grain-practice steels like A588, the grain size of hot-rolled product will be much the same as that of semikilled steel. The fine-grain practice itself, however, does impart better notch toughness.

In hot-rolled steels, the grain refinement method of strengthening is unique in that it concurrently improves impact properties. For example, Pickering and Gladman found that the yield and tensile strength of hot-rolled, low-carbon steels with ferritic-pearlitic structure increased and their fracture-appearance transition temperature decreased with grain size. The relationship between the aforementioned mechanical properties and $d^{-1/2}$, is linear [Pickering 1963].

Solid-Solution Strengthening. Solid solution of any alloying element in a metal strengthens the solvent metal. The differences in atom size and electronic structure between the solute alloying element and the solvent metal determines how much strengthening is obtained. In dilute solid solutions with up to 1 percent of any common solute and up to 1.8 percent manganese, the amount of strengthening is about proportional to the concen-

tration of the alloying element. If more than one solute element is present, the total strengthening is about the sum of the effects of each alloying element separately. The alloying elements of steels are either small enough to fit between the iron atoms in the crystal lattice, or they are of a size so that they can substitute for some of the iron atoms. They are termed interstitial and substitutional alloying elements, respectively. Carbon and nitrogen are examples of interstitial alloying elements. Other solid-solution elements, such as phosphorus, silicon, copper, manganese, molybdenum, and nickel are substitutional. Vanadium and columbium, on the other hand, are not solid-solution elements. They strengthen the steel mainly by precipitation.

To achieve a high-degree of solid-solution strengthening, a large difference between the atomic sizes of the solute and the solvent is required, so that the dislocations of the solvent metal are locked by its solute atmosphere. For this reason, elements like carbon, nitrogen, and phosphorus, which are relatively small compared with iron, increase the strength most, whereas nickel and chromium, which are comparable in size with iron, do not increase the strength. Figure 71 shows the effect of common alloying elements on the lower yield point and the tensile strength of weathering steels. Apparently, chromium weakens the steel because it removes some carbon and nitrogen from solid solution.

Solid-solution strengthening can be detrimental to impact properties. Additions of most alloying elements, especially interstitials as well as phosphorus, raise the impact transition temperature. Manganese increases strength as well as notch toughness and is inexpensive. For this reason, just about all the HSLA steels contain a high manganese content. Nickel improves toughness, but it has no effect on yield point and only a minor effect on tensile strength.

The A588 and A709 specifications set both lower and upper limits on alloying element content. The lower limits are meant

to ensure good corrosion resistance, and minimum specified lower yield point and tensile strength. More often than not, the upper limits provide simply suitable melting ranges for the elements or help to provide the specified strength in the heavier thicknesses. Transition temperature is sometimes a consideration in setting upper limits on alloying element content.

Precipitation Strengthening. Reactions leading to precipitation strengthening markedly improve the lower yield point of the steel beyond what might be obtained by solid-solution strengthening.

A number of alloying elements precipitate from supersaturated solid solution upon cooling. The microalloying elements, vanadium, columbium, titanium, and aluminum precipitate as nitrides, carbides, and carbonitrides. Copper precipitates by itself. The degree of strengthening depends primarily on volume fraction and also on the size and spacing of the precipitates. Aging time and temperature markedly affect the size and spacing of the precipitate particles, because they relate to the degree of supersaturation and the volume fraction of the precipitate particles. The necessary condition for precipitation strengthening in an alloy is a sloping solvus line in the phase diagram, so that there is appreciable solubility of the precipitating phase at elevated temperatures and a marked reduction in solubility at lower temperatures. Columbium and vanadium are the two most useful alloying elements for this purpose. They are used for strengthening many HSLA steels.

Vanadium causes marked precipitation strengthening and usually some grain refinement on normalizing, even in thick plates, because all of the vanadium is in solution at usual normalizing temperatures. In concentrations up to 0.10 percent, vanadium seems to have negligible deleterious effect on toughness. Also, in semikilled steels, vanadium combines with part of the nitrogen, thereby reducing the free nitrogen content and further lowering the impact transition temperature.

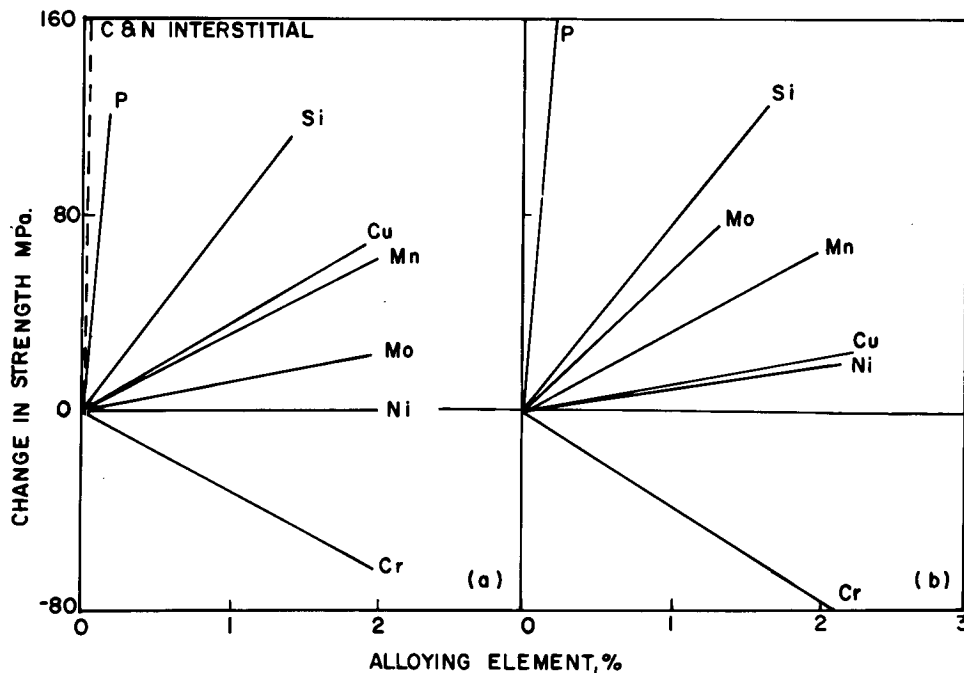


Figure 71. Effect of solid solution alloying elements on change in (a) lower yield point, and (b) ultimate tensile strength. [Pickering 1976]

Columbium provides marked precipitation strengthening in as-rolled steels. Its use in as-rolled steels, however, is limited to $\frac{3}{4}$ in. (19-mm) thick product (ASTM Specification A572), unless the steel is killed. In this case, there is no specified maximum thickness beyond those of the A572 specification.

When the steel is normalized at the usual austenizing temperature, columbium refines the grain, thereby improving toughness and lowering the impact-transition temperature. In this case, columbium does not appreciably strengthen the steel through precipitation.

To improve both strength and toughness of columbium-bearing steels, controlled-rolling would be required. Indeed, some weathering steels may be control-rolled to meet AASHTO notch toughness requirements.

Quenching and Tempering. Quenching and tempering is a heat treatment of steel that increases strength, hardness, and toughness by virtue of rapid cooling (quenching) from above the transformation range. Quenching produces martensitic or lower bainitic microstructures which is more desirable for strength and toughness than the ferritic-pearlitic microstructure.

After quenching, the steel is in a highly stressed condition. To avoid or minimize problems of cracking or distortion, the steel is then heated to an elevated temperature and air cooled, a process called tempering. The best combination of strength and toughness is usually obtained by suitably tempering a quenched microstructure consisting of a minimum of 80 percent martensite throughout the cross-section.

The A709 Grade 100W weathering steels are quenched and tempered.

ALLOYING ELEMENTS

Weathering steels may contain up to 15 different alloying elements in various combinations and proportions. It was previously noted that the presence of some of these elements in natural iron ore (i.e., sulfur) or their addition to steel (i.e., copper) may influence strength and toughness in addition to atmospheric corrosion resistance. Some elements are used because of the specific properties they impart to steel. Others rid the steel of impurities or render the impurities harmless. A third group counteracts harmful oxides or gases in the steel. The elements of this latter group do not remain in the steel to any great extent after it solidifies. Some elements fall into more than one of the aforementioned groups. The following discussion describes the effect of the more common alloying elements on strength and toughness.

Aluminum. Aluminum is generally used as a deoxidizer; usually along with silicon, when fine-grain-practice killed steel is specified. It does not have much, if any, effect on grain size in the as-rolled condition, but it does improve notch toughness. When such is normalized, however, the grain size decreases substantially and the transition temperature is lowered. Aluminum in concentrations greater than 0.1 percent is a solid-solution strengthener. However, because steels containing aluminum tend to have poor surfaces, this element is not used for solid-solution strengthening.

Carbon. The simplest and cheapest way to increase the tensile strength of steels is to raise the carbon content so as to provide a greater percentage of pearlite in the microstructure. The addition of carbon does not markedly increase the yield point, and impairs ductility, toughness, and weldability. Therefore, the car-

bon content of HSLA steels is generally kept below some maximum value that depends on the overall chemical composition and the intended application. The carbon content of A588 steels suitable for use in welded bridges is limited to about 0.20 percent maximum.

Chromium. Chromium can potentially increase strength by solid solution and by reducing the interlamellar spacing of pearlite. On the other hand, it removes carbon and nitrogen from solid solution. The net effect is that chromium weakens the steel, as shown in Figure 71.

Columbium. Columbium increases the yield point and the tensile strength of as-rolled steels by precipitation strengthening and, to a lesser degree, by solid solution. It also refines the grain size when the steel is control-rolled. Another advantage of alloying with columbium is the element's low affinity for oxygen, which maximizes the production yield of semikilled steels from the ingot. The impact transition temperature of conventionally rolled HSLA steels containing columbium is high, because most of the strength is achieved through precipitation. Hence, if low impact transition temperature is required, steels containing columbium are usually normalized or control-rolled.

Copper. Copper increases the yield point and the tensile strength primarily through solid solution. The effect of copper as a solid-solution strengthener is shown in Figure 71. To achieve precipitation strengthening, the steel has to be alloyed with at least 1.3 percent copper and also age hardened. Weathering steels contain much less copper and are, therefore, not strengthened by precipitation. Weldability and toughness are not affected by copper contents below 0.75 percent. Steels containing more than 0.50 percent copper frequently exhibit "hot shortness" during hot working, so that cracks or extremely roughened surfaces may develop during hot working. Adding nickel alleviates this problem. Copper, especially in amounts up to 0.25 percent, is the element most beneficial in improving atmospheric corrosion resistance.

Manganese. The chemical interaction of manganese with sulfur and oxygen makes it possible to roll hot steel. Manganese increases the yield point and tensile strength by substitutional solid solution, as shown in Figure 71, and improves toughness. In addition, it is a strong hardener, which is important to quenched and tempered steels. Higher ratios of manganese to carbon content prevent the formation of acicular transformation products, which are harmful to ductility and toughness. Furthermore, by refining the grain size, manganese compensates in part for the loss of ductility and toughness resulting from the addition of other alloying elements. To ensure good weldability, the manganese content should be kept below some maximum value that depends mainly on the content of carbon and other alloying elements. This is also discussed in Chapter Ten. The lower limits of manganese content specified for the various grades of A588 steel relate primarily to strength and notch toughness, the upper limits often relate to weldability.

Molybdenum. Molybdenum is an effective substitutional solid-solution strengthener. However, it reduces the effectiveness of columbium as a precipitation strengthener. High molybdenum contents harden the heat affected zone (HAZ), which is a disadvantage in welded steel construction. In as-rolled steel and possibly in normalized steel, molybdenum tends to impair toughness, even when present in relatively small amounts. It is a nonoxidizing element, making it highly useful in the melting of steels, such as quenched and tempered steels, whose hardness must be closely controlled.

Nitrogen. Free nitrogen, that is nitrogen in interstitial solid solution and not combined as a nitride, strongly strengthens the steel. But, it has the disadvantage of increasing the impact transition temperature by 50 F (28 C) for each 2.0 ksi (14 MPa) increase in yield strength [Fletcher 1979].

Nickel. Nickel marginally increases the tensile strength and has no effect on the yield point, as shown in Figure 71. Substantial amounts of nickel effectively lower the impact transition temperature. Nickel in an amount equal to at least one-half the copper content is very beneficial in preventing the hot shortness of steels containing about 0.50 percent or more copper.

Phosphorus. Phosphorus is a strong substitutional solid solution strengthener (see Fig. 71). Because it markedly increases the impact transition temperature, weathering steels used for bridges have a maximum phosphorus content of 0.04 percent. The mill scale on a copper-phosphorus steel is shed more quickly than on carbon steel [Chandler 1970].

Sulfur. Although sulfur markedly strengthens the steel by substitutional solid solution, it is very detrimental to toughness. Elongated stringers or ribbons of manganese sulphide, especially in more or less coplanar aggregates, cause anisotropy with regard to ductility and toughness. Reduction of sulfur content to very low values and sulfide inclusion-shape control helps to alleviate this anisotropy. With inclusion-shape control, a process that reduces the plasticity of inclusions by addition of titanium, zirconium, cerium or rare earth mixtures, or calcium, the inclusions remain globular or equiaxial during hot-rolling. Reduction of sulfur content lowers the impact transition temperature. Adding elements that globularize the sulfides, such as rare-earth metals or calcium, raises the Charpy V-notch shelf energy, particularly in the transverse direction where the CVN shelf energy may be low due to straightaway rolling. This elevation is reflected along the whole CVN curve, but to a lesser extent at the bottom of the curve.

Silicon. Silicon is commonly added to steel as a deoxidizer to produce killed steel. It also increases the yield point and tensile strength by substitutional solid solution. Silicon in amounts up to 0.30 percent lowers the impact transition temperature. Higher amounts reduce ductility and weldability. The upper limit on the range of silicon content in A588 steel varies from 0.30 percent to 0.90 percent. There appears to be a trade-off in the use of silicon as an economically attractive strengthener and its harmful effect on toughness, ductility, and weldability.

Titanium. In the past, titanium was added to HSLA steels primarily to refine the grain size and to improve sulfide morphology. More recently, it is being added to prevent grain coarsening of the HAZ and, hence, to ensure good toughness. Titanium reacts with oxygen, nitrogen, carbon, and sulfur in the steel, forming a number of compounds, including oxides, nitrides, carbides, and sulfides. Some of these compounds have desirable effects, but the presence of others is undesirable. The formation of titanium oxide is undesirable because it entails a loss of titanium which then is unavailable for grain refinement, precipitation strengthening, or shape control of sulfide inclusions. Hence, the molten steel must be thoroughly deoxidized before titanium is added.

Vanadium. Vanadium is an effective precipitation strengthener. In contrast to columbium, it forms precipitation phases in both as-rolled and normalized steels. A content of 0.08 percent vanadium strengthens steel about as much as 0.02 percent columbium. The combination of columbium and vanadium gives a higher yield strength in the conventionally hot-rolled condition than can be obtained with either element alone.

TOUGHNESS

Toughness of a steel can be defined as the ability to absorb energy by undergoing plastic deformation prior to fracture. The measure of this ability at a high rate of loading is called impact strength. Notches or stress concentrations are present in nearly all structures. If a steel is to resist fracture under impact loading, it must be able to absorb considerable energy or show notch toughness whenever stress concentrations occur. A component can fracture at points of stress concentration, even though the nominal stress is below design values, when the steel is subjected to static or impact loads at comparatively low temperatures.

Body-centered-cubic (BCC) or ferritic alloys exhibit a significant change in toughness over a range of temperatures. Figure 72 shows qualitatively the effect of temperature on toughness. At elevated temperatures, notched specimens fracture by a shear mechanism absorbing large amounts of energy. This is associated with a ductile, fibrous appearing fracture with plastic deformation. At low temperatures, notched specimens fracture by a cleavage mechanism absorbing little energy. This is associated with a brittle, granular-appearing fracture with little or no plastic deformation. At intermediate temperatures, steels exhibit a ductile-to-brittle fracture transition.

Fracture toughness testing to ASTM E399 specification is very time consuming and costly. For this reason, the impact Charpy V-notch test is used as an inexpensive and rapid way of characterizing the toughness of structural steels. Fortunately, because the curves for CVN energy-absorption and plane-strain fracture toughness, K_{Ic} , have a transition in the same temperature zone, the value of the latter in the transition region can be estimated from [AASHTO 1978]:

$$\frac{K_{Ic}^2}{E} = 5 \text{ (CVN)}$$

AASHTO has minimum CVN toughness requirements for fracture critical members at the minimum service temperatures corresponding to Zones 1 to 3. Table 37 summarizes these requirements for A588 steels. The requirements for A514 steel should also apply to A709 Grade 100 W weathering steel.

The toughness and temperature at which the transition from ductile to brittle fracture occurs can greatly vary from one steel to another. Good impact strength is promoted by: low carbon, phosphorus, and sulfur contents; high manganese-to-carbon ratio; small grain size; reduced section thickness; normalizing; and

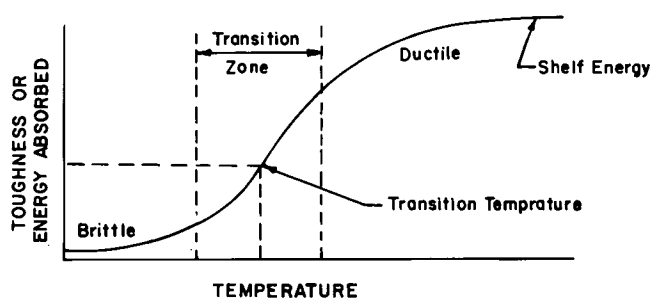


Figure 72. Effect of temperature on toughness or Charpy V-notch energy absorbed.

Table 37. Base metal Charpy V-notch requirements for fracture critical members of A588 steel. [AASHTO 1978]

Thickness	Zone 1 -17.8°C (0.°F)	Zone 2 -18.3°C to -34.4°C (-1°F to -30°F)	Zone 3 -35°C to -51.1°C (-31°F to -60°F)
Up to 102 mm (4 in.) mechanically fastened or up to 51 mm (2 in.) welded	33.9 J at 21.1°C (25 ft-lb at 70°F)	33.9 J at 44°C (25 ft-lb at 40°F)	33.9 J at -122.2°C (25 ft-lb at 10°F)
Over 51 mm (2 in.) to 102 mm (4 in.) welded	40.7 J at 21.1°C (30 ft-lb at 70°F)	40.7 J at 4.4°C (30 ft-lb at 40°F)	40.7 J at -12.2°C (30 ft-lb at 10°F)

quenching and tempering. It is, therefore, possible to equate a given type and grade of steel with a range of toughness, but not with a specific value of toughness. As an example, Figure 73 shows CVN toughness versus temperature curves for specimens cut from A588 steel plates of various thicknesses. It illustrates the large differences in transition temperature that may occur, and the need for measuring CVN toughness, to ensure that the steel is suitable for the intended application.

Figures 74 and 75 summarize the plane-strain fracture toughness data for 1-sec loading, K_{Ic} , and dynamic (about 1 msec) loading, K_{Ia} , that Crosley and Rippling are collecting for an ongoing FHWA study on fracture of bridge steels [Crosley 1983]. The data in Figures 74 and 75 are for specimens that met the ASTM E399 size requirements for plane-strain fracture toughness. Additional data for specimens that had higher toughness and did not meet the size requirements were not plotted. The 1-sec rate of loading is typical of that occurring in bridges. A preliminary analysis of all data collected by Crosley and Rippling appears to confirm that a correlation exists between the Charpy and dynamic fracture toughness measured at the transition temperature. The transition for 1-sec loading occurs at lower temperatures.

Roberts, et al., reported that A588 steels exhibit large variability in fracture toughness and a marginal temperature shift [Roberts 1974, Roberts 1977]. With regard to the fracture response of a welded bridge member, they found that the largest tolerable crack at the lowest service temperature, as provided for by the AASHTO toughness requirements, will generally be of the order of the flange plate thickness. Thus, one can conclude from these results that the specifications provide welded details that will nearly exhaust all fatigue life before a fatigue crack will become unstable at the lowest anticipated service temperature. This conclusion appears to be valid for bridge steels in general, excluding steels that are electro-slag welded.

In recent years, the ductility and the Charpy shelf energy values of HSLA steels in the transverse direction, and particularly in the through-thickness direction, have received much attention. This resulted in part from lamellar tearing encountered in welding and cracking or splitting (delamination) that accompanied certain bending operations in hot-rolled, flat-rolled products [Takeshi 1975, Heroe 1975]. The total ductility at fracture and the Charpy V-notch shelf energy of HSLA steels are reduced markedly by an increase in the volume fraction of nonmetallic inclusions, by greater elongation, and by decreasing

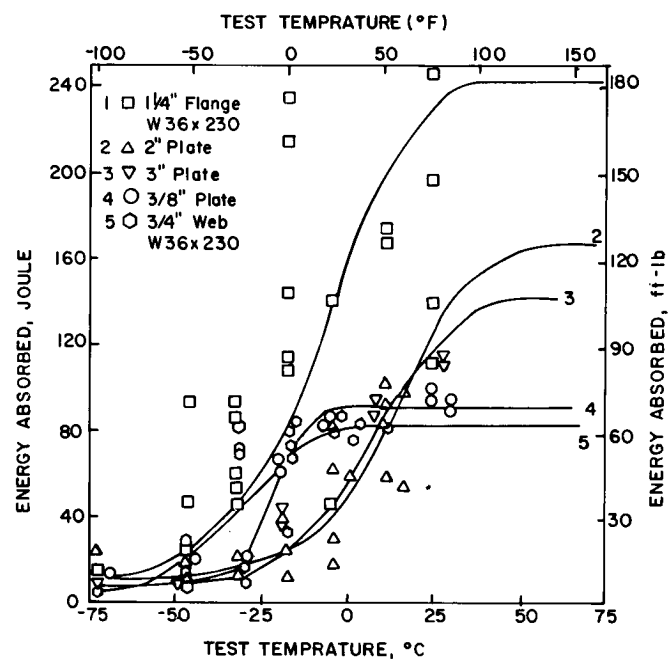


Figure 73. Charpy V-notch toughness data for A588 steels. [Roberts 1977]

separation of the inclusions. Those properties involve ductile failure in which the nonmetallic inclusions nucleate voids that grow and coalesce to form a major fracture. Elongated stringers of manganese sulfide, MnS, or discontinuous stringers of aluminum oxide inclusions (Al_2O_3) cause premature ductile fracture. Manganese sulfide is plastic at hot-rolling temperatures, and the particles tend to become elongated during rolling. The highly elongated inclusions, and particularly the more or less coplanar arrays, cause a marked anisotropy of the total ductility at fracture and of the Charpy shelf energy. Testing in the longitudinal direction (parallel to the rolling direction) gives much higher energy-absorption values than does testing in the transverse or through-thickness directions. This anisotropy is largely eliminated if the inclusions are present as small, isolated, non-deformed particles. Because most commercial steels have a much greater volume fraction of sulfides (MnS) than oxides (Al_2O_3), sulfides are the major problem. It can be alleviated by reducing

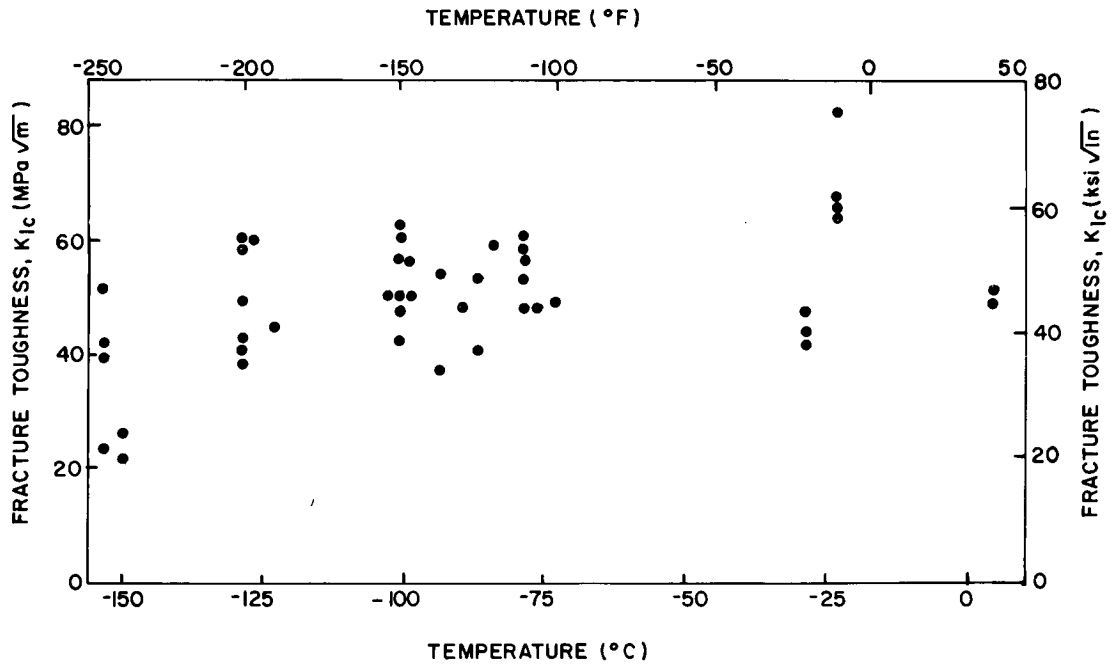


Figure 74. Fracture toughness of A588 steels under one-second loading. [Crosley 1983]

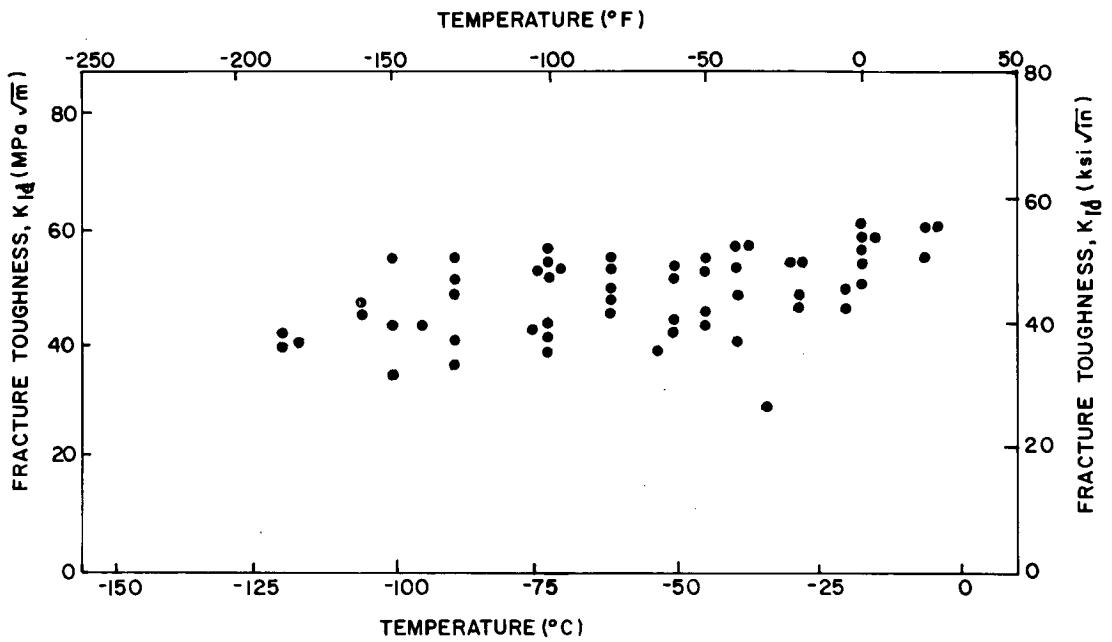


Figure 75. Fracture toughness of A588 steels under dynamic loading. [Crosley 1983]

the sulfur content to very low values and controlling the inclusion shape by calcium treatment or by adding microalloying elements such as rare metals, zirconium, and titanium [DuMond 1977, Fletcher 1979, Wilson 1982].

Figure 76 compares the Charpy V-notch energy of conventional (not calcium-treated) and calcium-treated A588 steels rolled alike. The specimens were oriented longitudinally. The notch extended in the two directions of interest for bridge members, simulating a crack through the thickness (LS) and across the width (LT) of an element. Table 38 summarizes the Charpy V-notch upper shelf energy of conventionally rolled and calcium treated A588 steels. The steel without the calcium treatment exhibited poor isotropy in the three major directions, whereas more isotropy was achieved through reduction and balling of the sulfides by calcium treatment.

Because ductility and toughness usually decrease as strength increases, these properties should be carefully optimized with due regard to the intended application.

ASTM does not specify the minimum impact strength of A242 and A588 steels. The Charpy V-notch impact test and the drop weight test are listed in the A242 and A588 specifications as standardized supplementary requirements for use at the option of the purchaser. ASTM specification A709 specifies the Charpy V-notch impact test as a supplementary requirement to be conducted in accordance with ASTM A673. Impact test requirements are in relationship to the lowest ambient temperature expected for the location where the steel is to be used, and are only applicable when specified in the contract and order.

Like any as-rolled steel, the notch toughness of as-rolled weathering steel can vary to a considerable degree depending on such features as individual cast or heat chemistry, rolling temperature and technique, product size, and the particular producing facilities involved. Therefore, when notch toughness is a matter of concern, desired minimum values must be incorporated as part of any inquiry or order.

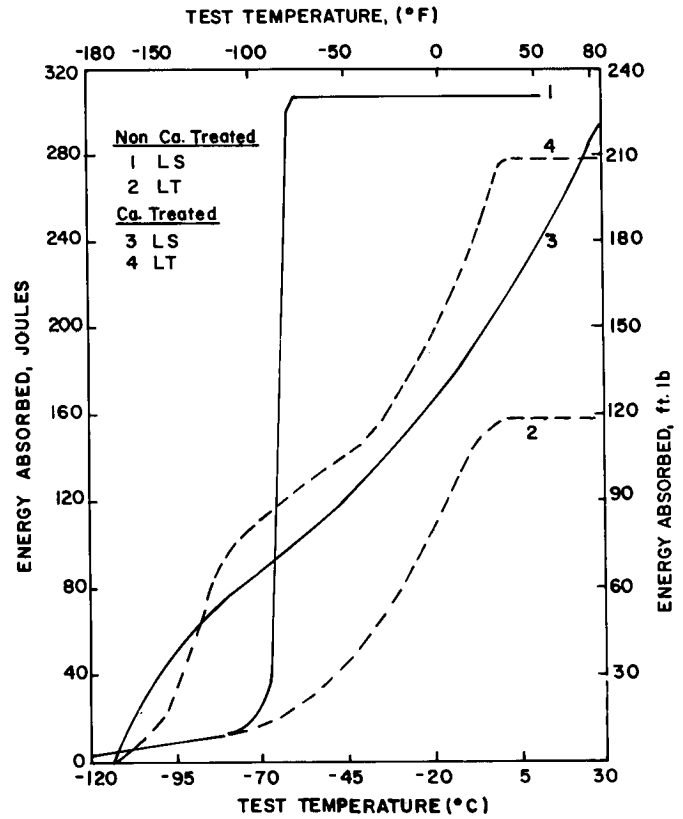


Figure 76. Charpy V-notch energy absorbed by non-calcium-treated and calcium-treated A588 steels. [Wilson 1982]

Table 38. Charpy V-notch upper shelf energy of conventional and calcium-treated A588 steels. [Wilson 1982]

Specimen Orientation	V-Notch Orientation	Charpy V-Notch Upper Shelf Energy [Joules (ft.-lb)]	
		Non-Calcium Treated Steel	Calcium Treated Steel
Longitudinal	Through thickness	313 (231)	290 (214)
Longitudinal	Transverse	160 (118)	279 (206)
Transverse	Through thickness	81 (60)	195 (144)
Transverse	Longitudinal	66 (49)	206 (152)
Through thickness	Transverse	30 (22)	140 (103)
Through thickness	Longitudinal	23 (17)	149 (110)

WEATHERING FATIGUE

This chapter examines all data reported in the literature for specimens that were weathered outdoors for a period of time and then fatigue tested to failure, a condition termed herein weathering fatigue. The uniform basis for comparing the data is defined first. Thereafter, the fatigue test data for corrosion resistant (weathering) steels and noncorrosion resistant steels are presented. The chapter concludes with a comparison of the data for all steels and an explanation of the observed losses and gains in life in terms of pitting and weld-toe rounding.

BASIS FOR COMPARING S-N DATA

The results of the fatigue tests are presented in S-N plots of the base-10 logarithm of stress range, f_r , versus the base-10 logarithm of number of cycles to failure, N , as shown in Figures 79–90. The features common to the S-N plots and the basis for their comparison are summarized in the following.

1. The mean regression lines for the sets of data points were calculated by the least-square method using the model:

$$\log N = b - m \log f_r \quad (29)$$

in which b equals the intercept and m equals the slope. All runout tests were excluded from the regression analysis.

2. The AASHTO allowable S-N lines, located two standard deviations to the left of the mean, are given by:

$$\log N_d = (b - 2s) - m \log F_{sr} \quad (30)$$

in which N_d equals the number of design cycles, F_{sr} equals the allowable stress range, and s equals the standard deviation on log of life. The allowable F_{sr} versus N_d values for redundant load path structures, given in Table 1.7.2A1 of the AASHTO fatigue specifications, are rounded coordinates of points that fall on the allowable S-N lines given by Eq. 30. The lines are cut off horizontally at the allowable stress range for over 2,000,000 cycles. Since AASHTO has set the allowable S-N lines at two standard deviations to the left of the mean, a sample of fatigue test data could be said to meet the fatigue strength of a given category if not more than 2.3 percent of the data fell below the allowable line.

3. The regression coefficients, b and m , and the standard deviation, s , that were used to calculate the AASHTO allowable S-N lines are summarized in Table 39.

4. The relative stress range after weathering is the ratio of the stress range after weathering to the stress range before weathering, f_{r1}/f_{r2} . Both are calculated at 500,000 cycles using the applicable regression equations. (See the open circles on the mean lines shown in Figure 77.) The relative loss in stress range is, therefore:

Table 39. Regression analysis and fatigue notch factors for the data base of current fatigue specifications.

Category	Type of Detail Tested	Number of Data Points		Regression ^b Coefficients		Standard Deviation	Stress Range at 500,000 Cycles (MPa)	Fatigue Notch Factor K_f
		Included	Excluded	Intercept b	Slope m			
A	Rolled beam	28	16	13.785	3.178	0.221	350	1.00
B	Welded beam	55	1	13.696	3.372	0.147	235	1.49
C*	Transverse stiffener	135	5	12.681	3.097	0.158	180	1.95
C	51-mm attachment	14	0	12.763	3.25	0.0628	149	2.35
D	102-mm attachment	44	8	12.177	3.071	0.108	129	2.72
E	Cover plate end	193	0	11.886	3.095	0.1006	100	3.51
E'	Cover plate end: ^a t > 20 mm (0.8 in)	18		11.849	3.2	0.1943	83.5	4.19

Notes

- Calculated as in Ref. [Albrecht 1981].
- Intercept C for stress range in MPa.

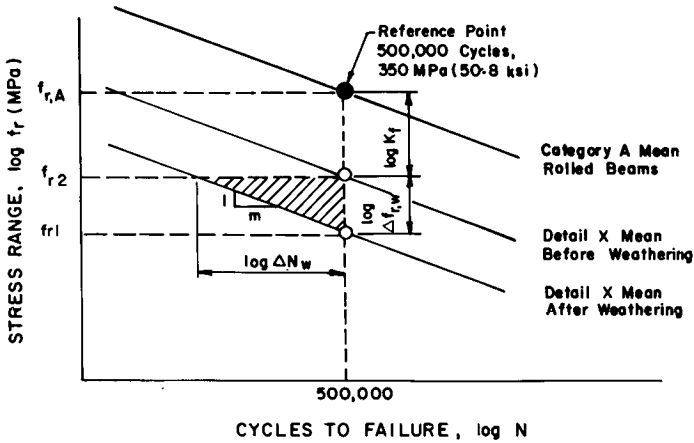


Figure 77. Definition of fatigue notch factor and computation of loss in stress range and fatigue life.

$$\Delta f_{r,w} = 1 - \frac{f_{r1}}{f_{r2}} \quad (31)$$

5. Since N is an m -th power exponential function of f_r , the corresponding relative fatigue life after weathering is $(f_{r1}/f_{r2})^m$. The relative loss in fatigue life is, therefore, given by:

$$\Delta N_w = 1 - \left(\frac{f_{r1}}{f_{r2}}\right)^m \quad (32)$$

The losses $f_{r,w}$ and N_w are sides of the shaded triangle in Figure 77. This procedure compares experimental data near its center of gravity, about midway in the range of cycles to failure usually measured in testing programs. It estimates losses in life in a manner that makes them insensitive to slope variations of the two S-N lines being compared.

6. The results of the regression analyses of all fatigue test data are summarized in Tables 40 and 41, respectively, for specimens that were fabricated from weathering steels and from noncorrosion resistant steels.

7. The calculated losses in stress range and fatigue life due to weathering are summarized in Tables 43 and 44. The values were calculated with Eqs. 31 and 32 using the mean slope, $m = 3.2$, of the five regression lines for the Category A to E data (499 points) given in Table 39. The mean slope of the regression lines for the data given in Tables 40 and 41 (1,572 points) is actually $\bar{m} = 3.39$. Slopes greater than 5.0 were excluded. In this study, a mean slope $\bar{m} = 3.2$ was assumed to make the calculations compatible with the AASHTO data base.

8. When the number of data points in a test series was insufficient to calculate a regression line, the loss in life, N_w , was determined as the ratio of the mean cycles to failure of specimens tested at the same stress range.

9. The S-N lines were compared at 500,000 cycles, in the manner illustrated in Figure 77. The fatigue notch factor is defined as the ratio of the mean stress range for Category A rolled beams, f_{rA} (solid circle), and the mean stress range of the detail under consideration, f_{r2} (open circle).

$$K_f = \frac{f_{rA}}{f_{r2}} = \frac{350 \text{ MPa}}{f_{r2}} = \frac{50.8 \text{ Ksi}}{f_{r2}} \quad (33)$$

It establishes the severity of the detail before weathering [Albrecht 1981] in terms of the factor, K_f , by which the mean S-N line for a detail falls below the mean S-N line for Category A rolled beams. It should be noted that K_f is not referenced to the mean S-N line for polished rotating beam specimens, as is done in other fields of application. The reference stress range was calculated by substituting in Eq. 29 the values of b and m for rolled beams given in Table 39, as follows: $f_{rA} = [10^{13,785} / 500,000]^{1/3.178} = 350 \text{ MPa (50.8 ksi)}$.

WEATHERING STEELS

Seven studies reported data for weathering steel specimens that were weathered outdoors, at most 11 years prior to fatigue testing [Kunihiro 1972; Albrecht 1980, 1982a, and 1983a; Blake 1982; Yamada 1983b; Barsom 1983; and Fisher 1983a]. The American studies were performed on A588 Grade A and Grade B steels, the Japanese studies on SMA 50 and 58 steels. The chemical requirements for these steels are given in Tables 4 and 10, respectively.

The following types of specimens, shown in Figure 78, were tested: base metal [Kunihiro 1972, Blake 1982], groove welds ground flush [Kunihiro 1972], groove welds with reinforcement not removed [Blake 1982, Barsom 1983], bead welds (Barsom 1983), transverse stiffeners [Albrecht 1980, 1982a, and 1983a; Yamada 1983b], 4-in. (102-mm) long attachments [Albrecht 1982a], 4-in. (100-mm) long gussets [Yamada 1983b], and cover plates [Fisher 1983a]. Most series of specimens were continuously weathered prior to fatigue testing; a few were alternately weathered and stress cycled.

To shorten the length of exposure, Albrecht [1977] tried to artificially weather A588 steel specimens in the laboratory by subjecting them to 1,800 wet-dry cycles of 1-hour duration. An examination of the surface and sections of the oxide coating with a scanning electron microscope (SEM) revealed, however, that the artificially grown oxide was porous, coarse, and poorly bonded to the steel base. On the other hand, the oxide on specimens of the same batch that had been weathered outdoors up to 18 months had the hard, dense, and well-bonded structure characteristic of natural exposure. The SEM work confirmed the need to naturally weather the specimens.

For ease of presentation, the fatigue test data are examined in the following by investigator.

Base Metal and Groove Welds

Kunihiro, et al., were the first to study the effect of weathering on the fatigue behavior of bare steel specimens [Kunihiro 1972]. They examined the following variables: (1) base metal and groove-welded ground flush specimens; (2) 0-year, 2-year, and 4-year weathering; and (3) 51-ksi (353-MPa) and 65-ksi (451-MPa) minimum specified yield point for SMA 50 and 58 steels. The specimens were 2 in. \times $\frac{1}{4}$ in. \times 18 in. (50 mm \times 19 mm \times 459 mm) long, as shown in Figure 78. They were fabricated and tested by seven participating laboratories. The specimens were fabricated from hot-rolled atmospheric corrosion resistant steels for welded structures and supplied to Japanese Industrial Standard G3114 [JIS 1981].

Table 40. Regression analysis of fatigue test data for specimens fabricated from weathering steels.

Reference	Type of Steel	Type of Detail	Weathering ^a Time (Years)	No. of Specimens Included	No. of Specimens Excluded	Regression Coefficients ^f		Standard Deviation s
						Intercept b	Slope m	
Kunihiro 1972	SMA 50 and SMA 58	Base metal	0	54	19 ^b	13.8677	3.176	0.3141
			2	49	17 ^b	15.1199	3.833	0.2167
			4	61	22 ^b	15.2587	3.874	0.2385
Kunihiro 1972	SMA 50 and SMA 58	Groove weld ground flush	0	45	19 ^b	17.4371	4.551	0.2860
			2	58	12 ^b	16.2795	4.311	0.1915
			4	63	19 ^b	15.0050	3.808	0.1752
Albrecht 1982a	A588 Grade A	Manually welded transverse stiffener	0	24	5 ^c	12.4209	3.007	0.1172
			2	15	5 ^c	11.6900	2.717	0.1008
			2 alt.	12	5 ^c	11.9734	2.797	0.0885
			4	15	5 ^c	11.2695	2.529	0.1172
			4 alt.	12	4 ^c	11.8110	2.762	0.0737
Albrecht 1982a	A588 Grade A	102-mm attachment	0	17	7 ^c	13.3987	3.416	0.0939
			2	15	0	13.3419	3.398	0.0905
			2 alt.	8	0	11.6714	2.597	0.1962
			4	20	0	12.9336	3.234	0.0976
			4 alt.	8	0	12.4304	2.982	0.1725
Albrecht 1980, 1983a	A588 Grade B	Automatically welded transverse stiffener	0	12	0	13.1897	3.226	0.1692
			3	12	4 ^c	12.4822	3.017	0.1010
			3 alt.	12	0	12.0186	2.810	0.1022
			8	16	6 ^c	12.7197	3.167	0.1142
Blake 1982	A588 Grade A	Base metal as received	0	15	3 ^b	19.6049	5.460	0.1556
			3	7	0	17.9190	4.883	0.0405
			6	16	0	19.0511	5.336	0.0817
Blake 1982	A588 Grade A	Base metal blast cleaned	0	7	5 ^b	24.1233	7.030	0.3860
			2	6	2 ^b	31.7058	10.070	0.1148
			4.5	8	0	18.4416	5.026	0.1003
Blake 1982	A588 Grade A	Groove weld	0	8	0	17.0948	4.745	0.2034
			2	6	0	10.5929	2.194	0.0562
Yamada 1983b	SMA 50	Transverse stiffener	0	6	4 ^c	12.6129	3.025	0.1149
			2	6	3 ^c	15.4230	4.152	0.1248
			4	3	7 ^c	--	--	--
Yamada 1983b	SMA 50	100-mm gusset	0	7	4 ^c	11.3791	2.610	0.0405
			2	9	3 ^c	13.9590	3.674	0.1859
			4	6	4 ^c	11.4457	2.587	0.1261
Yamada 1983a	SMA 50	Transverse stiffener	5.5	3	7	--	--	--
Barsom 1983	A588 Grade A	Groove weld	11-R ^d	15	0	24.8196	7.466	0.2429
			11-UI	16	0	20.3126	5.705	0.2621
			11-M	12	2 ^b	19.1370	5.234	0.2796
Barsom 1983	A242 Type 1	Groove weld	11-R	15	1 ^b	20.5061	5.774	0.3801
			11-UI	10	4 ^b	23.7322	7.014	0.3314
			11-M	11	1 ^b	28.8012	8.829	0.3698
Barsom 1983	A242 Type 1	Bead weld	11-R	14	2 ^b	19.7086	5.457	0.3853
			11-UI	16	1 ^b	27.2324	8.282	0.3599
			11-M	13	1	26.9339	8.098	0.2435
Barsom 1983	A588 Grade A	Bead weld	11-R 11-UI 11-M	(e)	(e)	(e)	(e)	(e)

Notes

- Specimens were continuously weathered, unless noted otherwise.
- Runouts were excluded from regression analysis.
- When a replicate specimen was a runout, others tested at the same stress range were also excluded from the regression analysis.
- R = rural, UI = urban-industrial, M = moderate marine
- Fatigue test data could not be obtained from the investigator.
- Intercept for stress range in MPa.

The significance of the variables could not be evaluated with statistical analysis techniques, because the experiment had not been designed with that purpose in mind. Still, by lumping the data from the various laboratories and for the two yield strengths, and plotting the results by type of detail, one can assess the effect of weathering time on fatigue life.

The data for base metal subjected to 2-year and 4-year weathering are plotted in Figures 79 and 80, respectively. Also shown are the means of the nonweathered and weathered specimens,

the mean of the Category A rolled beams [Fisher 1970] and the Category A allowable line. The data suggest the following conclusions:

- Both the 0-year weathered base metal and the 0-year weathered groove-welded specimens exhibited the fatigue strength of Category A, since only 2.7 percent (2 out of 73) and 1.6 percent (1 out of 64) of the data points, respectively, fell below that line. Their mean lives were slightly longer than the

Table 41. Regression analysis of fatigue test data for specimens fabricated from steels with no atmospheric corrosion resistance.

Reference	Type of Steel	Type of Detail	Weathering ^a Time (Years)	No. of Specimens Included	No. of Specimens Excluded	Regression Coefficients ^d		Standard Deviation _s
						Intercept _b	Slope _m	
Kunihiro 1972	SM 50 and SM 58	Base metal	0	44	18 ^b	10.9808	2.067	0.4029
			2	57	12 ^b	14.7022	3.693	0.2523
			4	62	23 ^b	13.1205	3.063	0.3083
Kunihiro 1972	SM 50 and SA 58	Groove weld ground flush	0	36	14 ^b	16.1164	4.071	0.3451
			2	56	15 ^b	14.6795	3.689	0.2756
			4	69	19 ^b	15.1260	3.893	0.1903
Nihei 1978	SM 50A	Notched plate	0	9	2 ^b	44.8791	14.490	0.3662
			3	7	4 ^b	16.0415	4.420	0.2143
Nihei	SM 50A	Notched plate R = -1	0	10	7 ^b	31.4776	10.698	0.2983
			3	7	2 ^b	27.5187	9.519	0.3804
Nihei 1978	SM 50A	Groove weld as welded	0	16	2 ^b	19.2248	5.454	0.1526
			3	7	3 ^b	13.8379	3.389	0.1127
Nihei 1978	SM 50A	Groove weld as welded, R = -1	0	30	11 ^b	15.3963	3.733	0.2373
			3	7	4 ^b	12.4359	2.754	0.1803
Yamada 1983b	SM 50	Transverse stiffener	0	6	3 ^c	13.9156	3.592	0.0853
			2	3	6 ^c	--	--	--
			4	3	7 ^c	--	--	--
Yamada 1983b	SM 50	100-mm gusset	0	10	0	11.7209	2.733	0.1099
			4	9	0	11.2160	2.487	0.1358

Notes:

- a. All specimens were continuously weathered.
- b. Runouts were excluded from regression analysis.
- c. When a replicate specimen was a runout, others tested at the same stress range were also excluded from the regression analysis.
- d. Intercept for stress range in MPa.

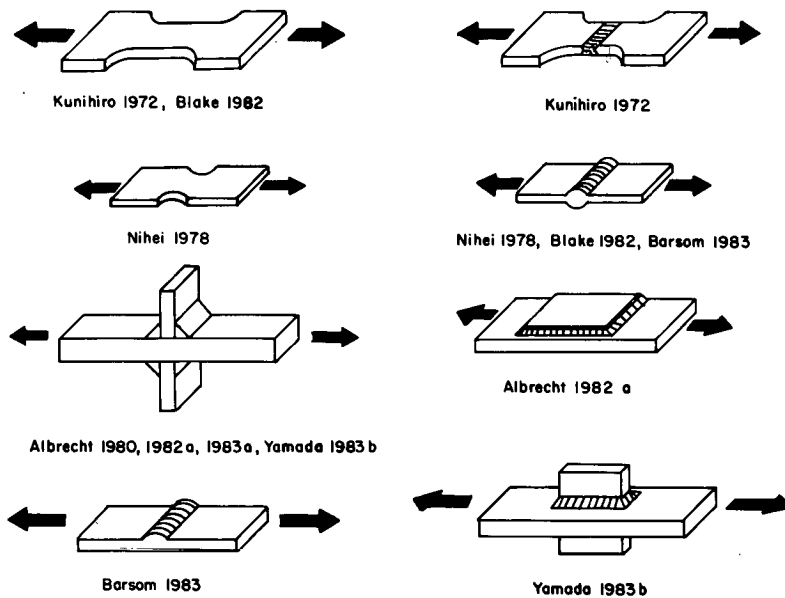


Figure 78. Types of fatigue tensile specimens.

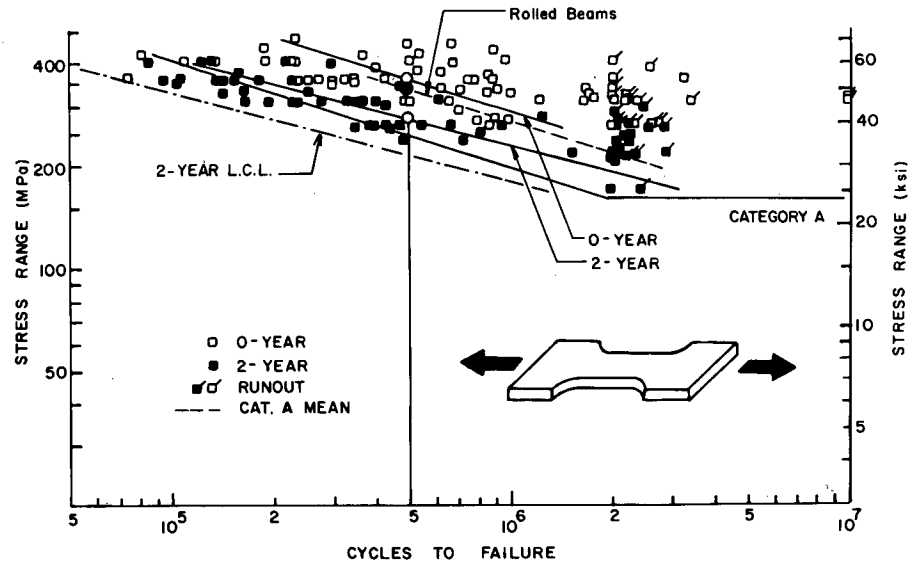


Figure 79. Fatigue strength of 2-year weathered base metal specimens fabricated from Japanese SMA 50 and SMA 58 weathering steels. [Kunihiro 1972]

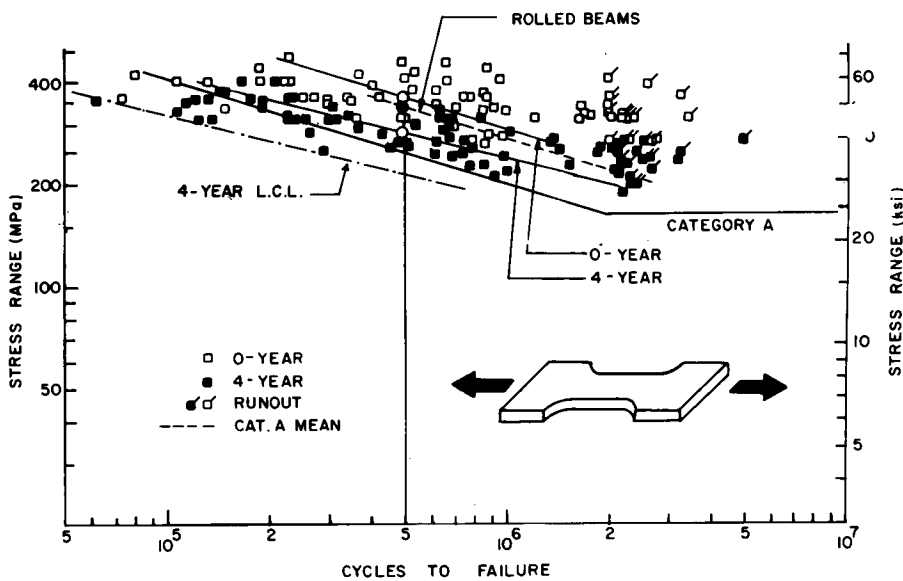


Figure 80. Fatigue strength of 4-year weathered base metal specimens fabricated from Japanese SMA 50 and SMA 58 weathering steels. [Kunihiro 1972]

Category A mean, as indicated by the fatigue notch factors, $K_f = 0.94$ and 0.92 , less than unity (Table 43).

2. The long life of the groove-welded specimens likely resulted from the extra care in welding caused by the element of competition between the seven fabricators.

3. Two years and 4 years of weathering decreased the mean fatigue life of the base metal specimens by 57 percent and 54 percent, respectively, with 16.7 percent (11 out of 66) and 10.8 percent (9 out of 83) of the corresponding data points falling below the Category A allowable line. (See Table 43 and points 1 and 2 in Figure 94.)

4. The losses in life of the groove-welded specimens were 60 percent and 63 percent, with 22.9 percent (16 out of 70) and

18.3 percent (15 out of 82) of the corresponding data points falling below the Category A allowable line. (See points 3 and 4 in Figure 94.)

5. The loss in life was about the same after 2 years and 4 years of weathering.

Manually Welded Stiffeners and Attachments

Albrecht, et al., reported the results of a study [Albrecht 1982a, Friedland 1982] in which the following variables were examined: (1) specimens with transverse stiffeners and 4 in. (102 mm) attachments; and (2) 0-year, 2-year, 2-year alternate, 4-

year and 4-year alternate weathering. The first specimen consisted of a 1-in. \times $\frac{3}{8}$ -in. \times 13-in. (25-mm \times 10-mm \times 330-mm) long main plate to which two 1-in. \times $\frac{1}{4}$ -in. \times 2-in. (25-mm \times 7-mm \times 50-mm) long transverse stiffeners were attached with $\frac{1}{4}$ -in. (7-mm) manual fillet welds. The other series consisted of a 2 $\frac{1}{2}$ -in. \times $\frac{3}{8}$ -in. \times 13-in. (64-mm \times 10-mm \times 330-mm) long main plate to which two 1 $\frac{3}{4}$ -in. \times $\frac{1}{4}$ -in. \times 4-in. (45-mm \times 7-mm \times 102-mm) long flat attachments were manually welded with $\frac{3}{16}$ -in. (5-mm) fillet welds. The specimens were made of A588 Grade A steel. The so-called alternate weathering consisted of repeatedly exposing the specimens for 6 months and applying either one-fourth or one-eighth of the estimated cycles to failure of the 2-year and 4-year weathered specimens, respectively.

As an example, the fatigue test data for the transverse stiffeners are plotted in Figure 81. The following conclusions can be drawn from the stiffener data:

1. The mean fatigue life of the 0-year weathered stiffeners ($K_f = 2.04$) was slightly shorter than that for Category C beams and girders with stiffeners welded to the web and flanges ($K_f = 1.95$) [Fisher 1974].
2. Two-year and 4-year weathering decreased the fatigue life by 21 percent and 22 percent respectively. (See Table 43 and points 5 and 6 in Figure 94.)
3. Two-year alternate weathering increased the life by 6 percent, whereas 4-year alternate weathering decreased it by 16 percent (points 7 and 8 in Figure 94).
4. All data points and, for the most part, the lower confidence limits at two standard deviations from the mean fell above the Category C allowable line.
5. The significance of the loss in fatigue life was examined with analysis of variance and with acceptance sampling [Ang 1975]. Two analyses were performed. In the first case, assuming a 5 percent risk to the bridge supplier and ignoring the risk to the bridge owner, the loss in life was found to be statistically insignificant. In the second case, assuming a 5 percent risk to the bridge supplier and considering the risk to the bridge owner, it was found that the owner's risk of a fatigue failure during

the service life would be 0.3 percent if he would accept the continuously weathered stiffeners.

6. Fatigue cracking occurred in nearly equal numbers from weld toes facing the sky or the ground. The effect of side of exposure on fatigue life was, in general, statistically insignificant.

The fatigue test data for the specimens with 4-in. (102-mm) attachments suggest the following conclusions:

1. The mean fatigue life of the 0-year weathered 4-in. (102-mm) attachments ($K_f = 1.95$) was equal to that of Category C transverse stiffeners ($K_f = 1.95$). It was longer than the life of the 4-in. (102-mm) attachments welded to beam flanges that have become the basis for Category D [Fisher 1974]. The reason for this behavior was the reduced degree of constraint provided by the main plate of the tensile specimens, as compared to that provided by the more rigid beam specimens. Still, the data gave additional information on the behavior of details with Category C fatigue strength.
2. Two-year and 4-year weathering decreased the fatigue life by 6 percent and 10 percent, respectively (points 9 and 10 in Figure 94).
3. Two-year alternate weathering increased the life by 8 percent, whereas 4-year alternate weathering decreased it by 9 percent (points 11 and 12 in Figure 94).
4. The loss in life was found by analysis of variance to be statistically insignificant at the 5-percent level.
5. The effect of side of exposure on fatigue life was statistically insignificant.

Automatically Welded Stiffeners

In a second study, Albrecht, et al., tested A588 Grade B steel specimens with transverse stiffeners identical to those of the first study, except that they were welded by the automatic submerged-arc process [Albrecht 1980 and 1983a]. The weathering times were 0-year, 3-year, 3-year alternate and 8-year. The alternately weathered specimens were exposed for 3 years, fol-

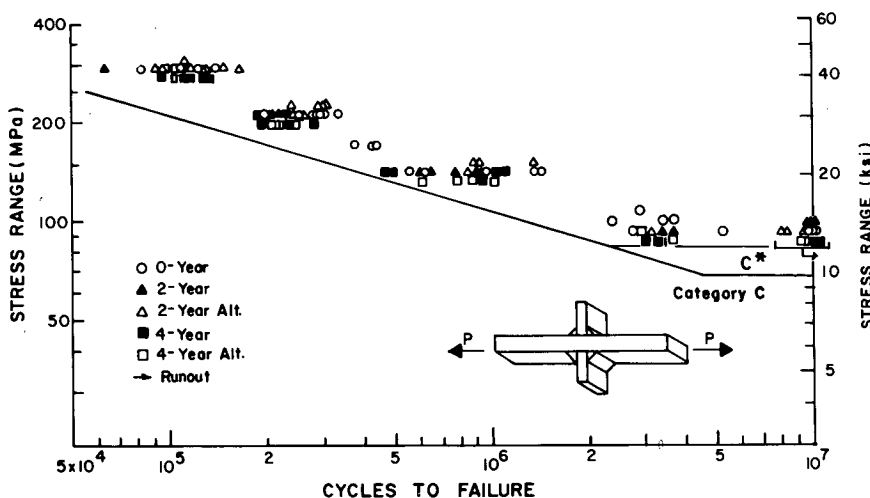


Figure 81. Fatigue strength of 2-year and 4-year weathered manually welded transverse stiffeners fabricated from A588 steel. [Albrecht 1982a, Friedland 1982]

lowed by alternate one-quarter-life cycling and 6-month weathering.

Figures 82 and 84 depict the gradual degradation of fatigue strength with exposure time. The 0-year weathered specimens ($K_f = 1.67$) had a fatigue strength closer to that of the Category B mean for welded beams ($K_f = 1.49$) than to Category C mean for manually welded stiffeners ($K_f = 1.95$), because the superior quality of the automatic submerged-arc welds increased the crack initiation portion of the fatigue life. Figure 83 compares the 3-year data against the "0-year rhomboid." The latter is the space between the two-standard-deviation confidence limits of the 0-year data. As can be seen, the 3-year data fell in the left half of the 0-year rhomboid. Figure 84 shows the 8-year mean falling on the lower confidence limit of the 0-year data, with one-half of the points falling short of the rhomboid. The fatigue strength degraded at a decreasing rate with increasing time of exposure.

The effect of side of exposure on fatigue life was statistically insignificant. The data points for the specimens that failed from

weld toes facing the sky or the ground are indeed well interspersed in Figures 83 and 84.

The data were compared with the current specification requirements in three ways. First, the data points were plotted against the Category C allowable line. Figure 85 shows that all 62 points fell above this line.

Secondly, the lower confidence limits at two standard deviations to the left of the mean were compared. For transverse stiffeners manually welded to beams and girders, Table 39 gives $b = 12.681$, $m = 3.097$, and $s = 0.158$. The corresponding values for the automatically welded stiffeners are given in Table 40. Substituting these sets of values into Eq. 30 yields the four lower confidence limits drawn with dashed lines in Figure 86, along with the solidly drawn limit for stiffeners welded to beams and girders (Category C*), and the limit for 2-in. (51-mm) attachments (Category C). The lines in Figure 86 are labeled in descending order of life. The four limits for the data fell above the Category C allowable line. The 8-year limit fell below the limit for stiffeners welded to beams and girders.

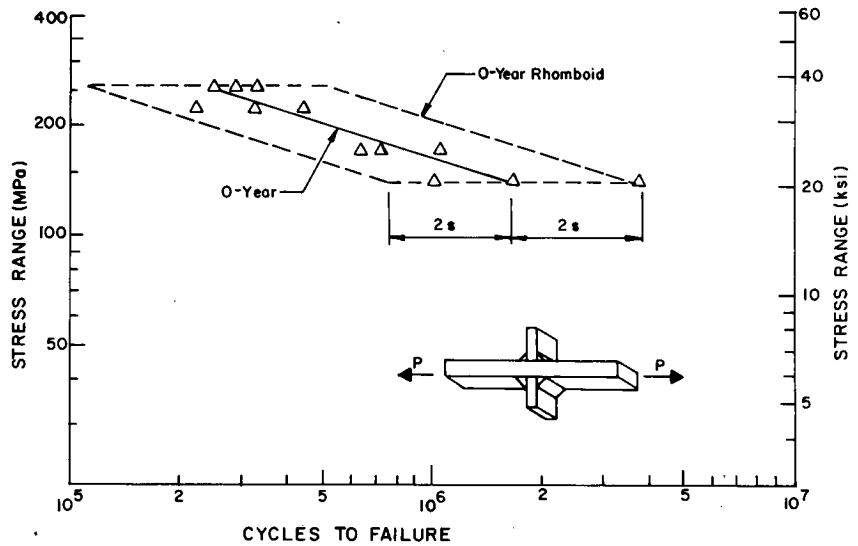


Figure 82. Fatigue strength of 0-year weathered automatically welded transverse stiffeners fabricated from A588 steel. [Albrecht 1980 and 1983a]

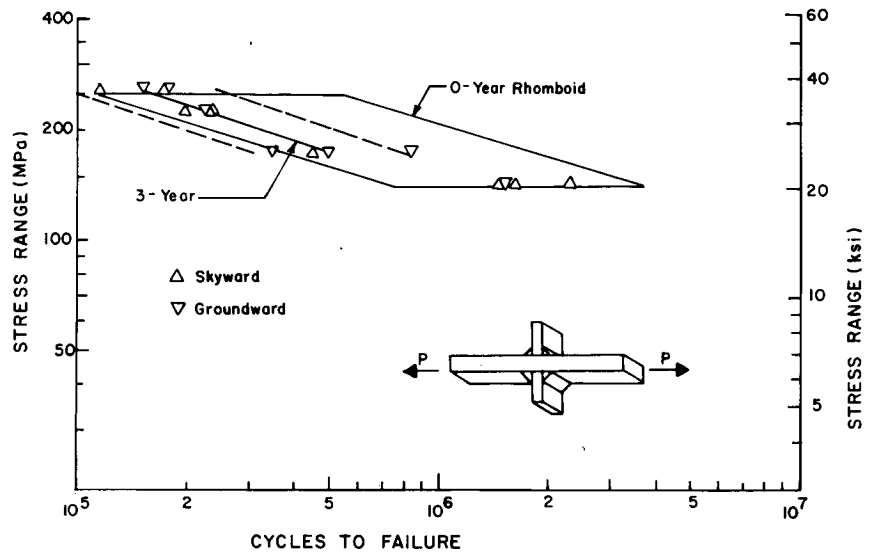


Figure 83. Fatigue strength of 3-year weathered automatically welded transverse stiffeners fabricated from A588 steel. [Albrecht 1980 and 1983a]

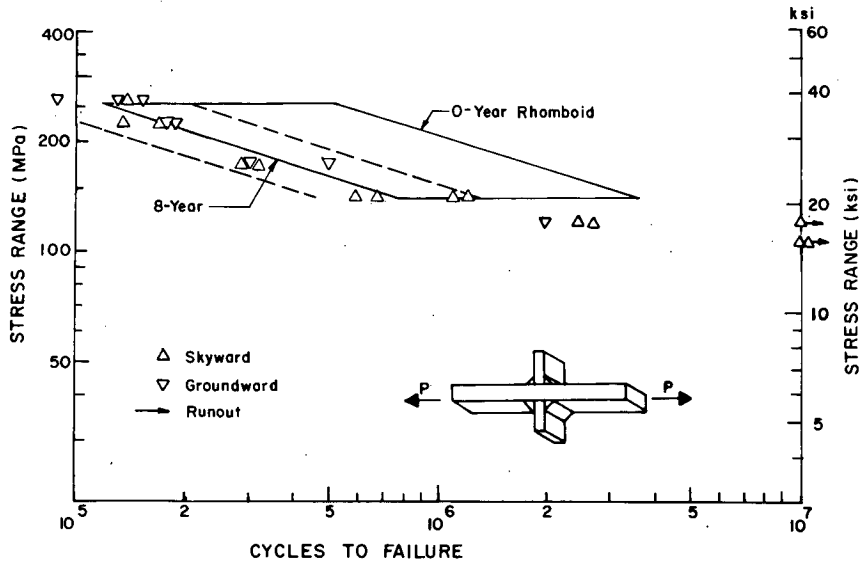


Figure 84. Fatigue strength of 8-year weathered automatically welded transverse stiffeners fabricated from A588 steel. [Albrecht 1980 and 1983a]

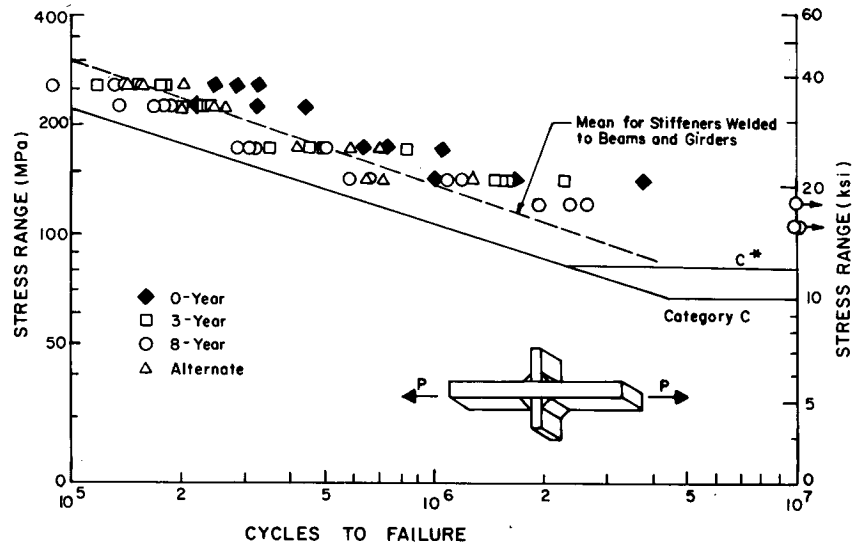


Figure 85. Comparison of all data for automatically welded transverse stiffeners fabricated from A588 steel. [Albrecht 1980 and 1983a]

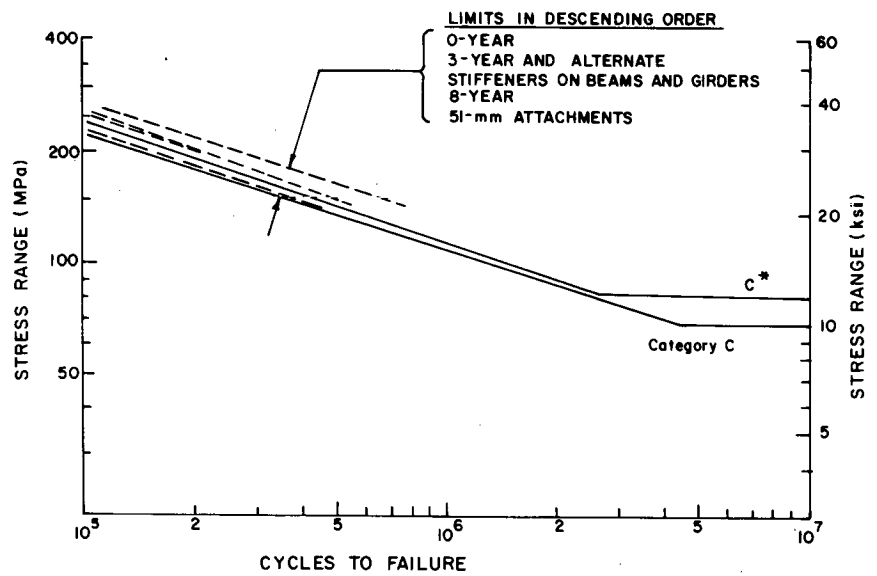


Figure 86. Comparison of lower confidence limits for automatically welded transverse stiffeners fabricated from A588 steel. [Albrecht 1980 and 1983a]

Thirdly, safety factors were compared. They were defined, for the purpose of this evaluation, as the ratio between the mean line of a data set and the Category C allowable line. Figure 87 shows that weathering reduced the safety factor on life of the automatically welded stiffeners from about 4.0 for the 0-year data to 2.5 for the 3-year and alternate data, and 1.9 for the 8-year data. These drops correspond to the reported 42 percent and 54 percent losses in life, respectively. The mean safety factor for the 8-year data was much lower than the safety factor of 2.5 for nonweathered stiffeners manually welded to beams and girders. In summary:

1. Three-year, 3-year alternate, and 8-year weathering reduced the life by 42, 42, and 54 percent, respectively. (See Table 43 and points 13, 14, and 15 in Figure 94.)

2. The data points and the lower confidence limits fell above the Category C allowable line for 2-in. (51-mm) attachments. This happened for two reasons: (1) the 0-year weathered specimens had a fatigue strength close to that of Category B; and (2) the standard deviation for the weathered specimens was smaller than for the nonweathered specimens.

3. The mean safety factor on life dropped from about 4.0 for the 0-year data to 1.9 for the 8-year data; that is, below the safety factor of 2.5 for the manually welded transverse stiffeners that are part of the data base for Category C.

4. The lower confidence limit continued to drop with increasing weathering time, although at a smaller rate. This suggests that the degradation of fatigue life slows down with time, but does not stop.

Base Metal and Groove Welds

Barsom quoted Blake as having tested the following three types of A588 Grade A steel plate specimens that were weathered up to 6 years: (1) base metal, as received from the mill; (2) blast-cleaned base metal; and (3) groove welds [Blake 1982, Barsom 1983]. The base metal specimens were subjected to stress ranges varying from 45 to 60 ksi (310 to 415 MPa). The results suggest the following conclusions:

1. The regression analysis of the data, excluding the runout specimens, indicated that at 500,000 cycles the mean strength of the nonweathered as-received steel ($K_f = 1.00$) was identical to the mean for Category A rolled beams ($K_f = 1.00$). The nonweathered blast-cleaned steel had higher strength ($K_f = 0.84$) because the residual surface compression stresses that were introduced by blast cleaning increased the crack initiation life.

2. Three-year and 6-year weathering reduced the life of the as-received steel specimens by 28 percent. Two-year and 4.5-year weathering reduced the life of the blast-cleaned specimens by 24 percent and 47 percent respectively. (See points 16–19 in Figure 94.)

3. The loss in life was much larger for the specimens cycled at or below the yield point than for those cycled above the yield point. If the data points for specimens cycled at more than 50-ksi (345-MPa) stress range were excluded, as they should be, the losses in life would be 69, 68, and 76 percent for the 3-year and 6-year as-received steel, and the 4.5-year blast-cleaned steels, respectively. (See points 16a, 17a, and 19a in Figure 94.)

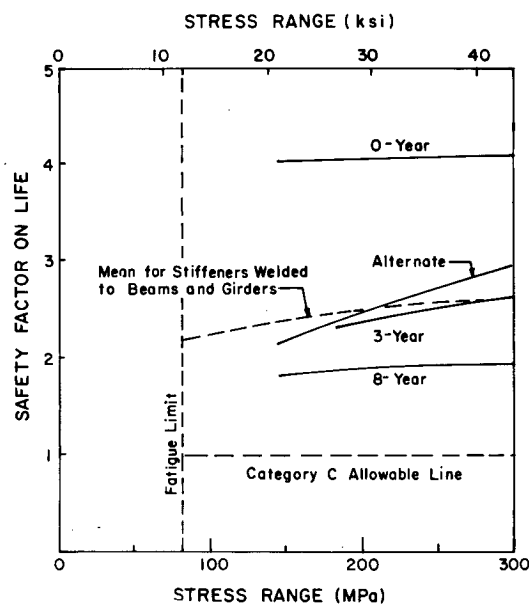


Figure 87. Comparison of safety factors for automatically welded transverse stiffeners fabricated from A588 steel. [Albrecht 1980 and 1983a]

The fatigue test data for the groove-welded plates with weld reinforcement left in place are plotted in Figure 88. The data indicate the following:

1. The nonweathered specimens had a mean fatigue strength ($K_f = 1.39$) higher than the mean of the Category B welded beams ($K_f = 1.49$).

2. Two-year weathering reduced the mean strength ($K_f = 2.07$) by 72 percent to less than the mean of the Category C transverse stiffeners ($K_f = 1.95$). The data are represented by point 20 in Figure 94.

3. For those specimens cycled at 45-ksi (310-MPa) nominal stress range, local yielding at the toe of the weld reinforcement greatly reduced the fatigue strength before weathering and, hence, also the loss in life after weathering.

Transverse Stiffeners and Longitudinal Gussets

Yamada and Kikuchi examined the effect of 0-year, 2-year, and 4-year weathering on the fatigue behavior of two types of specimens fabricated from Japanese SMA 50 weathering steel [Yamada 1983b]. One type consisted of a 3.15-in. \times 0.39-in. \times 24-in. (80-mm \times 10-mm \times 600-mm) long plate to which two 3.15-in. \times 0.39-in. \times 2.36-in. (80 mm \times 10 mm \times 60 mm) long transverse stiffeners were welded with 0.24-in. (6-mm) manual fillet welds. The other type of specimen consisted of a 3.15-in. \times 0.39-in. \times 28-in. (80-mm \times 10-mm \times 700-mm) long plate to which two 2-in. \times 0.39-in. \times 4-in. (50-mm \times 10-mm \times 100-mm) longitudinal gussets were welded all around with 0.24-in. (6-mm) manual fillet welds.

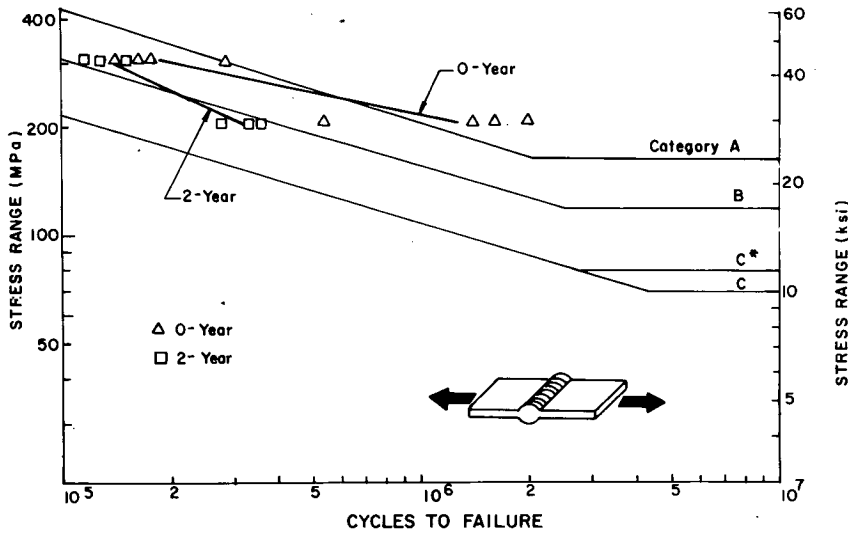


Figure 88. Fatigue strength of groove-welded plates fabricated from A588 steel. [Blake 1982]

The fatigue test data for the transverse stiffeners are plotted in Figure 89. Two-year and 4-year weathering increased the fatigue life of the stiffeners by 51 percent and 91 percent, respectively. (See points 21 and 22 in Figure 94.) The gain in fatigue strength after weathering pushed the data points for stiffeners above the Category B allowable line.

The data for the longitudinal gussets showed 70 percent and 39 percent increases in fatigue life after 2-year and 4-year weathering, respectively. (See points 23 and 24 in Figure 94.)

Yamada attributed the gain in life to weld-toe rounding due to corrosion which reduced the stress concentration factor and thus increased the crack initiation life. Measurements showed that the root radius at points along the toe of the stiffener weld increased from 0.028 in. (0.7 mm) for nonweathered to 0.060–0.120 in. (1.5–3.0 mm) for weathered specimens. According to Yamada, a gain in life could not always be expected because the weld toe radii did not increase along the full weld length.

Therefore, the scatterband of the weathered specimens data expanded upward. But the lower confidence limit for the weathered specimen data remained unchanged from that for the non-weathered specimens. The measured pit depths on the surface of the base metal increased from 0.004 in. (0.11 mm) before weathering to 0.008 in. (0.21 mm) after 4-year weathering. Pit depths were not measured near the weld toes where the fatigue cracks initiated.

An additional 10 specimens with transverse stiffeners were cut from a temporary SMA 50 steel bridge that had been used in Tokyo for 5.5 years. The specimen dimensions were identical to those of the transverse stiffeners fabricated for the research program. The fatigue test data fell above the Category C allowable line. Because there are no 0-year control data in this case, it cannot be determined if the bridge specimens gained or lost life after weathering.

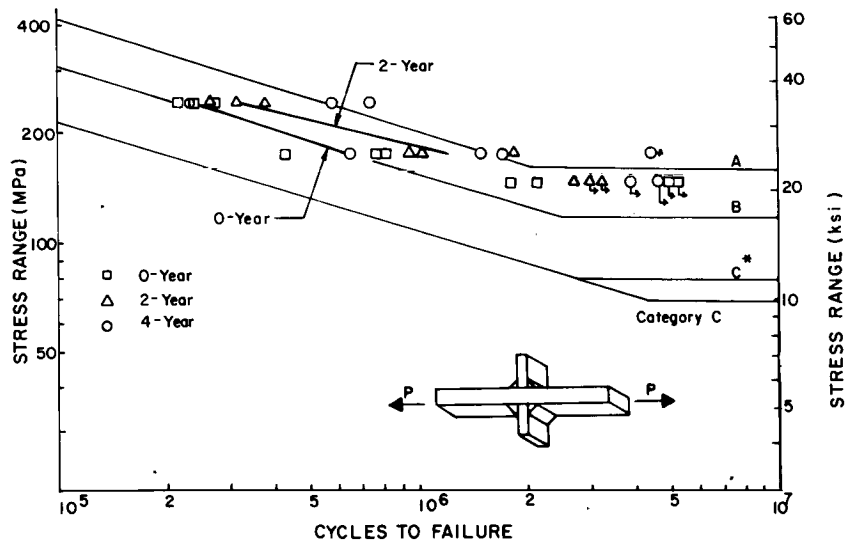


Figure 89. Fatigue strength of transverse stiffeners fabricated from SMA 50 steel. [Yamada 1983]

Bead Welds and Groove Welds

Barsom [1983] reported data for 86 groove-welded and over 47 welded specimens that were fatigue tested after 11-years exposure in rural, urban-industrial, and moderate marine environments. The reinforcements of the bead and groove welds were left in place. The specimens were made of A242 Type 1 and A588 Grade A steels.

Figure 90 shows, as an example, the fatigue test data for the groove-welded specimens made of A588 steel. Nine specimens tested at 70-ksi (483-MPa) stress range had lives shorter than 100,000 cycles. For this reason, their data points could not be plotted in the figure, but they were included in the regression analysis. Also shown, for comparison, is the Category A rhomboid. It encloses the 95.7 percent confidence space rolled beam fatigue test data from which the Category A allowable line was derived [Fisher 1970]. The findings are as follows:

1. Because no 0-year weathered specimens were tested, the effect of weathering on fatigue life cannot be determined from the data.
2. The groove welds had about equal fatigue lives after 11 years of weathering in the three environments, as indicated by the nearly identical fatigue notch factors ($K_f = 0.95 - 0.96$) in Table 43.
3. The data points fell above the Category A allowable line, although groove welds and bead welds are Category C details.

The reference [Barsom 1983] data raise questions about the validity of the results. Bead and groove welds with reinforcement not removed have Category C strength according to the AASHTO specifications. Yet, the reference [Barsom 1983] welds had Category A strength, that is 7.3 times longer allowable fatigue lives than Category C. At the 40-ksi (276-MPa) stress range level in Figure 90, where the data [Barsom 1983 and

Fisher 1970] overlap, the groove welds in the three environments had 2.0, 3.2, and 3.2 times longer mean fatigue lives than the Category A rolled beams. Even at the 70-ksi (483-MPa) stress range, that is 12 ksi (83 MPa) higher than the measured 58-ksi (400-MPa) yield strength of the A242 and A588 steel specimens tested in Barsom [1983], twice as many specimens fell above than below the Category A allowable line.

The lack of nonweathered specimen tests and the unlikely results make the data unsuitable for determining the effect of weathering on fatigue.

Cover Plates

Fisher weathered for 2 years one coverplated W36 beam fabricated from A588 steel [Fisher 1983a]. The coverplated tension flange faced the sky during weathering and the ground during fatigue testing. To artificially accelerate the weathering process, the cover plate ends were packed with rock salt (NaCl) once a month, a procedure that caused the ends to corrode very severely. Fatigue cracks initiated at the root of the end welds and propagated across the full cover plate width at both ends in 8,300,000 and 8,800,000 cycles of 6-ksi (41-MPa) stress range. The test was then discontinued. Root cracks are usually a sign of high fatigue strength. In this case, however, the two weathered cover plate ends had, on average, shorter fatigue lives than those on nonweathered beams that failed from weld toe cracks after 10,000,000 cycles or more. The conditions that existed in these tests are not relevant to bridges for the following reasons: (1) salt in the quantities deposited on the top flange of the upside-down beam cannot adhere to the underside of the bottom flange of a bridge girder; (2) artificial weathering cannot simulate natural weathering; and (3) bridges would not be built from non-painted weathering steel in an environment like the one created for the test.

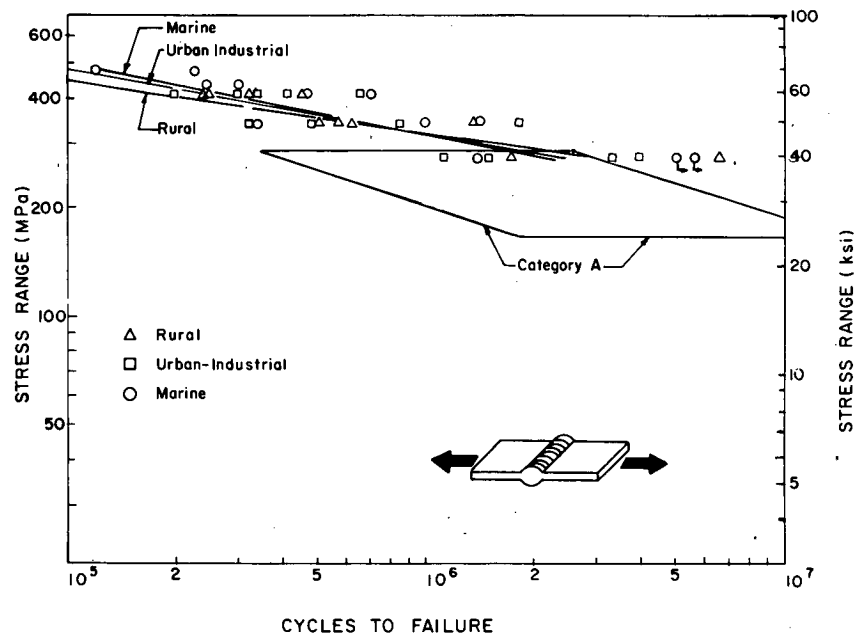


Figure 90. Fatigue strength of 11-year groove-welded specimens fabricated from A588 steel. [Barsom 1983]

NONATMOSPHERIC CORROSION RESISTANT STEELS

Three Japanese studies examined the effect of weathering on the fatigue life of specimens fabricated from rolled steel for welded structures supplied to Japanese Industrial Standard G3106, designation SM 50 and SM 58 [JIS 1977]. These steels do not have the atmospheric corrosion resistance of their SMA counterparts [JIS 1981]. They are comparable to ASTM A441 steel. The results of all regression analyses are given in Table 41. The calculations of change in stress range and life due to weathering are given in Table 43.

The following types of specimens, shown in Figure 78, were tested: base metal [Kunihiro 1972], groove weld ground flush [Kunihiro 1972], groove weld with reinforcement not removed [Nihei 1978], notched plate [Nihei 1978], transverse stiffeners [Yamada 1983b], and longitudinal gussets [Yamada 1983b]. The data are examined in the following by investigator.

Base Metal and Groove Welds

Kunihiro, et al., repeated for SM steels the same experimental program described earlier under "Weathering Steels" for SMA steels [Kunihiro 1972]. The plots of the data for the SM steels were very similar to those for SMA steels shown in Figures 79 and 80. The findings are as follows:

1. The mean fatigue lives of the nonweathered base metal and groove-welded specimens were slightly longer than the Category A mean, as indicated by the fatigue notch factors, $K_f = 0.98$ and 0.97 , less than unity (Table 44). Only 4.8 percent (3 out of 62) and 4.0 percent (2 out of 50) of the data points, respectively, fell below the Category A allowable line.
2. Both types of specimens exhibited about the same fatigue strength.
3. Two-year and 4-year weathering reduced the fatigue life of the base metal specimens by 58 percent and 62 percent, respectively, with 24.6 percent (17 out of 69) and 18.8 percent (16 out of 85) of the corresponding data points falling below the Category A allowable line. (See Table 44 and points 25 and 26 in Figure 94.)
4. The losses in life were 60 percent and 64 percent for the groove-welded specimens, with 23.9 percent (17 out of 71) and 28.4 percent (25 out of 88) of the corresponding data points falling below the Category A allowable line. (See points 27 and 28 in Figure 94.)
5. The loss in life was about the same after 2-year and 4-year weathering.

Notched Plates and Groove Welds

Nihei, et al., tested flat plate tensile specimens fabricated from SM 50 A steel [Nihei 1978]. The principal variables in the experiment were: (1) notched specimens versus groove-welded specimens; (2) minimum-to-maximum stress ratios of -1.0 and 0.0 ; and (3) 0-year versus 3-year weathering. The notches were 0.5 in. (12.5 mm) deep and had a 1.37-in. (35-mm) radius, for a theoretical stress concentration factor of 1.22. The weld reinforcement on the full penetration groove welds was not removed. The stress reversal data, $R = -1$, for the notched

specimens were analyzed in this study in terms of the tension portion of the stress cycle ($f_c/2$). The stress reversal data for the groove-welded specimens were analyzed in terms of the full stress range because residual tensile stresses elevate the applied stresses. All of weathered specimens were fatigue tested to failure after 3-years exposure. As an example, the data for the notched plate specimens, tested at $R = 0.0$, were plotted in Figure 91. The findings are as follows:

1. The fatigue strength of the nonweathered notched specimens ($K_f = 0.69$ for $R = 0.0$) exceeded that of Category A. Three-year weathering reduced the fatigue life by 74 percent and 58 percent for $R = 0$ and $R = -1$, respectively. (See points 29 and 30 in Figure 94.)
2. Three-year weathering reduced the fatigue life of the groove-welded specimens by 44 percent and 67 percent for $R = 0$ and $R = -1$, respectively (See points 31 and 32 in Figure 94.)

Transverse Stiffeners and Longitudinal Gussets

Yamada and Kikuchi repeated for SM steels the same experimental program described earlier under "Weathering Steels" for the SMA steels [Yamada 1983b]. The plots of the data were similar to that shown in Figure 89. The findings are as follows:

1. Two-year and 4-year weathering increased the fatigue life of the stiffeners by 120 percent and 91 percent, respectively. (See points 33 and 34 in Figure 94.)
2. Four-year weathering increases the fatigue life of the gussets by 11 percent. (See point 35 in Figure 94.) No 2-year weathered gussets were tested.

The reasons for the observed gains in life, given by the investigators, were already cited under "Weathering Steels."

EFFECT OF PITTING

The observed losses in life of specimens fabricated from weathering steels and nonatmospheric corrosion resistant steels are likely caused by pitting, as the work by Hiam and Pietrowski shows [Hiam 1978]. In that study, specimens were fabricated from a 0.06 percent C hot-rolled low-carbon (HRLC) mild steel of 36 ksi (250-MPa) yield strength and a silicon semikilled columbium-strengthened HSLA steel of 50-ksi (345-MPa) minimum specified yield strength, designation Dofascoloy 50W. The specimens were fatigue tested in the following conditions: (1) hot rolled surfaces; (2) laboratory pitted; (3) corroded under vehicle; and (4) corroded surfaces removed. The following conclusions were reported:

1. Fatigue cracks initiated from rust pits.
2. Pit shape and type of fatigue failure were similar for specimens that were laboratory pitted or corroded under a vehicle. Both produced similar losses in fatigue life.
3. The loss in life increased as the pit depth increased.
4. Removal of the corroded surfaces restored the original fatigue strength of the specimens.
5. The loss in life was greatest at low-strain amplitude, that is in the long-life regime (100,000 to 10,000,000 cycles).

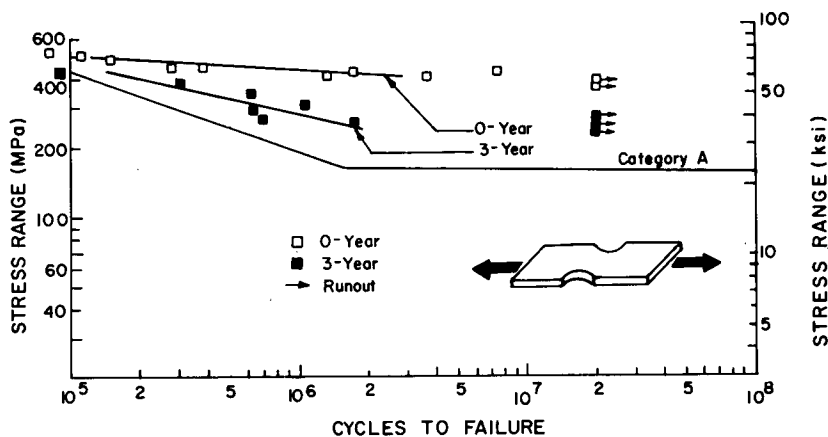


Figure 91. Fatigue strength of 3-year weathered notched plate specimens fabricated from SM 50 steel. [Nihei 1978]

6. There was no significant difference between the losses in life for the two steels.

Figures 92 and 93 give the results of the pit depth studies on the HRLC and HSLA steels, respectively. Both show the same trend of reduced fatigue life as pit depth increased. The summary of loss in life, given in Table 42, reveals that the two steels behaved alike, with losses of 78 percent and 84 percent at 0.009-in. (230- μm) pit depth [Hiam 1978]. In comparison, low-alloy steels reached that pit depth after less than 3-years exposure in industrial Bayonne, N.J., as shown in Figure 70 [Copson 1960].

The following data from other sources indicate the degree of pitting penetration that can be expected under various conditions: (1) 15 mils (370 μm) for low-alloy steel ideally exposed 18.1 years in industrial Bayonne, N.J. [Copson 1960]; (2) 28 mils (707 μm) for low-alloy steel ideally exposed 17.1 years in marine Block Island, R.I. [Copson 1960]; and (3) up to 250 mils (6,000 μm) for weathering steel bridges in Michigan exposed for 15 years to severe deicing salt environments [Culp 1980]. Evidently, pitting penetration of environmentally exposed weathering steel can be much larger than that of the reference [Hiam 1978] panels.

Since pitting penetration increases with time (Fig. 70), and loss in life increases with pit depth (Figs. 92 and 93), one would expect a continuing degradation of the fatigue strength of non-painted weathering steel bridges, although at a declining rate. The data [Albrecht 1983a] for automatically welded stiffeners show, in fact, a continuing degradation in mean fatigue strength (Figs. 82 to 84) and lower confidence limit (Fig. 86) in 3-year and 8-year weathered specimens. Sixteen-year data from current studies at the University of Maryland will not be available until 1992.

COMPARISON OF WEATHERING FATIGUE DATA

The losses in life for the 24 series of weathering steel specimens and the 11 series of nonatmospheric corrosion resistant steel specimens are summarized in Tables 43 and 44, respectively. The losses are plotted in Figure 94 against the fatigue notch

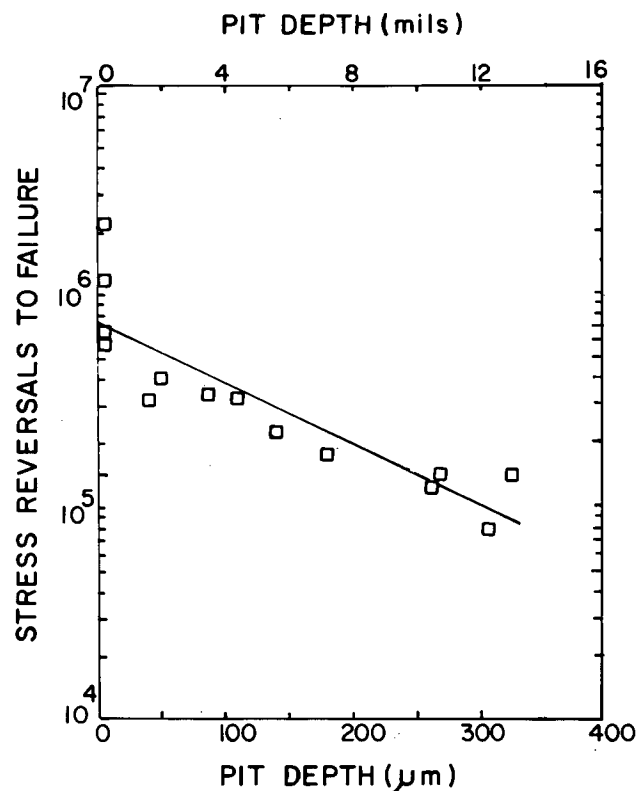


Figure 92. Fatigue life versus pit depth for HRLC mild steel.

factor (Eq. 33) of the corresponding nonweathered control specimens. The data points for the 24 A588 and SMA weathering steel series are shown with triangular symbols, those for the 11 SM steel series with circular symbols. Each point represents the average loss in life for one series of tests. These 35 points are based on an analysis of 1,431 fatigue tests, not counting over 132 fatigue tests performed by Barsom for which there were no 0-year weathered data. The data base for determining the effect

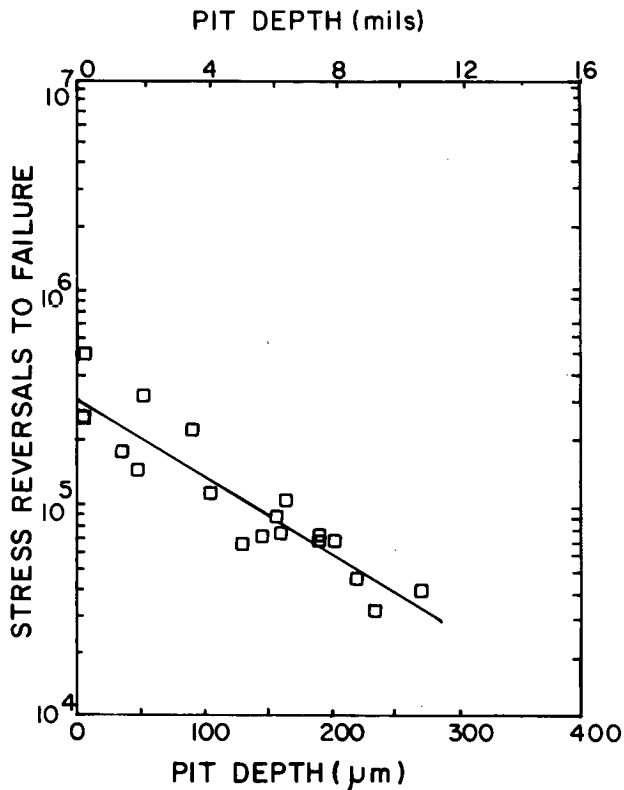


Figure 93. Fatigue live versus pit depth for HSLA steel, Dofacoly 50W.

Table 42. Loss in fatigue life due to corrosion pitting. [Hiam 1978]

Pit Depth (m)	HRLC Steel		HSLA Steel	
	Fatigue Life (cycles)	Loss in Life (%)	Fatigue Life (cycles)	Loss in Life (%)
0	740,000	--	292,000	--
50	529,000	29	194,000	33
100	378,000	49	129,000	56
230	164,000	78	46,900	84

of weathering on fatigue is very large. There are currently about three times as many tests as the 499 data points that were used to derive the Category A to E allowable lines [Fisher 1970 Fisher 1974].

The vertical grid lines in Figure 94 were drawn at the values of K_f for the Category A to E mean lines, given in Table 39. This allows one to compare the data for the weathered specimens with the data base for the current fatigue specifications. Note that the data for the 35 series were plotted against the actual experimentally determined fatigue strength of the nonweathered specimens, not against projected strength implied by the AASHTO classifications. For example, the points for the higher strength automatically welded stiffeners were plotted at $K_f = 1.67$, those for the lower strength manually welded stiffeners at $K_f = 2.04$.

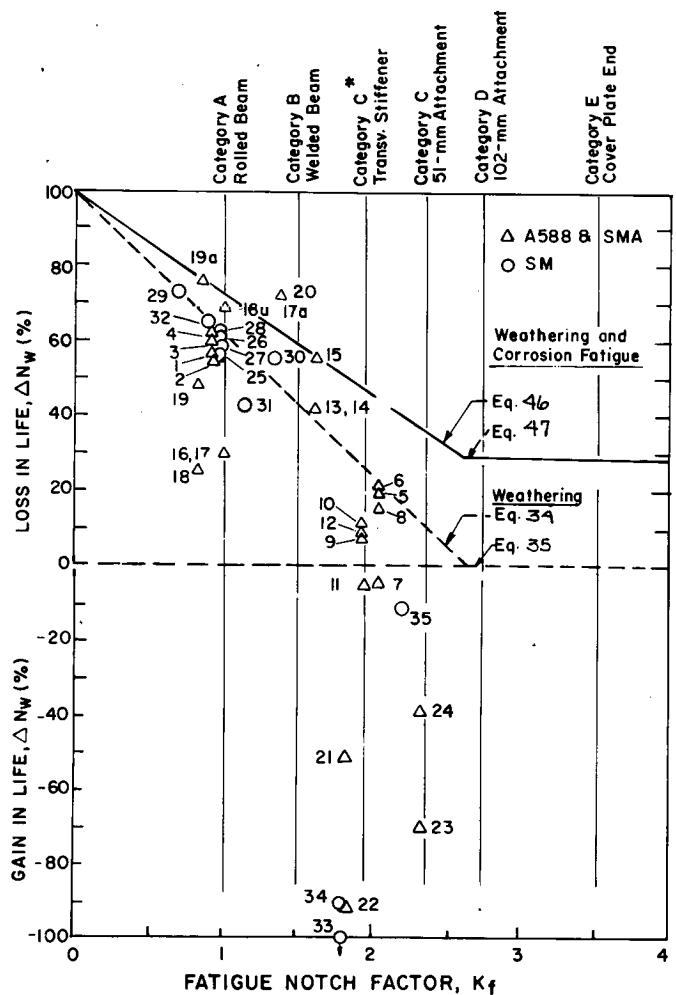


Figure 94. Loss in life due to weathering fatigue.

The comparison of the results from the 35 series of tests, shown in Figure 94, leads to the following conclusions:

1. The loss in life is highest for base metal and continuously decreases as the stress concentration of the detail increases. In other words, the higher the stress concentration of a detail, the less rust pitting reduces the life.
2. Atmospheric exposure affects weathering steels and regular steels alike.

Yamada's results [1983b] are not consistent with those obtained by the Japanese Ministry of Construction, University of Maryland, and U.S. Steel [Kunihiro 1972; Albrecht 1980, 1982a, and 1983a; Blake 1982]. His results show large gains in life, whereas all others show losses in life. The weld-toe rounding, to which Yamada attributed the gains in life, did not seemingly occur in the specimens tested by others. The scatter in his series of tests (points 21-24 and 33-35 in Figure 94) is much larger than other investigators have found (points 1-4, 5-8, 9-12, 13-15, 16-19, and 25-28). The weld-toe rounding may have been caused by galvanic corrosion between the weathering steel base and a less noble weld metal.

Table 43. Effect of weathering on fatigue life of specimens fabricated from weathering steels.

Symbol in Figure 94	Reference	Type of Detail	Weathering Time (Years)	Stress Range at 500,000 Cycles f_r (MPa)	Loss in Stress Range f_r^w (%)	Loss in Fatigue Life N_w (%)	Fatigue Notch Factor K_f
1	Kunihiro 1972	Base metal	0	373	--	--	0.94
			2	287	23	57	1.22
			4	294	21	54	1.19
3	Kunihiro 1972	Groove welds ground flush	0	380	--	--	0.92
			2	285	25	60	1.23
			4	278	27	63	1.26
5	Albrecht 1982a	Manually welded transverse stiffeners	0	172	--	--	2.04
			2	160	7	21	2.18
			4	159	8	22	2.19
			2 alt.	175	-2 ^a	-6 ^a	2.00
8	4 alt.	163	5	16	2.15		
9	Albrecht 1982a	102-mm attachments	0	179	--	--	1.95
			2	177	--	6	1.98
			4	173	4	10	2.03
			2 alt.	--	--	-8 ^a	--
12	4 alt.	--	--	--	9	--	
13	Albrecht 1980, 1983a	Automatically welded transverse stiffeners	0	210	--	--	1.67
			3	177	16	42	1.98
			3 alt.	177	15	42	1.98
			8	165	21	54	2.13
16	Blake 1982	Base metal as received	0	352	--	--	1.00
			3	318	10	28	1.10
			6	318	10	28	1.10
18	Blake 1982	Base metal blast cleaned	0	418	--	--	0.84
			2	382	9	24	0.92
			4.5	343	18	47	1.02
20	Blake 1982	Groove welds	0	252	--	--	1.39
			2	170	33	72	2.07
21	Yamada 1983b	Transverse stiffeners	0	193	--	--	1.82
			2	220	-14 ^a	-51 ^a	1.60
			4	--	--	-91 ^a	--
23	Yamada 1983b	100-mm gussets	0	150	--	--	2.83
			2	177	-18 ^a	-70 ^a	1.98
			4	167	-11 ^a	-39 ^a	2.10
24	Barsom 1983	Groove welds	11-R ^b	364	--	--	0.96
			11-UI	365	--	--	0.96
			11-M	369	--	--	0.95
25	Barsom 1983	Groove welds	11-R	366	--	--	0.96
			11-UI	372	--	--	0.94
			11-M	414	--	--	0.85
26	Barsom 1983	Bead welds	11-R	369	--	--	0.95
			11-UI	398	--	--	0.88
			11-M	419	--	--	0.84
27	Barsom ^c 1983	Bead welds	11-R	--	--	--	--
			11-UI	--	--	--	--
			11-M	--	--	--	--

Notes

- a. Gain in life and stress range.
b. R = rural, UI = urban industrial, M = moderate marine.
c. Fatigue test data could not be obtained from the investigator.

On the basis of the data available in 1981, Albrecht had drawn the following upper-bound envelope for relative loss due to weathering [Albrecht 1982a, Albrecht 1983a]:

$$\Delta N_w = 1 - 0.38 K_f \quad (34)$$

Equation 34 is based on data in the region of $0.69 \leq K_f \leq 2.04$. It intersects the abscissa ($\Delta N_w = 0.0$) at $K_f = 2.63$. Above that value, the high stress concentration of the detail was assumed to mask the loss in life due to rust pitting, so that for $K_f \geq 2.63$:

$$\Delta N_w = 0.0 \quad (35)$$

Equations 34 and 35 are plotted with dashed lines in Figure 94. The only data point (30) above Eq. 34 in Figure 94 had

been ignored because of the uncertainty in using the stress amplitude, $f_a = 0.5 f_r$, to calculate K_f for the notched plate specimens tested under stress reversal, $R = -1$ [Nihei 1978]. Fatigue test data reported since 1981 for base metal (points 16a, 17a, and 19a), groove welds (point 20) and stiffeners (point 15) have yielded five more points that exceed the losses predicted by Eq. 34. In light of the new data, this equation now appears to be a nonconservative upper-bound on the fatigue strength of weathering steel specimens that were weathered at most 8 years. It is not yet possible to quantitatively determine from the short-term exposure data the loss in fatigue life that bridges might exhibit after 50 years of service. Present indications are that the long-term losses will be greater.

The weathering fatigue data are used in Chapter Eight, in conjunction with the corrosion fatigue data, to determine the allowable stress ranges for fatigue design of weathering steel structures.

Table 44. Effect of weathering on fatigue life of specimens fabricated from SM steels with no atmospheric corrosion resistance.

Symbol in Figure 94	Reference	Type of Detail	Weathering Time (Years)	Stress Range at 500,000 Cycles f_r (MPa)	Loss in Stress Range Δf_r^w (%)	Loss in Fatigue Life ΔN_w (%)	Fatigue Notch Factor K_f
25	Kunihiro 1972	Base metal	0	359	--	--	0.98
			2	274	24	58	1.28
			4	265	26	62	1.32
27	Kunihiro 1972	Groove weld ground flush	0	362	--	--	0.97
			2	272	25	60	1.29
			4	264	27	64	1.33
29	Nihei 1978	Notched plate	0	506	--	--	0.69
			3	332	34	74	1.05
30	Nihei 1978	Notched plate R = -1	0	257 ^b	--	--	1.36 ^b
			3	196 ^b	24	58	1.79 ^b
31	Nihei 1978	Groove weld as welded	0	302	--	--	1.16
			3	252	17	44	1.39
32	Nihei 1978	Groove weld as welded R = -1	0	396 ^c	--	--	0.88 ^c
			3	279 ^c	30	67	1.25 ^c
33	Yamada 1983b	Transverse stiffener	0	194	--	--	1.81
			2	--	--	-120 ^a	--
34	Yamada 1983b	100-mm gusset	0	160	--	--	2.19
			4	165	-3 ^a	-11 ^a	2.12

Notes:

- a. Gain in fatigue life
- b. Based on stress amplitude
- c. Based on stress range

CHAPTER EIGHT

FATIGUE DESIGN

The total fatigue life consists of a crack initiation phase plus a crack propagation phase. Weathering, as discussed in Chapter Seven, reduced only the crack initiation phase, because the specimens were fatigue tested after the end of the weathering period. In reality, weathering steel bridges are exposed to the environment during both phases of fatigue cracking. The latter effect, termed corrosion fatigue, affects both the crack initiation and propagation phases.

Chapter Eight first presents the available corrosion fatigue data. This is followed by an estimation of total loss in life and an examination of allowable stress ranges.

CORROSIVE FATIGUE

The effect of a corrosive environment on fatigue life can be assessed by comparing data collected in clean air and in the environment of interest. The following types of data are available:

1. Corrosion fatigue crack initiation (CFCI) life.
2. Corrosion fatigue crack propagation (CFCP) rate.
3. Corrosion fatigue S-N (CFS-N) data.

The first two types are obtained from tests of standardized specimens. They provide information on crack initiation and crack propagation in a given structural steel. The combined data can be used, in conjunction with mathematical models of initiation and propagation lives to determine the effect of corrosion on the total fatigue life of a detail.

The third type of data is obtained by fatigue testing a detail to failure in the corrosive environment of interest. Such tests give directly the effect of corrosion on the total fatigue life. The three types of data are examined in the following to assess the corrosion fatigue life of weathering steel bridges.

Corrosion Fatigue Crack Propagation

Barsom and Novak published CFCP data in the form of plots of crack growth rate, da/dN , versus range of stress intensity factor, ΔK , for A36, A588 Grades A and B, and A514 steels. They were tested in air, distilled water, and 3 percent sodium chloride solution [Barsom 1977]. They presented no analysis of the data and have since discarded the raw data. Therefore, a total of 1,651 data points had to be digitized from the plots of the reference [Barsom 1977] for analysis in the present study

[Yazdani 1983]. The log-log mean regression lines were calculated for all steels and environments using the Paris equation:

$$\frac{da}{dN} = C \Delta K^n \quad (36)$$

in which C and n are material constants. The statistical difference between any two mean lines was determined with T-tests at the 5 percent level of significance.

The crack growth rate data for A588 steel in air and aqueous environments are plotted in Figures 95 and 96. The results of the regression analyses and statistical comparisons, given in Table 45, lead to the following conclusions:

1. The difference in crack growth rate of A588 steel in distilled water versus 3 percent sodium chloride solution was insignificant. The data for the two aqueous environments were, therefore, combined for comparison with the air data.
2. The crack growth rate for A588 steel in aqueous environments

$$\frac{da}{dN} = 3.77 \times 10^{-12} (\Delta K)^{3.320} \quad (37)$$

was significantly higher than the crack growth rate for A588 steel in the air environment

$$\frac{da}{dN} = 1.65 \times 10^{-12} (\Delta K)^{3.475} \quad (38)$$

Substituting ΔK in units of $\text{MPa}\sqrt{\text{m}}$ gives da/dN in units of m/cycle . The aqueous environments increased the crack growth rate by a factor of 1.48 at the lowest value, $\Delta K = 14.8 \text{ MPa}\sqrt{\text{m}}$ ($13.5 \text{ ksi}\sqrt{\text{in.}}$), at which data were collected.

3. The statistical conclusions for A588 steel, drawn from items 1 and 2, are also valid for A36 and A514 steels (Table 45).
4. The difference in crack growth rate for A36 and A588 steels in air or aqueous environments was insignificant (Table 45).

Including the previously mentioned data from Barsom (1977), Yazdani and Albrecht collected a total of 3,254 data points for

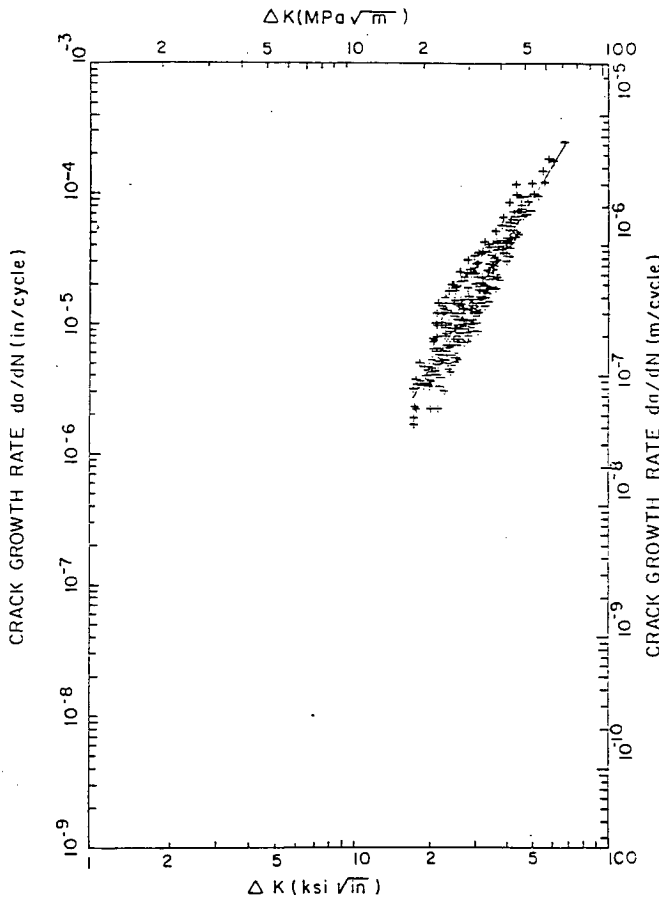


Figure 95. Crack growth rates for A588 steel in air. [Barsom 1977]

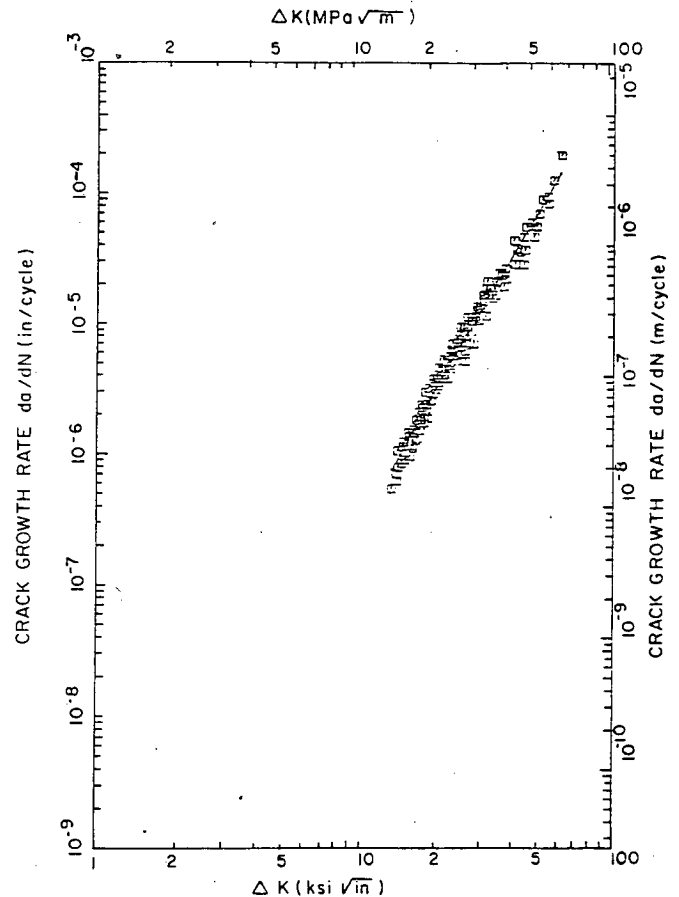


Figure 96. Crack growth rates for A588 steel in aqueous environments. [Barsom 1977]

Table 45. Regression analysis and statistical comparison of crack growth rate data for A36, A588 Grades A and B, and A514 steels.

Steel and Environment ^a	No. of Data Points	Results of Regression Analysis ^b		Standard ^c Deviation	T Statistic	T ^d Critical	Variation Significant
		c	n				
A36 H ₂ O vs. A36 NaCl	29 161	5.58 E-13 4.33 E-12	3.877 3.152	.0557 .1370	2.06 1.59	+2.08	NO
A588 H ₂ O vs. A588 NaCl	44 271	1.10 E-12 4.18 E-12	3.610 3.300	.0742 .1593	1.95 0.54	+2.00	NO
A514 H ₂ O vs. A514 NaCl	58 426	3.86 E-11 7.11 E-11	2.482 2.407	.0851 .1813	0.45 0.73	+1.97	NO
A36 Air vs. A36 Aqueous	92 190	1.05 E-12 4.71 E-12	3.609 3.221	.0811 .1336	9.25 8.46	+1.99	YES Aqueous faster
A588 Air vs. A588 Aqueous	168 315	1.65 E-12 3.77 E-12	3.475 3.320	.0732 .1552	2.73 1.83	+1.98	YES Aqueous faster
A514 Air vs. A514 Aqueous	372 484	5.28 E-12 6.02 E-11	3.026 2.420	.0883 .1563	27.15 11.27	+1.96	YES Aqueous faster
A36 Air vs. A588 Air	92 168	1.05 E-12 1.65 E-12	3.609 3.475	.0811 .0732	1.15 0.38	+1.99	NO
A36 Aqueous vs. A588 Aqueous	190 315	4.71 E-12 3.77 E-12	3.221 3.320	.1336 .1552	0.77 1.31	+1.97	NO

Notes:

- H₂O = distilled water; NaCl = 3% sodium chloride solution.
- Substituting K values in units of MPa√m yields crack growth rates, da/dN, in m/cycle.
- Standard deviation of log (da/dN).
- Critical value at 5% level of significance.

crack growth rate of mild steel (A36), high strength low alloy steels (A588, X-52), and quenched and tempered steel (A514) in air and aqueous environments [Yazdani 1983]. The aqueous environments consisted of distilled water and a solution of sodium chloride in distilled water. After a statistical analysis of the effects of type of steel, loading, environment and source, the data were combined into four sets. Correspondingly, the following crack growth equations were obtained for mild and HSLA steels in air:

$$\frac{da}{dN} = 1.54 \times 10^{-12} (\Delta K)^{3.344} \quad (39)$$

For mild and HSLA steels in aqueous environments:

$$\frac{da}{dN} = 4.16 \times 10^{-12} (\Delta K)^{3.279} \quad (40)$$

For quenched and tempered steels in air:

$$\frac{da}{dN} = 2.27 \times 10^{-11} (\Delta K)^{2.534} \quad (41)$$

For quenched and tempered steels in aqueous environments:

$$\frac{da}{dN} = 6.00 \times 10^{-11} (\Delta K)^{2.420} \quad (42)$$

The pairs of air and aqueous lines for each group of steels, plotted in Figure 97, are practically parallel. As in the case of the Barsom [1977] data for A588 steel alone (Eqs. 37 and 38), the crack growth rates for the combined mild and HSLA steels and for the quenched and tempered steels are higher in the aqueous environments than the corresponding growth rates in air. Both sets of equations suggest that structural details on nonpainted steel bridges have shorter crack propagation lives than those on painted bridges.

Fisher, on the other hand, stated that the investigation by Barsom and Novak [Barsom 1977] had concluded that the CFCP life of steel bridge components under actual operating (wet and dry) conditions in the aqueous environments tested should be equal to or greater than their fatigue life in room temperature air environments [Fisher 1983b]. This conclusion was based on an interrupted test of a single A514 steel specimen in salt water. In this test, the subsequent decrease in crack growth rate had originally been attributed to an overnight air exposure of the specimen during an equipment malfunction. However, according to Novak, the crack growth delay is now believed to have been caused by an overload accidentally applied on the specimen when the equipment malfunctioned [Novak 1983a, Albrecht 1983c]. Single overloads are known to delay crack growth under subsequent cycling at lower stress ranges [Abtahi 1976].

The greatest part of the fatigue life of bridge details is spent initiating fatigue cracks and growing them at values of ΔK lower than those at which corrosion fatigue crack growth rates were measured in most previous tests. To determine the effect of corrosion on the total fatigue life, it is necessary to also examine the available CFCI data and the available CFS-N data.

Corrosion Fatigue Crack Initiation

Novak determined the CFCI life of A36, A588 Grade A, and A517 Grade F steel specimens tested in a 3.5 percent sodium chloride solution under constant-amplitude cyclic loading and a realistic cyclic frequency of 12 cpm [Novak 1983b]. The CFCI life was defined as the number of cycles needed to initiate a crack from a notch with varying root radius (stress concentration) and to grow it to a surface length of between 0.030 and 0.070 in. (0.75 and 1.75 mm). The data were presented in plots of CFCI life versus the maximum stress range at the notch tip. The latter is equal to the nominal stress range times the theoretical stress concentration factor, $\Delta\sigma_{max} = \Delta\sigma_{nom} K_r$. An evaluation of the data showed that [Novak 1983b]:

1. The fatigue crack initiation (FCI) thresholds in air were $\Delta\sigma_{max} = 73, 90,$ and 124 ksi (505, 623, and 855 MPa), respectively, for A36, A588 Grade A, and A517 Grade F steels.
2. No CFCI thresholds were found for any of the steels tested

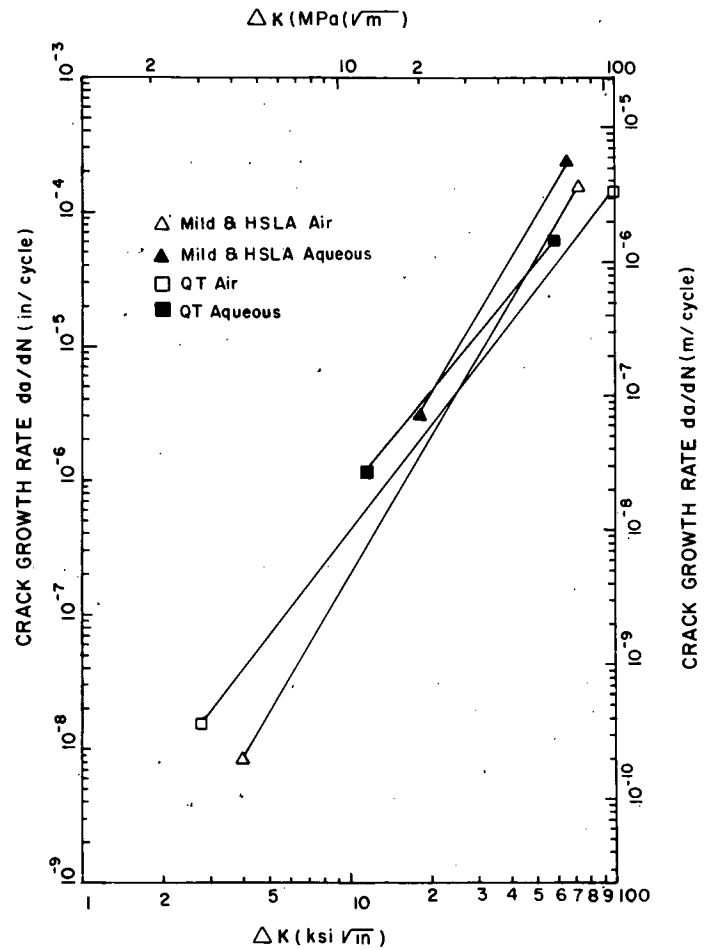


Figure 97. Comparison of crack growth rates for structural steels in air and aqueous environments. [Yazdani 1983]

in sodium chloride solution, despite strong attempts to characterize the long-life behavior of primary interest for structural applications such as bridges.

3. The long-life (3,000,000 cycles) cyclic strength for CFCI behavior was $\Delta\sigma_{max} = 34$ ksi (234 MPa) for all steels.

4. The degradation in cyclic load-carrying capacity, as determined by comparing the FCI and CFCI data, varied from a negligible amount at 1,000 cycles of loading to maximum amounts of about 54, 62, and 72 percent, respectively, for the A36, A588 Grade A, and A514 Grade F steels at 3,000,000 cycles.

Taylor and Barsom had reported similar findings for CFCI in A517 Grade F steel specimens [Taylor 1981].

The data show that the CFCI life of weathering steel details in an aqueous sodium chloride environment is much shorter than the fatigue crack initiation life in a clean air environment. There appears to be no fatigue limit in this aqueous environment. Therefore, the fatigue strength of long-life structures, such as bridges, in a corrosive environment may be much lower than the AASHTO fatigue limits for "over two million cycles" of loading which were based on the results of fatigue tests performed in clean laboratory air and intended for painted structures [Fisher 1970, Fisher 1974].

Corrosion Fatigue S-N Data

A further indication of how a corrosive environment might affect the total fatigue life (crack initiation plus propagation) of weathering steel details can be obtained by comparing the fatigue lives of specimens tested to failure in air and aqueous environments. Such S-N tests include crack initiation and crack propagation at all values of ΔK , from near threshold to failure.

Knight collected such data from the literature [Knight 1977]. Table 46 summarizes the data and the calculations performed herein. The fatigue notch factors were obtained by dividing the stress range in air at 2,000,000 cycles into the stress range for the Category A mean line at 2,000,000 cycles. The latter was calculated from Table 39:

$$f_{r,A} = (10^{13.785} / 2,000,000)^{1/3.178} = 32.8 \text{ ksi (226 MPa)}.$$

The loss in life was calculated from Eq. 32, using the stress ranges in air and in aqueous environments reported by Knight. The results, plotted in Figure 98, suggest the following conclusions:

1. Excluding the stress reversal data (points 5 and 10), the loss in life due to corrosion fatigue varied from 55 percent to 97 percent.
2. The loss in corrosion fatigue life did not significantly decrease with increasing fatigue notch factor, as was observed in the weathering fatigue tests (Fig. 94).
3. Salt-water environments reduced the corrosion fatigue life more than fresh-water environments.
4. Type of steel was not a significant variable.

As an example, Figure 99 shows the S-N data for welded cruciform joints fabricated from SM 50 structural steel corresponding to point 11 in Figure 98 (Serizawa 1973). In this case, salt-water immersion decreased the fatigue life by 84 percent at 2,000,000 cycles. It also drastically lowered the fatigue limit from about 18 ksi (123 MPa) in air to 7.5 ksi (52 MPa) in salt water, a finding consistent with the data for CFCI life [Novak 1983b, Taylor 1981].

Summary

All available data show large reductions in fatigue life of structural details in aqueous environments. This conclusion applies to all structural steels, not just weathering steel. No data reported in the literature show a beneficial effect of aqueous environments. It can be expected, therefore, that in corrosive environments weathering steel bridges will have a shorter fatigue life than painted bridges.

ESTIMATION OF LOSS IN LIFE

Estimates of the total fatigue life of weathering steel bridges should consider the effect of weathering fatigue on the crack initiation life, and the effect of corrosion fatigue on the crack initiation and crack propagation lives. The available data are of the following types:

1. Weathering fatigue life (Chapter Seven and Fig. 94).
2. Corrosion fatigue crack initiation life (Chapter eight; section under "Corrosion Fatigue").
3. Crack growth rates (Figs. 95, 96, 97).
4. Corrosion fatigue S-N data (Fig. 98).

Items 1 and 2 affect the crack initiation life, item 3 the crack propagation life, and item 4 the total life. These effects are not necessarily additive. The cyclic history of bridges is a complex phenomenon not easily modeled in short-term laboratory tests. Weathering steel bridges are subjected, at first, to weathering fatigue during construction and early years of service. If the stress cycles are sufficiently high to initiate fatigue cracks from weathering-induced rust pits and/or by corrosion fatigue crack initiation, the phenomenon gradually changes into one of corrosion fatigue crack propagation.

One can estimate the total corrosion fatigue life of a weathering steel bridge in two ways. In one, which parallels the current AASHTO approach of defining the life based on fatigue tests performed in air, one would repeat such tests in an aqueous environment. Figure 98 summarizes such early corrosion fatigue S-N data that were obtained from tests of specimens with Category A and B strength.

More recent data on welded joints with fatigue strengths comparable to those of Category C to E details were developed during extensive European research programs on offshore structures. As a result of such studies, the new British design rules for offshore structures now specify a 50 percent reduction in total fatigue life for all unprotected joints in sea water, that is [UK DOE 1981]:

$$\Delta N_r = 0.50 \quad (43)$$

Equation 43 is plotted in Figure 98. The same reduction for corrosion fatigue should also apply to bridges that are subjected to intermittent corrosion, because the crack creates its own environment at the tip, with a chemistry different from that of the bulk environment outside of the crack. In fact, differences in bulk environment were shown to have no significant effect on crack growth rate fatigue life as, for example, in the following steel-environment combinations:

1. Crack growth rates for BS 4360 Grade 500 structural steel in natural seawater, half diluted seawater, seawater with 2 mg/l HCO_3 , and 3.5 percent NaCl [Scott 1975].
2. Crack growth rates for A36, A588, and A514 steels in distilled water and 3 percent NaCl solution, Table 45 [Barsom 1977].
3. Corrosion fatigue (S-N) line for BS 4360 Grade 500 steel in seawater under freely corroding and intermittently corroding conditions [Booth 1983].
4. 5083-H116 aluminum, HY-80 steel, and 17-4 PH H1075 steel in natural seawater, ASTM seawater, and 3.5 percent NaCl solution [Bogar 1983].

Additional examples are given in Figure 98 and in the reference [Burnside 1983].

Furthermore, the capillaries in the oxide coating and the crack can retain humidity for long periods of time after precipitation or condensation has stopped. This humidity, in conjunction with

Table 46. Summary of corrosion fatigue data for specimens tested in both air and aqueous environments.

Point in Figure 98	Steel & Tensile Strength (MPa)	Type of Detail	Aqueous Environment	Stress Range at 2,000,000 Cycles		Loss in Life ΔN_{cf} (%)	Fatigue Notch Factor K_f
				In Air (MPa)	In Aqueous Environment (MPa)		
1	785	Base metal	Salt water immersion	430	235	86	0.53
2	785	Base metal	Dripping salt water	460	150	97	0.49
3	785	Base metal	Dripping fresh water	320	225	68	0.71
4	490	Base metal	Dripping salt water	300	160	87	0.76
5 ^a	0.35% C steel	Base metal	Synthetic tap water	454	423	20	0.50
6	785	Butt weld manual arc	Salt water immersion	180	110	79	1.26
7	785	Butt weld manual arc	Dripping fresh water	167	130	55	1.36
8	785	Butt weld submerged arc	Dripping fresh water	223	165	62	1.02
9	785	Butt weld manual arc toes ground	Dripping fresh water	266	192	65	0.85
10 ^a	Mild steel 410	Butt weld	Dripping salt water	200	162	49	1.13
11	490	Welded cruciform	Salt water immersion	153	87	84	1.48
12	Low-alloy 780	Welded "tee"	Dripping salt water	315	185	82	0.72

Notes

a. $R = -1$; all others $R = 0$.

chlorides and other corrosive contaminants trapped in the oxide coating, fosters corrosion beyond the time of surface wetness.

It is worth remembering that immersion in a 3 percent sodium chloride solution has been used in a past NCHRP study [Barsom 1977] and a current FHWA study [Roberts 1983] as a standard environment for determining the corrosion fatigue behavior of bridge steels. The water runoff from highway bridges can have a NaCl concentration higher than the 3.5 percent NaCl concentration in seawater. Therefore, the previously mentioned data and Eq. 43 are also indicative of the loss in life due to corrosion fatigue that can be expected for all structural details in weathering steel bridges.

Another method of estimating the total corrosion fatigue life of a weathering steel bridge was proposed by Albrecht. It assumes that weathering fatigue consumes the crack initiation life and corrosion fatigue reduces the crack propagation life, that is the net life after the loss due to weathering [Albrecht 1982a,

Albrecht 1983b]. From the data plots of Barsom [1977], Albrecht had estimated that aqueous environments increase the crack growth rate of A588 steel in air by a factor of $\sqrt{2} = 1.41$. Recent analysis of the data, presented earlier in this chapter under "Corrosion Fatigue," showed that the factor was 1.48 at the lowest ΔK values tested and more at higher ΔK values [Yazdani 1983].

The relative loss in life due to weathering is given by Eqs. 34 and 35. The relative loss in life due to corrosion fatigue is, therefore, $0.41/1.41 = 29$ percent of the relative net life after weathering. For $K_f \leq 2.63$:

$$\Delta N_{cf} = 0.29 (1 - \Delta N_w) = 0.11 K_f \quad (44)$$

and, for $K_f > 2.63$:

$$\Delta N_{cf} = 0.29 \quad (45)$$

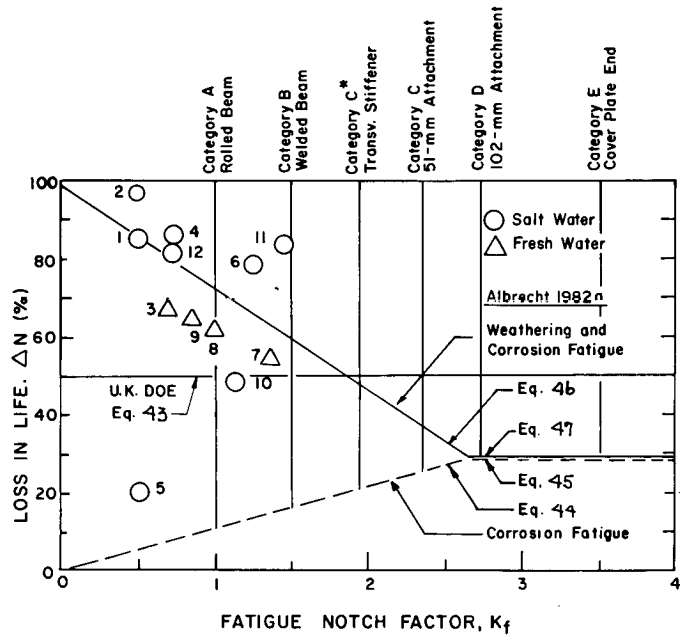


Figure 98. Loss in life due to corrosion fatigue.

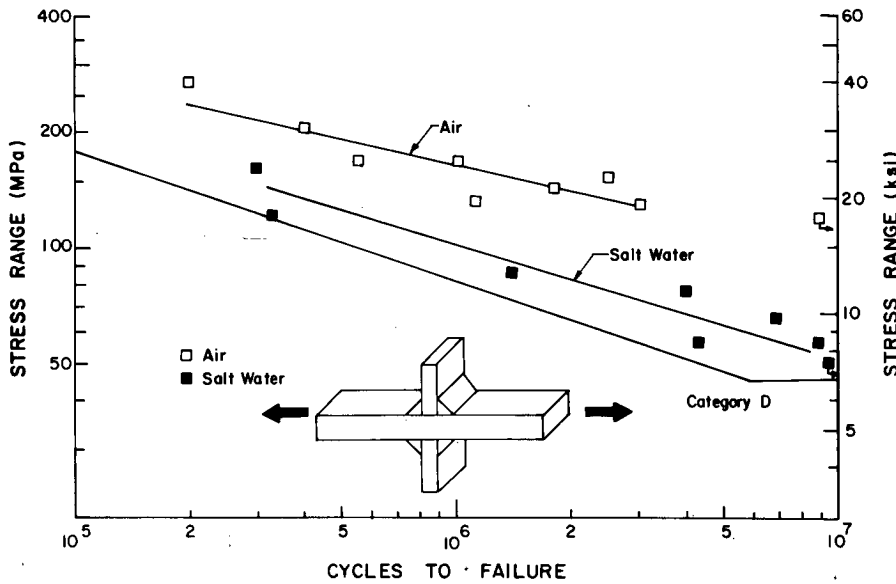


Figure 99. Corrosion fatigue strength of SM 50 welded cruciform joints immersed in salt water. [Serizawa 1973]

Equations 44 and 45 for loss in CFCP life are plotted with a dashed line in Figure 98. The loss predicted from crack growth rates greatly underestimates the loss in total corrosion fatigue life (crack initiation and propagation) that was determined from S-N data. Adding Eqs. 45 and 46 to Eqs. 34 and 35, respectively, gives the total relative loss in life due to weathering and corrosion fatigue. For $K_f \leq 2.63$:

$$\Delta N_t = \Delta N_w + \Delta N_{cf} = 1 - 0.27 K_f \quad (46)$$

and, for $K_f > 2.63$:

$$\Delta N_t = 0.29 \quad (47)$$

Equations 46 and 47, originally developed in 1981, were upper-bound estimates of the loss in life due to weathering and corrosion fatigue for data available at that time. Including the more recent weathering fatigue data and the corrosion fatigue data, respectively, Figures 94 and 98 show that Eq. 46 is no longer an upper-bound estimate of the loss in life due to weathering and corrosion fatigue. Equation 46 is nonconservative. It underestimates the experimentally determined losses.

Substituting the fatigue notch factors from Table 39 into Eq.

46 gives the following predicted losses in life: 73 percent for Category A, 60 percent for Category B, 36 percent for Category C, and 29 percent for Categories D to E prime details. These values are the intersection points of Eq. 46 with the vertical grid lines in Figure 98.

Subtracting Eqs. 46 and 47 from unity yields the total relative net life after weathering and corrosion fatigue. For $K_f < 2.63$:

$$N_i = 1 - N_i = 0.27 K_f \quad (48)$$

and, for $K_f > 2.63$:

$$N_i = 0.71 \quad (49)$$

Since fatigue life is an m -th power exponential function of stress range, the corresponding total relative net stress ranges are, for $K_f < 2.63$:

$$f_{r,t} = (0.27 K_f)^{1/m} \quad (50)$$

and, for $K_f > 2.63$:

$$f_{r,t} = (0.71)^{1/m} \quad (51)$$

In summary, Eq. 43 or Eqs. 46 and 47 estimate the total loss in life of weathering steel bridges. The former is nonconservative for Category A to B details. The latter better fits the data for Category A and B details, but is nonconservative for the Category C to E details.

FATIGUE DESIGN STRESSES

Design Equations

The only fatigue design equations proposed to date for weathering steel bridges are those given in Albrecht [1982a] and Albrecht [1983b]. Those equations and an alternative approach are presented in the following.

The allowable stress ranges, $F_{s,r}$, in the AASHTO fatigue specifications were obtained by shifting the mean S-N lines to the left by a distance equal to two standard deviations of the log of life. This gives:

$$F_{s,r} = \left(\frac{10^{b-2s}}{N_d} \right)^{1/m} \quad (52)$$

where N_d equals the number of design cycles, b equals the intercept, m equals the slope, and s equals the standard deviation. The data base comes from fatigue tests performed in clear air [Fisher 1970, Fisher 1974]. The regression coefficients and the standard deviation for the Category A to E' data are summarized in Table 39. Substituting them into Eq. 52 gives the allowable stress ranges for redundant-load path structures given in Table 47(a). The differences between those values and the ones listed in the AASHTO specifications arise from the round-off errors of 4 percent, at most, in the latter [Albrecht 1981].

To arrive at design stress ranges for weathering steel bridges, Albrecht shifted the current AASHTO design lines for clean air by an amount corresponding to the loss in life due to weath-

ering and corrosion fatigue, given by Eqs. 46 and 47. On that basis, the allowable stress ranges for weathering steel bridges, $F_{s,r}$, are equal to Eq. 52 multiplied by the relative net stress ranges, given by Eqs. 50 and 51. For $K_f \leq 2.63$:

$$F_{s,r,t} = \left(0.27 K_f \frac{10^{b-2s}}{N_d} \right)^{1/m} \quad (53)$$

and, for $K_f > 2.63$:

$$F_{s,r,t} = \left(0.71 \frac{10^{b-2s}}{N_d} \right)^{1/m} \quad (54)$$

The same reductions were also applied to the loading condition "for over 2,000,000 cycles." This seems reasonable, since Novak found a very large degradation in the cyclic load-carrying capacity at 3,000,000 cycles for all structural steels tested in a sodium chloride solution [Novak 1983b]. Other investigators have also reported a degradation of the fatigue limit in corrosive environments.

Substituting the values of b , s , m , and K_f from Table 39 into Eqs. 53 and 54 yields the allowable stress ranges for redundant-load-path structures after losses due to weathering and corrosion fatigue. The results are shown in Table 47(b). The allowable stress ranges for nonredundant load path structures, given in Table 48(b), were obtained in like manner.

Figure 100 compares the AASHTO allowable stress ranges at 500,000 cycles (open bars) with those proposed for weathering steel bridges after total losses due to weathering and corrosion fatigue (solid bars). As can be seen, the reductions vary from about one and one-half category at A to one-half category at E. For information, the cross-hatched bars show the allowable stress ranges after the loss due to weathering alone. The results are the same for the other AASHTO loading conditions of 100,000 and 2,000,000 cycles.

An alternative approach to that proposed in Albrecht [1982a] would be to apply to all structural details a uniform reduction in life, such as the 50 percent adopted by the U.K. Department of Energy for offshore structures [UK DOE 1981]. In this case, the allowable stress ranges would be given by combining Eqs. 43 and 52.

$$F_{s,r,t} = \left(0.50 \frac{10^{b-2s}}{N_d} \right)^{1/m} \quad (55)$$

Assuming the average slope of all S-N lines to be $m = 3.2$ (Table 39) yields a relative net stress range of $f_{r,t} = 0.50^{1/3.2} = 0.80$, that is a 20 percent loss in stress range. The corresponding allowable stress range levels are marked with arrows in Figure 100. In comparison with the equations proposed by Albrecht, Eq. 55 gives higher allowable stress ranges for Category A and B details and lower stress ranges for Category C to E' details. The uniform reduction approach is attractive for its simplicity.

Comments on Design Equations

The approach that was followed in deriving the fatigue design equations (Eqs. 53 and 54) for bare weathering steel structures consisted of shifting the AASHTO design lines (lower confidence

Table 47. Allowable stress ranges for redundant load path structures.

Category	Allowable Stress Range							
	For 100,000 Cycles		For 500,000 Cycles		For 2,000,000 Cycles		For Over 2,000,000 Cycles	
	ksi	MPa	ksi	MPa	ksi	MPa	ksi	MPa
(a) <u>Current AASHTO Specifications (Eq. 52)</u>								
A	61.2	422	36.9	254	23.9	164	24	165.47
B	45.0	311	27.9	193	18.5	128	16	110.31
C*	34.7	239	20.6	142	13.2	90.8	12	82.74
C	32.5	224	19.8	136	12.9	89.1	10	68.95
D	26.8	185	15.9	110	10.1	69.7	7	48.26
E	21.0	145	12.5	86.0	7.97	54.9	5	34.47
E'	15.2	104	9.17	63.2	5.94	41.0	2.6	17.93
(b) <u>After Weathering and Corrosion Fatigue (Eqs. 53 and 54)</u>								
A	40.6	280	24.4	169	15.9	109	15.9	109
B	34.4	237	21.3	147	14.1	97.4	12.2	84.2
C*	28.2	195	16.8	116	10.7	73.8	9.75	67.3
C	28.2	195	17.2	119	11.2	77.4	8.69	59.9
D	24.0	165	14.2	98.0	9.05	62.4	6.26	43.2
E	18.8	129	11.2	77.0	7.13	49.2	4.48	30.9
E'	13.6	93.9	8.23	56.8	5.34	36.8	2.34	16.1

* For transverse stiffener welds on girder web and flanges.

Table 48. Allowable stress range for nonredundant load path structures.

Category	Allowable Stress Range							
	For 100,000 Cycles		For 500,000 Cycles		For 2,000,000 Cycles		For Over 2,000,000 Cycles	
	ksi	MPa	ksi	MPa	ksi	MPa	ksi	MPa
(a) <u>Current AASHTO Specifications (See Table 1.7.2A1)</u>								
A	36	248.21	24	165.47	24	165.47	24	165.47
B	27.5	189.60	18	124.10	16	110.31	16	110.31
C	19	131.00	13	89.63	10, 12*	68.95, 82.74*	9, 11*	62.05, 75.84*
D	16	110.31	10	68.95	7	48.26	5	34.47
E	12.5	86.18	8	55.15	5	34.47	2.5	17.24
(b) <u>After Weathering and Corrosion Fatigue</u>								
A	24.4	169	15.8	109	15.9	109	15.9	109
B	21.3	147	14.1	97.4	12.2	84.2	12.2	84.2
C*	16.8	116	10.7	73.8	9.75	67.3	8.94	61.7
C	17.2	119	11.2	77.4	8.69	59.9	7.82	53.9
D	14.2	98.0	9.05	62.4	6.26	43.2	4.48	30.9
E	11.2	77.0	7.13	49.2	4.48	30.9	2.25	15.5

* For transverse stiffener welds on girder web and flanges.

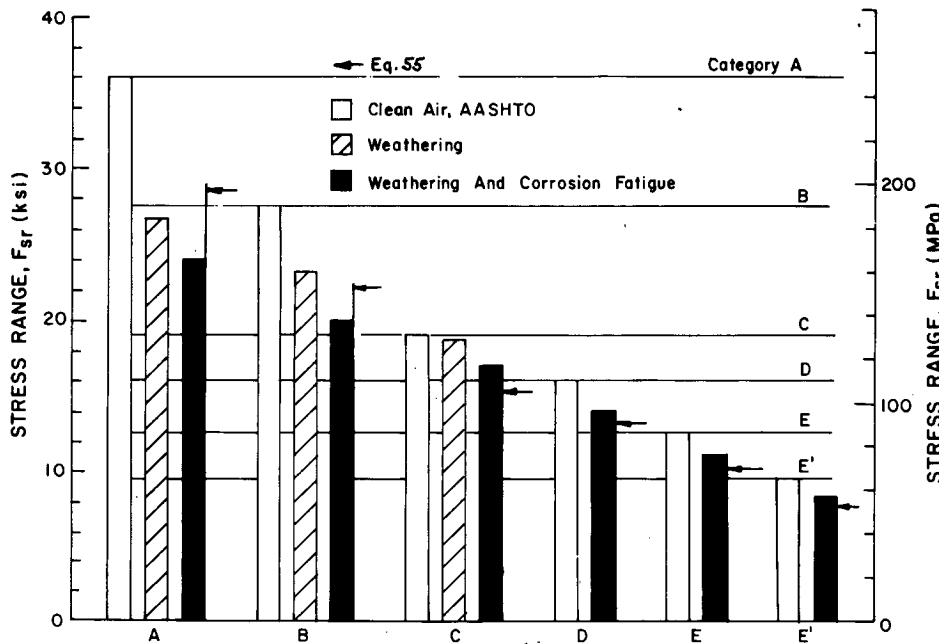


Figure 100. Comparison of allowable stress ranges for redundant load-path structures at 500,000 cycles: (1) current AASHTO, in clean air; (2) after weathering; and (3) after weathering and corrosion fatigue.

limits) to account for the loss in fatigue life caused by both the effect of weathering prior to load cycling and the effect of exposure to an aqueous environment during load cycling. These effects are additive because the former reduces the crack initiation life, while the latter reduces the crack propagation life.

In developing Eqs. 53 and 54, the standard deviation of the S-N data for weathered specimens was set equal to that for nonweathered specimens. This is a generally conservative assumption and is intended to compensate for several other factors that are not taken into account directly in these equations.

The results of the regression analysis, given in Tables 40 and 41, showed that the standard deviation after the initial 2 to 3 years of weathering:

- Decreased for Category A and B details.
- Did not change for Category C details.
- Increased for the details whose mean fatigue life increased with weathering.

Weathering times of 4 to 8 years did not change the standard deviation when compared with the values calculated after the initial 2 to 3 years. The following examples illustrate how the changes in the standard deviation affected the lower confidence limit (L.C.L. = mean minus two standard deviations):

- Figures 79 and 80: The L.C.L. for weathered base metal fell below the AASHTO allowable line for Category A.
- Figure 86: The L.C.L. for weathered transverse stiffeners fell on, or slightly above, the AASHTO allowable line for Category C.

It is worth noting that the L.C.L. continued to degrade with exposure times longer than 3 to 4 years (Fig. 86).

Data used to establish the mean loss in fatigue life for weathering steel are based on dry-air fatigue tests on specimens boldly

or ideally exposed for 4 years. As a result, the following factors might tend to cause Eqs. 53 and 54 to overestimate fatigue life for in-service bridge members.

1. Long-term loss in fatigue life caused by 50 or more years of weathering should be expected to exceed the loss after 4-years exposure. In fact, recent data for 8-year-weathered, automatically welded stiffeners show that their mean loss in life (point 15 in Fig. 94) exceeds the value predicted by Eq. 34 (dashed line in Fig. 94).
2. The corrosion penetration data presented in Chapters Four and Five show that the corrosion and pitting of structures in service is more severe than the corrosion of ideally and boldly exposed specimens.
3. Loss of net section due to corrosion increases the stress range.
4. Reported losses in crack propagation life (Fig. 97) caused by corrosion fatigue in an aqueous (as opposed to air) environment are larger than the assumed 29 percent (Eq. 45 and Fig. 98).
5. Results of the regression analysis, given in Table 45, show that the standard deviation of crack-growth-rate data is larger in an aqueous environment than in an air environment. Accordingly, the loss in corrosion fatigue life would actually be greater than that predicted from the loss in mean corrosion fatigue life.

In view of these factors, it does not appear to be sufficient to compare the lower confidence limit of the available weathering fatigue data against the AASHTO allowable S-N lines and recommend a one-category penalty on the fatigue resistance of Category A details fabricated from weathering steel, and no penalty on the fatigue resistance of Category C to E' details [Fisher 1983b]. The proposed design Eqs. 53 and 54 attempt to anticipate the long-term 50-year losses in fatigue life, the in-

creased corrosion under nonideal exposure conditions, the loss in net section, and the detrimental effect of aqueous environments.

With the information available in 1981, when the reference [Albrecht 1982a] was issued, it was believed that the conservative and nonconservative assumptions would compensate each other and would not lead to underestimates of the 50-year loss in life of weathering steel bridges. In retrospect, this expectation may not be realistic.

Category B Details

Some investigators stated that the Category B allowable line need not be lowered because most fatigue cracks at Category B details initiate at internal, subsurface flaws that are not affected by weathering and corrosion fatigue on the surface [Fisher 1983b, Barsom 1983].

In reality, most Category B details fail from external flaws, as the following data show [Albrecht 1983c]:

REFERENCE	TYPE OF DETAIL	NUMBER OF FAILURE INITIATIONS	
		INTERNAL	EXTERNAL
Fisher 1970	Welded beams	29	21
Fisher 1970	Flange splices with width transition	25	137
Yamada 1977	Flange splices with thickness transition	0	13

The Category B allowable line was based on the fatigue test data developed by Fisher, et al., for welded beams [Fisher 1970]. Hirt subsequently examined the fatigue failures of the 50 welded beams from the reference [Fisher 1970] and reported that the cracks initiated from internal porosities in 29 beams and from external discontinuities in 21 beams [Hirt 1971]. The two sets of welded beam data were evaluated herein with regression and statistical analyses. The regression line, standard deviation, and fatigue notch factor are: (1) for beams failing from internal porosities: $\log N = 12.8553 - 2.984 \log f_r$ (MPa), $s = 0.0717$, $K_f = 1.40$; and (2) for beams failing from external discontinuities: $\log N = 13.6820 - 3.394 \log f_r$ (MPa), $s = 0.01350$, $K_f = 1.56$. The 10 percent difference in fatigue strength of the two sets was insignificant at the 5 percent level of significance, as determined by a t-test of the differences between the intercepts and the slopes of the two regression lines. It follows that Category B welded beams are statistically as likely to fail from external as from internal discontinuities.

In a second series of tests on Category B details, Fisher, et al., obtained fatigue data for 162 welded beam flange splices with either straight or 2-ft (600-mm) radial transition in width [Fisher 1970]. Twenty-five of the 162 butt-welded splices failed from gas pockets, oxide inclusions, and internal cavities. The majority of the splices (137) failed from external flaws. The straight and curved transitions had fatigue notch factors of 1.47 and 1.35, respectively; hence, they had slightly longer mean lives than the plain welded beams for which $K_f = 1.49$. Both are Category B details, and most beams failed from external discontinuities.

Finally, Yamada and Albrecht reported fatigue test data for splices of plate girder flanges with 1:4 thickness transition [Yamada 1977]. All data points fell above the Category B allowable line. The 13 details failed from cracks that initiated at external flaws.

In summary, most (196 of 250) Category B details failed from external flaws. Since weathering and corrosion fatigue affect crack growth from external flaws, the loss in life must be applied to Category B details.

Other Viewpoints

In the opinion of some investigators, the allowable stress range need not be reduced so long as the weathering fatigue data for a detail do not fall below the applicable AASHTO allowable line [Fisher 1983b, Barsom 1983]. On that basis, Fisher agreed to a one-category reduction in fatigue strength for base metal, from A to B, but not for any other detail. Barsom concluded that the AASHTO allowable lines, including that for Category A, apply equally to weathered and nonweathered steel members.

In their opinion, rust pitting provides a notch condition not more severe than the worst flaw in a given detail. For this reason, they added, rust pitting lowers the mean life but not the lower confidence limit.

Both Fisher and Barsom are of the opinion that the detrimental effect of corrosion fatigue need not be accounted for in designing weathering steel bridges.

Other Specifications

The domestic and foreign specifications for fatigue design of aerospace structures, nuclear reactors, pipes, boilers, machinery, pipelines, and offshore structures provide for reduced fatigue strength when the structure is not protected against corrosion. Although the first weathering steel bridge in the United States was built in 1964, the AASHTO, AREA, AISC, and AWS specifications do not have provisions for fatigue design of non-painted steel structures.

European code writing bodies are more aware of the need to consider the detrimental effect of corrosion on fatigue than their American counterparts. In comparing the British and American fatigue design rules for welded structures, Gurney and Maddox stated that [Gurney 1980]: "Allowance for environment may also lead the reductions in allowable stresses, including the fatigue limit. In this context it may be noted that the British rules for the design of fixed offshore structures (DD55) allow for seawater corrosion by reducing the fatigue limit."

Current drafts of European codes and recommendations for fatigue design of steel structures state that "reduction in fatigue life due to corrosion" is not explicitly covered [Eurocode 1983, ECCS 1982]. The ECCS recommendations limit the application of the fatigue strength curves to structures in normal atmospheric conditions with suitable corrosion protection. They further state that more corrosive environments may significantly reduce the fatigue strength.

A Specialist Drafting Panel recommended revision of the fatigue sections of the U.K. Department of Energy's document "Offshore Installations: Guidance on Design and Construction" [UK DOE 1978, Tomkins 1982]. The revisions include an across

the board factor-of-two penalty on life (50 percent loss in life) for unprotected steel welded joints exposed to seawater. Protection is taken to include: (1) paint and other coatings, where these are known to adequately prevent contact of water with the steel; and (2) electrochemical potential. Furthermore, no fatigue limit is assumed to exist. The S-N design curve for seawater corrosion fatigue falls, therefore, a factor-of-two on life below the basic air curve (T) and extends as a straight line to 100 million cycles, at a slope of $-1/3$.

In the Federal Republic of Germany, special permits for building highway bridges with orthotropic steel decks and railway bridges from weathering steel are not granted because the authorities are concerned about the detrimental effect of corrosion on fatigue life.

The available data show that steel structures not protected against corrosion have shorter fatigue lives. Therefore, allowable fatigue stress ranges for weathering steel in aqueous environments should be lowered from their present levels for clean air environments.

APPLICATION TO CURRENT DESIGNS

It is of interest to examine the safety margin against fatigue of highway bridges designed to the current AASHTO specifications, in terms of the lower allowable stress ranges proposed previously under "Fatigue Design Stresses." These series of simple-span A588 steel bridges were investigated [Albrecht 1982a].

The following summarizes the types of bridges and the calculated stress ranges at the fatigue critical details. The latter are then compared in Figure 101 with the: (1) stress ranges allowed by AASHTO for redundant load path structures; and (2) values proposed in Table 47 for weathering steel structures. All bridges were designed for an average daily truck traffic, ADTT = 2,500. In this case, the fatigue design of Category A, B, and C details is always governed by the loading condition

"for 2,000,000 cycles" of truck loading on two or more traffic lanes, for which the wheel load distribution factor is $S/5.5$. Category D, E, and E' details are always governed by the loading condition "for over 2,000,000 cycles" of truck loading on one traffic lane, with distribution factor $S/7$ [Albrecht 1982b].

Example No. 1

Eight simple-span bridges; noncomposite, rolled beams; HS-20 loading; 25-ft to 60-ft (7.62-m to 18.30-m) spans; 30-ft (9.15-m) roadway; 8-ft 7-in. (2.62-m) girder spacing; 7-in. (0.18-m) concrete deck; A588 steel, $F_y = 50$ ksi (345 MPa); 4,000-psi (27.6-MPa) concrete; interior girders; working stress design [USS 1975a].

The stress ranges were computed for two details, both at midspan: (1) Category A base metal, extreme fiber of tension flange; and (2) Category C diaphragm gusset welded to the web, symmetrically about the centroid of the girder. The gussets were at most 9 in. (230 mm) long.

The calculated midspan stress ranges, shown in Figure 101 with triangular symbols, are much lower than the 24 ksi (165 MPa) allowed by AASHTO for Category A base metal, but they exceed the proposed 15.9 ksi (109 MPa) allowable stress range for weathering steel bridges. Fatigue would control the flexural design of this set of bridges. The Category C diaphragm gussets are in no case fatigue critical.

Example No. 2

Eight simple-span bridges; composite rolled beams; HS-20 loading; 25-ft to 60-ft (7.62-m to 18.30-m) span; 30-ft (9.15-m) roadway; 8-ft 7-in. (2.62-m) girder spacing; 7-in. (0.18-m) concrete deck; A588 steel, $F_y = 50$ ksi (345 MPa); 4,000-psi (27.6-MPa) concrete; interior girders; working stress design; unshored construction [USS 1975a].

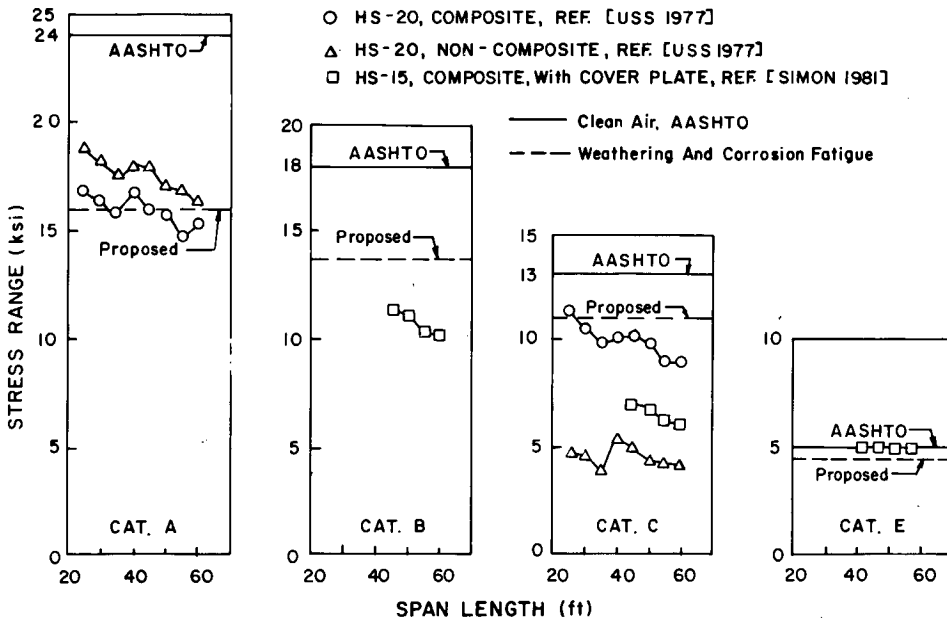


Figure 101. Comparison of calculated stress ranges in simple-span bridges with allowable stress ranges for: (1) clean air, current AASHTO; (2) weathering and corrosion fatigue.

As in the previous example, the midspan stress ranges were computed for Category A base metal and Category C diaphragm gussets. The calculated stress ranges are plotted in Figure 101 with circular symbols. The composite section raised the neutral axis and substantially increased the stress range at the lower end of the diaphragm gusset. It moderately lowered the stress range at the extreme fiber. Fatigue only controls the design of both details when the spans are short.

Example No. 3

Four simple-span bridges; composite rolled beams with cover plates; HS-15 loading; 45-ft to 60-ft (13.72-m to 18.30-m) span; 24-ft. (7.32-m) roadway; 6-ft 9-in. (2.06-m) girder spacing; 7-in. (0.18-m) concrete deck; A588 steel, $F_y = 50$ ksi (345 MPa); 4,000-psi (27.6-MPa) concrete; interior girders; working stress design; unshored construction [Simon 1981].

The stress ranges were computed for three details; (1) Category B cover plate-to-flange weld at midspan; (2) Category C diaphragm gusset at midspan, max. 9-in. (230 mm) long; and (3) Category E cover plate end.

The calculated stress ranges, plotted in Figure 101 with rectangular symbols, are much lower than the proposed allowables

for Category B, and C. But they exceed the proposed stress range for Category E by 0.5 ksi (3.6 MPa).

Comparison with Measured Stress Ranges

Another way of assessing the impact of lowering the allowable stress ranges on bridges already in service is to compare the highest stress range measured on bridge girders against those proposed herein for weathering steel bridges. Shaaban and Albrecht collected and analyzed 190 stress-range histograms that had been recorded on 40 bridges in Michigan, Alabama, Maryland, Virginia, Tennessee, Minnesota, Connecticut, Illinois, and Ohio [Shaaban 1983]. Table 49 compares the results for the most highly stressed detail in each category.

In Category A and B details, for which the observed reductions in fatigue strength are greatest, the highest measured stress ranges are lower than the proposed allowable stress ranges and well below the fatigue limits. In these cases, weathering is not expected to reduce the fatigue life of existing bridges. Stress-range histograms were not recorded at Category C and D details.

In category E and E' details, for which the proposed reductions are small, the highest measured stress ranges significantly exceed the proposed as well as the current AASHTO allowable stress ranges. In these cases, heavy truck loading, more than weathering, may shorten the fatigue life of bridges below the values intended by the AASHTO specifications.

Table 49. Comparison of allowable, computed, and measured stress ranges in highway bridges. [Shaaban 1983]

Stress Category	Type of Detail	Bridge	Allowable Stress Range		Computed Stress Range for HS20 (ksi)	Highest Measured Stress Range (ksi)
			Current AASHTO (ksi)	Proposed (ksi)		
A	Base metal at midspan	Shaffer Creek, F.A.I. Rt. 74, Illinois	24.0	15.9	16.3	8.93
B	Longitudinal flange-to-cover plate weld at midspan	Overpass for Southern RR, Sayre to Alden Rd., Jefferson County, Alabama	18.0	14.1	9.20	7.75
C	-----No data available-----					
D	-----No data available-----					
E	Flange at cover plate end	Over Alabama River on Road from Hunter Station to Prattmont, Montgomery and Elmore Counties, Alabama	5.0	4.5	6.20	7.25
E'	Flange at cover plate end	Overpass for Southern RR, Sayre to Alden Rd., Jefferson County, Alabama	2.5	2.2	14.10	7.25

PAINTING

This chapter deals with three aspects of painting. First, the performance of paint systems applied to a fresh weathering steel substrate are compared to the same paint systems applied to mild steel. Second, the Japanese practice of treating weathering steel with a sacrificial rust stabilizing system is described. Third, the results of laboratory tests of remedial paint systems for severely corroded weathering steel specimens are reviewed. Examples are given of paint systems that were used to rehabilitate weathering steel bridges.

INITIAL PAINTING OF NEW BRIDGES

Several investigators have examined the performance of paint systems applied to a nonweathered or fresh substrate of weathering copper (copper-bearing) and carbon steels. Some have found certain paint systems to be more durable on a weathering steel substrate [Copson 1959, Schmitt 1967, and Coburn 1978]. Others have found comparable durability of the same paint systems applied to both weathering and carbon steels [Nakayama 1972, Van Eijnsbergen 1979, and Scantlebury 1980]. The results of the aforementioned studies are reviewed in the following.

Copson and Larrabee examined the durability of paint on sheets of copper and HSLA steels on the sides of a railroad hopper car [Copson 1959]. After 5-years service, the paint on the copper-bearing steel sheets had come off in relatively large sheets, whereas the paint on the HSLA steel sheets continued to give adequate protection. Because the rust that formed on the HSLA steel is less voluminous, the paint film ruptured to a lesser extent and, thus, less moisture reached the steel to promote corrosion.

In the same study in which the performance of bare carbon, A242 Type 1, and A588 Grade A steels in chemical plant environments was evaluated (Chapter Five, section under "Industrial Pollutants"), Schmitt and Mathay also examined the durability of a paint system applied to these steels [Schmitt 1967]. The steel specimens were sandblasted, spray coated with a wash primer, and top coated with a 4-mils to 5-mils (100- μ m to 125- μ m) thick dry film of vinyl commonly used for chemical plant structures. A mark (X) was inscribed on the skyward surface of the coated specimens. The specimens were exposed at most 2 years in the environments listed in Table 35. The performance of the vinyl coating was judged by the extent of undercutting at the inscribed mark. The findings were as follows:

1. For exposure periods up to one year, the vinyl coating on the weathering steels tended to be less undercut than on the carbon steel.
2. After 2-years exposure at one sulfur and two chlor-alkali plants, the vinyl coating completely failed on all steels. In a

petrochemical plant, the vinyl coating was severely undercut on the carbon steel and slightly undercut on the weathering steel specimens.

Wonders evaluated the performance of paint on carbon and weathering type steels [Coburn 1978]. Standard specimens of both steels were exposed for several months in a coal coking plant atmosphere similar to those found in steel plants worldwide. In this type of environment, wind-blown moisture droplets carrying ammonium chloride deposit on metallic structures. They cause accelerated deterioration of the paint and attack the underlying steel. After the initial exposure, some specimens were sandblasted and others were hand wire brushed. The specimens were then coated with organic or inorganic zinc-rich alkyd, epoxy, or vinyl primer, followed by one of seven generic types of intermediate or top coat systems. All painted specimens were inscribed (X) down to the base metal and weathered for one year on racks facing the source of the pollutant at a 30-degree angle from the horizontal. The following results were obtained:

1. So long as both the organic or inorganic zinc-rich primers were in direct contact with the steel surface, regardless of the top coat, all specimens were in excellent condition.
2. The inorganic zinc-rich primer outperformed the organic zinc-rich primer when both were top coated with the same epoxy or vinyl paint.

For this series of test, Coburn [1978] did not assess the relative durability of paint on carbon and weathering steels, or the effect of initial exposure and cleaning method on paint durability.

Another series of one-year exposure tests examined the performance of 48 combinations of protective coating systems and substrates [Coburn 1978]. Table 50 summarizes the results. A rating of 10 indicates no rusting, while a rating of zero implies complete rusting. The data show that the paint systems consisting of an organic zinc-rich primer with or without a vinyl top coat, or an inorganic zinc-rich primer with or without an epoxy top coat were about equally durable on both the carbon and weathering steels. However, the paint systems that did not adequately protect the steel against corrosion lasted longer on the weathering steel than on the carbon steel, when both were exposed one year. The reason for this is that the rust on exposed areas, such as at scratches in the paint on the weathering steel, is less voluminous and, therefore, undercuts the paint film to a lesser degree.

When exposed to severe environments that cause rapid paint failure, such as continuous dampness and chlorides, the underlying weathering steels also corroded and produced flaky and voluminous rust comparable to that found on carbon steel. Under such conditions, there is no reason to expect that the paint on the two steels would behave differently. Table 51 shows the

Table 50. Relative performance of various coating systems on carbon and weathering steels in two coke plants. [Coburn 1978]

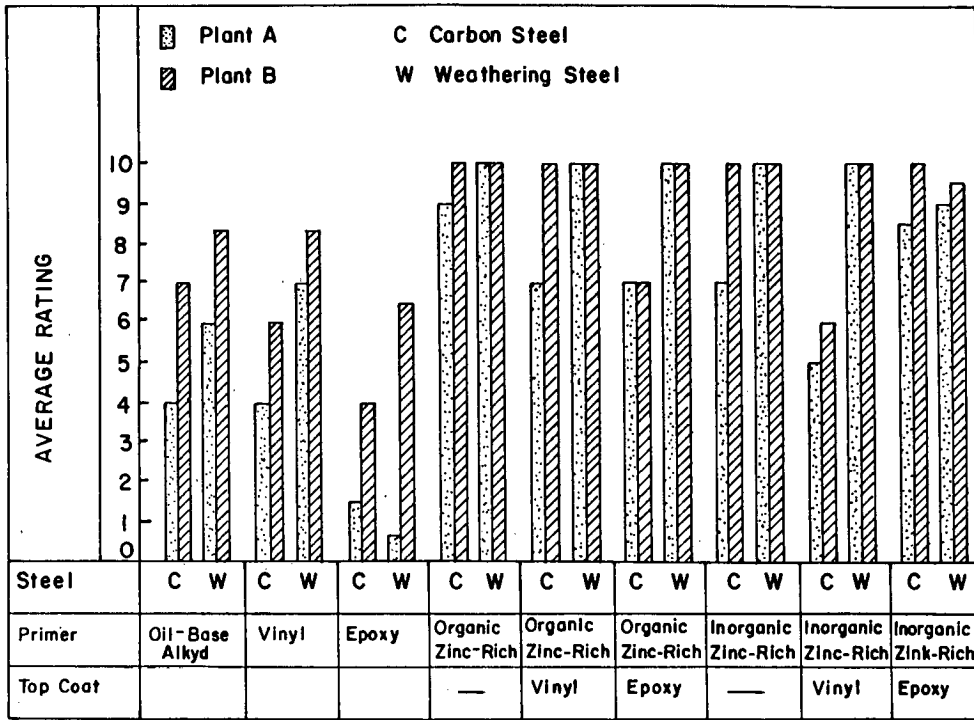
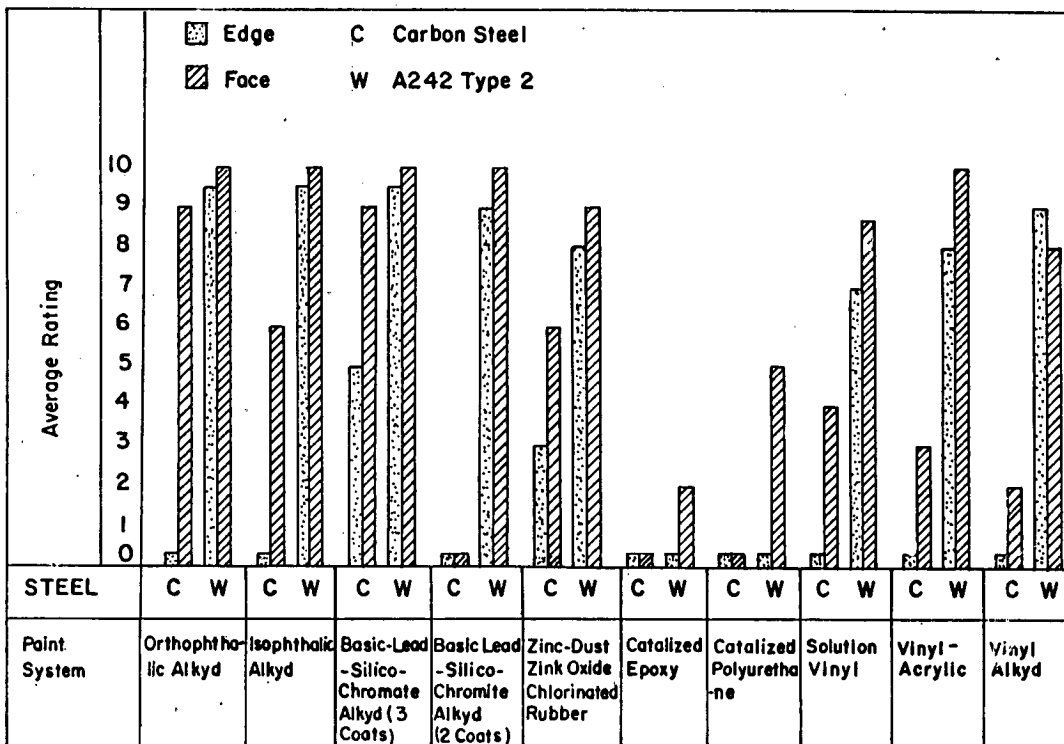


Table 51. Relative performance of various coating systems on carbon and weathering steels in Kure Beach (the 24-m lot), N.C. [Coburn 1978]



results of two additional paint durability tests performed at Kure Beach, N.C., 80 ft (24 m) from the ocean [Coburn 1978]. The first test involved a 15-year exposure of 10 paint systems applied on carbon and weathering steels. The specimens measuring 8 in. \times 12 in. (0.20 m \times 0.30 m) were sandblasted to white metal and painted with a 4.5 to 6.5 mils (115 to 165 μ m) thick two-coat or three-coat system of alkyds, vinyls, vinyl-acrylics, chlorinated rubber or catalyzed epoxy and polyurethane compositions. The paint began to deteriorate at the cut edges, and the rust gradually undercut the coating on the flat faces. Table 51 shows that all paint systems were more durable on the weathering than on the carbon steel.

The second test performed in Kure Beach, N.C., determined whether a surface free of mill scale is required for optimum paint performance. Specimens measuring 12 ft \times 1 ft (3.66 m \times 0.30 m) were fabricated from a carbon steel and a weathering steel used for sheet piling in seawater containing 0.5 percent copper, 0.5 percent nickel, and 0.1 percent phosphorus. Matched pairs were sandblasted and hand wire brushed, coated with a variety of thick mastics and thin coating systems and X-marked at the lower 5 ft (1.52 m). The plate specimens were mounted on a rack facing south at a 30-deg angle and exposed 80 ft (24 m) from the ocean in Kure Beach, N.C. The test showed that removing the mill scale by sandblasting ensured good paint performance, regardless of substrate steel composition. It also showed that paints were more durable on hand wire-brushed weathering steel than on similarly cleaned carbon steel.

One Japanese, two European, and one American study reported that paint systems applied on carbon and weathering steels were equally durable [Nakayama 1972, Van Eijnsbergen 1979, Scantlebury 1980 and Tinklenberg 1982].

Nakayama showed that after one-year exposure the damaged parts of painted atmospheric corrosion resistant steel corroded less than those of carbon steel [Nakayama 1972]. However, after 4-years exposure, no distinct difference in paint performance was observed. According to Nakayama, the use of atmospheric corrosion resistant steel does not necessarily simplify painting procedures.

Van Eijnsbergen, whose interests related to galvanized steel and shipping containers, investigated paint performance on both weathering steel and carbon steel, especially when damaged locally by scratches and dents [Van Eijnsbergen 1979].

He also investigated the claim that the use of an anticorrosive primer is unnecessary when applying a paint system on weathering steel surfaces, and that only a top coat is required. For this purpose, he exposed carbon and weathering steel specimens to the marine-industrial atmosphere of The Hague, Netherlands. All specimens were gritblasted and painted with one or two 1.2- to 1.6-mils (30- to 40- μ m) thick epoxy top coats. The results after 3 years of weathering were:

1. On all one-coat systems, considerable rust formed, irrespective of type of steel.
2. On all two-coat systems, slight or no rust formed.
3. On the back side of the specimens, the same degree of rust occurred. But on the areas where the specimens were supported by wooden beams rusting was more pronounced, especially with the one-coat systems.
4. In the 30- to 40-mils (0.8 to 1.0-mm) wide scratches penetrating to the steel, there was either no difference or only a

slight difference in the amount of paint undercutting. Filiform corrosion occurred in several cases, and there was practically no difference between carbon and weathering steels. These results were said to be in line with those obtained by other institutes in the Netherlands.

Van Eijnsbergen concluded that container roofs and other parts made of weathering steel must receive the same coating as other types of steels; that is gritblasting, followed by a coat of, preferably, zinc-rich primer and one or two coats of an epoxy ester or chlorinated rubber-based top coat with a total thickness of 3 to 4 mils (75 to 100 μ m). Inorganic zinc-rich coatings have a unique ability to prevent spreading and creeping of rust from bared areas adjoining the paint coat. It must be appreciated that it takes time for the weathering steel oxide films to stabilize. Three years of exposure is just about the point where further attack is stifled.

Scantlebury tested the corrosion inhibiting properties of aqueous extracts of zinc phosphate pigment on carbon steel and a low-alloy copper-bearing steel [Scantlebury 1980]. He found that durability of the paint was identical for both steels.

Tinklenberg [1982] initiated a study to learn whether coating systems applied to nonweathered or fresh A36 and A588 steel panels performed differently from each other in a chloride environment. The fresh panels were blast-cleaned to a near-white (SSPC 10) condition, coated with the eight paint systems listed in Table 52, and placed in a salt-fog cabinet for the durations indicated in the table. He found no significant difference between the performance of the various coating systems on shop-coated A36 and A588 panels in a moist chloride environment (salt fog). According to the ratings in Table 52, the zinc-rich systems performed best, followed by the urethane, alkyd and mastic systems.

On the basis of the published data, there appears to be convincing evidence that when a variety of paint systems are applied

Table 52. Paint systems applied to freshly blast-cleaned A36 and A588 steels and tested in salt-fog cabinet. [Tinklenberg 1982]

No.	Generic Type	Salt Fog, hours	A 36 vs. A588 Performance
1	Inorganic zinc-rich, vinyl	4,000	Equal-very good
2	Organic zinc-rich, epoxy	4,000	Equal-excellent
3	Aluminum epoxy mastic	2,500	A588 worse-both poor ^a
4	4-coat no lead alkyd	2,500	A36 worse-both poor ^b
5	3-coat urethane, epoxy	2,500	Equal-both good
6	4-coat urethane, epoxy	2,500	Equal-both good
7	3-coat red lead alkyd	2,500	Equal-both poor to fair
8	4-coat no lead alkyd ^c	2,500	Equal-both fair

Notes:

^aThe A588 panel had a thinner dry film thickness which caused the poorer rating.

^bThe A36 panel developed a large crack at 2,000 hours which lowered its rating.

Conditions (a) and (b) are not related to the type of steel.

^cDifferent pigment system than in No. 4.

on both carbon steel and weathering type steel compositions, and exposed in different environments long enough to exhibit degradation, the carbon steel panels will be the first to degrade. The previously cited 15-year exposure tests, performed by Coburn 80 ft (24 m) from the ocean in Kure Beach, N.C., confirm the results from the tests by Copson and Larrabee, in which the longer life of paints on corrosion resistant steels was first demonstrated [Coburn 1978, Copson 1959].

Some investigators tested paint systems over shorter periods of time and were unable to reach the same conclusions, because their exposure periods were too short for the paint to degrade on the carbon steel panel. Other investigators accelerated the testing time by imposing on the paint systems artificial environments in which the weathering steel was incapable of developing a protective oxide coating—the reason being that the test cabinets did not permit the necessary alternate wetting and drying cycles in the presence of sulfur oxides. Such artificial environments can only discriminate between paint systems, regardless of the substrate.

Weathering steels enhance the service life of paint, because the protective oxide coating in scratches and thin spots is sufficiently thin to avoid undercutting the adjacent intact paint film. With carbon steel, the voluminous rust continuously destroys the adhesive bond between the paint and the surface of the steel. On the other hand, when environmental conditions are so bad that the weathering steel would have to be painted, A572 steel would be a preferred economic choice.

Should a decision be made to paint the weathering steel, the same surface preparation procedures and standards applied to carbon steel should also be applied to the weathering steel. The service life of the paint is directly related to the amount of surface preparation.

RUST STABILIZING TREATMENT

Most weathering steel bridges in Japan were either painted or treated with an oxide stabilizing coating. The latter is designed to: (1) accelerate the formation of a dense and stable oxide coating; and (2) prevent dislodging and outflow of the initial rust. Such treatments are being used to ensure that a protective oxide coating develops in the Japanese climates often characterized by high humidity and high temperature, and that the concrete piers and abutments are not stained by rust-bearing water running off steel members.

To counter the aforementioned problems, Nippon Steel Corp. of Japan developed the Oxicoat and Weather-Coat surface treatment systems [Nippon 1974, Nippon 1976]. The Oxicoat system mainly accelerates the formation of a stable oxide coating, whereas the Weather-Coat system mainly inhibits outflow of rust. Both systems are sacrificial. They are only applied once, before the steel structure is erected.

The Weather Coat system consists of two coats. The primary coat is 0.4 to 0.6 mils (10 to 15 μ m) thick. Its main component is crystal zinc phosphate, $Zn_3(PO_4)_2 \cdot H_2O$. Its function is to accelerate the formation of the stable oxide coating. The primary coat is applied either by immersion or by spraying. The immersion method is suited for shop application to sheets and small structural members at relatively high temperatures. The spray method is suited for large structural members and for field touch-up of damaged coats.

The secondary coat, consisting of 0.16 to 1.6 mils (4 to 40

m) thick Prepalene, is a highly permeable film of acrylic resins that prevents dislodging of the initial rust products while the oxidation reactions progress.

The primary coat is gradually absorbed by the oxide, and the secondary coat vanishes with time.

As an example of how the Weather-Coat system is applied in practice, the treatment of the three-span continuous box girder of the Nadahama Bridge, Hyogo Prefecture, Japan, is described below. The following steps were carried out in the fabrication shop:

1. Blast cleaned steel surfaces.
2. Within 6 hours after blast cleaning, sprayed at least three times Weather-Coat #1000 solution at 122 F (50 C). Allowed 15-min drying time between two coats.
3. Rinsed surfaces with clean water immediately after step 2.
4. Allowed surfaces to dry naturally.
5. Masked surfaces that were to be treated with Prepalene (friction-type joints, etc.).
6. After 30 min to 24 hours drying, sprayed a 1.2-mils (30- μ m) thick film of dark brown Prepalene PS. Allowed it to dry for 30 min.
7. Sprayed a 0.8-mils (20- μ m) thick film of red brown Prepalene R.
8. Air dried surfaces for more than 12 hours.
9. Inspected appearance and average dry film thickness (minimum 2 mils (50 μ m)).

After the steel was erected, the following additional steps were carried out at the job site:

10. Removed surface contamination by concrete, oil, and dust. Removed rust from bolt-heads, etc.
11. Repaired damaged areas by repeating steps 2 to 9.
12. Sprayed a 0.8-mils (20- μ m) thick film of red brown Prepalene R finish.
13. Air dried surfaces for more than 12 hours.
14. Inspected appearance and average dry film thickness (minimum 2.8 mils (70 μ m)).

"Weather-Coating" is a lengthy and costly process. It adds about 10 to 20 percent to the basic cost of a weathering steel bridge in Japan [Yamada 1983a]. The Shiribetsu Railway Overpass Bridge, the first to be built of Weather-Coated weathering steel, was completed in November 1968. The exposure time of such bridges has not been long enough to conclusively evaluate the performance of the Weather-Coat process [Yamada 1983a]. Questions remain about how bridges treated with Weather-Coat perform in environments in which they are subjected to salt spray. The Nakashio Bridge in Tokyo, Japan, is an example of a Weather-Coated bridge constructed over the sea. In this case, after 2 years of exposure, a stable oxide coating was forming, except on internal beams which are rusting faster than anticipated [Nippon Steel].

REMEDIAL PAINTING OF EXISTING BRIDGES

The Michigan Department of Transportation and the Steel Structures Painting Council are currently working on conduct-

ing test programs on cleaning and painting corroded weathering steels. The only results available to date are those from the Michigan study [Tinklenberg 1982].

In comparison with shop coating new steels, the problem with remedial painting of severely corroded and pitted weathering steels lies in effectively removing all rust and salt trapped in dimples and pits. Unless this is achieved, paint life is reduced.

Continuing the work cited earlier in this chapter under "Initial Painting of New Bridges," Tinklenberg conducted three tests on remedial painting. In the first, A36 and A588 steel panels were exposed as described in Table 53. The environmentally exposed panels were cut from link plates that were removed from the Eight-Mile Road Bridge over I-75 in Detroit, Michigan. The panel material had been exposed in service for about 12 years. The joint in this area leaked badly and the steel was heavily corroded and pitted. The weathered panels were sand-blasted to a near-white (SSPC-10) condition and painted with the nine coating systems given in Table 54. Thereafter, they were exposed in a salt fog cabinet for 4,000 hours in accordance with the requirements of ASTM Specification B-117. Tinklenberg concluded the following:

1. If a problem is encountered with a laboratory panel, it could be worse with environmentally exposed A588 steel.

2. Three of the nine coating systems blistered uniformly on all five panels. Three of the coating systems blistered only on the face that had been environmentally exposed and retained salt residue in the surface crevices.

3. Five of the zinc-rich compositions performed very well. They showed little if any corrosion damage in the 4,000-hour test.

4. Natural weathering of A588 steel cannot be reliably duplicated in the laboratory with relatively short-time salt fog exposure.

5. Coating systems on environmentally exposed A588 panels tend to fail more rapidly than those on unexposed panels.

6. With proper preparation and application, it is possible to get excellent corrosion protection of A588 steel, even if it has been exposed to a leaky bridge joint environment.

7. On the basis of the observations from this group of tests and the tests described under "Initial Painting of New Bridges," one can conclude that the paint life is likely to be shorter on weathering steel previously exposed to even ideal environmental conditions, than on new steel.

In the second group of tests on remedial painting, Tinklenberg examined the effects of different surface preparation methods. An additional objective was to begin performance evaluation of various generic types of coatings. The nonweathered control panels, made of A36 steel, were blast cleaned to a near white (SSPC-10) condition. The weathered A588 steel panels were cut from hanger plates that had been removed from the Eight-Mile Road Bridge. The latter were prepared for painting by one of the five methods described in Table 55. All panels were then painted with the 10 coating systems given in Table 56 and placed in the salt-fog cabinet. Based on a preliminary analysis of the data, Tinklenberg concluded the following:

1. The washing cycle between the two blasting operations had little, if any, effect on paint performance. The green mold that formed on A588 steel had little effect on performance.

Table 53. Steel panels used to evaluate applicability of salt-fog cabinet testing to service conditions. [Tinklenberg 1982]

Type of Steel	Exposure Condition
A588	Environmentally exposed (Eight-Mile Road Bridge)
A588	Exposed uncoated in salt-fog cabinet for 1000 hours
A36 and A588	Exposed outdoors in rural atmosphere
A588	Not exposed

Table 54. Evaluation of coating systems on weathered A36 and A588 steels in salt-fog cabinet. [Tinklenberg 1982]

No. of Systems	Type of Coating
1	Four-coat lead free alkyd
2	Single-component organic zinc-rich primer with epoxy top coat
3	Multi-component (usually three) epoxy zinc-rich primer with epoxy topcoat
1	One-coat epoxy mastic
1	Single-component inorganic zinc-rich primer with vinyl topcoat
1	Two-component inorganic zinc-rich primer with epoxy topcoat

Table 55. Surface preparation of environmentally exposed A588 panels removed from Eight Mile Road Bridge over I-75 in Detroit, Michigan, and coated with systems given in Table 56. [Tinklenberg 1982]

No. of Panels	Surface Preparation
1	Blast-cleaned to near-white condition
1	Brush-off blasted, power washed and reblasted to near-white condition
1	Near-white blasted, power washed, and reblasted to near-white condition
1	Near-white blasted and exposed for 10 minutes to 100% relative humidity to produce the "green mold phenomenon"
1	Gradient blasted from brush-off to near-white condition

Table 56. Coating systems applied to steel panels given in Table 55 and to near-white-blasted A36 panels. [Tinklenberg 1982]

No. of Systems Tested	Type of Coating System	Rating
2	Multi-component organic zinc-rich with epoxy intermediate coat and an aliphatic urethane topcoat	1
1	Single component organic zinc-rich with a polyamide epoxy topcoat	2
3	Single component inorganic zinc-rich primer with polyamide-epoxy topcoats	3
1	Moisture-cured urethane with a polyamide epoxy topcoat	4
1	Polyamide epoxy primer with aliphatic urethane topcoat	5
1	4-coat chlorinated rubber system	6
1	4-coat no lead no chromate alkyd system	7

2. Blistering was generally worse in brush-off and commercially blasted areas.

3. In general, the appearance was worse on the side of the hanger plate that had been exposed to the environment. This is of concern as the interior surface was more severely pitted and much worse in initial appearance. Also, the outside surface more closely resembled the exposure and appearance of the entire bridge surface.

4. After 3,000 hours of salt-fog cabinet exposure, the generic coating systems that were applied to the A588 panels ranked in the order given in Table 56. The systems ranked 3rd, 4th, and 5th performed about equally.

A conclusive performance evaluation must await the final analysis of cost, film thickness, and ease of application under field conditions.

Based on the results of the two previously described test series, Tinklenberg initiated an extensive qualification program for coating systems. As before, the unexposed control panels of A36 steel were blasted to a near-white (SSPC-10) condition. The weathered A588 panels were cut from hanger plates removed from a badly corroded joint area of the Grand Boulevard Bridge over the C&O Railroad in Detroit, Michigan. They were cleaned to a near-white, white or gradient blasted condition. All steel panels were then coated with the 22 systems given in Table 57. The tests are in progress. When all results become available, the Michigan Department of Transportation will generate a

Table 57. Description of the 22 coating systems currently being evaluated in the salt-fog cabinet to determine their relative performance when applied to near-white, white, and gradient-blasted A588 steel panels and near-white blasted A36 steel panels. [Tinklenberg 1982]

No. of Systems	Type of Coating System
3	Single-component inorganic zinc-rich primers with hi-build vinyl topcoats
2	Single-component inorganic zinc-rich primers with polyamide epoxy topcoats
1	Single-component inorganic zinc-rich primer with a polyamide-epoxy intermediate coat and an aliphatic urethane topcoat
1	Multi-component inorganic zinc-rich primer with hi-build urethane topcoat
1	Multi-component inorganic zinc-rich primer with a hi-build epoxy intermediate and an aliphatic urethane topcoat
1	Single component organic zinc-rich primer with a polyamide epoxy topcoat
3	Multi-component organic zinc-rich primers (one of which is a urethane) with polyamide epoxy topcoats
1	Multi-component organic zinc-rich primer with a polyamide epoxy intermediate and aliphatic urethane topcoat
2	Moisture-cured urethanes with an aliphatic urethane topcoat
1	Moisture-cured urethane with a polyamide epoxy intermediate and an aliphatic urethane topcoat
1	Polyamide epoxy primer with an aliphatic urethane topcoat
1	Synthetic resin primer with a polyamide epoxy topcoat
1	Aluminum-filled epoxy-urethane mastic
1	Aluminum-filled epoxy mastic
1	Zinc-filled and aluminum-filled epoxy mastic with hi-build aliphatic urethane topcoat
1	4-coat, no lead, no chromate alkyd

qualified products list and specify cleaning procedures for salt-contaminated weathering steels. To date, it would appear that one-coat systems, alkyd systems, and chlorinated rubber systems are not suitable. Organic zinc-rich systems appear to protect best. They seem to tolerate a wider variation in surface preparation. Some of the other systems may be satisfactory, especially when cost is considered. It would appear that remedially painted bridges can perform well when suitable cleaning procedures and coating systems are selected [Tinklenberg 1982].

Table 58 describes the surface preparations and coating systems that were used for the remedial painting of nine weathering steel bridges. The bridge on Tuscarawas County Road 99 over Sugar Creek was painted 2 months after construction, as a precautionary measure, when it became apparent that the other weathering steel bridge in the County, on Road 37 over Little Still Water Creek, was corroding severely and had to be rehabilitated.

Other severely corroded weathering steel bridges are scheduled for remedial painting, namely: four bridges on Big Salt Lake Road, Prince of Wales Island, Alaska; and several bridges in Michigan. In these cases, the selection of painting systems is awaiting the results of the on-going Michigan study.

Initial experience with remedial painting shows that it takes more than the usual amount of sand to blast weathering steel clean, and more than the usual amount of paint to coat it [Baron 1983].

Table 58. Systems used for remedial painting of weathering steel bridges.

Year Opened to traffic	Year Painted	Surface Preparation and Paint System
<u>Brand Road Bridge, Franklin County, Ohio, [Circle 1983]:</u>		
1979	1983	- Sand blasted to bare metal - Two coats of red-lead primer - One coat of oil-alkyd paint
<u>Bridge on County Road 37 over Little Still Water Creek, Tuscarawas County, Ohio, [Smith 1983]:</u>		
1973	1979	- Sand blasted - One coat of zinc-rich primer - One coat of modified alkyd resin paint
<u>Bridge on County Road 99 over Sugar Creek, Tuscarawas County, Ohio, [Smith 1983]:</u>		
1979	1979	- Wire brushed as needed - One coat of zinc-rich primer - One coat of rubberized paint
	1983	- Wire brushed top coat - Touched up primer as needed - One coat of industrial safety green paint
<u>Doullut Canal Bridge, Empire, Louisiana [Garrido 1983, Dunn 1983]:</u>		
1975	1983	- Sand blasted to near-white condition (SSPC-10) - Two coats of organic zinc-rich primer, 115 μm (4.5 mils) thick - One coat of vinyl, 38 μm (1.5 mils) thick
<u>Six Bridges in Butler County, Ohio [Brooks 1983]:</u>		
After 1971	1983	- Sand blasted - One coat of asphalt-base paint

CONNECTIONS

ARC WELDING

Most welded connections of bridge members are made by the shielded-metal-arc (SMA), submerged-arc (SAW) and flux-cored arc (FCA) processes. A few bridges have electroslag welded flanges, but the use of this process is no longer permitted for main structural tension members.

The weldability of steel is a complex property involving: (1) the sensitivity to weld and heat-affected-zone (HAZ) cracking; and (2) the toughness in both the weld and HAZ, as required by service conditions and temperatures. The first is a welding-fabrication problem, the second a service problem. Most welding cracks are termed cold cracks or delayed cracks because they usually occur at temperatures lower than 392 F (200 C). They consist of toe cracks, root cracks, and underbead cracks, and are caused by a combination of: (1) lower ductility due to hardening of the HAZ; (2) embrittlement due to diffusion of hydrogen into the HAZ, and (3) residual welding stresses in high restraint details and thick plates.

The so-called carbon equivalent (CE) is a convenient index of the cold cracking susceptibility of welds. Experimentally determined CE formulas take several forms, depending mainly on their applicability to certain types of steel. The Dearden-O'Neill formula, developed for the safe welding of carbon-manganese steel, shows how the various alloying elements influence hardenability.

$$\text{Carbon Equivalent (CE)} = C + \frac{\text{Mn}}{6} + \frac{\text{Cr} + \text{Mo} + \text{V}}{6} + \frac{\text{Ni} + \text{Cu}}{16} \quad (56)$$

The numbers to be substituted for the elements are the alloy contents in weight percent. As Eq. 56 shows, some elements used for enhancing atmospheric corrosion resistance and for increasing strength through solid-solution or precipitation strengthening also increase the susceptibility to cold cracking. In HSLA steels, this adverse effect is offset in part by limiting the carbon content.

Table 59 shows weldability rating scales in terms of CE, used by two Canadian steel producers [SCC 1973, DFSL 1975]. They are quite similar. The first producer reported 0.44 mean CE and 0.50 maximum CE for its multiple-alloy weathering steel. According to Table 59, the mean CE value implies good weldability.

The Canadian Standard G40.21 for Structural Quality Steel cites Eq. 56. The ASTM specifications A242, A588, and A709 and the product literature issued by American steel manufacturers do not mention CE values. If one were to substitute in Eq. 56 the maximum alloy contents for the various grades of weathering steel, as specified in the A588 specification, one would obtain CE values no greater than 0.71 percent. Actual

Table 59. Weldability rating based on carbon equivalent values.

Weldability Rating	Range of Carbon Equivalent Values	
	Producer A	Producer B
Excellent	CEV \leq 0.40	CEV \leq 0.35
Very good		0.36 \leq CEV \leq 0.40
Good	0.41 \leq CEV \leq 0.45	0.41 \leq CEV \leq 0.45
Fair	0.46 \leq CEV \leq 0.52	0.46 \leq CEV \leq 0.50
Poor	CEV $>$ 0.52	CEV $>$ 0.50

values are much lower because the content of all alloying elements need not be simultaneously maximized to achieve the desired strength and corrosion properties. The carbon equivalent of a specific heat of weathering steel can be determined from the chemical composition given in the mill report.

Hydrogen embrittlement of the HAZ can be minimized by controlling the hydrogen content of the weld metal. This is achieved by using low-hydrogen electrodes, proper drying of electrodes fluxes and gases, and other suitable techniques for minimizing the hydrogen content of the molten weld metal.

Residual welding stresses and applied stresses, in combination with cold cracks, can produce a condition critical for unstable crack extension, particularly in thick sections and restrained welds. To control this possibility, A242, A588, and A709 Grade 100W steel plates are preheated before welding, so as to retard the rate of cooling after welding [AWS 1984]. Thicker sections dissipate heat faster and are, therefore, preheated to higher temperatures. The higher the hardenability of the steel, the greater is the need for preheating, because higher hardenability promotes the formation of martensite and increases the probability of cold cracking. Heat input is also important because it affects the rate at which the weld cools after the arc has passed on. The more heat is introduced by the arc, the slower the cooling rate. Heat input is related to arc voltage, welding current, and travel speed.

Another form of cracking, called solidification or hot cracking, may occur when during the solidification of the weld metal the interdendritic liquid (the last region to freeze) freezes at a substantially lower temperature than the bulk solidified dendrite. Under these conditions, shrinkage stresses produce interdendritic cracks. This type of cracking is most commonly caused by the presence of low-melting iron sulfides and alloy sulfides that wet the dendritic surfaces. Carbon, phosphorus, and sulfur are harmful alloys. The A588 specification, for example, limits the maximum content to about 0.20 percent C, 0.04 percent P and 0.05 percent S. Hot cracking is controlled by limiting the amount and type of sulfides that form and the amount of minor alloying elements that may promote it. Manganese-to-sulfur ratios of 30 or more help to prevent hot cracking of weld metal. Maximum amounts of manganese and sulfur, listed in the A588 specification, give about this ratio.

The previous discussion briefly highlighted some of the more important factors that need to be considered in welding low-alloy steels. They help to emphasize the importance of following proper welding procedures so that a sound weld is obtained.

Weathering steels supplied to ASTM Specifications A242, A588, and A709 Grades 50W and 100W are weldable. Their weldability is comparable to that of their A572 and A514 counterparts. Weathering steels can be welded to other structural steels without difficulty.

The welding technique is of fundamental importance. The adopted procedure must be suitable for the steel being welded and its intended use.

SPECIFICATIONS

Weathering steels, especially those for bridge applications, are weldable with the use of good welding practice and the proper choice of filler metal. Carbon steel or low-alloy steel electrodes or electrode-flux combinations may be used, provided that the welding procedure ensures a sound weldment that matches the mechanical properties and the atmospheric corrosion resistance of the base steel.

All weathering steels require a minimum preheat and interpass temperature. This is a characteristic of all high strength steels in thick sections. In addition, A709 Grade 100W steel has limitations on maximum preheat and interpass temperatures and limitations on welding heat input. The provisions of the American Welding Society (AWS) "Structural Welding Code" (latest edition) and the AASHTO "Standard Specifications for Welding Structural Steel Highway Bridges" (latest edition), in conjunction with the steel producer's recommendations, ensure that the filler metal, preheat temperature, and other relevant welding parameters provide suitable welded joints for the intended application [AWS 1984, AASHTO 1981].

Filler Metal Requirements

The filler metal requirements for welding of weathering steels are given in Table 60. Both AWS and AASHTO specify low hydrogen electrodes for SMA welding of weathering steels. Covered carbon steel electrodes (AWS A5.1) and bare carbon steel electrode-flux combinations (AWS A5.17) listed in Table 60 are used for: (1) underlying passes of multiple-pass weldments which need not be corrosion resistant; (2) single-pass SMAW up to 0.25-in. (6.4-mm) maximum weld dimension; (3) single-pass SAW, GMAW, and FCAW up to $\frac{3}{16}$ -in. (8.0-mm) maximum weld dimension; and (4) painted structures. Covered low-alloy steel electrodes (AWS A5.5) and bare low-alloy steel electrode-flux combinations (AWS A5.23) listed in Table 60 are used for exposed, bare applications of ASTM A242, A588, and A709 Grade 50W steel requiring weld metal with atmospheric corrosion resistance and coloring characteristics similar to the base metal. At least two layers of the exposed surfaces and edges of multiple-pass weldments must be deposited with these electrodes provided the underlying layers are deposited with one of the filler metals specified in Table 60.

Minimum Preheat Interpass Temperature

The minimum preheat and interpass temperatures for welding of weathering steels are listed in Table 61. When the base metal

temperature is below the temperature given in this table for the thickness of material being welded, it shall be preheated (except as otherwise provided) in such manner that the parts on which the weld metal is being deposited are above the specified minimum temperature for a distance equal to the thickness of the part being welded but not less than 3 in. (76 mm) in all directions from the welding point. Quenched and tempered steel A709 Grade 100 W steels should not be preheated or welded at an interpass temperature above a maximum temperature, depending on grade and thickness, or the steel would lose some of its strength and toughness. The AASHTO specification requires higher preheat temperatures for fracture critical nonredundant members made of A588 and A709 Grade 100W steels than for redundant members made of the same steels [AASHTO 1978].

ELECTROSLAG WELDING

The electroslag welding (ESW) is a butt-welding process wherein coalescence is produced by molten slag which simultaneously melts the filler and the parent metals. The slag shields the full cross-section of the joints and remains molten by its resistance to electric current passing between the electrode and the metal. Welding is done in the vertical or near vertical position, and joints are completed in a single pass for any thickness by using one or more electrodes.

As the molten slag pool and the weld pool rise in the joint, they are contained by two molds or shoes made of copper or sometimes steel plates. The copper shoes are either solid (dry shoe) or hollow for cooling water circulation (cooled shoe). One study indicates that reducing the weld cooling rate during and after welding by the use of dry versus cooled shoes reduced the number of grain boundary separations in an A588 electroslag weldment [Konkol 1979]. Other reports indicate that slow cooling of electroslag weldments prolongs the thermal cycle and results in "coarse casting" of the weld metal and the HAZ. Electroslag weldments can be made with consumable and non-consumable guide tubes. The consumable guide tube method is used more often because of equipment and set-up simplicity.

The main advantage of electroslag welding is the savings in manpower, time, and welding consumables. In addition, the joint does not require machine edge preparation, and a proper welding procedure eliminates the necessity for preheating. The very high heat input and prolonged thermal cycle, with accompanying slow solidification and cooling rates, results in a "coarse casting" type of weldment structure with anisotropic and nonhomogeneous large-grained weld metal and extremely large HAZ. Several studies showed that the HAZ had at the center both a lower yield point and a lower Charpy V-notch energy than the surrounding weld metal, and sometimes lower than the base metal [Paton 1962a, 1962b, Culp 1976; Noel 1972]. Others have confirmed these results and have further reported a loss in ductility in the weld metal as compared to the base metal [Norcross 1965, Woodley 1966, Campbell 1970].

Following the failure near Pittsburgh, Pa., of the Interstate I-79 bridge made of A588 steel, the Federal Highway Administration banned the use of the electroslag welding process for main structural tension members on any Federal-aid project. The AWS "Structural Welding Code-Steel" also prohibits the use of this process for welding quenched and tempered steels and for welding bridge members subjected to tension or stress reversal [AWS 1984].

Table 60. Filler metal requirements for welding weathering steels. [AWS 1984]^g

ASTM Steel Designations ^{a,b}	Matching Filler Metal Requirements		Filler Metal Requirements for Exposed Base Applications ^c	
	Shielded Metal	Submerged	Shielded Metal	Submerged
	Arc	Arc	Arc	Arc
A242 ^d , A588 ^d and A709 Grade 50W	<u>AWS A5.1</u>	<u>AWS A5.17 or A5.23</u>	<u>AWS A5.5</u>	<u>AWS A5.23^f</u>
	E7015	F7X-EXXX	E7018-W	F7AX-EXXX-W
	E7016		E8018-W	
	E7018		E8016-C3 or E8018-C3	F7AX-EXXX-Ni ^e
	E7028		E8016-C1 or E8010-C1	F7AX-EXXX-Ni ^e
A709 Grade 100W t ≤ 63 mm (t ≤ 2.5 in)	<u>AWS A5.5</u>	<u>AWS A5.23</u>	E8016-C2 or E8016-C2	F7AX-EXXX-Ni ^e
	E11015	F11X-EXXX ^f	E7016-C1L or E8018-C1L	F7AX-EXXX-Ni ^e
	E11016		E7016-C2L or E8018-C2L	F7AX-EXXX-Ni ^e
	E11018		E8018-B2L ^f	
A709 Grade 100W 63 mm < t ≤ 102 mm (2.5 in < t ≤ 4 in)	<u>AWS A5.5</u>	<u>AWS A5.23</u>		
	E10015	F10X-EXXX ^f		
	E10016			
	E10018			

Notes

- In joints involving base metals of two different groups low-hydrogen filler metal requirements applicable to the lower strength group may be used. The low-hydrogen processes used shall be subject to the technique requirements applicable to the higher strength group.
- When welds are to be stress-relieved, the deposited weld metal shall not exceed 0.05 % vanadium.
- Deposited weld metal shall have the following chemical composition: C, max 0.12%, Mn 0.50%/1.30%, P, max 0.04%, S max 0.04%, Si 0.35/0.80%, Cu 0.30/0.75%, Ni 0.40/0.80%, Cr 0.45/0.70%.
- Special welding materials and procedures (e.g., E80XX low alloy electrodes) may be required to match the notch toughness of base metal (for applications involving impact loading or low temperature), or for atmospheric corrosion and weathering characteristics.
- The use of the same type of filler metal having next higher tensile strength as listed in AWS specifications is permitted.
- Deposited weld metal shall have a minimum impact strength of 27.1 J (20 ft. lb) at -18°C (0°F) when Charpy V-notch specimens are used. This requirement is applicable only to bridges.
- See AWS specifications for filler metal requirements for gas metal arc and flux cored arc welding.

Table 61. Minimum preheat and interpass temperature for welding of weathering steels. [AWS 1984]

ASTM Steel Designation	Welding Process	Thickness of Thickest Part at Point of Welding mm (in)	Minimum Temperature ^{a,c} °C (°F)
A242, A588 and A709 Grade 50W	SMAW with low hydrogen electrodes, SAW, GMAW and FCAW	$t < 19$ (0.75)	None
		$19 (0.75) < t \leq 38$ (1.50)	10 (50)
		$38 (1.50) < t \leq 64$ (2.50)	66 (150)
		$64 (2.50) < t$	107 (225)
A709 Grade 100W ^b	SMAW with low hydrogen electrodes, SAW with carbon or alloy steel wire, neutral flux, GMAW or FCAW	$t < 19$ (0.75)	10 (50)
		$19 (0.75) < t \leq 38$ (1.50)	50 (125)
		$38 (1.50) < t \leq 64$ (2.50)	80 (175)
		$64 (2.50) < t$	107 (225)

- Welding shall not be done when the ambient temperature is lower than -18°C (0°F). When the base metal is below the temperature listed for the welding process being used and the thickness of material being welded, it shall be preheated (except as otherwise provided) in such manner that the surfaces of the parts on which weld metal is being deposited are at or above the specified minimum temperature for a distance equal to the thickness of the part being welded, but not less than 76 mm (3 in) in all directions from the point of welding. Preheat and interpass temperatures must be sufficient to prevent crack formation. Temperature above the minimum shown may be required for highly restrained welds.
- The maximum preheat and interpass temperature shall not exceed 205°C (400°F) for thickness up to 38 mm (1.5 in) inclusive, and 230°C (450°F) for greater thickness. The heat input shall not exceed the steel producer's recommendations.
- In joints involving combinations of base metals, preheat shall be as specified for the higher strength steel being welded.

The bar on the use of electroslag welding in bridges is basically a technological matter. It was believed that the electroslag welding technology had not reached the point where a defect free welding could be made. Separations larger in size than the critical flaw were detected. Reinspection of electroslag welds in highway bridges revealed a high percentage of welds containing unacceptable weld defects, and some were judged to be structurally unsafe [Atteridge 1982].

In order to yield consistent and defect-free welds, substantial changes are needed in ESW fabrication and inspection technology. One study sponsored by the FHWA developed and assessed fabrication methods aimed at improving ESW technology [Atteridge 1982]. The results indicated that defect-free ESW could repeatedly be made if certain welding procedures were followed.

It was concluded that commercial operations tend to weld at too low a slag level and at too high a current. The customary technique of adding relatively large amounts of flux to the slag pool accentuates the welding problem. Relatively simple solutions for quality control were suggested by Atteridge et al. as follows [Atteridge 1982]:

- Use a sliding shoe(s), as in the nonconsumable welding method, to allow constant access to the slag pool.
- Add a simple but accurate current monitoring device.
- Continuously add flux.

Also suggested were the following fabrication changes to upgrade the technology and reduce anisotropy and nonhomogeneity of electroslag weldments:

- Use a consumable winged guide tube instead of a cylindrical guide tube to eliminate guide tube misalignment.

Table 62. Tensile requirements for high strength bolts.

ASTM (AASHTO) Specification	Bolt size mm (in)	Tensile Strength MPa (ksi)	Proof Stress, Length Measurement Method MPa (ksi)	Alternative Proof Stress, Yield Strength Method MPa (ksi)
A325 (M164)	$12 < t \leq 25$ ($1/2 < \bar{t} \leq 1$)	825 (120)	585 (85)	635 (92)
	$29 < t \leq 38$ ($1-1/8 < \bar{t} \leq 1-1/2$)	725 (105)	510 (74)	560 (81)
A490 (M253)	$12 < t \leq 38$ ($1/2 < \bar{t} \leq 1-1/2$)	1035-1170 (150-170)	825 (120)	895 (130)

- Use narrow gap ($3/4$ in. (19 mm)) high current (1,000 Amp) welding in combination with a winged guide tube to: (a) decrease the specific heat input; (b) increase the welding speed; and (c) most importantly, to improve the mechanical properties of the "weld centerline at midthickness location.

- Use a thin quartz shroud for guide tube shielding and intense mechanical stirring of the weldpool to achieve grain refinement and homogeneous weldment throughout the length. This however requires further research.

HIGH-STRENGTH BOLTED JOINTS

High-strength bolts are furnished to the requirements of ASTM specifications A325 and A490. Both are made of quenched and tempered steel. Their minimum tensile requirements are summarized in Table 62.

Table 63. Chemical requirements for Type 3 bolts based on product analysis.

Class	Composition %									
	C	Mn	P, max	S, max	Si	Ni	Cr	Mo, max	Cu	V, min
A325 Type 3 Bolts (M164)										
A	.31	.86	.045	.055	.13	.22	.42		.22	
	.42	1.24			.32	.48	.68		.48	
B	.36	.67	.06		.25	.47	.47	.07	.17	
	.50	.93	.125	.055	.55	.83	.83		.43	
C	.14	.76	.040	.045	.13	.22	.27		.17	.010
	.26	1.39			.32	.53	.53		.53	
D	.14	.36	.045	.055	.20	.47	.45	.11	.27	
	.26	1.24			.55	.83	1.05		.53	
E	.18	.56	.045	.045	.13	.27	.55		.27	
	.27	1.04			.32	.63	.95		.63	
F	.19	.86	.045	.045	.13	.17	.42		.17	
	.26	1.24			.32	.43	.68		.43	
A490 Type 3 Bolts (M253)										
	.19	.37	.045	.055		.17	.42	(a)	.63	
	.55	min	max	max		min	min.		max	

Note:

a. 0.17% Ni min can be replaced by 0.14% Mo min.

The bolts come in three types. Type 1 and Type 2 are intended for use in painted construction. Type 3 bolts have atmospheric corrosion resistance and weathering characteristics comparable to those of weathering steels covered in the A242, A588, and A709 specifications. The chemical requirements are given in Table 63. A325 Type 3 bolts come in five classes, each with a distinct chemical composition. A490 Type 3 bolts come in one general class, with minimum and maximum limits set on each alloying element.

High-strength bolts must be tightened to a minimum tension equal to 70 percent of the specified bolt tensile strength. The high installation forces introduce high prestresses between the contact surfaces in a region equal to about two bolt diameters. Within this region, the joints are well seated. The joints are designed to resist shearing forces either as friction-type or bearing-type connections. Slip along the contact surfaces is prevented in the former but not in the latter case.

The crevice between the plies of a bolted weathering steel joint usually seals itself with oxide if the joint is tight and does not move. If the joint moves, the surfaces should be coated with a protective material and filled with a suitable sealant to avoid progressive corrosions [Bethlehem 1983].

The proper design of high-strength bolted weathering steel joints needs to consider: (1) corrosion of the contact surfaces; and (2) slip resistance of friction-type joints with mill scale and blast cleaned surfaces. The first aspect is examined next; the second aspect is examined in the sections under "Friction-Type Joints with Mill Scale Surfaces" and "Friction-Type Joints with Blast-Cleaned Surfaces."

Experience with bolted joints in exposed frameworks of

weathering steel has indicated that if the stiffness of the joint is adequate and the joint is tight, the crevice between two contact surfaces seals itself as corrosion products form around the periphery of the joint. However, if the joint design does not provide sufficient stiffness, continuing crevice corrosion and subsequent accumulation of corrosion products in the crevices induce expansion forces which can: (1) deform the connected elements, and (2) cause large tensile loads on the bolts.

Brockenbrough inspected weathering steel transmission towers with joints connected by bolts furnished to another specification, but having a measured tensile strength approaching that of A325 bolts [Brockenbrough 1983]. The bolts were only snug tightened. It was found from these inspections that the plates between two bolts did not significantly bow when the pitch did not exceed 14 times the plate thickness or 7 in. (178 mm). Similarly, the plate edges did not significantly lift up when the edge distance was limited to 8 times the plate thickness or 5 in. (127 mm). The limits on maximum pitch and edge distance are comparable to those specified by AASHTO for all steel structures. These guidelines were confirmed by observations made on 45 bolted joint specimens exposed about 7 years at Kure Beach, North Carolina, and Monroeville, Pennsylvania [Brockenbrough 1982]. Joint designs that met the suggested maximum pitch and edge distance developed little bowing between bolts or edge distortion.

To determine the expansive force of corrosion products, Brockenbrough analyzed and tested tower joints removed from service. Each joint consisted of $6 \times 6 \times \frac{5}{16}$ in main angles spliced with $6 \times 6 \times \frac{5}{16}$ in splice angles $23\frac{3}{4}$ in. (594 mm) long. Each leg was connected with four $\frac{5}{8}$ -in. (16-mm) diameter

staggered bolts on either side of the splice center line on a 6-in. (152-mm) pitch and 2¼-in. (57-mm) gage. Corrosion products caused the corners of the angles to lift up by 0.43 in. (11 mm). Comparison of the measured lift-up with the displacement predicted by a finite element analysis showed that the corrosion products developed an equivalent uniform pressure of 1.2 ksi (8.3 MPa). At this pressure, the average measured bolt tensile stress was about 70 percent of the measured bolt tensile strength, reaching 95 percent for the bolt closest to the corner. Apparently, the bolts would have failed had corrosion continued. It should be kept in mind that these bolts were tightened snug and widely spaced.

Yura and Frank reported a 0.8-mil (20- μ m) average increase in elongation of A325 Type 3 bolts in A588 steel joints after one year weathering [Yura 1981]. At that rate, and considering the ductility of the bolt, it would take about 100 years to fail the bolt in tension. They concluded from this observation that the expansive force caused by corrosion of the contact surfaces will not adversely affect the bolt performance in A588 steel joints with bolts placed in compact patterns.

The initial bolt tension, the AASHTO limitations on bolt spacing, and the generally thicker sections should make bridge girder joints less susceptible to crevice corrosion than the tower joints examined by Brockenbrough.

Knotkova et al. reported that sufficiently tightened bolted joints on girder models exhibited no significant corrosive attack even when exposed in highly aggressive environments [Knotkova 1982a]. The character of the external and internal joint surfaces, after pickling, was not significantly different. The rate of corrosion within the joint was not excessive. The quantity of rust measured on the internal surfaces was smaller than that on the external surfaces. The most suitable bolting material was found to be high-strength low-alloy weathering steel. Chrome-plated high-strength bolts were less suitable, and standard structural steel bolts were least suitable. Galvanized bolts were also reported to be unsuitable for weathering steel joints and are not recommended by the investigators for use in heavily polluted atmospheres.

FRICION-TYPE JOINTS WITH MILL SCALE SURFACES

Slip Resistance

The slip resistance of high-strength bolted joints depends on the surface condition of the steel plates. The slip coefficient is determined from experimental data using the expression:

$$k_s = \frac{P_s}{nmT_i} \quad (57)$$

in which: P_s equals measured slip load, n equals number of bolts, m equals number of shearing planes, and T_i equals initial bolt tension. The slip load can also be expressed in terms of a fictitious slip shear stress on the gross area of all bolts, that is $P_s = nm f_s A_b$. The specified minimum bolt tension is $T_i = 0.70 F_u A_s$, in which A_s is the tensile stress area, and F_u is the tensile strength of bolt. Substituting P_s and T_i in Eq. 57, and recalling that $A_s/A_b \cong 0.75$, gives:

$$k_s = \frac{f_s}{0.525 F_u} \quad (58)$$

The allowable shearing stress for friction-type joints is obtained by substituting the allowable slip coefficient in Eq. 58 and solving for:

$$F_v = 0.525 F_u k_{s,all} \quad (59)$$

The tensile strength for A325 bolts is $F_u = 120$ ksi (830 MPa) for ½ to 1 in. (12 to 25 mm) diameter, and $F_u = 105$ ksi (725 MPa) for 1⅞ to 1½ in. (28 to 38 mm) diameter. For A490 bolts, it is $F_u = 150$ ksi (1,035 MPa).

AASHTO Allowable Shear Stresses

In 1957, AASHTO specified, for the first time, an allowable shear stress of 13.5 ksi (93 MPa) for A325 bolts. In 1965, they specified an allowable shear stress of 18 ksi (124 MPa) for A490 bolts. These values, shown in Figure 102 with solid and dashed lines, respectively, were based on early data for A7 and A440 steels.

In 1974, Fisher and Struik surveyed the literature and reported that for 180 tests of A7, A36, and A440 steel specimens with clean mill scale surfaces the average slip coefficient was $\bar{k}_s = 0.322$ and the standard deviation $s = 0.062$ [Fisher 1974]. They also reported that the mean clamping force measured in laboratory tests was higher than the minimum specified bolt tension, by 35 percent for turn-of-nut tightening, and by 13 percent for calibrated wrench tightening. On the basis of the Fisher [1974] findings, and assuming a 5 percent slip probability, the allowable slip coefficient was calculated from

$$k_{s,all} = 1.13 (\bar{k}_s = 1.645 s) \quad (60)$$

Accordingly, AASHTO in 1979 increased the allowable shear stress for clean mill scale surfaces of all steels to $F_v = 0.525 \times 120 \times 1.13 (0.322 - 1.645 \times 0.062) = 15.7$ ksi \cong 16 ksi (110 MPa) for A325 bolts, and to $F_v = 20$ ksi (138 MPa) for A490 bolts (Eq. 59). These values are plotted in Figure 102.

In 1981, Frank and Yura developed statistically reliable slip resistance data for bolted joints with coated contact surfaces. In that study, they also proposed that the allowable shear stress for friction-type joints should be based on a 1 percent slip probability or a mean safety factor of 1.45, whichever gave the smaller value [Frank 1981], that is:

$$k_{s,all} = \bar{k}_s - 2.326 s \quad (61)$$

or:

$$k_{s,all} = \frac{\bar{k}_s}{1.45} \quad (62)$$

In their opinion, the possible increase of the clamping force above the minimum specified bolt tension should not be considered in the development of design recommendations.

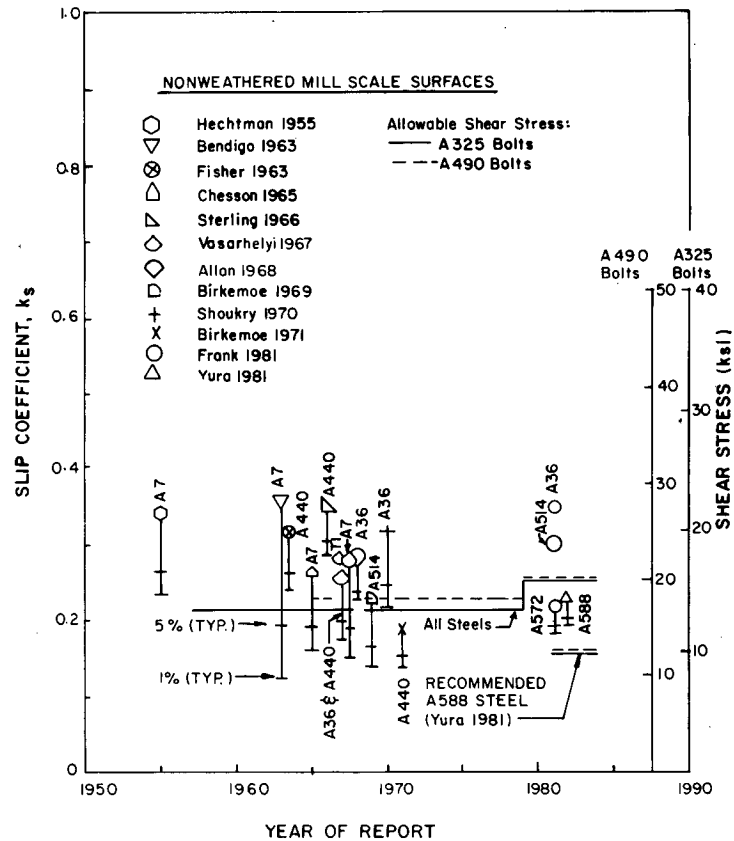


Figure 102. Slip coefficients and allowable shear stresses for friction-type bolted joints with nonweathered mill scale surfaces.

In a companion study, Yura et al. [Yura 1981] found that the mill scale on A588 weathering steel was, on average, much less resistant to slip ($\bar{k}_s = 0.23$) than the mill scale on A7, A36 and A440 steels ($\bar{k}_s = 0.322$) [Fisher 1974]. After analyzing their data with Eqs. 59, 61, and 62, Yura, et al., recommended that a new Class of Surface J for A588 clean mill scale be added to Table 1.7.41C2 of the AASHTO specifications, with $F_v = 10$ ksi (69 MPa) for A325 bolts and $F_v = 13$ ksi (90 MPa) for A490 bolts. These proposed values are also plotted in Figure 102.

Review of Previous Data

The decrease in allowable shear stress for A588 steel with clean mill scale surfaces is due in large part to a reduced slip resistance and in small part to a change in the criteria for calculating the allowable shear stress. To place the values in proper perspective, it was necessary to collect and analyze all slip resistance data available to date.

The first part of Table 64 summarizes the data for nonweathered mill scale surfaces that were obtained from 254 tests of A7, A36, A440, A572, A588, and A514 steel specimens. They were obtained from 12 domestic studies. For each series of tests entered in the table, a data point was plotted in chronological order in Figure 102. The vertical dimension line below each point indicates the data scatter. The tick marks were drawn at

the 5 percent (1.645 s) and 1 percent (2.326 s) levels of slip probability.

The second part of Table 64 summarizes the same data by type of steel. The allowable shear stresses were calculated for each series of tests and for various combinations of type of steel. The column for 5 percent slip probability, excluding the factor of 1.13 for increased bolt tension (Eq. 60), corresponds to the AASHTO criterion. The columns for 1 percent slip probability (Eq. 61) and 1.45 safety factor (Eq. 62) correspond to the criteria proposed by Frank and Yura [Frank 1981].

The analysis of all previous data in terms of the new criteria suggests the following allowable shear stresses for clean mill scale surfaces.

TYPE OF STEEL	ALLOWABLE SHEAR STRESS F_v , [ksi (MPa)]	
	A325 BOLTS	A490 BOLTS
<i>Based on Frank and Yura's criteria:</i>		
A36 carbon steel	13.5 (93)	17.0 (116)
A572 and A588		
HSLA steels	10.0 (69)	12.5 (86)
A514 quenched and tempered steel	9.0 (62)	11.3 (76)
<i>Based on Fisher and Struik's criteria:</i>		
All steels	16.0 (110)	20.0 (138)

Table 64. Slip coefficients and allowable shear stresses for friction-type bolted joints with nonweathered mill scale surfaces.

Reference	Type of Steel	Weathering Time (days)	No. of Tests n	Average Slip Coefficient k_s	Standard Deviation s	Allowable Shear Stress for A325 Bolts ^c			
						5% (ksi)	1% (ksi)	FOS = 1.45 (ksi)	
Nonweathered Specimens									
Hechtman	1955	A7	0	58	0.342	0.046	16.8	14.8	14.8
Bendigo	1963	A7	0	20	0.359	0.101	12.1	7.8	15.6
Fisher	1963	A440	0	14	0.316	0.033	16.5	15.1	13.7
Chesson	1965	A7	0	11	0.262	0.043	12.0	10.2	11.4
Sterling	1966	A440	0	8	0.348	0.026	19.2	18.1	15.1
Vasarhelyi	1967	A36, A440	0	6	0.254	0.033	12.5	11.1	11.0
		A7	0	17	0.278	0.054	11.9	9.6	12.1
		T1	0	2	0.28	--	--	--	12.2
Allan	1968	A36	0	8	0.285	0.014	16.5	15.9	12.4
Birkemoe	1969	A514	0	10	0.226	0.037	10.4	8.8	9.8
Shoukry	1970	A26	0	39	0.32	0.044	15.6	13.7	13.9
Birkemoe	1971	A440	0	19	0.18	0.018	9.5	8.7	7.8
Frank	1981	A36	0	6	0.22	--	--	--	15.2
		A572	0	6	0.22	0.015	12.3	11.7	9.6
		A514	0	2	0.30	--	--	--	13.0
Yura	1981	A588	0	20 ^a	0.250	0.015 ^d	14.2	13.5	10.9
			0	11 ^b	0.20	0.012 ^d	11.3	10.8	8.7
Summary of Nonweathered Specimens									
		A7, A36, A440	0	203	0.306	0.071	11.9	8.9	13.3
		A36	0	52	0.313	0.043	15.3	13.4	13.6
		A572	0	6	0.217	0.015	12.1	11.5	9.4
		A588	0	31	0.23	0.034	11.0	9.5	10.0
		A572, A588	0	37	0.23	0.015	12.9	12.3	10.0
		A514	0	14	0.244	0.043	10.9	9.0	10.6

NOTES: a. Specimens tested at Federal Highway Administration.
b. Specimens tested at University of Texas.
c. For A490 bolts multiply shear stresses by 150 ksi/120 ksi = 1.25.
d. Based on sample means and sample standard deviations [Natrella 1963].

The data for A7 and A440 steels were deleted because: (1) ASTM discontinued these specifications in 1968 and 1979, respectively; (2) these steels have not been used in bridge construction for many years; and (3) data for obsolete steels should not be allowed to influence the results for other steels when, as it seems, slip resistance is to some degree a function of the mill scale that forms on different types of steel. That leaves A36, a carbon steel, with the same allowable shear stress used prior to 1979. All others would have lower allowable stresses.

Both HSLA steels, A572 and A588, should have the same allowable shear stress. For convenience of limiting the number of classes of surface, A514 steel could be lumped with the HSLA steels. The allowable shear stresses for A709 Grade 100W weathering steel should be set at the same level as those for A514 steel.

The aforementioned discussion showed that the lower mean slip resistance and a change in criteria have led to the reduced allowable shear stresses for A588 steel proposed by Yura et al. There is an obvious need to choose one set of criteria for calculating allowable shear stresses and to apply them uniformly to all steels, not just weathering steel.

Weathered Mill Scale Surfaces

Slip resistance data for steel plates with weathered mill scale surfaces are available from two studies [Kuperus 1966, Yura 1981]. The former was performed on European Fe 37 and Fe 50 steels with no enhanced atmospheric corrosion resistance, the latter on A588 steel. The specimens were: (1) bolted and then weathered; or (2) weathered and then bolted.

The data are summarized in Table 65 and plotted in Figure 103. The mean slip coefficient for A588 plates that were bolted and then weathered at most to one year did not appreciably change with time of exposure. However, the mean slip coefficient increased significantly when the plates were first weathered and then bolted prior to testing. The rate of increase was much greater for Fe 37 and Fe 52 steels than for A588 steel. Evidently, it takes more time for A588 to lose its hard mill scale through corrosion.

The results of the calculations summarized in Table 65 show that the allowable shear stresses for nonweathered mill scale surfaces can also be applied to their weathered counterparts. The small apparent losses exhibited by the bolted and then weathered A588 steel specimens could reflect normal variations between small samples rather than effect of weathering. There appears to be no need to protect the mill scale contact surfaces against corrosion, from the time when the steel members leave the shop until they are bolted at the bridge site.

FRICION-TYPE JOINTS WITH BLAST-CLEANED SURFACES

AASHTO Allowable Shear Stresses

Prior to 1979, the allowable shear stresses for friction-type joints with blast-cleaned surfaces were the same as those for mill scale surfaces. These values of $F_v = 13.5$ ksi (93 MPa) for A325 bolts and $F_v = 18$ ksi (124 MPa) for A490 bolts, shown in Figure 104 with solid and dashed lines, respectively, were based on early data for A7 and A36 steels.

Table 65. Slip coefficients and allowable shear stresses for friction-type bolted joints with weathered mill scale surfaces.

Reference	Type of Steel	Weathering Time (days)	No. of Tests n	Average Slip Coefficient k_s	Standard Deviation s	Allowable Shear Stress for A325 Bolts ^c		
						5% (ksi)	1% (ksi)	FOS = 1.45 (ksi)
Bolted and Then Weathered Specimens								
Yura ^b 1981	A588	0	6 ^b	0.20	0.015	11.0	10.4	8.7
	A588	180	5	0.18	0.004	0.91	10.7	7.8
	A588	360	5	0.19	0.011	10.8	9.7	8.2
Weathered and Then Bolted Specimens								
Kuperus 1966	Fe 37	0	4	0.35	0.055	16.3	14.0	15.2
	Fe 37	5	4	0.37	0.040	19.1	17.4	16.1
	Fe 37	14	4	0.48	0.020	28.1	27.3	20.8
	Fe 37	60	4	0.55	0.020	32.5	31.7	23.9
	Fe 52	0	4	0.25	0.020	13.7	12.8	10.9
	Fe 52	5	4	0.27	0.010	16.0	15.5	11.7
	Fe 52	14	4	0.35	0.030	18.9	17.6	15.2
	Fe 52	60	4	0.51	0.06	25.9	23.3	22.1
Yura 1981	A588	0	5 ^b	0.20	0.009	11.7	11.3	8.7
	A588	180	4	0.27	0.009	16.1	15.7	11.7
	A588	360	5	0.37	0.068	16.2	13.3	16.1

NOTES: a. Specimen tested at Federal Highway Administration.
 b. Specimens tested at University of Texas.
 c. For A490 bolts multiply shear stresses by 150 ksi/120 ksi = 1.25.

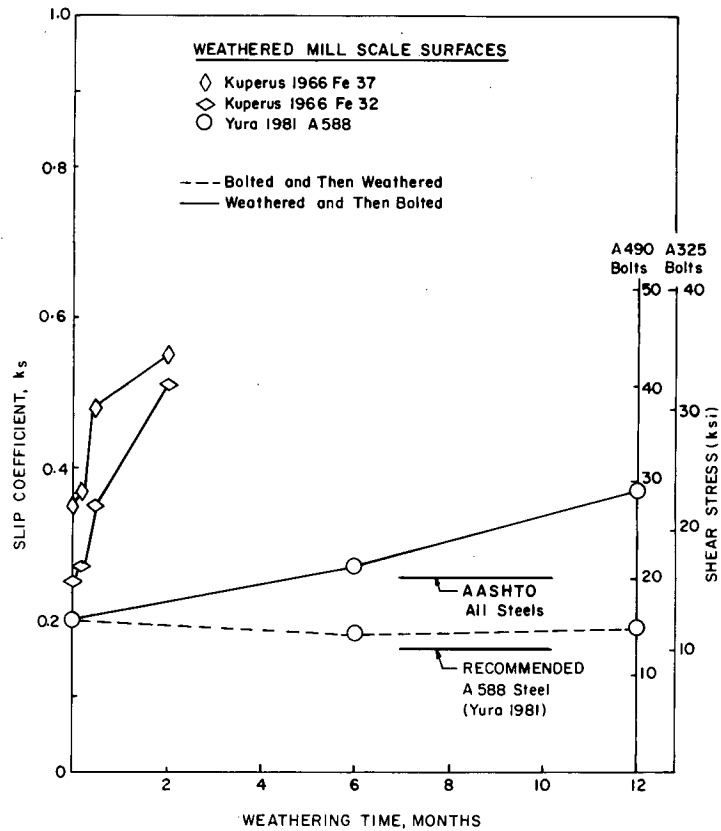


Figure 103. Slip coefficients and allowable shear stresses for friction-type bolted joints with weathered mill scale surfaces.

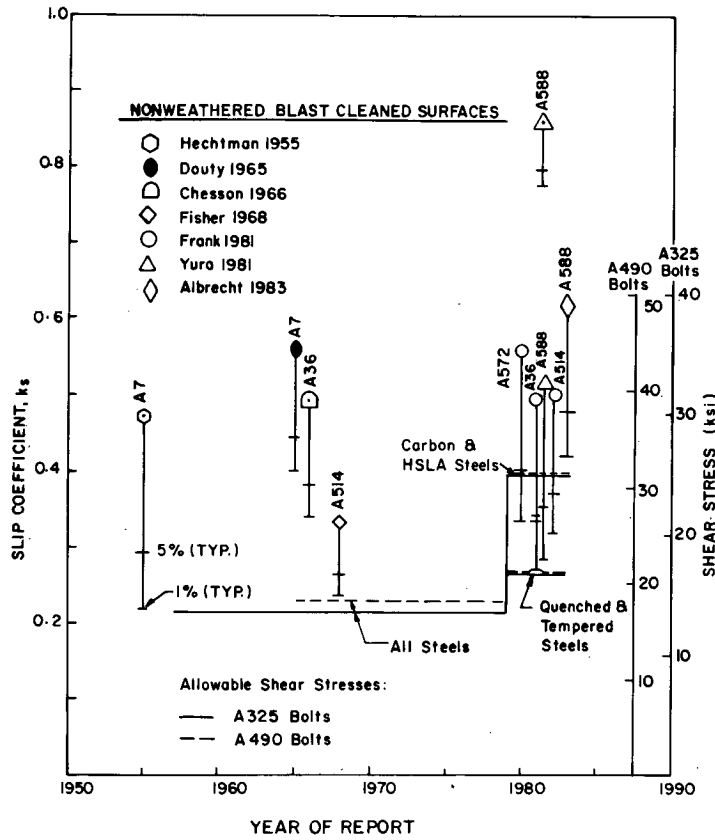


Figure 104. Slip coefficients and allowable shear stresses for friction-type bolted joints with nonweathered blast-cleaned surfaces.

In 1974, Fisher and Struik reported that the average slip coefficient of 168 grit-blasted A7, A36, and European Fe 37 steel specimens tested in several studies was $k_s = 0.493$, with standard deviation $s = 0.074$ [Fisher 1974]. The corresponding values for 19 grit-blasted A514 steel specimens were $k_s = 0.331$ and $s = 0.043$.

It appears that the allowable shear stresses in AASHTO's 1979 Interim Specification were calculated from the aforementioned data, assuming a 5 percent probability of slip and neglecting the factor of 1.13 increase in bolt tension over the minimum specified value. Substituting the means and standard deviations in Eq. 60, one obtains from Eq. 59 for Class of Surface B blast-cleaned carbon and low-alloy steel and A325 bolts:

$$F_v = 0.525 \times 120 (0.493 - 1.645 \times 0.074) = 23.4 \text{ ksi} \approx 25 \text{ ksi (172 MPa)}$$

and for Class of Surface C blast-cleaned quenched and tempered steel and A325 bolts:

$$F_v = 0.525 \times 120 (0.331 - 1.645 \times 0.043) = 16.4 \text{ ksi} \approx 17 \text{ ksi (117 MPa)}$$

These values are also shown in Figure 104.

Review of Previous Data

All slip resistance data for nonweathered blast-cleaned surfaces were collected and are summarized in Table 66 for pur-

poses of comparison. For each series of tests entered in the first part of the table, a data point was plotted in Figure 104 in chronological order. The vertical dimensions below each point indicate the data scatter, with tick marks drawn at the 5 percent and 1 percent levels of slip probability.

The second part of Table 66 summarizes the data by type of steel. The calculations of allowable shear stresses were based on the criteria described in the previous section. On the basis of the recommendations by Frank and Yura [Frank 1981], the data indicate the following allowable shear stresses for blast-cleaned surfaces:

TYPE OF STEEL	ALLOWABLE SHEAR STRESSES, F_v [ksi (MPa)]	
	A345 BOLTS	A490 BOLTS
<i>Based on Frank and Yura's criteria:</i>		
A36 carbon steel	17.6 (121)	22.0 (152)
A572 and A588 HSLA steels	22.2 (153)	27.8 (191)
A514 quenched and tempered steel	14.4 (99)	18.0 (124)
<i>Based on Fisher and Struik's criteria:</i>		
Carbon and low-alloy steels	25.0 (172)	31.0 (214)
Quenched and tempered steels	17.0 (117)	21.0 (145)

Table 66. Slip coefficients and allowable shear stresses for friction-type bolted joints with nonweathered blast-cleaned surfaces.

Reference	Type of Steel	Weathering Time (days)	No. of Tests n	Average Slip Coefficient k_s	Standard Deviation s	Allowable Shear Stress for A325 Bolts		
						5% (ksi)	1% (ksi)	FOS = 1.45 (ksi)
<u>Nonweathered Specimens</u>								
Hechtman 1955	A7	0	3	0.47	0.108	18.4	13.8	20.4
Douty 1965	A7	0	7	0.560	0.068	28.2	25.3	24.3
Chesson 1966	A36	0	5	0.488	0.064	24.1	21.5	21.2
Kulak 1968	A514	0	17	0.332	0.041	16.7	14.9	14.4
Frank 1981	A36	0	21	0.495	0.097	21.1	17.0	21.5
	A514	0	24	0.50	0.077	23.5	20.2	21.7
	A572	0	68	0.561	0.097	25.3	21.1	24.4
Yura 1981	A588	0	5 ^b	0.86	0.038	50.2	48.6	37.3
		0	20 ^a	0.518	0.10	22.2	18.0	22.5
Albrecht 1983d	A588	0	9	0.621	0.086	30.2	26.5	27.0
<u>Summary of Nonweathered Specimens</u>								
	A7, A36	0	36	0.507	0.089	22.7	18.9	22.0
	A36	0	26	0.492	0.091	21.5	17.6	21.4
	A572	0	68	0.561	0.097	25.3	21.1	24.4
	A588	0	34	0.596	0.090 ^d	28.2	24.3	25.9
	A572, A588	0	102	0.573	0.095 ^d	26.3	22.2	24.9
	A514	0	41	0.43	0.106	16.1	11.5	18.7

- Notes:**
- Specimens tested at Federal Highway Administration.
 - Specimens tested at University of Texas.
 - For A490 bolts multiply shear stress by 150 ksi/120 ksi = 1.25.
 - Based on sample means and sample standard deviations [Natrella 1963].

The data for A7 steel were deleted for the reasons cited in the previous section. The HSLA steels have a higher slip resistance than carbon steels.

The data for A514 steels came from two series of tests with vastly different results, namely: $k_s = 0.332$ and $s = 0.041$ [Fisher 1968], versus $k_s = 0.50$ and $s = 0.077$ [Frank 1981]. The combination of the two sets would give an unrealistically high standard deviation and low allowable shear stress. For this reason, the allowable shear stresses for A514 steel in the previous table were based on Fisher's data alone. These stresses should also apply to A709 Grade 100W steel.

The need for uniform criteria for determining allowable shear stresses, cited under "Friction-Type Joints with Mill Scale Surfaces," is again evident.

Weathered Blast-Cleaned Surfaces

Four studies examined the slip resistance of blast-cleaned Fe 37, Fe 52, and A36 Grade 50 steels with no enhanced atmospheric corrosion resistance; as well as A588 and British weathering steels [Kuperus 1966, Lee 1969, Moss 1979, Yura 1981]. Most specimens were weathered and then bolted; only one series of specimens was bolted and then weathered. Weathering times were at most one year.

The data are summarized in Table 67 and plotted in Figure 105. The trend for weathering steel specimens that were weathered and then bolted indicates: (1) an increase in mean slip resistance with weathering time when the 0-year control spec-

imens had low slip resistance; and (2) a decrease when the control specimens had high slip resistance.

The data for nonatmospheric corrosion resistant steels Fe 37, Fe 52, A36 and Grade 50 do not exhibit a consistent trend. Because bolted weathering steel joints of bridge members would normally remain unassembled for, say, at most 3 months, corrosion of unprotected blast-cleaned contact surfaces should not affect the slip resistance.

The slip resistance of joints that were bolted and then weathered (dashed line in Figure 105) decreased slightly over a one-year weathering period. Longer exposures are needed to assess the slip behavior of bolted joints in service.

PIN AND HANGER CONNECTIONS

The crevice between two plies of a bolted weathering steel joint seals itself with corrosion products around the periphery of the joint, if the bolt is tight and the joint stiffness is adequate (see section under "High-Strength Bolted Joints"). However, progressive crevice corrosion occurs when the joints are loose. This was found in pin-and-hanger joints of bridge girders and in bolted highway guardrail joints with widely spaced bolts [Culp 1980, McCrum 1980].

The space between the contact surfaces and between the pin and the hole of pin-and-hanger connections was reported to be tightly packed with dense rust, which could not be removed without disassembling the joint. This has occurred on bare weathering steel bridges after only 7 years of service. In some

Table 67. Slip coefficients and allowable shear stresses for friction-type bolted joints with weathered blast-cleaned surfaces.

Reference	Type of Steel	Weathering Time (days)	No. of Tests n	Average Slip Coefficient k_s	Standard Deviation s	Allowable Shear Stress for A325 Bolts		
						5% (ksi)	1% (ksi)	FOS = 1.45 (ksi)
Bolted and then Weathered Specimens								
Yura ^d 1981	A588	0	5	0.86	0.038	50.2	48.6	37.3
	A588	180	5	0.77	0.035	44.8	43.3	33.4
	A588	360	5	0.75	0.020	45.1	44.3	32.6
Weathered and then Bolted Specimens								
Kuperus 1966	Fe 37	0	16	0.549	0.060	28.3	25.8	23.8
	Fe 37	7	16	0.510	0.050	26.9	24.8	22.1
	Fe 37	14	16	0.540	0.035	30.4	28.9	23.4
	Fe 52	0	16	0.649	0.077	32.9	27.7	28.2
	Fe 52	7	16	0.522	0.068	25.8	22.9	22.7
	Fe 52	14	16	0.600	0.052	32.4	30.2	26.0
Lee 1969	A36	60	1	0.43	--	--	--	18.7
		180	1	0.47	--	--	--	20.4
		360	1	0.39	--	--	--	16.9
Moss 1979	Grade 50	0	3	0.422	0.031	23.4	22.0	18.3
	Grade 50 ^e	360	3	0.490	0.085	22.0	18.4	21.3
	Grade 50 ^f	360	3	0.545	0.016	32.6	32.0	23.7
	Weathering	0	3	0.367	0.019	21.1	20.3	15.9
	Weathering	360	2	0.473	--	--	--	20.5
Yura ^a 1981	A588	0	20	0.518	0.062 ^d	26.2	23.5	22.5
	A588	90	20	0.835	0.066 ^d	45.7	42.9	36.2
Yura ^b 1981	A588	0	5	0.86	0.038	50.2	48.6	37.3
	A588	180	5	0.74	0.043	42.1	40.3	32.1
	A588	360	5	0.59	0.080	28.9	25.4	25.6

- Notes: a. Specimens tested at Federal Highway Administration.
 b. Specimens tested at University of Texas.
 c. For A490 bolts multiply shear stress by 150 ksi/120 ksi = 1.25.
 d. Based on sample means and sample standard deviations [Natrella 1963].
 e. Specimens weathered at industrial site.
 f. Specimens weathered at marine site.

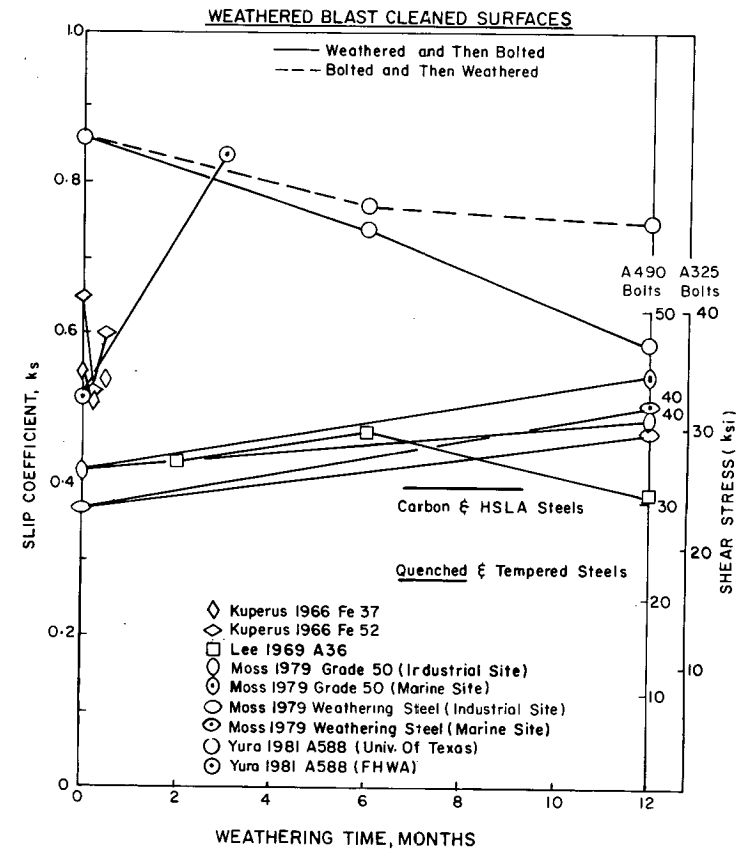


Figure 105. Slip coefficient and allowable shear stresses for friction-type bolted joints with weathered blast-cleaned surfaces.

cases, corrosion was found to have frozen the joint, thus causing other structural damage [Culp 1980].

Pin-and-hanger connections should not be used in weathering steel bridges without adequate corrosion protection. Provisions for cleaning, maintaining and painting such joints must be made.

HIGHWAY GUARDRAILS

An examination of a weathering steel test stretch of guardrail after 15 years of exposure revealed that the lapped surfaces of the bolted joints had bulged out under the internal pressure of the accumulated corrosion products [McCrum 1980]. To assess the corrosion-induced strength reduction of such joints, weathering steel guardrail and galvanized guardrail sections were removed from a Lansing, Michigan, highway site after 15.5 years of exposure, and from a Detroit, Michigan, urban site after 4.5 years. Some joints were removed intact in order to evaluate the effect of the prestressed condition of the joint from tight packing of corrosion products. Others were disassembled and rebuilt from end sections or interior sections of the guardrail beams in order to determine how differences in the two types of exposure (lapped versus freely exposed guardrail) affect joint strength. Control joints were also built from new guardrail beams for comparison with the joints in service.

The spliced guardrail beams were tested in tension. Several modes of failure, and combinations thereof, were observed. The most common failure occurred as the metal surrounding the lap joint bolts sheared and bent until one of the bolts could freely pass through the deformed holes of the lapped beams. Some of the joints failed in tension, usually along a line through the bolt holes. The following conclusions were obtained from the test results:

1. The loss in strength of the galvanized guardrail after 15.5 years of exposure at the Lansing site was negligible. The galvanized coating remained intact.
2. The weathering steel guardrails lost 20 percent and 11 percent, respectively, of their initial strength when exposed 15.5 years at the Lansing site and 4.5 years at the Detroit site.
3. The joints that were removed intact had about the same strength as those that were disassembled and rebuilt. The expansive forces induced by the corrosion products did not seemingly affect joint strength.

Guardrails can be built from weathering steel. But the contact surfaces at the lap joints should be painted to protect them against crevice corrosion.

CHAPTER ELEVEN

STRUCTURAL DETAILS

BRIDGE DECK JOINTS

Flow of runoff water through open bridge deck joints or leaking sealed joints has been one of the major causes of extensive maintenance and costly remedial work on bridges in general. The corrosion in these critical areas is particularly severe for weathering steel bridges that have no protective coating.

The desirable characteristics of a joint are water tightness, smooth rideability, low noise level, wear resistance, and resistance to damaging snowplow blades. In reality, the performance of many joint systems is disappointing. When subjected to traffic and bridge movement, they fail in one or more important aspects, notably water tightness. [NCHRP 1979].

In some bridges, troughs were placed below open expansion joints to collect the runoff water and discharge it through drain pipes away from the structure. This solution does not seem to be viable because it introduces an additional item to clean and maintain. The original problem reoccurs as soon as accumulated road debris clogs the troughs and pipes and causes the runoff water to overflow.

Conscious of the shortcomings of open expansion joints, many designers have sought ways to reduce or even eliminate the

number of such joints, by making multispan bridges continuous and sealing any deck joints that cannot be avoided. In recent years, cellular neoprene compression seals have been extensively used for sealed deck expansion joints of short-span bridges when the total movement does not exceed 2 to 2½ in. (51 to 64 mm) and the skew angle is not extreme. For expansions greater than 2 in. (51 mm), designers have to rely on proprietary joint seals. Field experience indicates, however, that a large number of these seals have performed poorly. The lack of water tightness has brought the most complaints, although this feature is supposed to be the most desired attribute of sealed joints [NCHRP 1979].

To prevent progressive corrosion of weathering steel bridges, several states have been painting the girder ends on either side over a length of one beam depth (Pennsylvania), 2 ½ beam depths (Illinois), or 5 to 10 ft (1.5 to 3.0 m) (Colorado, Louisiana, Maine, Maryland). Massachusetts is also painting the cross beams and stringer ends.

Weathering steel bridges must have water-tight joints. Otherwise the steel below the joint must be painted. To improve the rate of success in achieving water tightness, designers need to be knowledgeable about joint systems, inspectors must strictly

enforce construction quality control, and maintenance organizations should be responsible, capable, and adequately funded.

WATER AND DEBRIS ACCUMULATION

Water ponds and debris accumulate on horizontal surfaces and in corners formed by horizontal and vertical plates. The most susceptible locations are: top of lower beam flanges and truss chords, gusset plates for horizontal bracing, bolted splices of horizontal and sloped members, at bearing and intermediate stiffeners, and inside nonsealed box sections. To avoid water ponding and debris accumulation it is important to:

1. Slope horizontal surfaces longitudinally or transversely.
2. Avoid reentrant corners.
3. Seal box sections and provide drainage holes at the lowest point.
4. Design details for easy discharge of water.

Implementation of these measures is illustrated with the following examples of a truss bridge and a plate-girder bridge built in Japan. The information on these bridges was provided by Yamada [1983a].

Railroad Truss Bridge

In 1980, the Japanese National Railways built their first non-painted weathering steel railroad bridge, No. 3 Ohkawa Bridge, in Fukushima Prefecture, Northern Japan (Fig. 106). The 207-

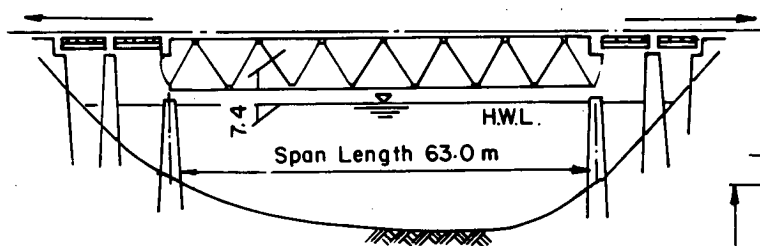


Figure 106. No. 3 Ohkawa Railroad Bridge.

ft (63-m) truss bridge is located away from industrial centers, in an area with 79 percent average relative humidity, 53-in. (1,340-mm) annual rainfall, and 52 F (11.3 C) average temperature. The designers have taken the following measures to ensure that the protective oxide coating would form:

1. Sloped the main trusses 5 percent out of plumb (Fig. 107).
2. Transversely sloped the lower flange of the floor beams and the stringer flanges; and longitudinally sloped parts of the upper flange of the floor beams (Figs. 108 and 111).
3. Selected box sections for all main truss and bracing truss members (Figs. 108 to 110).
4. Sealed the box sections at both ends, and drilled drainage holes at the lowest point of the main truss diagonals (Fig. 112).
5. Extended the top flange of the upper chord beyond the webs; extended the webs of both chords below the underside of the bottom flange (Fig. 109).
6. Mounted the rails on the stringers, without wooden railway ties (Fig. 111).
7. Increased the thickness of the top flanges of stringers and floor beams by 80 mils (2 mm) as a corrosion allowance. The Japanese National Railway specifications call for a 40-mils (1-mm) thickness increase when members are painted.
8. Fabricated the conventional bearings from cast carbon steel and coated them with tar-epoxy-resin paint.

Highway Plate Girder Bridge

In 1980, the Hanshin-Express Highway Authority fabricated and assembled the Dejima Off-Ramp Bridge in Osaka, Japan.

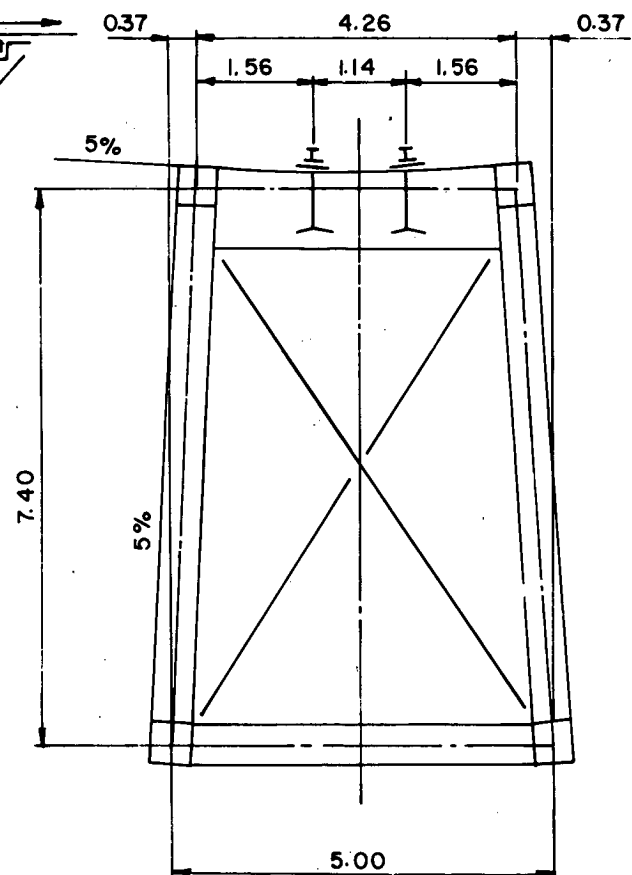


Figure 107. Cross-section of Ohkawa Railroad Truss Bridge.

Figure 108. Cross-section of top chord, stringer, and floor beam.

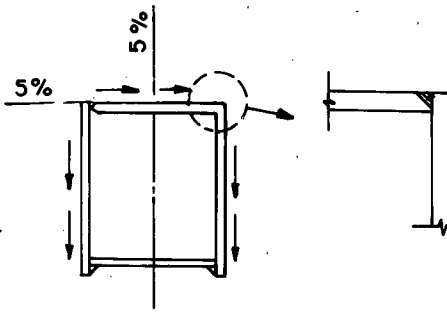
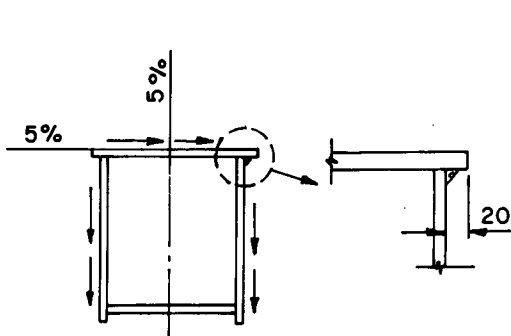
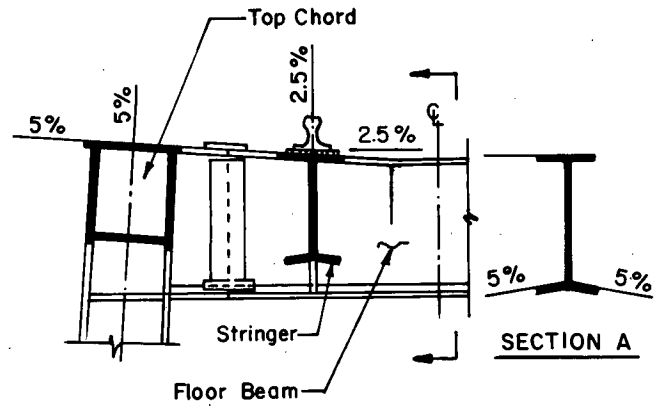


Figure 109. Cross-section of top chord (left) and bottom chord (right).

Figure 110. Cross-section of top (left) and bottom (right) chord splices.

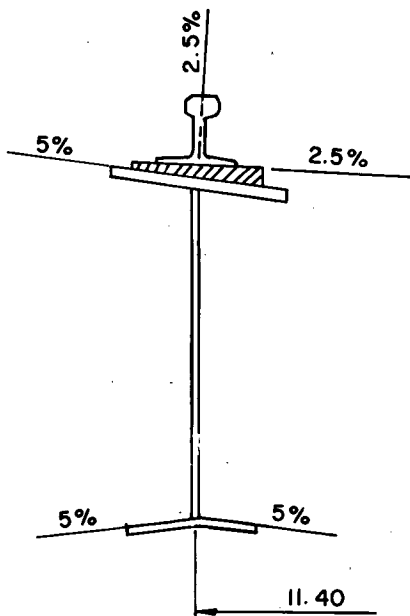
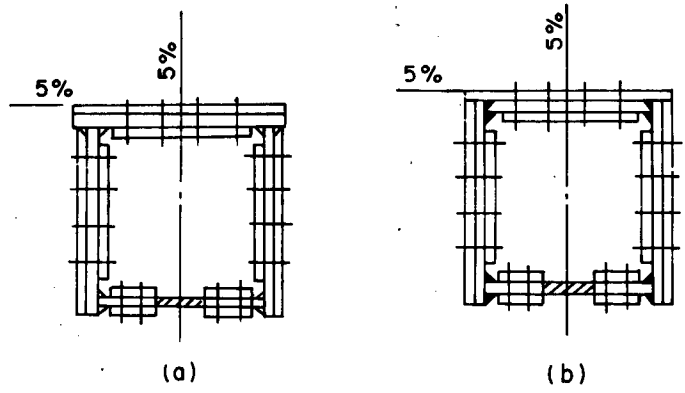


Figure 111. Cross-section of stringer.

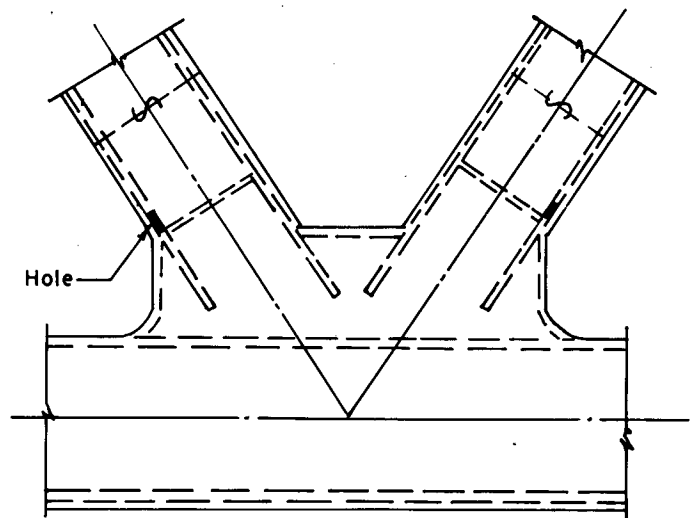


Figure 112. Detail of bottom chord joint.

This three-span continuous weathering steel bridge is located in an industrial area. The following structural details are different from those used in painted bridges:

1. Changed the cross sectional area of the lower flange by increasing thickness rather than width. Past experience had shown that uneven water flow at transitions in width caused nonuniform appearance and corrosion (Fig. 113).
2. Increased the thickness of the lower flange plates by 40 mils (1 mm) as a corrosion allowance.
3. Increased the gap between the flange ends at bolted splices to 0.8 in. (20 mm). Provided separate splice plates on the underside of the bottom flange for ease of drainage (Fig. 114).
4. Eliminated reentrant corners at the bearing stiffeners by welding sloped plates, and provided sealed access holes (Fig. 115).
5. Cut off the intermediate stiffeners 1.2 in. (30 mm) above the inside of the bottom flange, and coped the fitted stiffeners with 2-in. (50-mm) radius (Fig. 116).
6. Bolted the tee-sections of the longitudinal bracing to the underside of the gusset plates. Enlarged the opening in the gusset plate through which the vertical stiffener passes (Fig. 117).
7. Increased the height of the shoe base at the supports for better air circulation.

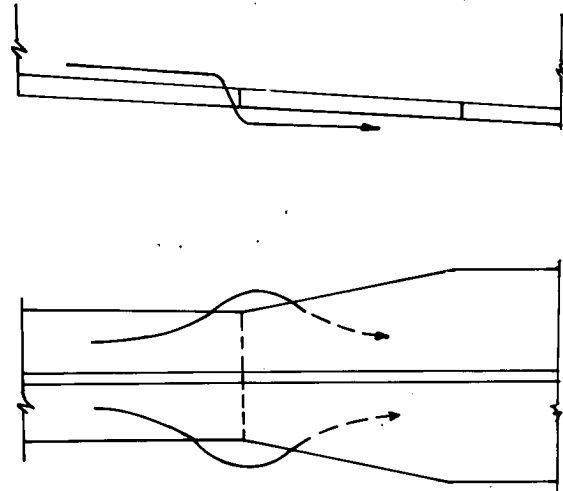


Figure 113. Water flow top-to-bottom surface at flange width transition.

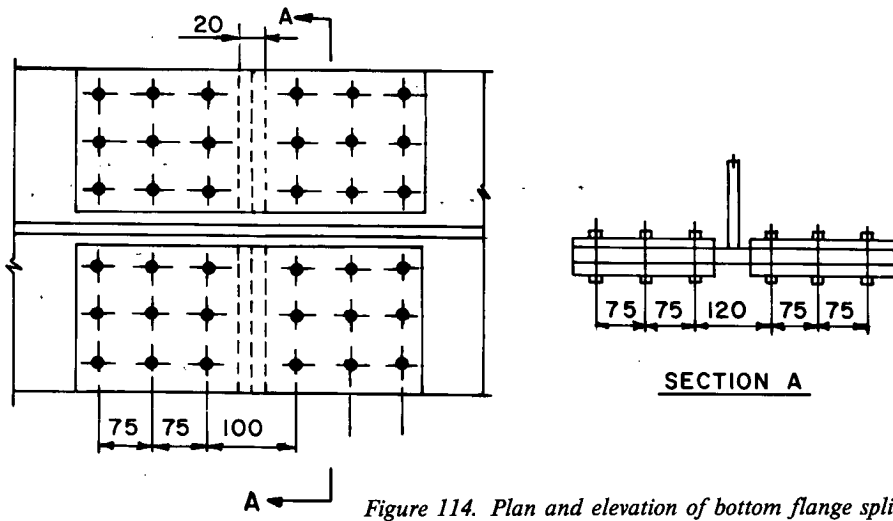


Figure 114. Plan and elevation of bottom flange splice.

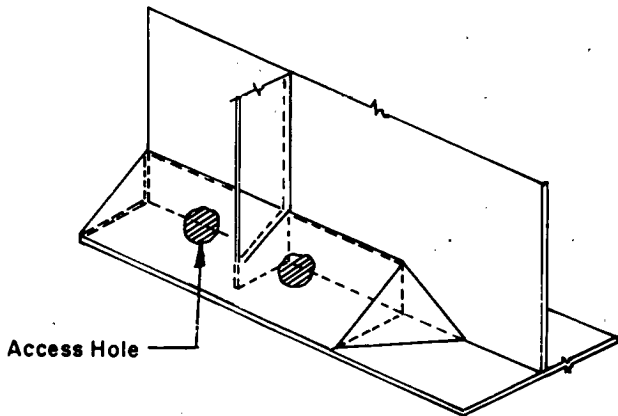


Figure 115. Closed-box detail at bearing stiffener.

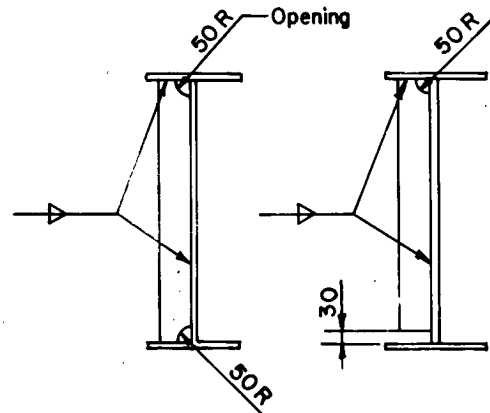


Figure 116. Intermediate stiffener detail.

STAINING OF ABUTMENTS AND PIERS

Weathering steel releases minute oxide particles of corrosion when water washes or drains over its surface. This oxide-laden runoff water stains the adjacent concrete abutments and piers, particularly during the early years of exposure. The rusty stains and streaks are unsightly. For aesthetic reasons every effort should be made to minimize them when the concrete supports are visible to the public.

The best way to minimize staining is to incorporate permanent design details that will divert the runoff water away from the concrete. The following measures can be taken to prevent staining of concrete piers and abutments in new bridges:

1. Cover the pier caps with a polyethylene sheet until the bridge deck has been poured. The sheets can be removed once the deck is in place and a permanent system installed to carry away the rust-laden water.

2. Build a parapet wall on top of piers and abutments. Drain the water through a pipe embedded in the concrete, or channel the overflow with a V-groove on vertical surfaces (Fig. 118) [Yamada 1983a].

3. Slope the abutment cap towards the retaining wall and drain the water through a pipe into the dry well behind the wall (Fig. 119) [Bethlehem 1983]. In solutions (2) and (3) drain pipes must be maintained clean to avoid clogging.

4. Install drip pans made from steel or fiberglass under the bearings and cantilever them out from the pier. When piers are very high, wind can blow the dripping rust-laden water against the pier surface.

5. Attach a drip plate to the top of the lower flange and to the girder web to divert water off the steel before it drips on the concrete pier (Fig. 120). The Michigan experience shows that drip plates do not prevent capillary advancement of salt-laden water [Arnold 1984].

6. Reduce the number of joints by making the bridge continuous.

7. Coat concrete surfaces with liquid silicon sealer or other formulations to prevent rust penetration into the concrete pores. This method is not necessarily reliable. In some cases the treatment may discolor the concrete and oxide deposits may still stain the concrete despite the treatment. Coatings break down with time. Two penetrant sealers examined in an experimental study did not adequately protect the concrete against oxide stains and painted graffiti [Cosaboom 1978].

Stained surfaces of concrete piers and abutments of existing bridges can be either cleaned with chemical stain removers, sandblasted, or painted dark brown. Allowing the stain to occur freely for several years, and then treating the concrete in the aforementioned ways, is not a satisfactory solution for new bridges because it adds one more maintenance task.

RECOMMENDATIONS FROM ONTARIO STUDY

In a study conducted by the Ontario Ministry of Transportation and Communications, the following recommendations were made on how to improve design details so as to minimize the accelerated corrosion of weathering steel bridges [Manning 1984]:

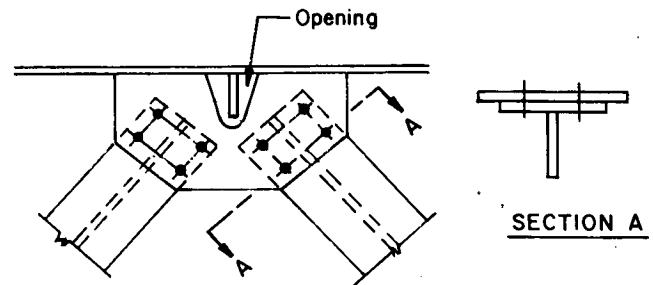


Figure 117. Detail of longitudinal bracing joint.

1. Provide an 8-in. (200-mm) gap between the beams and the ballast wall to improve ventilation and reduce the effects of moisture-induced deterioration on both steel and concrete bridges.

2. Do not use I-sections in grade separations because of the possible accumulation of salt spray on the girders, especially in areas of heavy salting. Box sections are permissible in grade separations.

3. Waterproof and pave concrete deck of box girders to prevent leakage through cracks in the deck slab.

4. Close diaphragms of box girders to keep out vermin and moisture. Ventilate box with a series of small holes in the bottom flange. Insert a short tube in each hole, flush with the top surface of the bottom flange, so that any seepage can escape without running on the underside of the structure. Fit the tube with a screen to prevent insects nesting in the box.

5. Locate inspection hatches in accessible places. Avoid locations over the roadway or over ground which would not support the inspection vehicle. For structures in grade separations, the preferred location is directly above the shoulder of the highway.

6. Extend the web of box girders below the bottom flange in a manner similar to that shown in Figure 109.

7. Given the potential for galvanic pitting corrosion along breaks in the mill scale and the very long time needed to form a stable oxide film under sheltered conditions, blast clear the entire structure.

8. Avoid ledges and horizontal surfaces where moisture and debris can accumulate.

9. Ventilate the superstructure well.

10. Protect the weathering steel in the area of the expansion joints against corrosion either by controlling the flow path of runoff or by the use of a protective coating.

11. Do not route drainage through closed sections.

Manning concluded that since remedial action is neither desirable nor always effective, it is important to avoid those sites, section geometries, and design details that may result in poor corrosion performance.

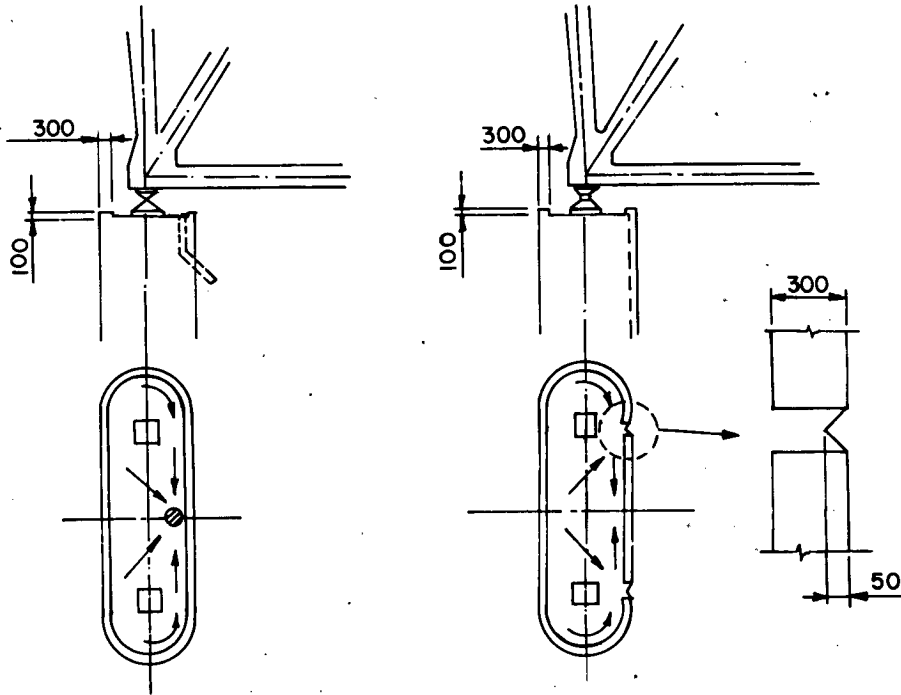


Figure 118. Sloping pier caps with a 4-in. (100-mm) high parapet wall. [Yamada 1983]

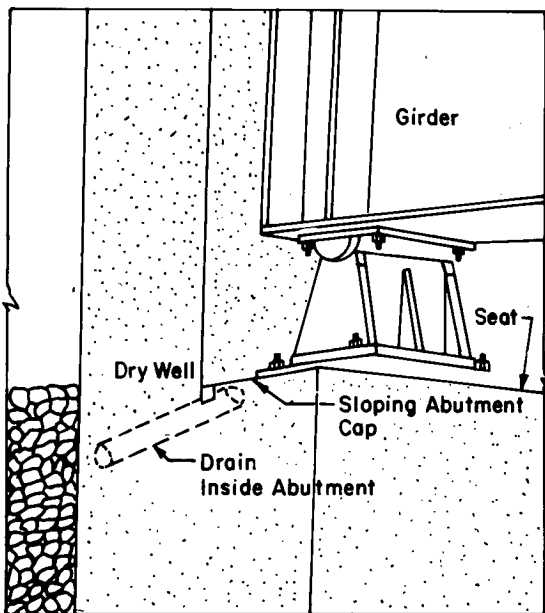


Figure 119. Sloping abutments and piers with drain. [Bethlehem 1983]

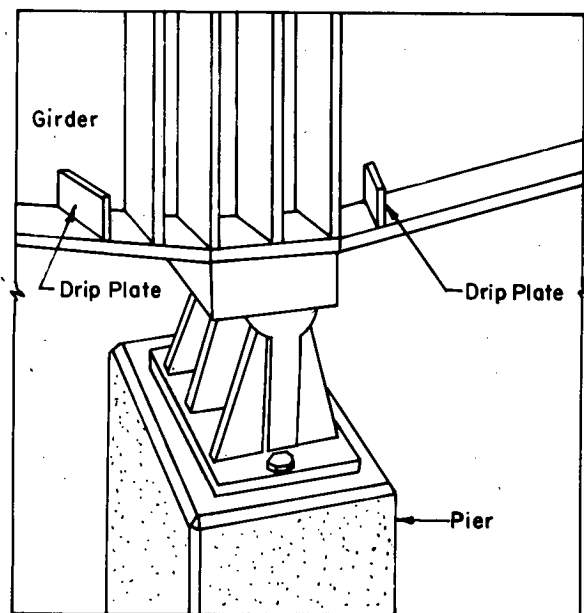


Figure 120. Drip plates attached to the bottom flange and the web. [Bethlehem 1983]

CONCLUSIONS AND RECOMMENDED RESEARCH

CONCLUSIONS

The majority of the weathering steel bridges are in good condition, although localized areas of accelerated attack can be found in many structures. A number of bridges in several states are experiencing excessive corrosion because of salt contamination and/or prolonged time of wetness. Weathering steel can provide a satisfactory service life with limited maintenance if the structural details are designed in a manner that prevents accelerated attack, vulnerable areas are painted, and contamination with chlorides is avoided.

Extensive information on the performance of weathering steel and its use in highway bridges has been assembled in this report. The practitioner can use this body of knowledge as a general guide for designing new bridges and for trouble shooting. The second phase of research under NCHRP Project 10-22 will develop specific guidelines for design, construction, maintenance, and rehabilitation of weathering steel bridges. The guide should include, as a minimum: (1) a quantitative definition of corrosion resistance of weathering steels that is independent of the corrosion resistance of reference steels; (2) tolerable levels of corrosion penetration; (3) allowance for loss of section due to corrosion; (4) reduction in fatigue life due to pitting and corrosion fatigue; and (5) monitoring corrosion and pitting penetration of bridges in service.

RECOMMENDED RESEARCH

There are five major areas in which additional studies are needed to properly assess the performance of nonpainted weathering steel and to establish the conditions under which the steel can be used to its best advantage. These areas are corrosion, fatigue, painting, joints, and specifications. The most important topics that need to be studied are as follows:

1. Develop predictive models for estimating the corrosiveness of a site and the expected corrosion penetration of ideally exposed standard size coupon of weathering steel from known climatological and air pollution data. Determine how time-of-wetness can be correlated with climatological data.

2. Relate the corrosion resistance measured under standard conditions to the corrosion resistance of a large-size bridge girder exposed under partially or fully sheltered conditions.

3. Determine the tolerable limits on time-of-wetness, air pollution, and deicing salt contamination for satisfactory performance of weathering steel.

4. Perform corrosion fatigue tests of typical welded joints under conditions representative of bridge service environments. This should include large specimens, realistic cycling rates, variable amplitude loading, and intermittent spray. S-N data, in

addition to crack growth rate data, should be obtained for typical details. In this way, the effect of corrosion on the total life (initiation plus propagation) can be reliably determined in the same test.

5. Evaluate joint seals for water tightness under movements that typically occur in straight, skewed, and curved bridges.

6. Determine the slip resistance and the effect of corrosion-induced expansive forces on the behavior of high-strength bolted joints after long-term exposure.

7. Develop effective methods of cleaning and painting corroded weathering steel bridges, particularly those contaminated with salt. Assess the performance of the remedial painting system with natural exposure tests. Any such work should build on the results of the on-going studies being performed by the Michigan DOT and the Structural Steel Painting Council.

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