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HIGHWAY RESEARCH PROGRAM REPORT **314**

GUIDELINES FOR THE USE OF WEATHERING STEEL IN BRIDGES

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

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GUIDELINES FOR THE USE OF WEATHERING STEEL IN BRIDGES

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TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
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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.

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FOREWORD

By Staff
Transportation Research Board

This report contains guidelines for the design, construction, maintenance, and rehabilitation of weathering steel used in bridges. A previous phase of research documented the body of knowledge on weathering steel in *NCHRP Report 272,* while follow-on research, as represented in this report, improved the state of practice through specific guidance on the use of weathering steel. The guidelines will be of particular interest to structural engineers, maintenance engineers, materials engineers, specification writers, and structural steel producers and suppliers. Based on statistical analyses of data from a variety of sources on the fatigue life of weathering steel, as well as laboratory tests on weathered steel transverse stiffener specimens, changes to the AASHTO bridge specifications have also been recommended. A supplemental report, not included herein but available on request, documents the statistical analyses of the fatigue test data and the results of the laboratory tests.

Weathering steel has been used in the construction of over 2,000 bridges in the United States since the mid-1960s. Under proper conditions, this material is expected to form its own protective surface layer to significantly retard the rate of corrosion and, thereby, eliminate the need for painting. Consequently, it offers the potential for considerable savings in life-cycle costs. However, under certain conditions, corrosion has continued at a rate more rapid than anticipated, causing concern over the long-term performance of this material. As a means to better understand the problem, research was performed under NCHRP Project 10-22, "The Performance of Weathering Steel in Bridges," by Sheladia Associates, Inc., Riverdale, Maryland.

An initial phase of research was conducted to assemble and evaluate information on the use and performance of weathering steel and to document the state of practice. In July 1984, the results of the first phase, covering a wide range of issues, were published in *NCHRP Report 272,* "Performance of Weathering Steel in Bridges."

Soon after the completion of the first phase, a second phase was initiated to develop practical guidelines for the design, construction, maintenance, and rehabilitation of weathering steel bridges. The guidelines are included in this report. In addition, laboratory tests on weathered steel transverse stiffener specimens and statistical analyses of fatigue test data from many sources were performed as part of the second phase. The details of the laboratory tests and the statistical evaluations are not included in this published report but are documented in an agency report titled, "Fatigue Strength of 8-Year Weathered Stiffeners in Air and Salt Water." That report is available for loan or purchase ($6.00 per copy) upon written request to the NCHRP.

In this report, the guidelines are divided into sections that specifically deal with the performance, design, construction, inspection, maintenance, and rehabilitation of weathering steel in bridges. The application of weathering steel in the 50 states and the District of Columbia is described in the performance section of the report. Many case studies on performance are also presented in an appendix to develop a better understanding of how to avoid past problems in the design of new bridges. The design section provides guidelines on site selection, economic analysis of the benefits of choosing weathering steel versus painted steel, good detailing for improved corrosion resistance, use of welded and bolted connections, and criteria for fatigue design. The construction section of the report addresses the proper handling and storage of weathering steel members prior to erection, the final cleaning of the steel after erection, and the protection of the concrete substructure against rust staining. The inspection and maintenance section includes guidelines for evaluating the condition of the oxide coating, the underlying steel base, and the structure as a whole. It also gives examples of the periodic maintenance needed to ensure satisfactory performance of the weathering steel. Finally, the last section of the report recommends a method for the remedial painting of weathering steel bridges that have not performed satisfactorily.

The appropriate use of weathering steel has been debated vigorously throughout the 1980s. Although many of the factors that limit the use of weathering steel to certain applications and environments are now agreed upon, there are differences of opinion on how prevalent these factors are in actual practice. Accordingly, these guidelines are intended to help make informed decisions on the appropriate use of weathering steel for new bridges and in the maintenance and rehabilitation of existing bridges.

Chapter Nine, "Fatigue," includes recommendations for revisions in the *AASHTO Standard Specifications for Highway Bridges* relating to fatigue strength of bare, exposed weathering steel in bridges. Recommendations for percentage reductions in allowable stress ranges are made. The need for such reductions has also been debated in various forums, but without resolution. The recommendations are based on evaluations of existing data and represent the researchers' best judgment on the need for modifying the standard specifications. At the time of publication, the researchers' recommendations contained in this report are being reviewed by a task force of the AASHTO Highway Subcommittee on Bridges and Structures; a decision, and its effect on the standard specifications, is still pending.

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ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 10-22 by Sheladia Associates, Inc. Professor Pedro Albrecht, Department of Civil Engineering, University of Maryland, was the principal investigator. The other authors of this report were: Mr. Seymour K. Coburn, retired from United States Steel Corp., and now President of Corrosion Consultants, Inc., Pittsburgh, Pennsylvania; Dr. Fateh Wattar, formerly with Sheladia Associates, Inc., and now President of Matrix Development, Inc., Washington, D.C., Mr. Gary Tinklenberg, formerly with Michigan Department of Transportation, and now President of Corrosion Control Consultants and Laboratory, Lake Odessa, Michigan; and Dr. W. Patrick Gallagher, formerly with United States Steel Corp., and now Senior Materials Engineer, Westinghouse Electric Corporation, Sunnyvale, California. Mr. Gopal Pillai, Vice President of Sheladia Associates, Inc., was the Coordinator and Project Manager on the project.

The principal authors of the various chapters of this report were: Chapter One—Albrecht, Wattar, and Coburn; Chapter Two—Gallagher; Chapter Three—Coburn; Chapter Four—Albrecht and Wattar; Chapter Five—Wattar, Albrecht, and Tinklenberg; Chapters Six, Seven, and Eight—Albrecht and Wattar; Chapter Nine—Albrecht; Chapter Ten—Coburn; Chapter Eleven—Coburn and Tinklenberg; and Chapter Twelve—Tinklenberg. All authors reviewed and contributed comments to the twelve chapters.

GUIDELINES FOR THE USE OF WEATHERING STEEL IN BRIDGES

SUMMARY

This report is part of NCHRP Project 10-22, "The Performance of Weathering Steel in Bridges," which is directed toward assembling and evaluating existing experimental and field performance data with respect to the use of weathering steel in bridges. The research has been accomplished in two phases. The objectives of Phase I were: (1) to assemble a systematic body of information on the performance of weathering steel, and (2) to document and evaluate the current state of practice. The final report on that phase was published as *NCHRP Report 272,* "Performance of Weathering Steel in Bridges."

The specific objectives of the second phase were: (1) to fatigue test 8-year weathered A588 transverse stiffener specimens under constant loading in air and aqueous environments, and (2) to develop practical guidelines for design, construction, maintenance, and rehabilitation of weathering steel bridges. The results of the fatigue tests and analyses that were performed as part of this project in fulfillment of the first objective in Phase II are documented in the report entitled "Fatigue Strength of Weathering Steel in Bridges" (refer to *Foreword* for availability).

The findings summarized in the following, and described in full in the main report, relate primarily to the second objective of the second phase—namely the guidelines. The subject matter is discussed under the four broad areas pertaining to: behavior, design, construction and maintenance, and rehabilitation.

Behavior

Characteristics. When the structural details are properly designed and the bridge is located in a suitable environment, weathering steel forms a protective oxide coating that inhibits the steel's tendency to continuously rust in the atmosphere.

The advantage of using weathering steel is the potential it offers for savings in life cycle cost by eliminating the need for the initial painting and periodic repainting of the entire superstructure, as is required for ordinary steels that do not have enhanced atmospheric corrosion resistance. By eliminating the need for repainting, the closing of heavy traffic lanes below grade separation structures can be prevented as can safety hazards to painters and motorists. Paint jobs over difficult terrains, bodies of water, and electrified railways can also be avoided.

However, weathering steel does not provide a service life for the superstructure that is free of maintenance. Proper detailing and choice of a suitable environment can reduce the need for some, but not all, maintenance.

Weathering steel comes in yield strengths of 345 to 690 MPa (50 to 100 ksi). A242 Type 1 steel is used only for architectural applications because the high phosphorus content impairs the weldability and toughness required of bridge steels. The low-phosphorus version, A242 Type 2, which corresponds to the A588 and A709 Grade 50W steels, is suited for bridge applications. A588 steel is now available in nine proprietary grades. Each grade is a variation of the same basic chemical composition which contains copper, chromium, nickel, and silicon for enhanced atmospheric cor-

rosion resistance. A709 is a general specification for bridge steels. The two grades with enhanced atmospheric corrosion resistance, 50W and 100W, correspond to A588 steel and a modified A514 alloy steel. A852 is a new quenched and tempered weathering steel with 485 MPa (70 ksi) minimum yield strength and a generic chemical composition.

The weathering steel specifications A242, A588, A709, and A852, first issued in 1941, 1968, 1974, and 1986, respectively, have undergone a number of changes over the years. The most important change affecting the corrosion resistance was made in the mid-1970's when it became apparent that the results of long-term exposure tests failed to confirm the atmospheric corrosion rating of four relative to carbon steel. The compositions of A588 Grade A to E steels were then enriched with nickel, chromium, and silicon. At the same time the atmospheric corrosion rating of A242 Type 2 was lowered from "at least" to "approximately" four times that of carbon structural steel without copper.

Applications. According to a 1987 telephone survey some 2,300 weathering steel bridges were built on state highway systems alone, not counting county, city, toll road, or mass transit bridges. Among the 50 states and the District of Columbia there are 4 nonusers of weathering steel, 14 former users, and 33 remaining users. The four states that do not have weathering steel bridges are Arizona, Hawaii, Nevada, and South Dakota. Hawaii does not use it because concrete is more economical than steel. In the other three states the climate is so dry that paint systems on their ordinary steel bridges last indefinitely. Weathering steel in these arid states would not weather uniformly for a long time.

The 14 former users no longer specify weathering steel for new bridges because of the following reasons: (a) excessive corrosion of bridges in their state—Indiana, Iowa, Michigan, Washington, and West Virginia; (b) concerned by experience in other states—Alabama, Florida, Georgia, Oklahoma, and South Dakota; (c) aesthetic reasons—New Mexico and South Carolina; and (d) economic reasons—California and North Dakota. Some states have more than one reason for no longer using weathering steel. For example, New Mexico and North Dakota also have a semi-arid climate in which paint systems on ordinary steel last indefinitely.

Among the 33 remaining users there is a great variety in degree of usage for new bridges, from almost exclusively (Vermont) to practically none (Pennsylvania and Tennessee). Recognizing the limitations of the material, most remaining users now follow their own design criteria for enhancing the corrosion performance such as: using weathering steel mainly in rural areas, remote areas, or where the bridge is not visible to the public (8 states); using it mainly over streams (5 states) and over railroad tracks (3 states); not using it for grade separation structures (4 states), nor in cities and where the average daily traffic exceeds 10,000 (1 state); not using it along the coast (6 states), on heavily salted highways (4 states), or high humidity areas (3 states); painting steel 1.5 to 3 m (5 to 10 ft) on each side of joints (10 states); blast cleaning all steel before erection (2 states); keeping drainage water from running over substructure and protecting concrete against rust staining (5 states); galvanizing scuppers, bearings, and expansion devices (1 state); galvanizing finger plates (1 state); not using hinges or sliding plates (1 state); and making decks jointless, and building bridges integrally with abutments where conditions permit (1 state).

The successful application of weathering steel for bridges requires on the part of the designer a clear understanding of the structural details and the microenvironment that initiate and support corrosion at a specific bridge site. An evaluation of the macroenvironment and the simple substitution of weathering steel for ordinary steel in a set of standard design drawings is simply not sufficient. This point must be emphasized.

Experience. Experience over the last 22 years has revealed many good applications as well as problems with some bridges located in the snowbelt states of the Midwest and Northeast, in Gulf Coast states, and in high rainfall and foggy regions along the West Coast. The problems were generally caused by contamination of the steel structure with deicing salts, exposure to onshore breezes laden with sea salt, and extended periods of wetness in areas of persistent high relative humidity. These types of problems have raised questions concerning the long term performance of weathering steel bridges.

As a result of poor performance, several states have remedially painted some bridges (Iowa, Louisiana, Michigan, and Ohio) or are in various stages of planning such work (Alaska, Indiana, Texas, and Washington). In other states where weathering steel bridges are excessively corroding only in limited areas, the steel near joints was remedially painted (Kentucky, Maryland, and Missouri), or such work is being planned (Massachusetts and New Jersey Turnpike).

Experience with bridges in service since 1965 has shown that weathering steel is not a maintenance free material and has distinct limitations with regard to the environmental conditions under which it can be used. In essence, weathering steel does not develop a protective oxide coating if the steel remains wet for a long time or is contaminated with salt from any source.

Direct precipitation of rain or snow is not needed for a steel surface to become wet. Moisture can be deposited by (a) nightly condensation when the surface temperature of the steel falls below the dew point; (b) radiant cooling of the skyward deck surface in a clear night, causing moisture to condense on the upper portions of the steel girder and run down the web along drip lines; (c) capillary moisture absorption by the porous oxide coating and crevices in structural details; (d) adsorption by corrosion products particularly in the presence of salt; (e) leaking bridge deck joints; and (f) traffic spray kicked up in the wake of high-speed traffic and settling on the overhead members of a bridge. High relative humidity, nightly fog, moisture evaporating from bodies of water, and poor air circulation enhance the deposition of moisture.

Salt can be deposited on bridges built in marine environments or where salt is spread on highways to melt ice and snow. Salt particles found in oceanic air are the residual salt from evaporated droplets formed by bubbles bursting at the sea surface. These particles are mixed upwards 2 km (6,500 ft) into the marine atmosphere over the ocean. Onshore winds carry the salt-laden air inland where particles are deposited by wind impingement, gravitational fall-out, and diffusion, or are washed out by precipitation. As a result, the salt concentration in the air and the rate of deposition is reduced with distance from the sea.

Waves breaking along the beach release many bubbles. The larger bubbles are rapidly deposited by gravitational fall-out, making the marine environment more severe near the shore line than away from it.

Deicing salt is deposited on weathering steel bridges by salt-laden runoff water leaking through the deck joints and by salt-laden traffic spray being kicked up behind trucks and settling on the overhead bridge members.

The most serious and common cause of corrosion problems in weathering steel bridges is caused by runoff water leaking through the deck and wetting the diaphragm girders, and bearings in the vicinity of the joints. Such runoff water can migrate for long distances along the bottom flange and wick about 150 mm (6 in.) up the web. The resulting severe corrosion of the steel and the locking of expansion bearings create major bridge maintenance problems.

Salt deposition by traffic spray poses a severe corrosion hazard under combinations of low clearance, high traffic speed, and heavy use of deicing salt. It is particularly severe in depressed and half-tunneled highways where the lateral confinement helps to direct the spray onto the overpassing bridge and prevents winds from clearing the spray. Traffic spray deposits deicing salts on large areas of all steel members, with large amounts being deposited over the traffic lanes and lesser amounts over the shoulders.

A clear understanding of the conditions that interfere with the proper development of the protective oxide coating is essential to the successful application of weathering steel. Several case studies illustrate various conditions where limitations on the use of weathering steel are exceeded. These experiences form the basis for recommending guidelines for the design, fabrication, maintenance, and rehabilitation of weathering steel bridges.

Corrosion Performance. The ability of weathering steel to resist atmosphere corrosion is vital to its being used successfully. However, weathering steel is resistant only to specific types of atmospheric corrosion in a limited range of environments. Therefore, the proper design of weathering steel structures must be based on an understanding of the material's corrosion-resisting characteristics.

An important requirement for the successful performance of weathering steel is an adequate content of alloying elements that lead to protective oxide formation and a consequent decrease in the corrosion rate. The most important elements for increased corrosion resistance are copper, nickel, chromium, phosphorus, and silicon.

It is possible to develop weathering steels with alloy contents that would give the steel very high atmospheric corrosion resistance. Such steels would resist atmospheric corrosion in much more aggressive atmospheres than are suitable for the present grades of weathering steel for bridges. However, the available weathering steels are limited in their atmospheric corrosion resistance because an increase in alloy content that is beneficial to corrosion resistance can adversely affect other required properties such as toughness and weldability. The practical upper limit on alloy content restricts the types of environments in which weathering steel can be used for bridges.

To ensure satisfactory performance of weathering steel, the environment must provide adequate periods of drying, must not provide excessive periods of wetting, and must not contain excessive amounts of corrosive contaminants. The environment of a bridge is a combination of factors related to the general climate of the region plus factors related to the specific bridge site. These local climatic environments, often called microenvironments, must be considered in the selection of a suitable site and the design of the bridge.

An adequate design is necessary to ensure that a protective oxide film develops on members exposed to the specific microenvironment. The design must allow for alternate wetting and drying cycles, must not create prolonged conditions of wetness, and should avoid connections and details subject to crevice and galvanic corrosion.

Atmospheric corrosion data are useful in determining the effects of types of atmospheres and exposure conditions on weathering steel corrosion. However, such data do not usually match the corrosion rates of weathering steel in bridges exposed to the same type of atmosphere. This is because the test data were obtained from small coupons exposed fully to the sun and the weather (called "boldly exposed" in corrosion literature). In contrast, structural bridge members have greater mass and hence different thermal behavior than test coupons. In addition certain bridge members may not be boldly exposed, and may be subjected to additional environmental influences such as spray from roadway water. Also, much data on atmospheric corrosion labeled "weathering steel" relate to A242 Type 1 steel. This type of weathering steel has a higher alloy content and exhibits lower atmospheric corrosion rates than the A588 steel which is commonly used in bridges.

In addition to the effects of the atmosphere types in the macroenvironment, the corrosion performance of weathering steel bridges is also affected by several factors of the microenvironment such as shelter, orientation, contamination with local airborne pollutants, accumulation of debris, deposition of moisture, structural details, and geometry. The combined effects of the micro- and macroenvironments tend to lower the corrosion resistance of weathering steel in bridges as compared to that of boldly exposed test coupons.

The corrosion penetration per surface found for weathering steel in a variety of bridges in the contiguous United States that are performing satisfactorily can be contained in an envelope with upper and lower bound mean corrosion rates of 7.5 μm/yr/surface (0.3 mil/yr) and 3 μm/yr/surface (0.12 mil/yr). Weathering steel corroding at a rate higher than 7.5 μm/yr/surface (0.3 mil/yr) cannot be expected to develop a protective oxide coating.

Site Analysis. To determine if weathering steel is suitable for use in a bridge at a specific site, the engineer must evaluate the corrosivity of the macroenvironment, evaluate the corrosivity of the microenvironment, and verify suitability by measurement of corrosion resistance and levels of contaminants. If sufficiently short time-of-wetness and low contamination of the steel are found to create favorable conditions for the formation of a protective oxide coating, weathering steel may be considered for use at the bridge site.

The factors that play a critical role in the corrosivity of a macroenvironment are climate, atmospheric pollutants, and airborne chlorides originating from industrial activity, deicing salt use, and proximity to salt water.

Climatological factors of the macroenvironment can interact with the local factors of the microenvironment in the vicinity of the planned bridge site to influence the corrosion behavior of weathering steel. Among the important factors to be considered in an evaluation of the corrosivity of the microenvironment are the performance of existing steel structures, site topography, local industrial plants, and deicing salt use. A study of the microenvironment must include a visit to the site.

Because several years are needed to plan and design highways and bridges, there is sufficient time to measure, if needed, the corrosivity of the environment and determine whether the weathering steel will perform satisfactorily. The most helpful tests are those that measure the corrosion penetration of the steel, sample the atmosphere for the presence of salinity, monitor time-of-wetness of the steel, and monitor atmospheric sulfur dioxide content. These test methods and practices are now standardized by the American Society for Testing and Materials.

Design

Economic Analysis. In choosing a type of steel engineers should also consider the life cycle cost of the structure. Calculations of life cycle cost must include the initial cost, cost of maintenance, repaint interval, and time value of money. Examples of such calculations are provided for three bridges, each having five alternate systems of steel type and corrosion protection.

The calculation of life cycle cost shows that for the three bridges used in this particular example the bare, maintenance-free A588 steel (initial cost only) should be the most economical. But experience with bridges in service and current manufacturers' literature show that weathering steel is not a maintenance-free material. Of the remaining four alternates, painted A572 steel was the least expensive, followed by remedially painted A588 steel, painted A36 steel, and bare periodically hosed A588 steel. Engineers should perform a similar cost analysis to determine the most economical alternate for a specific bridge.

Design Details for Enhanced Corrosion Performance. The most important considerations in designing a weathering steel bridge are preventing water ponding, diverting the flow of runoff water away from the steel superstructure, preventing the accumulation of debris that traps moisture, and avoiding environments in which the bridge would be contaminated with salt. Under conditions of prolonged wetness or contamination with salt, weathering steel does not form a protective oxide coating. In fact, it may corrode at about the same rate as carbon steel and, thus, offers no advantage in corrosion resistance.

Leaking joints are the most serious and common cause of corrosion problems in weathering steel bridges. Runoff water leaking through the deck joints persistently wets the bearings, flanges, webs, stiffeners, and diaphragms in the vicinity of the joint where it can migrate for long distances along the bottom flange and wick about 150 mm (6 in.) up the web. The resulting excessive corrosion of the superstructure and the freezing of expansion bearings created major bridge maintenance problems. To avoid the problems created by leaking joints, bridges should have a continuous superstructure, fixed or integral bearings at piers and abutments, and no bridge deck expansion joints unless absolutely necessary. When expansion joints cannot be avoided, they should be provided only at the abutments.

Open expansion joints should not be used in weathering steel bridges. When soil conditions permit, joints can be eliminated altogether by making the deck continuous over the piers and integrating the girder ends with the abutment. Hanger plate-and-pin connections of girders at open deck joints must not be used in bare, exposed weathering steel bridges. They are susceptible to crevice corrosion and cannot be maintained.

Whenever possible, box and tubular members should be constructed tight. If tight construction is not feasible, a sufficient number of hatches, vents, or openings should be provided to create a draft. The inside surfaces should be painted, and the coating around the openings should be extra heavy.

Box girders cannot be sealed against moisture entry. Therefore, the box must be adequately drained and ventilated to reduce the potential for moisture entrapment and accelerated corrosion. Furthermore, if the box girder is inaccessible for inspection and maintenance, the interior surfaces must be painted. Accessible box girders may be left bare but the inside must be periodically inspected for evidence of corrosion. Because of its smooth contour, the exterior of a box girder is less prone to corrosion than an I-girder on which debris, moisture, and salt spray accumulate more easily.

In I-girder bridge members, water ponds and debris accumulate on horizontal surfaces and in corners formed by horizontal and vertical plates (reentrant corners), fostering excessive corrosion. The most susceptible locations are bottom flanges, gusset plates for horizontal bracing, longitudinal stiffeners, bolted splices of horizontal and sloped members, and intersections of bearing and intermediate stiffeners with flanges and gusset plates. To avoid water ponding and debris accumulation, it is important to: minimize the number of horizontal surfaces on which water can pond and debris can accumulate, minimize the number of reentrant corners that may entrap windblown dust and debris that prevent drainage, design details for self-cleaning and easy discharge of water, and avoid crevices. Similar design criteria apply for truss bridge members.

The lapped surfaces of guardrails are susceptible to crevice corrosion. The crevices can accumulate salt, soak up moisture by capillary wicking, and corrode at a much faster rate than the freely exposed faces of the guardrail. Guardrails may be built from weathering steel, but the contact surfaces at the lap joints should be painted to protect them against crevice corrosion.

Water on the approach roadway must be intercepted to prevent its flow onto the bridge deck. Downspouts for deck drains must be located such that runoff water is discharged away from any part of the bridge, and must extend below the adjacent members. Bridge decks should be drained with downpipes only when no acceptable alternative is available. Downpipes must be of rigid corrosion-resistant material and have cleanouts in sufficient numbers and at convenient locations to permit access to all parts of the downpipe system. Drain pipes must not be routed through box members.

Runoff water carrying suspended particles of iron oxide released by weathering steel will stain concrete surfaces if it is allowed to drain over abutments and piers, particularly during the early years of exposure. For aesthetic reasons, every effort should be made to minimize unsightly rust staining of concrete supports visible to the public by preventing runoff water from draining over the weathering steel and onto the concrete. Continuous jointless decks and integral abutments are most effective in preventing such staining. When joints cannot be avoided, they should be sealed and maintained properly.

Combinations of high speed traffic, low clearance, and heavy use of deicing salt on the lower roadway pose a severe corrosion hazard, because the spray plume kicked up by passing trucks is about twice as high as the truck and settles on the overhead grade separation structure. Weathering steel bridges for grade separation structures must have adequate clearance to keep most spray from reaching the bridge. It is equally important to provide adequate clearance over bodies of water so that vapor emanating from the water surface does not condense on the weathering steel. The required clearance depends on the degree of air circulation and the topography at the site.

Corrosion Resistance. The ASTM specifications A242, A588, and A709 state that the atmospheric corrosion resistance of weathering steel is approximately equal to two times that of carbon structural steel with copper, which is said to be equivalent to four times that of carbon structural steel without copper (max. 0.02 percent Cu).

The ASTM requirement is based on analysis of pre-1968 data for which the corrosion resistance was calculated as the ratio of the corrosion rate of the reference steel (carbon or copper steel) to that of weathering steel, over an increment of exposure time. This implicitly accepted rating criterion was shown to be unreliable and misleading because it did not consistently discriminate between good and poor performance of weathering steel in a variety of environments ranging from rural to moderate marine. The rating numbers do not reliably reflect the actual corrosion resistance of a weathering steel in a given environment. Thus, statements of relative corrosion rates are not helpful for purposes of stress analysis of weathering steel members. Instead, members must be designed for average, uniform corrosion penetration (loss of metal thickness) per surface expected at the end of the service life of the bridge.

Weathering steel performing satisfactorily can be expected to corrode at a rate not exceeding 7.5 μm/year (0.30 mil/year), depending on the type of exposure. For members 38 mm (1.5 in.) or greater in thickness, production tolerances will provide sufficient steel to compensate for these corrosion rates. For members less than 38 mm (1.5 in.) thick, the thickness should be increased by 0.8 mm (32 mil) per exposed surface above that arrived by stress calculation. Where corrosion rates greater than 7.5 μm/year (0.3 mil/year) are anticipated, weathering steel should not be used in the bare condition.

Welding. The welding procedure for weathering steel is similar to that for ordinary steel with comparable strength. The main difference is that for bare, exposed applications of weathering steel the weld metal must have similar corrosion behavior and coloring characteristics as does the base metal. As with any other structural steel, good shop or field practice must be followed to obtain sound welds of desired strength and ductility.

Combinations of any approved weathering steel base may be welded together. Any weathering steel base may also be welded to a structural steel base that does not have atmospheric corrosion resistance such as the A36, A572, and A514 steels.

A588 steel and A709 Grade 50W steel meeting the chemical and mechanical properties of A588 are weldable by the prequalified procedures of AWS D1.1. The weldability of A242 steel must be investigated by the engineer and that of A709 Grade 100W must be established by the producer.

Shielded metal arc (SMAW), submerged arc (SAW), gas metal arc (GMAW), and flux cored arc (FCAW) welding procedures, which conform to the provisions of AWS D1.1, may be used for welding weathering steel.

Electroslag and electrogas welding procedures may be used only for weldments of A242 and A588 steel members subjected to compressive stresses. These procedures must not be used for welding A709 Grade 100W steel or for welding bridge members of any steel that are subjected to tensile stresses or reversal of stresses.

Bare, exposed A242 and A588 steel structures must be welded with electrodes or electrode-flux combinations matching the strength, corrosion resistance, and color of the base metal. In multiple-pass welds only the two outer layers on all exposed surfaces and edges must match the strength and corrosion resistance of weathering steel, provided the underlying layers are deposited with a filler metal matching the strength but not the corrosion resistance of weathering steel.

In single-pass welds, absorption of alloying elements from the steel base may give the weld metal a corrosion resistance and coloring similar to that of the steel base. Accordingly, filler metal matching the strength but not the corrosion resistance of weathering steel may be used for single-pass SMAW welds up to 6.4 mm ($\frac{1}{4}$ in.) and SAW, GMAW, and FCAW welds up to 8.0 mm ($\frac{5}{16}$ in.) in size.

Bare, exposed A709 Grade 100W steels must be welded with filler metal containing one or more of the elements Ni, Cr, Cu, and Si in quantities sufficient to match the corrosion resistance and color of the base metal.

The allowable stresses for the static design of welds for weathering steel bridges are the same as those for ordinary steel bridges. Reductions in allowable stress range are recommended for the fatigue design of welded details.

Joints must be continuously welded on all sides to prevent moisture intrusion and corrosion in the crevice formed by the contact surfaces. Fillet welds in built-up members must also be continuous.

Bolting. Mechanical fasteners made of steel whose chemical compositions provide atmospheric corrosion resistance and weathering characteristics comparable to those of the A242, A588, and A709 weathering steels are commonly available as A325 and A490 Type 3 weathering steel bolts. The mechanical requirements for A325 and A490 Type 3 bolts are identical to those of their Type 1 and Type 2 counterparts which do not have atmospheric corrosion resistance. Other types of fasteners such as regular bolts, unfinished bolts, and rivets may not be readily available in weathering steel grades.

Bolts made of dissimilar metals, such as stainless steel, may be used if the bolt metal is more noble than weathering steel, and the bolted joint does not act as a crevice which may hold water and debris. Zinc and cadmium coated bolts, including galvanized steel bolts, should not be used in weathering steel bridges because in time the coating is sacrificed through cathodic corrosion, leaving an exposed carbon-steel bolt less resistant to atmospheric corrosion than the weathering steel.

The tightening of high-strength bolts must be controlled (a) by the turn-of-the-nut method, (b) with a calibrated wrench, or (c) with a load indicating device. The load indicating devices available for use in bolted joints are the load indicator washer and the tension-control bolt.

Load indicator washers with epoxy coated and either mechanically galvanized or mechanically cadmium coated surfaces are supplied for use with Type 3 high-strength bolts. The gap between the washer and the bolt head, which is maintained by the flattened protrusions on the washer, may act as a crevice that serves as a path for water ingression to the shank and threads of the bolt. This condition may lead to accelerated crevice corrosion of the weathering steel bolt and plates particularly in the presence of salt water. Lacking long-term exposure data, load indicator washers are not recommended for use in joints of bare weathering steel bridges unless the joint area is painted.

Tension control bolts conforming to the requirements of Specification A325 are available in Type 1 alone. They do not have the atmospheric corrosion resistance and weathering characteristics of Type 3 bolts and must, therefore, be protected by paint.

Bolted joints of weathering steel members show good corrosion performance if the bolts are initially tensioned to 70 percent of the tensile strength, and the plate thickness and specified bolt spacing provide adequate stiffness. Under these conditions the joint remains tight, and the space between the contact surfaces of two weathering steel plates seals itself with corrosion products that form around the periphery of the joint.

High-strength bolts in tension or bearing may be designed to the allowable stresses given in the AASHTO specifications. Bolts in slip-critical connections of mill-scaled weathering steel members must be designed to lower allowable shear stresses because the mill scale is more slippery and adheres more tightly to the underlying weathering steel base than the mill scale of an ordinary steel base. Therefore, a new Class D for allowable shear stress for slip-critical connections was added to cover contact surfaces of clean, mill-scaled high-strength low-alloy steel, and quenched and tempered steel. If the mill scale is removed, the allowable shear stress for slip-critical connections of weathering steel members is the same as those for ordinary steel members.

Good detailing of bolted connections is important. Any detail that traps water and damp debris in pockets or crevices facilitates accelerated pitting or crevice corrosion. The designer must, therefore, detail such elements with extreme care to ensure there is no possibility of moisture entrapment. If this condition cannot be avoided, all such unexposed surfaces, including the contact surfaces between the plies of the connection, are to be treated like ordinary steel and must be protected by painting.

Fatigue. Unlike painted bridges in which the steel is protected against the environment, weathering steel bridges are concurrently exposed to aqueous environments and subjected to truck-induced stress cycling during their service life. The combinations of exposure and loading are complex. Before the bridge is opened to traffic and during the initial years of service, weathering creates rust pits from which cracks may eventually initiate. During the service life the aqueous environment enhances crack initiation and accelerates the rate of crack propagation. Therefore, the exposure conditions of a weathering steel bridge lead to a reduction in fatigue strength caused by the effect of weathering and corrosion fatigue on the crack initiation life plus the effect of corrosion fatigue on the crack propagation life. The effects are cumulative. These findings are supported by a vast amount of data on weathering fatigue, corrosion fatigue crack initiation life, corrosion fatigue crack growth rate, and corrosion fatigue life.

Accordingly, reductions in allowable stress range are recommended for the fatigue design of weathering steel bridges. The reductions account for type of detail and environment. The reductions are highest for Category A base metal and continuously decrease in the direction of Category D through Category E details. They are higher for details exposed in environments of high corrosivity than for those in medium corrosivity environments.

The recommended reductions in allowable stress range are not applicable to weathering steel structures exposed in environments of very high corrosivity in which long periods of wetness or high levels of contamination with salt deeply pit the surface and significantly reduce the net section, causing much higher reductions in fatigue strength than are recommended herein.

The allowable stress ranges need not be reduced for painted weathering steel structures whose paint system is properly maintained.

Construction and Maintenance

Construction. Exposed weathering steel must be handled carefully in shipment, storage, and erection. It should be treated with the amount of care required by a finished architectural product. To prevent damage from occurring during handling, members must be padded in appropriate places, blocked in transit, and handled with slings instead of chains. Exposed weathering steel must be kept free and clean of all foreign substances both in the shop and on the job site. Cover cloths may be necessary for protection of the material during construction.

Weathering steel members are best stored at a construction site in a boldly exposed condition on an incline so as to facilitate drainage of any rainwater, melted snow, or condensed dew. To prevent unsightly and excessive corrosion, the following conditions should be avoided or minimized: long transit times in railcars or open trucks; ponding of water at the construction site; contact with the soil; and entrapment of water through nesting of I-beams, girders, angles, gusset plates, and the like. Intimate contact with treated or untreated lumber used in blocking or support must be avoided by insertion of plastic sheeting at the contact points. Care must be taken to avoid deposits of soil and other surface contaminants.

The color and texture of the mill scale do not match those of the corroded steel surface. Unless the mill scale is removed, the steel surface will appear mottled, flaky, and nonuniform for several years depending on the degree of exposure and the aggressiveness of the local environment. Therefore, all surfaces visible to the public should be blast-cleaned for aesthetic reasons.

The mill scale should also be removed when it is possible that parts of or the entire structure may be exposed to long periods of wetness or contaminated with salt. Under these conditions the less noble weathering steel corrodes galvanically along the numerous cracks in the mill scale and where flecks of scale are missing. As a result deep corrosion pits forming at the scale free anodic sites may lead to large reductions in fatigue strength.

The mill scale adheres much tighter to a weathering steel base than to an ordinary steel base, and its removal will require a greater effort. Near-white blast cleaning is recommended for weathering steel surfaces exposed to public view and commercial blast cleaning for surfaces not visible to the public.

Foreign substances that adhere to the steel and inhibit the formation of the protective oxide must be removed soon after erection. Some materials, such as mud and concrete dust, will normally be displaced by the corrosion products during the natural weathering process. Paint or wax-based crayons should not be used for marking weathering steel because these materials do not wash or weather away even after many years of exposure. Oil, grease, cutting compounds, and similar insoluble contaminants can be removed from steel surfaces by solvent cleaning.

Minor contamination of the steel surface with salt and other water soluble compounds during the early stages of exposure can be removed by fresh water hosing. More severe contamination requires steam cleaning, using detergents and cleaners, followed by a steam or fresh water wash to remove detrimental residues.

Water insoluble material such as loose mill scale, loose rust, loose paint (markings), rust scale, and weld slag can be removed by hand or power tool cleaning.

During construction, before the concrete deck is cast, water running over the weathering steel superstructure and onto the piers and abutments will stain the concrete. To prevent staining during this period, the vulnerable concrete surfaces should be draped, wrapped, or otherwise sheltered with heavy-gage polyethylene sheeting. After construction and during the service life of the bridge, the concrete surfaces continue to be rust stained for an indefinite period should water running or dripping off the girders reach the piers and abutments. To reduce the penetration by rust stain, the concrete can be coated with liquid silicone-based sealers. Rust stains on concrete surfaces can be removed with proprietary chemical stain removers or, if the stained areas are large, by abrasive blast cleaning.

Inspection. An effective inspection and maintenance program is essential to ensuring that a weathering steel bridge reaches its design life. The inspection of a bare weathering steel bridge differs from, and is more difficult than, the inspection of a painted ordinary steel bridge. Unlike painted structures where rust is undesirable and its appearance serves as a warning of incipient paint failure, the entire weathering steel bridge is covered with rust. Areas where weathering steel has not developed a protective oxide coating can be detected only at close range.

Weathering steel bridges must be inspected every 2 years as is required for painted bridges. The appearance of the oxide film (color and texture) indicates the degree to which the oxide that forms on weathering steel is protective. However, a test of visual appearance alone can be deceptive. Therefore, the oxide film must be tapped with a hammer and vigorously wire brushed to determine whether it still adheres to the underlying steel base or has debonded in the form of granules, flakes, or laminar sheets. The inspector must be familiar with the various colors, textures, and general appearance that the oxide film assumes when exposed to different macro and microenvironments.

When the appearance indicates that the oxide film is nonprotective, the uniform corrosion penetration and pit depth should be measured with an ultrasonic thickness gage or a depth gage at selected locations on a member. In addition to inspecting the oxide and the steel, the inspector should also examine the joints, deck, abutments, and substructure. The condition of these elements can provide information helpful in determining the causes and severity of a corrosion problem.

The principal sign of existing or impending distress that the inspector should look for are the appearance of a nonprotective oxide film; accumulations on horizontal surfaces and in sheltered corners of windblown dust, debris, and oxide particles shed during weathering; water streaks and drain patterns on vertical and sloped surfaces; leaky expansion joints; and rust packout in crevices.

The visual detection of fatigue cracks in painted steel structures is made easier by the contrast between the rust stain and the color of the paint along the crack, and the streaks of moisture-laden rust oozing from the crack. This advantage is absent in weathering steel structures. Furthermore, the crevice formed by the crack in a weathering steel member completely fills with rust during the long service exposure, thus hiding the crack from view. As a result, fatigue cracks in weathering steel bridges are not likely to be found by visual inspection until the member has fractured. The reliability of detecting cracks in weathering steel members with other methods such as ultrasonics, accoustic emission, and radiography has not yet been evaluated.

Maintenance. Weathering steel is not a maintenance-free material. Experience has shown that highway bridges by their nature and use accumulate much debris; become wet from condensation, leaky joints, and traffic spray; and are exposed to salts and atmospheric pollutants. Different combinations of these three major factors may create exposure conditions under which weathering steel cannot form a protective oxide coating. Therefore, the bridges must be maintained properly.

The following examples illustrate the type of periodic maintenance that may be needed: removing loose debris with a jet of compressed air or vacuum cleaning equipment; scraping off sheets of delaminated rust; high-pressure hosing wet debris and aggressive agents from the steel surfaces; tracing leaks to their sources; repairing leaky joints; installing drainage systems, drip plates, and deflector plates that divert runoff water away from the superstructure and abutments; cleaning drains and downspouts; cleaning and caulking all crevices; and remedially painting areas of excessive corrosion.

Rehabilitation

Remedial Painting. Weathering steel bridges that are undergoing a high degree of uniform or local corrosion must be remedially painted to protect the affected areas against further section loss.

Remedial painting of weathering steel bridges, like repainting of ordinary steel bridges, involves the following steps: selecting a good coating system, developing sound and practical specifications, estimating the cost of painting, inspection of the painting job by an expert, and proper maintenance after the painting is completed.

Coating systems for remedially painted weathering steel must tolerate large dry film thickness variations caused by the roughness of the steel substrate, be insensitive to residues of rust and chemical contaminants that are difficult to remove from the deeply pitted surface, and have a low water vapor transmission rate to prevent osmotic blistering of the film.

The roughness of the surface and the presence of numerous pits make it practically impossible to remove all visible rust products. Some rust products often remain in the bottom of the pits even after the surface has been thoroughly cleaned. Because specifications for painting steel structures usually require near-white blast-cleaned surfaces (meaning removal of all visible rust products), the difficulty in achieving this condition must be addressed in the specification. One method is to specify that the surface meet the requirements of the SSPC visual standards at a viewing distance of 0.6 m (2 ft), deleting the verbal description of near-white blast cleaning.

The roughness of the surface makes it difficult to determine the quantity and thickness of the primer. A great deal of primer is required to fill the profile. Experience with remedially painted weathering steel bridges has shown that one gallon of primer covers only about one-fourth of the area given in the manufacturers' product data sheet.

Performance testing of generic paint systems that were applied on hanger plates, which had been removed from severely corroded joints on weathering steel bridges in Detroit, Michigan, has shown the following ranking, from best to worst: (1) multicomponent organic zinc-rich, (2) single-component organic zinc-rich, (3) single-component inorganic zinc-rich, (4) moisture-cured urethane, (5) epoxy primer, (6) chlorinated rubber, and (7) alkyd systems.

A similar study on remedial painting of A588 steel panels that were exposed 6 months atop a bridge near the Gulf Coast in southern Texas showed that: overall, the organic zinc-rich systems (epoxy primer, epoxy intermediate coat, and vinyl topcoat) were the best; the barrier type systems (e.g., those that did not contain zinc) were the best over flash rusted panels; flushing with water improved overall performance; only zinc-rich systems were not undercut; and it is more difficult to clean weathered A588 steel than A36 steel.

To summarize the results of the paint studies, coating systems with demonstrated good performance over weathered A588 steel consist of an epoxy zinc-rich primer, an epoxy polyamide intermediate coat, and either a urethane or vinyl topcoat. This hybrid system of galvanic and barrier protection has good tolerance to film thickness variation, surface contaminants, and application errors. Appendix A lists the Michigan DOT Specification for Cleaning and Coating Existing Steel Structures, including a qualified product list.

A comparative cost analysis of painting bridges in service yielded a unit cost of $2.29/ft^2$ for repainting an A36 steel bridge and $3.55/ft^2$ for remedially painting an A588 steel bridge. The Michigan experience with remedially painting four weathering steel bridges confirmed these estimates.

Inspection and Maintenance. Coating systems must be inspected and maintained. Some painted areas fail relatively early during the projected life of a coating system, typically within 5 years. To extend the life of the system and reduce local corrosion losses, paint failures must be repaired. Regular maintenance has the added benefit of reducing the life-cycle cost of a coating system.

The major section losses result from accelerated corrosion in crevices and exposure to long periods of wetness. Both can be compounded by chloride contamination. The crevices that are formed between back-to-back angles, intermittently welded members, and between hanger plates and the web at some expansion joints are particularly vulnerable. The corrosion rate is much higher within crevices than on exposed surfaces of the steel. Crevices must be rehabilitated before the structure is remedially painted. Appendixes B through E list specifications for rehabilitating hanger plate and pin connections.

CHAPTER ONE

INTRODUCTION

1.1 CHARACTERISTICS OF WEATHERING STEEL

"Weathering steel" is a term applied to a carbon steel base that is alloyed with about 2 percent of copper, nickel, chromium, and silicon. This addition inhibits the steel's tendency to continuously rust in the atmosphere. When the bridge is located in a suitable environment and the structural details are properly designed, weathering steel forms a protective oxide coating. Under these conditions, weathering steel need not be painted.

The primary advantage of using weathering steel is the potential it offers for savings in life cycle cost by eliminating the need for the initial painting and periodic repainting of the entire superstructure, as is required for ordinary steels that do not have enhanced atmospheric corrosion resistance. (Unless noted otherwise, the term "ordinary steel" refers to painted ASTM A36, A572, and A514 steels that do not have atmospheric corrosion resistance. The term "weathering steel" refers to bare, exposed A242, A588, and A709 Grade W weathering steels.) By eliminating the need for repainting, the closing of heavy traffic lanes below grade separation structures can be prevented as can safety hazards to painters and motorist. Additionally, paint jobs over difficult terrains, bodies of water, and electrified railways can also be avoided.

However, weathering steel does not provide a service life for the superstructure that is free of maintenance. The following maintenance items cannot be avoided: resealing expansion joints, painting the steel near leaking expansion joints, cleaning water drains at open joints, removing debris and dust that accumulate on horizontal surfaces, and periodic washing when the structure is excessively contaminated with deicing salts. Proper detailing and choice of a suitable environment can reduce the need for some, but not all, maintenance.

Experience over the last 22 years has revealed problems with some bridges located in the snowbelt states of the Midwest and Northeast, in the Gulf Coast states, and in high rainfall and foggy regions along the West Coast. The problems are generally caused by contamination of the steel structure with deicing salts, exposure to onshore breezes laden with sea salt, and extended periods of wetness where high relative humidity persists. These types of problems have raised questions concerning the long term performance of weathering steel bridges.

Nonetheless, as with all new developments, the service performance of existing weathering steel bridges has helped to identify design problems and corrosive environments whose significance had not been anticipated. The performance of the bridges in service has shown which design features need special attention and where an initial paint system may be necessary because of local conditions that inhibit the formation of the protective oxide coating.

The successful application of weathering steel for bridges requires a better understanding on the part of the engineer of the structural details and the microenvironment that initiate and support corrosion at a specific bridge site. An evaluation of the macroenvironment and the simple substitution of weathering steel for carbon steel in a set of standard design drawings is simply not sufficient. The criticality of these factors must be recognized.

Experience with ordinary steel bridges greatly helps in understanding how a weathering steel bridge would perform at the same site. The locations on an ordinary steel bridge, where a properly applied paint system fails early, are the same as those where local corrosion problems would occur if the bridge were fabricated from weathering steel. Furthermore, environments that reduce the service life of a paint system can be similarly aggressive toward weathering steel. Persistent wetness and heavy contamination by industrial pollutants or deicing salts are detrimental to paint systems and weathering steel alike.

1.2 APPLICATIONS

The 1982 survey of the use of bare weathering steel for highway bridges on the federal system [Albrecht and Naeemi, 1984] was updated with information obtained in phone conversations

Table 1. Survey of use of bare weathering steel for bridges on federal-aid system.

	Weathering Steel Bridges in Service				New Weathering Steel Bridges	
State	Previously Used	No. of Bridges	Used Since	Comment	Future Use	Comment
1. Alabama	Yes	2	Late 1970's	No major problem; concerned about aesthetics and future performance.	No	Concerned about experience in Michigan and elsewhere.
2. Alaska	Yes	40	1977	6 of 7 bridges in SE coastal area are corroding severely and need to be remedially painted. Problems with continuous moisture and sea water spray.	Yes	Policy: do not use along coastal area with high precipitation and salt water spray.
3. Arizona	No	...	...	Not needed; paint systems last indefinitely in arid climate; would weather nonuniformly.	No	...
4. Arkansas	Yes	75	1970	Concerned about aesthetics and concrete staining.	Yes	Keep drainage water off steel; protect piers or coat them when aesthetics are important.
5. California	Yes	1	1978	Initial rust scaling of sections shipped from overseas.	No	Prestressed concrete is more economical.
6. Colorado	Yes	30	1975	Good experience, except for some severe corrosion at leaking joints; blends in with environment; dry climate.	Yes	Blast clean to get uniform weathering sooner; paint steel 8 ft on each side of expansion joint.
7. Connecticut	Yes	40	1970	Problems with concrete staining; concerned about experience in Michigan.	Yes	Policy: use only over railroads. Protect piers during construction; install fiberglass deflectors under bearings and weld drip bars to bottom of bottom flange.
8. Delaware	Yes	10	1975	Salt water runoff corroding painted steel at leaking joints; concerned about concrete staining and experience in Michigan.	Yes	Policy: do not use in areas of high humidity or salt contamination.
9. District of Columbia	Yes	10	1972	Concerned about concrete staining and experience elsewhere.	Yes	Install troughs of expansion joints; galvanize finger plates; cover concrete during construction; seal concrete. All new bridges are over railroad tracks, because maintenance is difficult.
10. Florida	Yes	1	1972	Not visible to public; undesirable concrete staining; concerned about experience elsewhere.	No	Policy: use painted steel only.
11. Georgia	Yes	70	1978	No severe problems; some pitting and continuous corrosion near joints.	No	Policy: Use painted steel only.
12. Hawaii	No	...	...	Built last steel bridge in 1965.	No	Concrete is more economical.

with design and maintenance officials in the 50 states and the District of Columbia. The results are summarized in Table 1. The data for the bridges on the New Jersey Turnpike, a major weathering steel user, are shown for information only in the table and are not discussed in the following paragraphs.

According to the present survey, some 2,300 weathering steel bridges were built on state highway systems alone, not counting county, city, toll road, or mass transit bridges. Among the 50 states and the District of Columbia there are 4 nonusers of weathering steel, 14 former users, and 33 remaining users. The four states with no weathering steel bridges are Arizona, Hawaii, Nevada, and South Dakota. Hawaii does not use it because concrete is more economical than steel. In the other three states the climate is so dry that paint systems on ordinary steel bridges last indefinitely, and weathering steel would not weather uniformly for a long time.

The 14 former users no longer specify weathering steel for new bridges because of the following reasons: (a) excessive corrosion of bridges in their state—Indiana, Iowa, Michigan, Washington, and West Virginia; (b) concerned by experience

Table 1. Continued

	Weathering Steel Bridges in Service				New Weathering Steel Bridges	
State	Previously Used	No. of Bridges	Used Since	Comment	Future Use	Comment
13. Idaho	Yes	25	1970	Blends in with environment; some joints leak salt water onto beam seats.	Yes	...
14. Illinois	Yes	90	1968	No problems to date; concerned about long-term performance.	Yes	Policy: use only for stream crossings; painted steel 10 ft on each side of expansion joint.
15. Indiana	Yes	40	1968	May remedially paint some bridges that are corroding continuously and look bad. Concerned about experience in Michigan.	No	No policy, but avoiding use of bare weathering steel.
16. Iowa	Yes	5	1965	Remedially painted Racoon River Bridge in 1984.	No	Policy: do not use bare weathering steel.
17. Kansas	Yes	20	1970	Problems with rolling flaws and fabrication; continuous corrosion at expansion joints in urban environments. Concerned about experience in Michigan.	Yes	Policy: use in rural areas, for stream crossings, and over railroad tracks; do not use for grade separation structures. Minimize number of expansion joints and paint steel near joints. Would not use A588 in general; only for continuous rolled girder bridges in nonagressive environments.
18. Kentucky	Yes	2	1970	On one bridge, severe corrosion of bearings and steel due to salt water leaking through expansion joint. Remedially painted steel at joints.	Yes	Policy: use in rural areas where there is no salt; do not use in aggressive environments.
19. Louisiana	Yes	20	1975	Remedially painted Doullut Canal Bridge. Carefully monitoring other bridges along Gulf Coast. Initial rust scaling of sections shipped from overseas.	Yes	Provide drainage and prevent debris accumulation; paint steel 10 ft on each side of expansion joint.
20. Maine	Yes	80	1968	Positive experience; some concrete staining.	Yes	Policy: use in rural areas for stream crossings; do not use for overpasses, industrial areas, or near ocean.
21. Maryland	Yes	66	1973	Section loss at girder ends below leaking expansion joints, due to prolonged wetness; concrete staining. Remedially painted girder ends on one bridge, and may have to do it on more bridges.	Yes	Policy: use in narrow structures with good clearance for proper drying; paint steel 10 ft on each side of piers and abutments.
22. Massachusetts	Yes	20	1972	Severe corrosion problems due to deicing salts and leaking expansion joints. Preparing contract to remedially paint girder ends.	Yes	Policy: do not use where exposed to salt.

Table 1. Continued

	Weathering Steel Bridges in Service				New Weathering Steel Bridges	
State	Previously Used	No. of Bridges	Used Since	Comment	Future Use	Comment
23. Michigan	Yes	480	1965	Severe corrosion problems due to deicing salt, leaking expansion joints, and traffic spray below bridge. Remedially painted 34 weathering steel bridges to date and will paint 36 more in 1989.	No	Policy: first to ban all usage of unpainted weathering steel on state highway system (March 1980).
24. Minnesota	Yes	100	1970	Good overall performance; some corrosion problems at joints.	Yes	Policy: do not use in the cities of Minneapolis St. Paul, Duluth, and in municipalities where average daily traffic under bridge exceeds 10,000. Paint steel 5 ft on each side of joints.
25. Mississippi	Yes	22	1972	Satisfactory experience, but concerned about experience elsewhere. Dirty steel with inclusions from the mill causing problems for fracture critical members.	Yes	Policy: use in remote areas; do not use steel in grade separation structures in cities to avoid unpleasant concrete staining; do not salt bridges.
26. Missouri	Yes	50	1973	Satisfactory experience; some corrosion problems where joints were provided; remedially painted joint areas; waterproofed deck; treated substructure to avoid staining.	Yes	Policy: use in rural environments over stream crossings; not for grade separation structures. Build continuous jointless bridges with integral abutments.
27. Montana	Yes	25	1973	Good experience; dry climate causes nonuniform weathering; minor concrete staining.	Yes	Policy: do not use hinges or sliding plates.
28. Nebraska	Yes	20	1969	Used mainly in rural areas; treated concrete against staining.	Yes	Policy: do not use in areas of heavy salting.
29. Nevada	No	...	...	Dry climate; paint lasts indefinitely.	No	...
30. New Hampshire	Yes	200	1970	Good experience. Treated concrete before steel erection and cleaned it thereafter. Used deflectors on flanges to divert water off piers.	Yes	Policy: do not use in coastal environments.
31. New Jersey	Yes	1	...	...	No	Policy: use painted steel only.
New Jersey Turnpike	Yes	200	1964	Used almost exclusively. Continuous corrosion under leaking expansion joints. Planning remedial painting of critical areas.	Yes	Painted steel 5 ft on each side of joint with shop coat.
32. New Mexico	yes	1	1983	Dry climate, paint systems have long life; nonuniform weathering.	No	Do not use for aesthetic reasons.

in other states—Alabama, Florida, Georgia, Oklahoma, and South Dakota; (c) aesthetic reasons—New Mexico and South Carolina; and (d) economic reasons—California and North Dakota. New Mexico and North Dakota also have a semi-arid climate in which the paint systems on their steel bridges last indefinitely.

Among the 33 remaining users there is a great variety in the degree of usage for new bridges, from almost exclusively (Vermont) to practically none (Pennsylvania and Tennessee). Recognizing the limitations of the material, most remaining users now follow design criteria intended to enhance the corrosion performance. When asked whether they followed any special guidelines for designing weathering steel bridges, officials cited the following (see also last column in Table 1):

- Using weathering steel mainly in rural areas, remote areas, or where bridge is not visible to public (8 states).
- Using it mainly over streams (5 states) and over railroad tracks (3 states).

Table 1. Continued

State	Weathering Steel Bridges in Service: Previously Used	No. of Bridges	Used Since	Comment	New Weathering Steel Bridges: Future Use	Comment
33. New York	Yes	100	1969	Good experience. Used in rural areas, over railroad tracks, in areas not visible to public, and on some expressways. Drip bars on top of bottom flange were not always effective.	Yes	Guidelines: use over railroad tracks and areas not visible to public.
34. North Carolina	Yes	215	1968	Very good experience. Did not use in marine environments, industrial areas with high acidity, near smoke stacks, and under tunnel like conditions.	Yes	...
35. North Dakota	Yes	2	1970	Excellent experience; semi-arid climate.	No	Concrete is more economic; paint lasts indefinitely.
36. Ohio	Yes	100	1971	Mostly favorable experience; few severe corrosion problems requiring remedial painting. Sub-drains not low enough; some problems due to salt. Severe corrosion on county bridges having corrugated steel deck with asphalt riding surface and low clearance over streams.	Yes	Trend towards using less weathering steel. Was previously first choice. Paint steel a distance of two girder depths on each side of all joints.
37. Oklahoma	Yes	5	1975	Satisfactory experience, but concerned about experience in Michigan. Nonuniform weathering.	No	Policy: use painted steel.
38. Oregon	Yes	4	1975	Good experience; limited accelerated corrosion where debris and moisture are trapped. Concrete staining.	Yes	Blast clean for uniform weathering. Try to avoid details that trap moisture.
39. Pennsylvania	Yes	40	1968	Excessive corrosion of 10-year old bridges at joints due to moisture and salt; but not yet critical. Material sensitive to lack of maintenance; not suitable for state's environment.	Yes	Use discouraged. Limited applications, mostly in upstate rural areas of low deicing salt usage.
40. Rhode Island	Yes	13	1974	Good experience; some concrete staining.	Yes	Use with caution.
41. South Carolina	Yes	1	...	Dislike appearance.	No	...
42. South Dakota	No	...	...	Dry climate; paint lasts indefinitely. Concerned about experience elsewhere.	No	...

- Not using it for grade separation structures (4 states); nor in cities, and where the average daily traffic exceeds 10,000 (1 state).
- Not using it along the coast (6 states), on heavily salted highways (4 states), or in high humidity areas (3 states).
- Painting steel 1.5 to 3 m (5 to 10 ft) on each side of joints (10 states).
- Blast cleaning all steel before erection (2 states).
- Keeping drainage water from running over substructure and protecting concrete against rust staining (5 states).
- Galvanizing scuppers, bearings, and expansion devices (1 state); galvanizing finger plates (1 state); not using hinges or sliding plates (1 state).
- Making decks jointless and building bridges integrally with abutments where conditions permit (1 state).

In several states weathering steel bridges have not performed as had been anticipated. As a result, several states have remedially painted bridges (Iowa, Louisiana, Michigan, and Ohio) or are in various stages of planning such work (Alaska, Indiana, Texas, and Washington). In other states where weathering steel bridges are excessively corroding only in limited areas, the steel near joints was remedially painted (Kentucky, Maryland, and Missouri), or such work is being planned (Massachusetts and New Jersey Turnpike).

Table 1. Continued

State	Weathering Steel Bridges in Service: Previously Used	No. of Bridges	Used Since	Comment	New Weathering Steel Bridges: Future Use	Comment
43. Tennessee	Yes	6	1974	Satisfactory performance, but dislike appearance and concrete staining.	Yes	General consensus: do not specify weathering steel.
44. Texas	Yes	50	1974	Satisfactory performance away from coast. One bride 2 miles from coast corroding continuously. Nonuniform appearance in dry areas.	Yes	...
45. Utah	Yes	15	1970	Used mainly in rural areas, in the western part of state, and areas free of salt and pollution. Decks are continuous where possible and joints are maintained. Nonuniform appearance; used more for economical than aesthetic reasons.	Yes	Use selectively.
46. Vermont	Yes	97	1973	Good experience; used almost exclusively. Some areas are painted for corrosion protection. Drain tubes leaked water onto bottom flange and were plugged. Only problem where salt leaks onto girders.	Yes	Paint steel for protection against moisture where needed. Paint steel 5 ft on each side of expansion joints. Seal joints. Galvanize scuppers, bearings, and expansion devices.
47. Virginia	Yes	40	1972	Generally good experience except for excessive corrosion at leaking joints. One bridge is severely corroding.	Yes	Policy: paint steel 5 ft at each side of expansion joints and from ends; paint exterior of facia girder. Anticipating future use on rare occasions.
48. Washington	Yes	12	1966	Some bridges performing unsatisfactorily; flaking, lamellar corrosion and pitting. Considering remedial painting of three bridges in near future.	No	Policy: do not use west of mountains where humidity is high; may stop using it statewide.
49. West Virginia	Yes	11	1971	Deicing salts causing continuous corrosion. Salt water was leaking through expansion joints of New River Gorge Bridge. Instituted maintenance program.	No	Policy: do not use weathering steel.
50. Wisconsin	Yes	30	1976	Satisfactory experience. Some problems with excessive corrosion at joints, short drains discharging water onto lower lateral bracing, concrete staining and nonuniform weathering of mill-scaled surfaces.	Yes	Use primarily on stream crossings and grade separation structures in remote areas.
51. Wyoming	Yes	20	1975	Good experience. Slow development of uniform appearance in dry climate.	Yes	Limited use, unavailable in smaller quantities.

1.3 EXPERIENCE

Although many weathering steel bridges are performing well and 33 states continue to specify weathering steel for new bridges, experience with bridges in service since 1965 has shown that weathering steel is not a maintenance free material and has distinct limitations with regard to the environmental conditions under which it can be used. Weathering steel does not develop a protective oxide coating if the steel remains wet for a long time or is contaminated with salt, or debris, from any source.

A clear understanding of the conditions that interfere with the proper development of the oxide coating is essential to the successful application of weathering steel. The case studies in Appendix F illustrate various conditions where these limitations are exceeded. The purpose of the discussion presented in Appendix F is to provide, from the experiences of others, a better understanding of how to avoid past problems. The guidelines recommended in the remaining chapters of this report, for the design, fabrication, maintenance, and rehabilitation of weathering steel bridges, were developed in light of these experiences.

CHAPTER TWO

CORROSION PERFORMANCE

2.1 CORROSION PRINCIPLES

2.1.1 Importance of Corrosion Characteristics of Weathering Steel

The successful use of a structural material depends on its resistance to environmental degradation throughout the life of the structure. The resistance to deterioration may be intrinsic to the structural material or may be achieved by added protection such as coatings. In most applications weathering steels are used in a bare, exposed condition without added protection against the environment. Thus, the well-demonstrated ability of weathering steel to resist atmospheric corrosion is vital to its performance. However, weathering steel is resistant only to specific types of corrosion in a limited range of environments. The proper design of weathering steel structures, therefore, must be based on an understanding of the material's corrosion-resisting characteristics.

2.1.2 Corrosion Reaction Components

Corrosion is the deterioration of a metal through chemical or electrochemical reaction with the environment. For corrosion to occur there must be an active metal, a corrodent to react with the metal, and a conductive medium in which the corrosion reaction takes place. For weathering steel corrosion, the active metal is the weathering steel itself; the corrodent is a combination of oxygen from the atmosphere, water, and contaminants; and the conductive corrosion medium is water with dissolved contaminants.

The basic reaction in corrosion engineering is the familiar dry cell battery. Any combination of metals can form a battery, such as the zinc-carbon couple in the dry cell. However, many conditions can create "batteries" or "cells" that do not require two different metals, such as in corrosion of weathering steel members. Differences in condition on a weathering steel member or in the electrolyte in different locations in contact with the steel can create a battery and result in the member corroding preferentially at some locations. Thus, engineers need to understand the components that make up a battery when designing structures against premature corrosion. The forms of corrosion that are easily recognizable include uniform, pitting, crevice, and galvanic corrosion.

2.1.3 Types of Corrosion

Atmospheric and Immersed Corrosion. For bare steels it is important to distinguish between resistance to atmospheric corrosion and resistance to immersed corrosion. Atmospheric corrosion occurs when a metal exposed in the atmosphere is wet by a thin film of moisture. Immersed corrosion occurs when a metal is either submerged in water or exposed to water retained by some feature of the structure. The performance of weathering steel is superior to that of ordinary steel in atmospheric corrosion, but not in immersed corrosion where the protective oxide does not form.

For both weathering and ordinary steels, the rate of atmospheric corrosion is generally lower than their immersed corrosion rate. The lower atmospheric corrosion rate of steel relative to its immersed corrosion resistance is due in part to the properties of the thin film of moisture in which atmospheric corrosion takes place. The thin film in atmospheric corrosion is much less conductive than the same solution present in bulk in immersed corrosion. Also, in atmospheric corrosion the products of the corrosion reactions cannot diffuse away through the corrosion medium as they can during immersed corrosion. Therefore, the corrosion products are present to serve as raw material for the development of a protective oxide layer. In many atmospheres a protective layer develops on both ordinary and weathering steels. However, the protective layer on weathering steel is much more effective in reducing atmospheric corrosion than the protective oxide that forms on ordinary steel.

A major factor in determining the severity of atmospheric corrosion is the "time-of-wetness." This term refers to the length of time a metal remains wet enough to corrode at an appreciable rate. The wetting that promotes corrosion may be due to precipitation, condensation, runoff from another part of the structure, or absorption from the atmosphere at high relative humidities. A thin, invisible film of moisture that supports corrosion forms on the surface of corroded weathering steel when the relative humidity exceeds about 70 percent in a salt-free environment and 55 percent in a salt-laden environment. Above these critical relative humidities the atmospheric corrosion rate increases sharply as the humidity increases.

General Versus Localized Corrosion. There are notable differences between localized and general corrosion of metals. Localized corrosion takes place in only a relatively small area of the total exposed surface, whereas general corrosion takes place over the entire exposed surface. For the same amount of metal corroded away, localized corrosion may produce a more deteriorated condition than general corrosion because the depth of attack is greater and a more irregular surface is produced. In localized corrosion specific anode sites remain active for extended periods of time, producing metal loss at these areas only. In general corrosion each part of the entire corroding surface is at some time anodic, causing metal loss over the entire surface.

Pitting Corrosion. Pitting corrosion is a form of localized corrosion that produces a pitted profile on the metal's surface. The profile of the pits can vary from broad and shallow to deep and narrow. Because pits can serve as initiation sites for fatigue cracks, the pitting of weathering steel in bridges should be minimized.

Crevice Corrosion. Crevice corrosion occurs where crevices provide an environment that is different from the environment of the bulk of the structure and the environment produced leads to accelerated attack. Different metals have characteristic susceptibilities to crevice corrosion. In weathering steel, crevice corrosion can occur where moisture and contaminants are retained by inadequately designed details, such as intermittently welded joints or open narrow crevices between adjoining members. Crevice corrosion is also a problem for painted steel structures with such crevices.

Galvanic Corrosion. Galvanic corrosion occurs when a dissimilar conductive material is in electrical contact with a metal as well as in contact with the conductive corrosion medium. The dissimilar conductive material can be another metal or nonmetals such as mill scale.

The driving force of galvanic corrosion depends on the nature of the dissimilar material coupled with the reference metal. Materials that provide additional anodic area are called "active" relative to the reference metal; those that provide more cathodic area are called "noble" relative to the reference metal. The coupling of weathering steel with more active metals, such as zinc or ordinary steel, results in accelerated attack of the active metals; the coupling with more noble metals, such as stainless steel, copper alloys, or bronze, results in accelerated attack on the weathering steel.

Galvanic corrosion is limited by the conductivity of the corrosion medium. Thus, the effects of galvanic corrosion are not as destructive in air as under immersed condition. However, the combination of galvanic corrosion and crevice corrosion, as with a bronze washer forming a crevice with a weathering steel member, can be quite destructive.

Another factor that controls the extent of galvanic corrosion concerns the relative areas of the anode and cathode. When the cathodic area greatly exceeds the anodic area, the loss of metal from the anodic area, required to balance the reaction of the corrodent on the cathodic area, can lead to unacceptably high corrosion rates [Ellis and LaQue, 1951]. Figure 1 shows the increasing corrosion of carbon steel coupled with an increasing relative area of weathering steel immersed in seawater. The data illustrate why weathering steel members must not be joined with carbon steel fasteners or weld deposits having the same composition as that of carbon steel. (A single-pass weld made with a carbon steel electrode on weathering steel may be diluted enough by the base metal so that a weld deposit is produced that presents no galvanic corrosion hazard.)

Coatings may be used to protect against galvanic corrosion. When organic coatings are used it is necessary to coat both the active and noble areas, because the driving force for galvanic corrosion can be so high that coating only the active metal usually leads to breakdown of the coating and subsequent corrosion of the active metal.

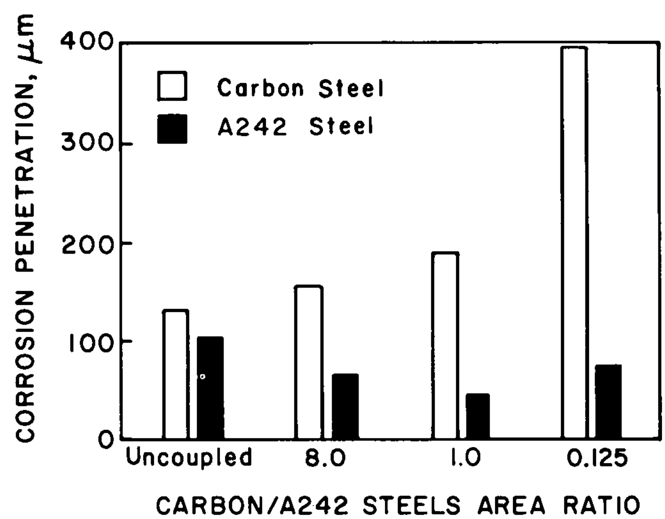

Figure 1. Corrosion penetration of uncoupled and coupled steels after 6 months in seawater. [Ellis and LaQue, 1951]

2.2 PROTECTIVE OXIDE ON WEATHERING STEEL

2.2.1 Formation of Protective Oxide

When weathering steel is exposed to the atmosphere, a powdery orange rust forms during the first wetted period. This rust resembles the rust on ordinary steel exposed to the atmosphere. A large part of the iron corrosion products is retained in the rust, which is formed by the reaction of oxygen present in the air with the soluble ferrous salts that the corrosion reaction produces. However, some soluble corrosion products are not incorporated in the rust as is evidenced by the iron oxide stains on concrete piers subjected to water runoff from the exposed weathering steel. Subsequent wet periods produce additional rust on the weathering steel and soluble corrosion products in the runoff.

As weathering steel continues to be exposed to the atmosphere, the rust layer on the surface recrystallizes and consolidates. The rust becomes darker and bluer with time. After the consolidated rust layer forms on weathering steel, the atmospheric corrosion rate is significantly lower than the corrosion rate on first exposure.

The composition of the consolidated rust layer has been studied extensively. Its major components are iron oxyhydroxides of complex structure. Other components may be iron oxides, iron salts, and ash and other materials deposited from the environment. The consolidated rust layer is often called "protective oxide."

Experiments and field observations show that periodic wetting and drying periods are necessary for the development of the characteristic protective oxide on weathering steel. The protective oxide does not form under immersion conditions. Without the formation of the protective oxide the corrosion rate of weathering steel does not decrease with time.

In some environments the corrosion rate of weathering steel is so low that the steel does not corrode enough to form a protective oxide. Lack of formation of protective oxide in such cases will not cause significant corrosion loss if the corrosivity of the environment remains low. However, the appearance of the steel in the absence of the normal protective oxide will resemble ordinary rusty steel and will generally not be acceptable.

Contaminants affect the protective oxide formation on weathering steel. Salt from many possible sources and acidic sulfur oxides from the combustion of fuels containing sulfur are the most commonly encountered contaminants.

When present in sufficient quantity these contaminants, in combination with atmospheres of otherwise acceptable corrosivity, may produce new corrosion products on the metal-rust interface that push away the overlying protective oxide layer. As a result, the outer oxide layer may detach in flakes or sheets. In environments of high corrosivity the weathering steel may never form a stable protective oxide, whereas in environments of lower corrosivity a protective oxide will form after a period of several years. A protective oxide coating may detach itself if the steel is exposed to an intermittent period of highly corrosive environment.

2.2.2 Requirements for Protective Oxide Development

Steel Composition. An important requirement for the successful performance of weathering steel is an adequate content of alloying elements that lead to protective oxide formation and a consequent decrease in the corrosion rate. The effect of steel composition on atmospheric corrosion rates after several years of exposure is illustrated in Figure 2.

With the detailed knowledge now available on the effects of alloy content on atmospheric corrosion resistance, it is possible to develop weathering steels with alloy contents that would have very high atmospheric corrosion resistance [Copson 1960; Larrabee and Coburn, 1962]. Such steels would resist atmospheric corrosion in much more aggressive atmospheres than are suitable for the present grades of weathering steel for bridges. However, the available weathering steels are limited in their atmospheric corrosion resistance because an increase in alloy content that is beneficial to corrosion resistance can lower the toughness and weldability. The practical upper limit on alloy content restricts the corrosivity of environments that are suitable for weathering steel bridges.

Environment. A suitable environment is necessary for the successful use of weathering steel. As discussed below, the environment must provide adequate periods of drying, must not provide excessive periods of wetting, and must not contain excessive corrosive contaminants. The environment of a bridge is a combination of factors related to the general climate of the region plus factors related to the specific bridge site. These local climactic conditions are often referred to as the microenvironment.

Design. An adequate design is necessary for the protective oxide to form on members exposed to corrosive environments. The design must allow for alternate wetting and drying cycles, it must not allow immersion conditions to exist, and it should avoid connections and details susceptible to crevice corrosion and galvanic corrosion. Suitable design details are recommended in Chapter Six.

2.2.3 Characteristics of the Protective Oxide

The normal protective oxide on bridge steels may exhibit a wide range of colors and textures. The color may vary from maroon to black, in contrast to the bright orange color of new rust on ordinary steel. The protective oxide incorporates ash and other contaminants so the oxide layer that forms in industrial atmospheres is generally darker than that in rural atmospheres. The texture may be smooth or may provide small particles of rust when rubbed with the hand. The normal protective oxide will adhere to the underlying steel and will not separate in flakes or sheets.

2.3 EFFECTS OF ENVIRONMENTS ON CORROSION

2.3.1 Atmosphere Types

Various atmosphere types have been identified as suitable and convenient for the atmospheric corrosion testing of metals. Corrosion data from these sites show the effects of atmosphere type and other variables on weathering steel corrosion. However, data from such tests do not usually match the corrosion rates

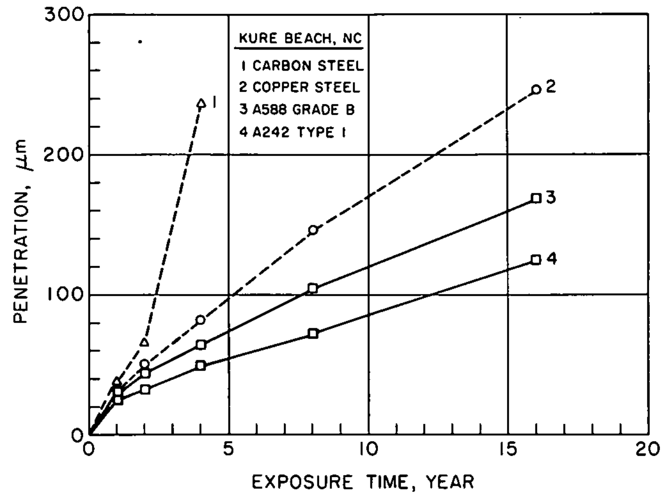

Figure 2. Effect of steel composition on atmospheric corrosion rate. [Shastry et al., 1988]

of weathering steel in bridges exposed to the same atmosphere type because the test data are obtained from small coupons exposed fully to the sun and the weather (called "boldly exposed" in corrosion literature). In contrast, the thermal behavior of bridge structural members is different from that of the test coupons in that the members may not be boldly exposed and are subjected to additional environmental influences, such as spray from roadway water. Also, much data on atmospheric corrosion labeled "weathering steel" is related to A242 Type 1 steel, which has a higher alloy content and exhibits lower atmospheric corrosion rates than A588 steel commonly used in bridges.

Data shown in the following paragraphs are largely from sites in the contiguous United States. Data from test sites in northern Europe generally show higher corrosion rates because the levels of atmospheric contamination ("acid rain") are higher and the time-of-wetness is longer at the northern latitude sites.

Rural Atmospheres. Atmospheres identified as rural in corrosion testing are atmospheres remote from urban centers and industrial plants, which are a source of contaminants that affect the corrosion rate of susceptible metals. The corrosion rates in rural atmospheres are generally lower than in other atmosphere types. Typical corrosion test performances of A588 steel in the rural atmospheres identified in Table 2 are shown in Figure 3. (The U.S. Steel technical reports referenced in Tables 2, 3, and 4 are not available in the open literature.)

Industrial Atmospheres. Atmospheres identified as industrial in corrosion testing are those close to industrial facilities in which the deposition of materials emitted by industrial plants affects the corrosion rate of susceptible metals. Typical corrosion test performances of A588 steel in the industrial atmospheres identified in Table 3 are shown in Figure 4. Urban atmospheres often are classified in the same category as industrial atmospheres because the corrosion rates in urban atmospheres can approach those in industrial atmospheres.

Marine Atmospheres. Atmospheres identified as marine in corrosion testing are atmospheres in which the deposition of salt originating from a body of salt water affects the corrosion rate of susceptible metals. Marine atmospheres are often distin-

Table 2. Description of corrosion penetration curves for A588 steel exposed in rural environments. See also Figure 3.

Curve No.	Type of Steel	Exposure Time (yr)	Exposure Site	Reference
1	A588 Gr. A	7.5	South Bend, Pa.	Gallagher 1978 & 1982, Vrable 1985
2	A588 Gr. A[a]	7.5	South Bend, Pa.	Gallagher 1978 & 1982, Vrable 1985
3	A588 Gr. A	7.5	Potter County, Pa.	Gallagher 1978 & 1982, Vrable 1985
4	A588 Gr. A[a]	7.5	Potter County, Pa.	Gallagher 1978 & 1982, Vrable 1985
5	A588 Gr. A	15.5	South Bend, Pa.	Schmitt 1965, Gallagher 1976, Komp 1987
6	A588 Gr. B	16.0	Saylorsburg, Pa.	Cosaboom 1979, Townsend 1982, Shastry 1988

Note:

a. New composition of A588 Grade A adopted in 1978.

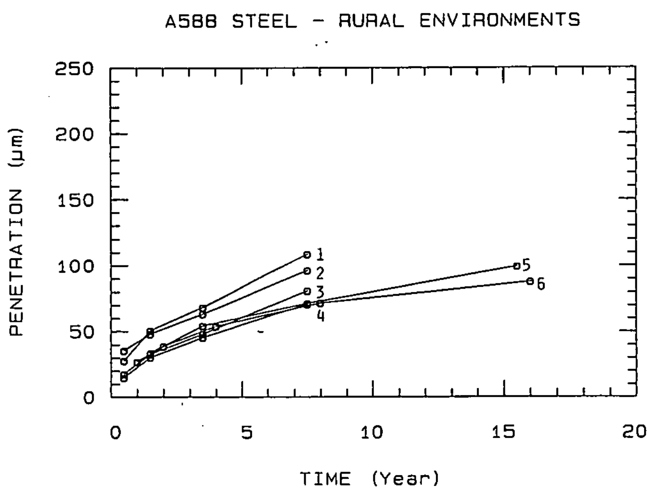

Figure 3. Corrosion penetration of A588 steel exposed in rural environments. See also Table 2.

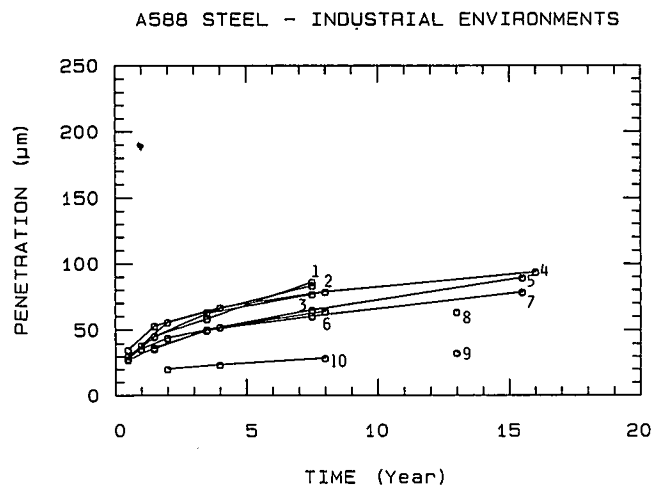

Figure 4. Corrosion penetration of A588 steel exposed in industrial environments. See also Table 3.

Table 3. Description of corrosion penetration curves for A588 steel exposed in industrial environments. See also Figure 4.

Curve No.	Type of Steel	Exposure Time (yr)	Exposure Site	Reference
1	A588 Gr. A	7.5	Monroeville, Pa.	Gallagher 1978 & 1982, Vrable 1985
2	A588 Gr. A[a]	7.5	Monroeville, Pa.	Gallagher 1978 & 1982, Vrable 1985
3	Low alloy	7.5	Kearny, N.J.	Larrabee 1953
4	A588 Gr. A[b]	16.0	Bethlehem, Pa.	Cosaboom 1979, Townsend 1982, Shastry 1988
5	A588 Gr. A	15.5	Monroeville, Pa.	Schmitt 1965, Gallagher 1976, Komp 1987
6	A588 Gr. B	8.0	Newark, N.J.	Cosaboom 1979, Townsend 1982
7	A588 Gr. A	15.5	Newark, N.J.	Schmitt 1965, Gallagher 1976, Komp 1987
8	A588 Gr. A,G,H	13.0	Los Angeles, Ca.	Reed 1982
9	A588 Gr. A,G,H	13.0	Sacramento, Ca.	Reed 1982
10	A588 Gr. B	8.0	Detroit, Mi.	Tinklenberg 1986b
11	A588	to 17	Michigan bridges[b]	McCrum 1985

Note:

a. New composition of A588 Grade A adopted in 1978.

b. Urban and rural beams not exposed to traffic spray.
Average corrosion rate of all beams: 10 μm/yr (0.4 mil/yr).

guished as mild, moderate, or severe. This indicates the relative extent to which salt deposition increases the corrosion rate in comparison to that in an uncontaminated atmosphere. Typical corrosion test performances of A588 steel in the marine atmospheres identified in Table 4 are shown in Figure 5.

2.3.2 Factors Affecting Microclimates

Shelter. Shelter affects the atmospheric corrosion rate of weathering steel by a variety of mechanisms. Sheltering can, in some instances, prevent rain from washing corrosive contaminants from a surface or it can prevent the deposition of corrosive contaminants. Also, it can lower or raise the time-of-wetness of the steel. Thus, it is not possible in every case to predict the effect that shelter will have on atmospheric corrosion rate. Observations on bridges have shown that sheltered weathering steel contaminated with salt and subjected to periods of condensation or high humidity will corrode at unacceptably high rates.

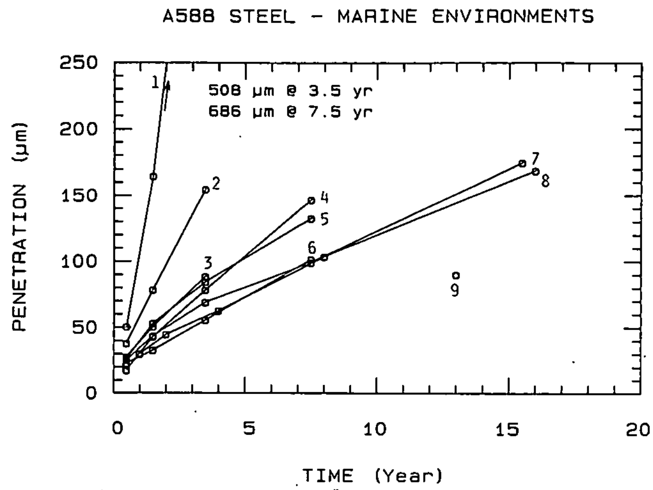

Figure 5. Corrosion penetration of A588 steel exposed in marine environments. See also Table 4.

Table 4. Description of corrosion penetration curves for A588 steel exposed in marine environments. See also Figure 5.

Curve No.	Type of Steel	Exposure Time (yr)	Exposure Site	Reference
1	A588 Gr. A	7.5[b]	Kure Beach, N.C.	Schmitt 1965, Gallagher 1976, Komp 1987
2	A588 Gr. A	3.5[c]	Kure Beach, N.C.	Gallagher 1978 & 1982, Vrable 1985
3	A588 Gr. A[a]	3.5[b]	Kure Beach, N.C.	Gallagher 1978 & 1982, Vrable 1985
4	A588 Gr. A	7.5[c]	Kure Beach, N.C.	Gallagher 1978 & 1982, Vrable 1985
5	Low alloy	7.5[c]	Kure Beach, N.C.	Larrabee 1953
6	A588 Gr. A[a]	7.5[c]	Kure Beach, N.C.	Gallagher 1978 & 1982, Vrable 1985
7	A588 Gr. A	15.5[c]	Kure Beach, N.C.	Schmitt 1965, Gallagher 1976, Komp 1987
8	A588 Gr. B	16.0[c]	Kure Beach, N.C.	Cosaboom 1979, Townsend 1982, Shastry 1988
9	A588 Gr. A,G,H	13.0	Point Reyes, Ca.	Reed 1982

Note:

a. Specimen exposed at 24-m lot.
b. Specimen exposed at 240-m lot.
c. New composition of A588 Grade A adopted by U.S. Steel in 1978.

Orientation. Like shelter, orientation can increase or decrease the atmospheric corrosion rate of weathering steel, depending on other environmental factors at the site. In general, the atmospheric corrosion rate of weathering steel on surfaces exposed to the ground is higher than it is on surfaces exposed to the sky (Fig. 6), and it is also higher on surfaces exposed to the north than it is on surfaces exposed to the south. These effects can be attributed to the longer time-of-wetness of the groundward and northward orientations because the sun dries the steel exposed to these orientations less than it does to steel exposed to the sky and the south.

Airborne Contaminants. Where present, contamination by airborne chlorides and sometimes by sulfur compounds causes problems for weathering steel bridges. The corrosion rate of weathering steel can be significantly affected by these and other contaminants originating from a variety of sources, for instance, fumes from chemical process plants, vapors from polluted waterways, splash from roadway water containing deicing salts, and acid rain.

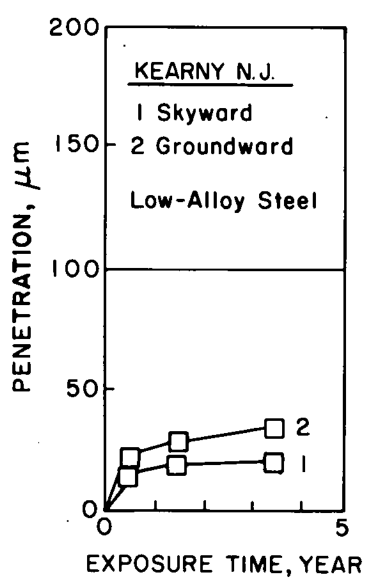

Figure 6. Effect of skyward versus groundward exposure on corrosion of weathering steel in Kearny, N.J., industrial atmosphere. [Larrabee 1944]

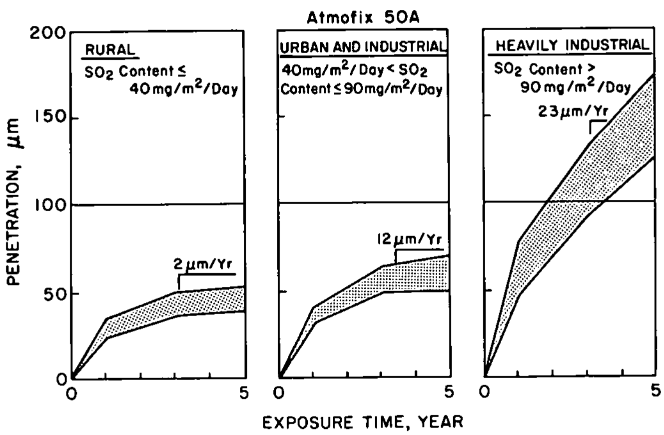

Figure 7. Effect of sulfur dioxide content on corrosion of weathering steel exposed to various atmospheres in Czechoslovakia [Knotkova 1982].
(**Note**: 1.0 mg SO_2/m^2/day deposition rate $\simeq$ 0.8 μg SO_2/m^3 volumetric concentration. [International Standards Organization 1988])

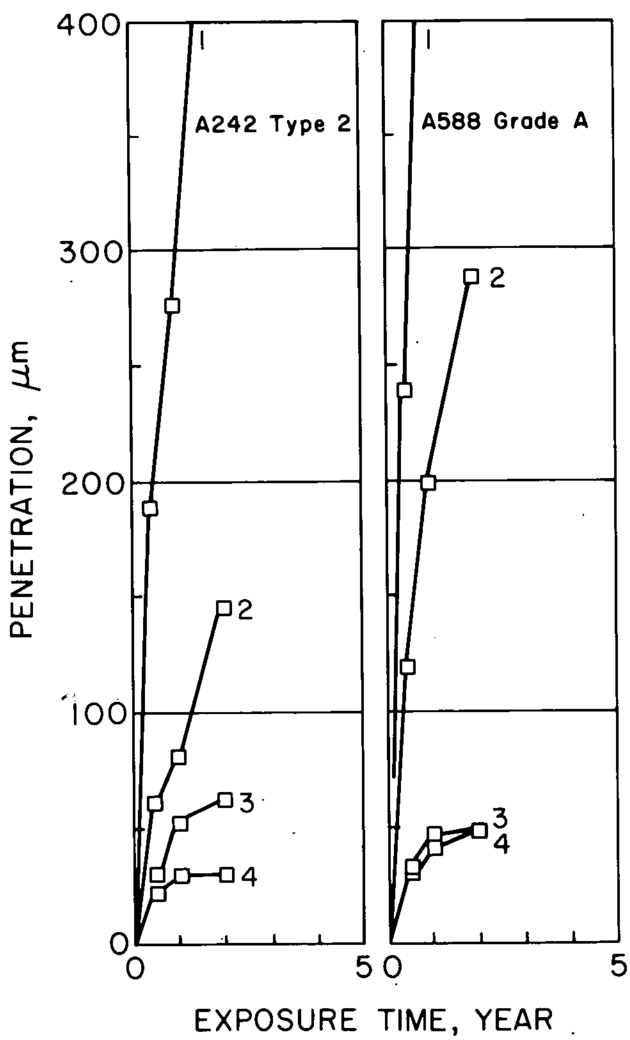

Figure 8. Corrosion penetration of carbon and weathering steels exposed to various chemical plant atmospheres. See also Table 5. [Schmitt and Mathay, 1967]

Table 5. Corrosion penetration of weathering steel exposed to various chemical plant atmospheres. See also Figure 8.

Curve No.	Type of Plant	Atmospheric Constituents
1	Sulfur	Chlorides, sulfur and sulfur compounds
2	Chlor-alkali	Mositure, chlorides and lime
3	Chlor-alkali	Moisture, lime and soda ash
4	Petrochemical	Chlorides, hydrogen sulfide and sulfur dioxide

The effect of different atmospheric concentrations of sulfur oxides on the corrosion rate of carbon and weathering steels in a European country is shown in Figure 7 [Knotkova et al., 1982]. As can be seen, weathering steel corrodes more, the higher the sulfur content in the atmosphere.

The effects of several chemical plant atmospheres are shown in Figure 8 (see also Table 5). The relative corrosivities of the contaminants are likely to persist in lower concentrations. Concentrations of the contaminants sufficient to produce the high corrosion rates shown in Figure 8 are not likely to be encountered outside the plant [Schmitt and Mathay, 1967].

Debris. Debris can affect the corrosion performance of weathering steel bridges according to the extent to which it provides a trap for moisture or contaminants. In general, debris such as gravel, glass, or other hard materials does not provide sites for poor corrosion performance, whereas porous, absorbant materials that retain moisture do provide such sites.

Continuous Moisture and Thermal Effects. As noted earlier, periods of drying are necessary for weathering steel to develop a protective oxide. Long periods of moisture provide a continuously present corrosive medium that prevents the development of a protective oxide. It can lead to unacceptably high corrosion rates especially with moisture of low pH. Test data on the effect of continuous moisture are shown in Figure 9a [Larrabee 1953].

For bridges, continuous moisture can result from natural sources such as coastal fogs, high banked streams or rivers, leaking expansion joints, water trapped by crevices or other construction details. Hollow members such as box sections can accumulate water if not properly drained or sealed.

When the temperature of a bridge member is near or below the dew point of the surrounding atmosphere, the critical relative humidity for corrosion will be exceeded. At sufficiently low temperatures, moisture will condense to the extent that the condensed water will drain down the web. Rust formed on the path of this drainage will provide a visible trace (Fig. 9b).

The mass of metal in a bridge acts as a heat reservoir. It is commonly observed that bridge members retain the heat of the day and remain warm in the early evening. They also retain the cold of the night into the morning. This thermal lag leads to the time-of-wetness of bridge members being offset to that of smaller metal samples such as corrosion test coupons. The interaction between the variation in humidity during the daily cycle and the different periods of time-of-wetness can lead to differences in the corrosion behavior of bridge members and corrosion test coupons. Therefore, bridge members may corrode at somewhat higher or lower rates than corrosion test coupons, depending on unpredictable, complex interactions with other features of the microclimate.

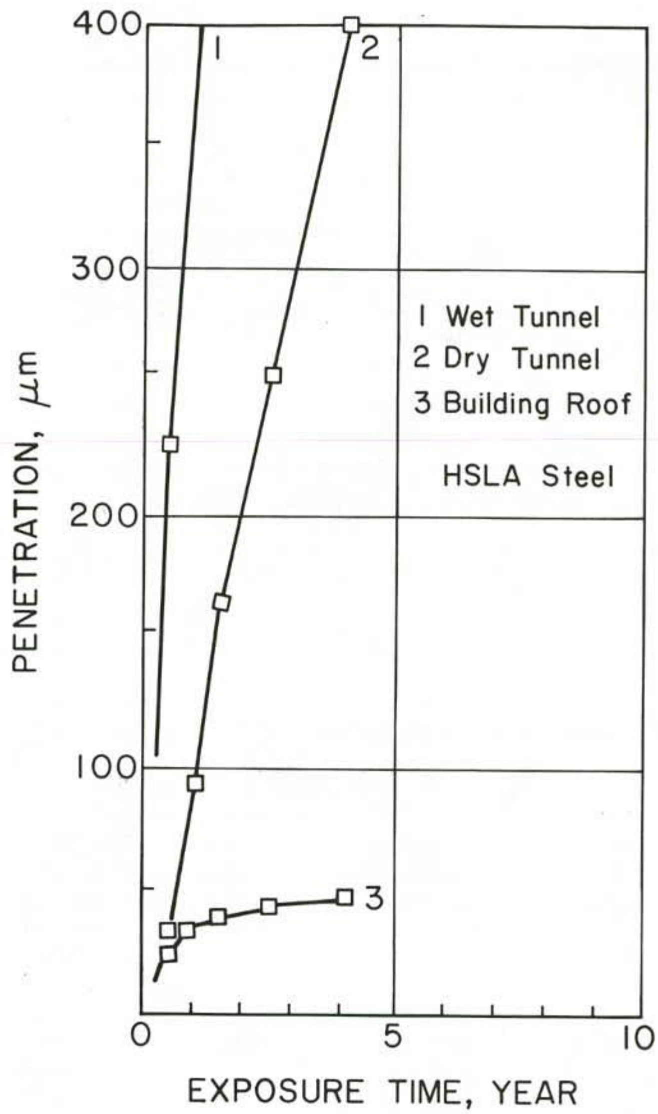

Figure 9a. Effect of continuously moist condition on corrosion of weathering steel. [Larrabee 1953]

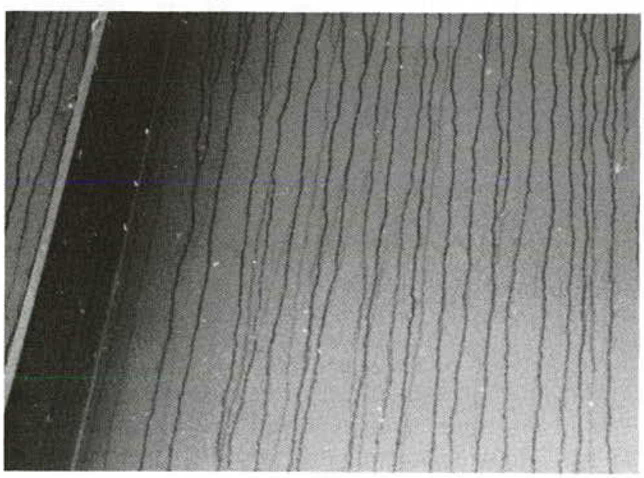

Figure 9b. Condensed water draining down web of plate girder.

Buried Steel. Because no protective oxide forms under buried conditions, weathering steel offers no advantage over carbon steel in underground corrosion resistance. If weathering steel is buried, it should be protected against corrosion to the same extent as is appropriate for carbon steel in the same soil. The protection can be provided by coatings, cathodic protection, or a combination of both.

Structural Details and Geometries. Details that retain rust flakes, windblown dirt, and moisture will lead to corrosion. Confined geometries of grade separations that permit the deposition of water spray from the lower roadway onto the overhead bridge can intensify corrosion rates, particularly if the water contains deicing salts. In such geometries the exterior girder facing the oncoming traffic corrodes more than the interior girders sheltered from the spray [McCrum et al., 1985].

2.3.3 Corrosivity of Atmospheres

The corrosion rate of weathering steel boldly exposed in an atmosphere free of significant amounts of contaminants is high during the initial period of exposure and, after several years, approaches a steady-state value. For practical purposes, the steady-state corrosion rate can be assumed to be constant for very long exposure periods.

The ISO Draft Proposal 9224 for Corrosion of Metals and Alloys classifies the corrosivity of atmospheres in five categories [International Standards Organization 1988]. Table 6 gives for each category the range of steady-state corrosion rate of weathering steel.

2.4 SERVICE CORROSION OF WEATHERING STEEL BRIDGES

2.4.1 Expected Performance

Corrosion Penetration. The corrosion penetration found for weathering steel in a variety of bridges in the contiguous United States can be contained in the envelope, shown in Figure 10, with an upper bound of:

$$C = 50 + 7.5\ (t-1) \qquad (1)$$

and a lower bound of

$$C = 25 + 3\ (t-1) \qquad (2)$$

where C = corrosion penetration per side in μm, and t = time of exposure in years. (It is noted that all corrosion penetration values given in this report as well as in *NCHRP Report 272* [Albrecht and Naeemi, 1984] are per exposed surface.) The corrosion rates for the upper and lower bounds (7.5 and 3.0 μm/year/surface) are equal to the average steady-state corrosion rates for the ISO high and medium corrosivity categories (Table 6).

The data in Figure 10 include bridge and industrial exposure locations as shown in Table 3. These losses are shown as average penetration, whereas examination of the underlying steel shows that some shallow pitting is occurring in conjunction with general corrosion. However, the use of the average penetration is adequate for many structural calculations.

Table 6. Guiding values of steady-state corrosion rates of weathering steel for corrosivity categories of atmospheres. [International Standards Organization 1988]

Corrosivity Category	Steady-State Corrosion Rate	
	(μm/year)	(mil/year)
Very low	less than 0.1	less than 0.004
Low	0.1 to 1	0.004 to 0.04
Medium	1 to 5	0.04 to 0.2
High	5 to 10	0.2 to 0.4
Very high	10 to 80	0.4 to 3.2

Weathering steel exposed in bridges that are performing satisfactorily can be expected to have corrosion rates approaching 7.5 μm/year/surface (0.3 mil/yr/surface). According to Eq. 1, the expected loss in thickness for a 100-year service life extrapolates to (all thickness loss values are the sum of the corrosion penetration for both surfaces of a plate):

$$\Delta t = (50\ \mu\text{m} + 100\ \text{yr} \times 7.5\ \mu\text{m/yr}) = 0.8\ \text{mm/surface (32 mil/surface)} \quad (3)$$

For members 38 mm (1.5 in.) or greater in thickness, production tolerances will provide sufficient steel to compensate for these losses. For members less than 38 mm (1.5 in.) in thickness, the corrosion loss in a 100-yr service life would affect the structural integrity of the member. Therefore, the thickness of weathering steel members less than 38 mm (1.5 in.) thick should be increased by 0.8 mm (32 mils) above that arrived at by stress calculations.

The latest guidelines for fatigue design of highway bridges suggest a minimum service life of 75 years but preferably 100 years [Moses et al., 1987]. Such service lives also apply to corrosion design.

Weathering steel corroding at a rate higher than 7.5 μm/year (0.3 mil/year) cannot be expected to develop a normal, protective oxide coating. When such corrosion rates are anticipated, weathering steel should not be used in bare condition.

Appearance. Properly performing weathering steel bridges viewed from a distance provide a dark, visually uniform appearance well accepted by the public. Under close examination the color and texture of the protective oxide are seen to vary from structure to structure and from member to member in one structure. The appearance is a function of exposure to the elements of weather, contaminants, microenvironment, and runoff patterns of the structure. Generally, members boldly exposed to the weather show a darker, more uniform appearance and a smoother texture than sheltered members.

Staining. The water runoff from weathering steel structures will produce rust stains on rough or porous materials such as unsealed concrete. This staining will continue for the life of the weathering steel structure. (Chapter Six contains recommended design details to prevent staining.) In addition, weathering steel bridge members will shed small rust flakes that look like coarse ground pepper. These flakes can accumulate to block inadequately sized drainage holes or form a moisture retaining poultice that can lead to corrosion.

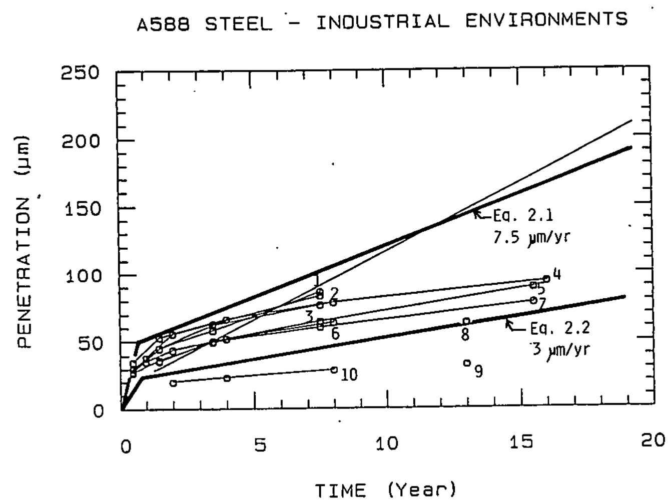

Figure 10. Comparison of expected performance of weathering steel bridges with corrosion penetration of A588 steel in industrial environments. See also Table 3.

2.4.2 Unsatisfactory Performance

Direct losses due to unsatisfactory corrosion performance of bridges include the cost of any required remedial action, which may range from painting, portions of or the entire structure, to replacing excessively corroded members. The remedial painting of weathering steel bridges is more expensive and time consuming than the repainting of ordinary steel bridges because the tenacious nature of the oxide that forms on weathering steel makes surface preparation by abrasive blast cleaning more difficult. As with any bridge repair, the repair of weathering steel bridges may require extensive traffic control or diversion, which leads to associated costs and safety problems.

When rust scaling gives weathering steel bridges an unsatisfactory appearance, although excessive corrosion losses warranting remedial action may not have occurred, the public may still display a lack of confidence in the safety of the bridge.

Service experience with existing weathering steel bridges [AISI 1982; Albrecht and Naeemi, 1984] and other weathering steel structures has shown that almost every case of unsatisfactory performance is due to one or more of a limited number of causes. These causes and the range of unsatisfactory performance to which they lead are discussed below.

Marine Environment. Aggressive marine environments may provide sufficient salt contamination and moisture to prevent the development of the protective oxide on weathering steel. The consequences are unacceptably high corrosion rates and continual shedding of rust films.

Humid Environment. Such environments may provide a long time-of-wetness that may lead to periodic shedding of rush films, thus preventing the formation of a protective oxide. An unsightly appearance and higher than normal corrosion rates result.

Industrial Contaminants. These contaminants can adversely affect the formation of a protective oxide and/or produce unsightly deposits or stains on weathering steel.

Deicing Salts. The widespread use of deicing salt in the snow belt poses a hazard to all constructional materials. The effect of deicing salt on weathering steel depends in part on the amount of salt reaching the exposed steel, producing in some cases a corrosion rate slightly higher than normal and a negligible change in appearance; and in other cases, a significant increase in corrosion rate and an unsightly appearance. The magnitude of the effect is a complex function not only of salt deposition rate but also of the microenvironment and design features of the bridge. However, observations show that a combination of high deicing salt use, high speed traffic, and a low clearance above the roadway will very likely produce significantly accelerated corrosion of the overhead weathering steel bridge. Geometries of a grade separation that produce tunnel-like conditions have been specifically cited as a cause of nonadherent flaky rust [AISI 1982]. Salt can also be deposited on the members of a bridge from salt-containing spray thrown up from the deck above the members.

Leaking Expansion Joints. Roadway water leaking through expansion joints can be a significant problem for weathering steel bridges. Factors that increase the hazard are deicing salts and traps for water such as crevices or horizontal surfaces where water can collect. Expansion joints made of weathering steel, such as pin-and-link connections and rocker-and-plate supports, can severely corrode in a few years to the extent that they will not longer freely permit movement.

Mill Scale Pitting. A weathering steel member partially covered by mill scale presents conditions for galvanic corrosion, wherein the scale-free steel surface serves as the anode and the mill-scaled surface as the cathode. This produces pitting of the exposed steel at the juncture. In the case of a thin or nonadherent mill scale, the pitting undercuts and completely removes the scale after some time. However, if the mill scale is thick and adherent, as is the case for heavy rolled sections that take long to cool and thus build up a thicker scale, pitting can continue to a depth much greater than the anticipated loss of section due to atmospheric corrosion. Such pits can serve as initiation sites for fatigue cracks. Thus, it is advisable to use heavy sections covered with mill scale only in atmospheres of very low or low corrosivity (Table 6).

Poultices. Poultices of granular or fibrous material that hold water on weathering steel structures can lead to conditions resembling immersion and corrosion rates much higher than in atmospheric corrosion. A common cause of poultice formation is the collection, in some design details, of the granular flakes of rust that weathering steel continually sheds. If a poultice remains dry, there is no hazard to the steel. Thus, bird nests present no problem to a weathering steel bridge if they are in sheltered locations that do not periodically become wet.

Crevices. Crevices between weathering steel members that can collect water also may show very high corrosion rates typical of immersion exposure, which are unacceptable for most bridge designs. However, tight crevices between stiff members, such as are produced in high-strength bolted joints of bridge members, become sealed with rust and further corrosion is negligible. Where weathering steel and an adjacent metal such as bronze or stainless steel form a crevice that is periodically wet, galvanic corrosion will cause high thickness losses to weathering steel [Culp and Tinklenberg, 1980].

Designers of weathering steel structures should be aware that crevices between weathering steel members that lack sufficient stiffness can allow corrosion to occur to the extent that the pressure of the continually formed rust, called "packout", will distort the members. As a result, the crevice will not seal itself and corrosion will continue indefinitely. Joints on the main structure of bridges are not likely to distort because of the code requirements for member thickness and bolt spacing. But they could distort in an ancillary structure, such as a guardrail, if the design is not adequate [McCrum and Arnold, 1980; Brockenbrough and Gallagher, 1985].

Contamination During Shipping or Storage. Deicing salt from roadway spray or salt from other sources can contaminate weathering steel members during storage or shipment. This contamination can lead to higher than anticipated corrosion rates and an unsightly appearance of the steel after erection.

CHAPTER THREE

SITE ANALYSIS

3.1 APPROACH

The approach that must be followed to determine whether bare, exposed weathering steel is suitable for use in a bridge at a specific site consists of three major steps: (1) evaluate the corrosivity of the macroenvironment; (2) evaluate the corrosivity of the microenvironment; and (3) verify the suitability by measurement of corrosion resistance and levels of contaminants. If short time-of-wetness and low contamination levels are found to create favorable conditions for the formation of the protective oxide coating, weathering steel may be considered for use at the bridge site.

The term "macroenvironment" refers to the regional or local environment of a city or town, or that within a radius of a few miles or even a few hundred meters of a bridge site. The term "microenvironment" refers to the environment at the bridge site. It can also be used to characterize the exposure conditions at some part of the bridge, such as close to an abutment, beneath an expansion joint, or at a structural detail.

3.2 CORROSIVITY OF THE MACROENVIRONMENT

The factors that play a critical role in the corrosivity of the macroenvironment are climate, atmospheric pollutants, and airborne chlorides originating from a body of salt water.

3.2.1 Climate

Valuable information on climate can be obtained from two publications issued by the National Oceanic and Atmospheric

Administration, which are available at the local library or the Weather Bureau. The annual publication entitled "Local Climatological Data: Annual Summary With Comparative Data" contains a narrative climate summary describing the seasons, temperature, precipitation, relative humidity, and influence of mountains and rivers. It has a page devoted to monthly records of temperature, rainfall, and snowfall; and a page devoted to the meteorological data for the current year, including the monthly average relative humidity at 0100, 0700, 1300, and 1900 hours.

The "Local Climatological Data Monthly" provides daily measurements of several parameters at 3-hour intervals beginning at 0100 hours. Of significance are the measurements of air temperature, wet bulb temperature, dewpoint, relative humidity, and wind speed.

Climatological data for rural and agricultural areas can be obtained by referring to the reports for the nearest city where there is an airport and a branch of the National Weather Service.

From these climatological reports one can deduce the time-of-wetness, that is, the time during which an invisible film of electrolyte forms on the surface of the steel causing a sharp increase in the atmospheric corrosion rate. Studies show that the time-of-wetness of panels exposed under standard conditions was equivalent to the cumulative time during which the relative humidity exceeded 84 percent in clean air, 70 percent in air with 0.01 percent sulfur dioxide, or 55 percent when the panel had previously corroded in a 3 percent sodium chloride solution. The corrosivity of the macroenvironment increases with increasing time-of-wetness.

3.2.2 Atmospheric Pollutants

The atmospheric contaminants most important to the corrosion of weathering steel are sulfur oxides that enter a macroenvironment of the region by way of the prevailing winds. Such data are available from the regional offices of the Environmental Protection Agency. While steel corrodes more as the sulfur oxide pollution increases (Fig. 8), the level of these pollutants in domestic macroenvironments is not high enough to be of concern with regard to the corrosion of weathering steel.

3.2.3 Atmospheric Sea Salts

The level of atmospheric sea salt and the corrosivity of coastal ares vary greatly with location, distance from the sea, direction of prevailing winds, climate, topography, and time. Table 7 shows the large variations in chloride levels in rainwater samples that were measured in coastal areas [Junge and Gustafson, 1957]. In another example, despite a chloride level that was tenfold lower in rain samples gathered on the Louisiana Gulf Coast than those gathered on the Texas Gulf Coast, the Doullut Canal Bridge in Empire, Louisiana, was corroding more than the High Island Bridge near Port Arthur, Texas. The weathering steel did not develop a protective oxide coating in either bridge.

This exemplifies the difficulty of categorizing marine atmospheres. Therefore, corrosion penetration data from one coastal area should not be applied to another coastal area without careful verification. To ensure that the environment at a site near the coast is safe for weathering steel, the level of atmospheric sea salt should be measured or steel panels should be exposed for a minimum of 2 years.

Table 7. Nationwide chloride levels in coastal areas and interior. [Junge et al., 1957]

Coast	States	Chloride (mg/l)
West Coast	Southern California	2.0
	Northern California	0.6
	Washington State	2.0
	Puget Sound	22.6
East Coast	Maine	8.9
	Cape Code	5.9
	Carolina Coast	6.9
	South Florida	2.4
Gulf Coast	Louisiana-Mississippi	0.4 - 0.8
	Texas	4.0 - 8.0
Interior	Middle Atlantic States	0.23 - 0.40
	Middle West States	0.012
	Mountain States	0.09 - 0.16

3.3 CORROSIVITY OF THE MICROENVIRONMENT

Climatological factors of the macroenvironment, cited in section 3.2, can interact with the local factors of the microenvironment in the vicinity of the potential bridge site to influence the corrosion behavior of weathering steel. Among the important factors to be considered in an evaluation of the corrosivity of the microenvironment are the performance of existing steel structures, site topography, local industrial plants, and deicing salt use. A careful site inspection is basic to generating the necessary information. The site should be inspected when the climatic conditions are most adverse.

3.3.1 Performance of Existing Steel Structures

During the site visit the condition of various types of metal installations should be carefully inspected and their maintenance history determined. For example, the performance of pole line hardware in utility applications, painted ordinary steel light standards, sign poles, gasoline station appurtenances, farm fencing, newspaper mail boxes, and a variety of other metal items such as air conditioners, store fronts, pickup trucks, automobiles, and so forth, can supply clues about the corrosivity of the microenvironment. Such an inspection should be made for a distance of several miles around the site. Residents, building contractors, and hardware suppliers should be consulted for their views concerning the effects of the weather.

It is particularly helpful to know how a weathering steel structure would perform in that or a similar environment. Often major steel suppliers can identify nearby weathering steel structures which the engineer can then inspect as part of the site visit.

3.3.2 Topography

The site visit must identify features of the local topography that might cause the weathering steel to remain wet for extended periods of time, without intermittent drying, and thus prevent the formation of the protective oxide coating. Free air circulation is needed for the steel to dry properly.

Conditions that must be avoided are unfavorable orientation, low clearance over bodies of water, surrounding dense vegetation, and other obstacles that shelter the site, all of which tend to curtail free air flow and limit access to the sun. Under such circumstances the site may have to be cleared and the bridge height over a body of water increased so as to provide greater air circulation below the deck.

3.3.3 Local Industrial Plants

Weathering steel bridges should not be built near industrial plants emitting dust, gases, or vapors that may react with the weathering steel under certain conditions of temperature and relative humidity. The area near the site should be inspected for possible sources of atmospheric pollution.

3.3.4 Deicing Salts

Finally, it is necessary to learn the local practices of snow removal and deicing salt use. The best source of such information is the local authority.

Additional information on the use of sodium chloride, calcium chloride, abrasives, and deicing salt solutions for snow and ice control across the United States is available from the Salt Institute in Alexandria, Virginia. The data include information on the quantity of salt used by state, county, city, toll road authority, and other highway agencies.

According to the Salt Institute's past experience, the type and frequency of snow, sleet, freezing rain, frost, and ice changes greatly from place to place. Also, varying weather conditions, equipment type and number, and familiarity of the local public with hazardous driving conditions influence spreading policies. Snow depth, number of storms, and length of highway system do not correlate with the amount of deicing materials spread on highways.

Therefore, engineers should rely more on information from the local authority than on statistical data when assessing the likelihood of the bridge being contaminated with deicing salts. Such information is needed for selection of bridge type, span continuity, deck joint configuration, clearance, and the like.

3.4 Testing

Because several years are needed to plan and design highways and bridges, there is sufficient time to evaluate the corrosivity of the environment at the selected bridge site and determine whether the weathering steel will perform as anticipated. The most helpful tests are those that measure the corrosion penetration of the steel, sample the atmosphere for the presence of salt and sulfur oxides, and monitor the time-of-wetness of the steel. These test methods and practices now are standardized by the American Society for Testing and Materials.

3.4.1 Corrosion Penetration

The ASTM standard specification G1 for "Preparing, Cleaning, and Evaluating Corrosion Test Specimens" gives procedures for preparing metal specimens for exposure tests, removing corrosion products after the test has been completed, and evaluating the corrosion damage that has occurred. Emphasis is placed on procedures for evaluating corrosion by mass loss and pitting measurements. The average corrosion penetration per side is calculated from the mass lost during the time of exposure.

The test specimens must be fabricated from the same type of steel as the bridge. They must be exposed for a minimum of 2 years, preferably 3 years, before it can be determined whether the weathering steel will develop a protective oxide coating. In marine environments it is important to expose the corrosion test specimens at an elevation equal to the height of the bridge.

3.4.2 Atmospheric Salinity

ASTM Committee G1 on Corrosion of Metals is currently standardizing a practice for using a "wet candle" to determine the degree of atmospheric salinity. The wet candle device consists of a narrow-mouth flask filled with distilled water and closed with a rubber stopper. A general purpose test tube is inserted through a hole in the stopper so that the lip (top opening) of the tube is at the bottom of the stopper. A roll of bandage gauze is wrapped over all exposed areas on the protruding end of the test tube, and the two ends of the gauze are threaded into the flask through open channels on opposite sides of the stopper. The two ends of the gauze are dipped in the liquid and serve as a wick.

The wet candle is placed in a support stand and exposed for a predetermined length of time. At the end of the exposure time, the chloride ion content of the gauze is measured. Dividing this quantity by the exposed area of the gauze and the exposure time gives the chloride ion concentration per square meter per day. Monthly wet candle tests should be done over a 24-month period in conjunction with the exposure tests.

The chloride ion levels present in the air can provide useful data that help to identify the corrosivity of ocean coastal areas, inland lakes and marshes with high salt levels, industrial plants, and heavily salted highways. The test method provides only semiquantitative values, and experience is needed to determine from the results of a wet candle test whether the measured atmospheric salinity will prevent the formation of a protective oxide coating.

3.4.3 Atmospheric Sulfur Dioxide

ASTM Standard Practice G91 for "Monitoring Atmospheric SO_2 Using the Sulfation Plate Technique" describes a practice for measuring the amount of sulfur dioxide captured by a sulfation plate exposed for 30 days. The results are reported in terms of milligrams of sulfur dioxide per square meter per day. The results of this practice correlate approximately with volumetric sulfur dioxide concentrations, that is, deposition rate (mg $SO_2/m^2/day = 0.8 \times$ concentration ($\mu g\ SO_2/m^3$) [Gullman 1985].

The amount of sulfur compounds present in the atmosphere can also be measured in accordance with the ASTM Standard Method D2010 for "Evaluation of Total Sulfation Activity in the Atmosphere by the Lead Dioxide Candle." Lead dioxide candles have been used to estimate the potential for sulfuric acid damage on structures such as bridges and buildings.

Permissible ambient levels of sulfur dioxide have been established by law.

3.4.4 Time-of-Wetness

ASTM Standard Practice G84 for "Measurement of Time-of-Wetness on Surfaces Exposed to Wetting Condition as in Atmospheric Corrosion Testing" describes a technique for detecting and recording surface moisture conditions. The deposited moisture serves as an electrolyte that generates a potential in a moisture sensing element (galvanic cell) mounted on the surface of the steel specimen. The recorded output from this cell gives the time that moisture is present on the sensing element. Experience has shown that the sensing element and the surface on which it is mounted react alike to factors that cause wetness.

The time-of-wetness gage measures the total time during which moisture is deposited on the surface by atmospheric or climatic phenomena such as direct precipitation of rain or snow, condensation, deliquescence (or, at least, the hygroscopic nature) of corrosion products or salt deposits on the surface. A direct measure of atmospheric or climatic factors responsible for moisture deposition without a time-of-wetness gage does not necessarily give an accurate indication of time-of-wetness.

Long-term studies have shown that the time-of-wetness experienced annually by panels exposed under standard conditions is equivalent to the cumulative time the RH is above a given threshold value [Guttman 1968]. This time value varies with location and with other factors. Probability curves for top and bottom surfaces of a standard panel at one location give the probable times that a surface will be wet as a percentage of the cumulative time the relative humidity is at specific levels [Sereda et al., 1982]. While there are no data to show what length of time-of-wetness is acceptable for bare, exposed weathering steel, the practice is, nevertheless, useful for determining the corrosivity of an environment.

CHAPTER FOUR

WEATHERING STEEL

4.1 TYPES OF WEATHERING STEEL

Weathering steel is supplied to the requirements of ASTM specifications A242, A588, A709, and A852. These specifications were first issued in 1941, 1968, 1974, and 1986, respectively, and have undergone a number of changes over the years.

A242 Type 1 steel is used only for architectural applications because the high phosphorus content impairs the weldability and toughness required of bridge steels. The low-phosphorus version, A242 Type 2, corresponds to the A588 and A709 Grade 50W steels. It is suited for bridge applications. *A588 steel* is available in nine proprietary grades. Each grade is a variation of the same basic chemical composition which contains copper, chromium, nickel, and silicon for enhanced atmospheric corrosion resistance. *A709* is a general specification for bridge steels. The two grades with enhanced atmospheric corrosion resistance, 50W and 100W, correspond to A588 steel and a modified A514 alloy steel. *A852* is a new quenched and tempered weathering steel with 485 MPa (70 ksi) minimum yield strength and a generic chemical composition.

This chapter presents the scope and requirements for weathering steel and summarizes the changes that were made in the specifications since their first adoption. The current requirements are needed for the design of new bridges. Knowledge of the changes made over the years helps to assess the performance of the older bridges.

4.2 ASTM A242 STEEL

The ASTM A242 standard specification for "High-Strength Low-Alloy Structural Steel" corresponds to the AASHTO Standard No. M161. Since its first publication in 1941, 19 editions of the A242 specifications have been issued. The current edition is dated 1985.

4.2.1 Scope

The 1941 edition covered "Low Alloy Structural Steel" suitable for welding and riveting, and was intended primarily for use as main stress-carrying material of structural members. The material was limited to thickness not under 5 mm ($\frac{3}{16}$ in.) and not over 50 mm (2 in.). 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 was changed to "High-Strength Low-Alloy Structural Steel." Steel shapes, plates, and bars were included in thicknesses up to 100 mm (4 in.).

In 1960, bolted construction was added to the scope. The atmospheric corrosion resistance was quantified as being equal to or greater than that of structural carbon steel 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 the type of steel furnished.

In 1968, the year when the A588 specification was first adopted, several changes were concurrently made in the A242 specification. The atmospheric corrosion resistance was specified as being at least two times that of structural carbon steel with copper, which was said to be equivalent to four times structural carbon steel without copper (max. 0.02 percent Cu). The statement attributed to the A242 steels a higher corrosion resistance than that of the A588 steels, which were specified to have atmospheric corrosion resistance approximately (not at least) four times that of structural carbon steel without copper. When required, the manufacturer had to supply evidence of corrosion resistance satisfactory to the purchaser.

Recognizing the fundamental importance of welding technique, the A242 specification presupposed that a welding procedure suitable for the grade of steel and intended use or service would be utilized.

In 1975, the atmospheric corrosion resistance was reduced from "at least" to "approximately" four times that of structural carbon steel without copper, as is also stated in the A588 and A709 specifications. The scope was last changed in 1985 when the SI units were added.

4.2.2 Chemical Requirements

The first edition, A242-41, specified the maximum alloying content of carbon, manganese, and sulfur given in Table 8. 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. The manufacturer had to analyze each melt and report the percentages of carbon, manganese, phosphorus, sulfur, and any other alloying elements present.

The 1942 edition intended that the selection of the alloying elements should 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 raised to 1.40 percent (Table 8).

In 1960, the manufacturer was required to identify the type of steel by determining and reporting the content of all alloying elements found by a ladle analysis. The purchaser had to indicate and consult with the manufacturer when he wanted higher atmospheric corrosion resistance than that of structural carbon steel with copper.

In 1968, the same year when the requirement for atmospheric corrosion resistance was raised to "at least four times that of carbon structural steel without copper," the chemical requirements were changed as follows: (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 0.50 percent minimum each, the 0.20 percent minimum copper requirement did not apply.

The chemical requirements for Types 1 and 2 are given in Table 8. 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, however, has remained unchanged to date; namely, to permit the manufacturer to select the alloying elements, in combination with those prescribed within limits, that gave the required atmospheric corrosion resistance and mechanical properties.

4.2.3 Tensile Requirements

The first part of Table 9 gives the tensile requirements for A242 steel that were specified in the original 1941 edition for material thicknesses varying from 8 mm (5/16 in.) to 50 mm (2 in.). The basic values of 345-MPa (50 ksi) yield point and 485-MPa (70 ksi) tensile strength at the lowest thickness range are

Table 8. Chemical requirements for A242 steel, 1941–1984.

		Composition, % (Ladle Analysis)				
Designation	Type	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]

Notes:

a. If the chromium and silicon contents are 0.50% min each, then the 0.20% min copper 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%.

the same as the present values. In 1950, the minimum elongation in 200-mm (8 in.) 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 50-mm (2 in.) gage length was added for the thickness range over 40 mm (1½ in.) to 50 mm (2 in.).

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 9. The elongation in 200-mm (8 in.) gage length was lowered to 19 percent for the thickness range 40 to 100 mm (1½ to 4 in.). 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.

In 1970, the elongations for all thicknesses of plates, bars, and all groups of structural shapes were set at a uniform 18 percent in 200-mm (8 in.) gage length and 21 percent in 50-mm (2 in.) gage length. With that last change, all tensile requirements had become identical to those of the A242-81 edition listed in the second part of Table 9.

4.2.4 General Requirements

The current edition of the A242 specification refers the purchaser to the ASTM A36 specification entitled "General Requirements for Rolled Steel Plates, Shapes, Sheet Piling, and Bars for Structural Use," for optional, standardized, and supplementary requirements. Those considered suitable for use with the A242 specification are: S2 Product Analysis; S3 Simulated Post-Welded Heat Treatment of Mechanical Test Coupons; S5 Charpy V-Notch Impact 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 1975 (S2, S3, S5, S6, and S8) and 1979 (S15).

4.3 ASTM A588 STEEL

The ASTM A588/A588M standard specification for "High-Strength Low-Alloy Structural Steel with 345 MPa (50 ksi) Minimum Yield Point to 100 mm (4 in.) Thick" corresponds to the AASHTO Standard No. M222. Since its first publication

Table 9. Tensile requirements for A242 steel.

ASTM Specification	Structural Shapes	Plates & Bars[a] Thickness, mm (in.)	Tensile Strength MPa (ksi)	Yield Point MPa (ksi)	Elongation in 200 mm (8 in.) min, %	Elongation in 50 mm (2 in.) min, %
A242-41T	...	13 ≤ t ≤ 19 (1/2≤t≤3/4)	485 (70)	345 (50)	21	...
	...	19<t≤38 (3/4<t≤1-1/2)	455 (66)	310 (45)	23	...
	...	38<t≤51 (1-1/2<t≤2)	435 (63)	275 (40)	24	...
A242-81 Type 1&2	...	t≤20 (t≤3/4)	480 (70)	345 (50)	18[b,d,e]	...
	...	20<t≤40 (3/4<t≤1-1/2)	460 (67)	315 (46)	18[d,e]	21[e]
	...	40<t≤100 (1-1/2<t≤4)	435 (63)	290 (42)	18[d,e]	21[e]
	Groups 1 & 2	...	480 (70)	345 (50)	18[b]	...
	Group 3	...	460 (67)	315 (46)	18	...
	Groups 4 & 5	...	435 (63)	290 (42)	18	21[c]

Notes:

a. For plates wider than 600 mm (24 in.), the test specimen is taken in the transverse direction.

b. For material under 8 mm (5/16 in.) in thickness or diameter, as represented by the test specimen, a deduction of 1.25 percentage points from the percentage of elongation in 200 mm (8 in.) specified in Table shall be made for each decrease of 0.8 mm (1/32 in.) of the specified thickness or diameter below 8 mm (5/16 in.).

c. For wide flange shapes over 634 kg/m (426 lb/ft) elongation in 50 mm or 2 in. of 18% minimum applies.

d. Elongation not required to be determined for floor plate.

e. For plates wider then 600 mm (24 in.) the elongation requirement is reduced two percentage points.

in 1968, 14 editions of the A588 specification were issued. The current edition is dated 1985.

4.3.1 Scope

The A588 specification covers high-strength low-alloy structural steel shapes, plates, and bars for welded, riveted, or bolted construction, but is intended primarily for use in welded bridges and buildings where savings in weight or added durability are important. The atmospheric corrosion resistance of A588 steel is approximately two times that of structural carbon steel with copper, which is said to be equivalent to four times structural carbon steel without copper (max. 0.02 percent Cu). Welding technique is of fundamental importance. It is assumed that the welding procedure will be suitable for the steel and the intended service. This specification is limited to material up to 200 mm (8 in.) thick. The scope has remained unchanged to date except that SI units were added in 1985.

4.3.2 Chemical Requirements

A588 steel comes in nine grades A to K, excluding G and I, each proprietary to a producer. Table 10 identifies the producers of each grade and the proprietary names of their A588 steels. 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 1984. Bankruptcies and mergers of steel companies have greatly reduced the number of available grades.

Table 11 summaries the chemical requirements specified in the current (1985) edition of the A588 specification. Of the elements listed in the table copper, nickel, chromium, and silicon contribute most to improving atmospheric corrosion resistance, with various combinations of these elements providing comparable improvements. It should be remembered that these elements also influence, in combination with others, the tensile and impact properties of the steel, and its weldability. The producers balance the concentration of the alloying elements to achieve the desired overall behavior.

The results of long-term exposure tests of A588 Grade A steel, reported in 1976, failed to confirm the atmospheric corrosion rating of four relative to carbon steel [Gallagher 1976]. Recent analysis of published corrosion penetration data shows that other grades of A588 and A242 also did not meet the corrosion resistance requirements in a variety of exposure sites (see also section 6.11.1) [Albrecht and Naeemi, 1984].

In 1977, the chemical composition of A588 Grade A was modified to improve its corrosion resistance (Table 12). In 1977, 1978, and 1984 the compositions of A588 grades B, C, D, and E were similarly changed.

4.3.3 Tensile Requirements

Table 13 gives the current tensile requirements for A588 steel. The tensile strength and yield point remained unchanged since 1968 with one exception. In 1974, the tensile strength and yield point of Group 5 structural shapes were increased from 460 and 315 MPa (67 and 46 ksi), respectively, to the uniform values of 485 and 345 MPa (70 and 50 ksi) that apply today to all groups. In addition, the elongation of plates and bars in 200-mm (8 in.) gage length was lowered from the original 19 percent to 18 percent; and the elongation of structural shapes in 50-mm (2 in.) gage length was raised from the original 19 percent to 21 percent.

4.3.4 General Requirements

The bend test requirement in the A588 specification was mandatory until 1974. Thereafter, it became optional at the request of the purchaser. The additional standardized requirements given in ASTM A6 specification for use at the option of the purchaser were adopted in 1977 (S2, S3, S5, S6 and S14) and in 1979 (S15 and S18).

Table 10. Producers and proprietary names of A588 steel.

Grade[b]	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[a]	Armco	High Strength A
H	Kaiser Steel	Kaisalloy 50
J	Jones & Laughlin	Jal-Ten
K	Republic Steel	Dura Plate 50

Note:

a. Grade G was deleted in 1984.

b. Bankruptcies and mergers of steel companies have greatly reduced the number of available grades.

Table 11. Chemical requirements for A588 steel.

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 -1.35	.04 ...	.05 ...	.15 -.50	.50 max	.40 -.70	... ...	.20 -.40	.01 -.10	... ...	... ...	... ...
C	.15 max	.80 -1.35	.04 ...	.05 ...	.15 -.40	.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[a]	.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 ...	.04 ...	.25 -.75	.30 -.60	.10 -.25	.15 max	.20 -.35	.02 -.10	... ...	... ...	.005 -.030
J	0.20 max	.60 -1.00	.04 ...	.05 ...	.30 -.50	.50 -.70	... ...	... ...	.30 min	... ...	... ...	... ...	.030 -.050
K	0.17 max	.50 -1.20	.04 ...	.05 ...	.25 -.50	.40 max	.40 -.70	.10 max	.30 -.50	... ...	... ...	.005 -.05[b]	... ...

Notes:

a. Grade G was deleted in 1984.

b. For plates under 12.7 mm (1/2 in.) in thickness, the minimum columbium required is waived.

Table 12. Changes in chemical requirements for A588 steel, 1968–1984.

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	C	0.10 - 0.20	0.20 max
	1977a	Ni	0.25 - 0.50	0.50 max
	1980a	Mn	0.75 - 1.25	0.75 - 1.35
		Si	0.15 - 0.30	0.15 - 0.50
C	1984	Si	0.15 - 0.30	0.15 - 0.40
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

4.4 ASTM A709 STEEL

The ASTM A709 standard specification for "Structural Steel for Bridges" conveniently allows the purchaser to select from a single document a bridge steel that has the desired tensile properties, impact test properties, and atmospheric corrosion resistance. Since its first publication in 1974, ten editions of the A709 specification have been issued. The current edition is dated 1986.

4.4.1 Scope

The A709 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. Five grades are available in three yield strength levels (250, 345, and 690 MPa (36, 50, and 100 ksi)). Grades 36, 50, and 100 are also included in specifications A36, A572, and A514 respectively.

The grades with suffix "W", 50W and 100W, are also included in specifications A588 and A514. They provide a level of atmospheric corrosion resistance approximately two times that of carbon structural steel with copper, which is said to be equivalent to four times carbon structural steel without copper (max. 0.02 percent Cu). When the purchaser so requires, the manufacturer must supply satisfactory evidence of corrosion resistance.

4.4.2 Chemical Requirements

A709 Grade 50W steel comes in grades A, B, C, and F, which are equivalent to the A588 specification grades A, B, C, and F, respectively. A709 Grade 100W steel comes in grades E, F, P, and Q, which are equivalent to the A514 specification grades E, F, P, and Q. The chemical requirements for Grade 50W and Grade 100W, given in Table 14, were added in 1984 to the A709 specification. Prior to that time limits had been set on the contents of only carbon, manganese, phosphorus, and silicon.

4.4.3 Tensile Requirements

The tensile requirements for Grade 50W and Grade 100W steels, given in Table 15, are the same as those for A588 and A514 steels.

Table 13. Tensile requirements for A588 steel.[a]

Structural Shapes	Plates & Bars Thickness mm (in.)	Tensile Strength min, MPa (ksi)	Yield Point min, MPa (ksi)	Elongation in 200 mm (8 in.) min, %	Elongation in 50 mm (2 in.) min, %
	$t \le 100$ ($t \le 4$)	485 (70)	345 (50)	18[b,c,d]	21[c,d]
	$100 < t \le 125$ ($4 < t \le 5$)	460 (67)	315 (46)	...	21[c,d]
	$125 < t \le 200$ ($5 < t \le 8$)	435 (63)	290 (42)	...	21[c,d]
All groups		485 (70)	345 (50)	18[b]	21[e]

Notes:

a. For plates wider than 600 mm (24 in.), the test specimen is taken in the transverse direction.

b. For metal under 8 mm (5/16 in.) diameter, as represented by the test specimen, a deduction of 1.25 percentage points from the percentage of elongation in 200 mm or 8 in. specified in the Table shall be made for each decrease of 0.8 mm (1/32 in.) of the specified thickness or diameter below 8 mm (5/16 in.).

c. Elongation not required to be determined for floor plate.

d. For plates wider than 600 mm (24 in.), the elongation requirement is reduced two percentage points.

e. For wide flange shapes over 634 kg/m (426 lb/ft) elongation in 50 mm (2 in.) of 18% minimum applies.

Table 14. Chemical requirements for A709 Grade 50W and Grade 100W steels.[a]

Type	C	Mn	P	S	Si	Ni	Cr	Mo	Cu	V	B	Ti
	Composition, %											
						Grade 50W Steels						
A	0.19 max	0.80-1.25	0.04 max	0.05 max	0.30-0.65	0.40 max	0.40-0.65	---	0.25-0.40	0.02-0.10	---	---
B	0.20 max	0.75-1.25	0.04 max	0.05 max	0.15-0.50	0.50 max	0.40-0.70	---	0.20-0.40	0.01-0.10	---	---
C	0.15 max	0.80-1.35	0.04 max	0.05 max	0.15-0.40	0.25-0.50	0.30-0.50	---	0.20-0.50	0.01-0.10	---	---
F	0.10-0.20	0.50-1.00	0.04 max	0.05 max	0.30 max	0.40-1.10	0.30 max	0.10-0.20	0.30-1.00	0.01-0.10	---	---
						Grade 100W Steels						
E	0.12-0.20	0.40-0.70	0.035 max	0.04 max	0.20-0.40	---	1.40-2.00	0.40-0.60	---	(b)	0.001-0.005	0.04-0.10
F	0.10-0.20	0.60-1.00	0.035 max	0.04 max	0.15-0.35	0.70-1.00	0.40-0.65	0.40-0.60	0.15-0.50	0.03-0.08	0.0005 0.006	---
P	0.12-0.21	0.45-0.70	0.035 max	0.04 max	0.20-0.35	1.20-1.50	0.85-1.20	0.45-0.60	---	---	0.001-0.005	---
Q	0.14-0.21	0.95-1.30	0.035 max	0.04 max	0.15-0.35	1.20-1.50	1.00-1.50	0.40-0.60	---	0.03-0.08	---	---

Notes:

a. Weldability data for these types have been qualified for use in bridge construction.

b. May be substituted for part or all of titanium on a one for one basis.

Table 15. Tensile and hardness requirements for A709 Grade 50W and Grade 100W steels.[a]

Grade	Plate Thickness mm (in)	Structural Shapes	Yield Point Strength[b] min, MPa (ksi)	Tensile Strength, MPa (ksi)	Minimum Elongation,[c,d] % in 200 mm (8 in)	Minimum Elongation,[c,d] % in 50 mm 2 (in)	Reduction of Area[e,d] min, (%)	Brinell Hardness Number
50W	t≤102 (4)	Groups 1-5	345 (50)	485 (70) min	18	21[g]	. . .	. . .
100W	t≤64(2-1/2)	. . .	690 (100)[b]	760-895 (110-130)	. . .	18	40-50	235-293[f]
100W	64<t≤102 (2-1/2<t≤4)	. . .	620 (90)[b]	690-895 (100-130)	. . .	16	50	. . .

Notes:

a. For plates wider than 610 mm (24 in.), the test specimen is taken in the transverse direction.

b. Measured at 0.2% offset or 0.5% extension under load.

c. For plates wider than 610 mm (24 in.), the elongation requirement is reduced two percentage points.

d. Elongation and reduction of area not required to be determined for floor plates.

e. For plates wider than 610 mm (24 in.), the reduction of area requirement, where applicable, is reduced five percentage points.

f. Brinell requirements apply to material 9 mm (3/8 in.) and thinner.

g. For wide flange shapes over 634 kg/m (426 lb/ft) elongation in 50 mm (2 in.) of 18% minimum applies.

4.4.4 General Requirements

The purchaser may specify supplementary requirements regarding fine grain practice, Charpy V-Notch impact tests, evidence of atmospheric corrosion resistance, and ultrasonic examination. When such supplementary requirements are specified, the grades of A709 steel exceed the requirements of specifications A36, A572, A588, and A514.

4.4.5 Impact Test Requirements

Table 16 gives Charpy V-notch impact test requirements for Grade 50W and Grade 100W steels. The values of minimum average energy absorbed by the CVN specimen are higher for fracture critical members (suffix "F") than for nonfracture critical members (suffix "T"). Fracture critical members or member components are defined as tension members or tension components of members whose failure would be expected to result in collapse of the bridge [AASHTO 1978].

The minimum average energy values are the same for all three temperature zones, in which the lowest ambient service temperatures are: −18°C (0°F) for Zone 1; below −18°C to 34°C (0 to −30°F) for Zone 2; and below −34°C to −51°C (−30°F to −60°F) for Zone 3. However, as the lowest ambient service temperature drops by zone, the minimum average energy must be achieved at increasingly lower temperatures provided in the footnotes of Table 16. The test temperatures for quenched and tempered Grade 100W steel are lower than those for high-strength low-alloy Grade 50W steel.

The impact test requirements for Grade 50W and Grade 100W steels are the same as those for the Grade 50 and Grade 100 steels respectively, which do not have enhanced atmospheric corrosion resistance.

Table 16. CVN impact test requirements for A709 Grade 50W and Grade 100W steels.

Grade	Plate Frequency Testing	CVN-Impact Test Temperature	Thickness, in. (mm) and Joining Method	Minimum Average Energy ft-lbf (J)
			Non-Fracture Critical Members	
50WT	a	d,e	to 4 (102) incl., mechanically fastened	15 (20)
			to 2 (51) incl., welded	15 (20)
			over 2 to 4 (51 to 102) incl., welded	20 (27)
100WT	b	f	to 4 (102) incl., mechanically fastened	25 (34)
			to 2-1/2 (64) incl., welded	25 (34)
			over 2-1/2 to 4 (64 to 102) incl., welded	35 (48)
			Fracture Critical Members	
50WF	c	d,e	to 4 (102) incl., mechanically fastened	25 (34)
			to 2 (51) incl., welded	25 (34)
			over 2 to 4 (51 to 102) incl., welded	30 (41)
100WF	c	f	to 4 (102) incl., mechanically fastened	35 (48)
			to 2-1/2 (64) incl., welded	35 (48)
			over 2-1/2 to 4 (64 to 102) incl., welded	45 (61)[g]

Notes:

a. The CVN-impact testing shall be "H" heat frequency testing except for material greater than 1-1/2" which shall be "P" plate frequency testing in accordance with Specification A673.

b. The CVN-impact testing shall be "P" plate frequency testing in accordance with Specification A673.

c. The CVN-impact testing shall be "P" plate frequency testing in accordance with Specification A673 except tests shall be on each end of each plate. For material greater than 1-1/2" in thickness, the required test temperature shall be reduced by 20°F (7°C).

d. The CVN-impact test temperature for Grade 50W steels shall be: 70°F (21°C) for Zone 1; 40°F (4°C) for Zone 2; and 10°F (-12°C) for Zone 3.

e. If the yield point of the Grade 50W steel exceeds 65 ksi (450 MPa), the CVN-impact test temperature shall be reduced by 15°F (8°C) for each increment of 10 ksi (70 MPa) above 65 ksi (450 MPa). The yield point is the value given on the certified "Mill Test Report".

f. The CVN-impact test temperature for the Grade 100W steels shall be: 30°F (-1°C) for Zone 1; 0°F (-18°C) for Zone 2; and -30°F (-34°C) for Zone 3.

g. Grade 100WF steel in thickness of 2-1/2 to 4 in. (64 to 102 mm) incl. may not be used for welded fracture critical members in Zone 3.

4.5 ASTM A852 STEEL

The ASTM A852/A852M standard specifications for "Quenched and Tempered Low-Alloy Structural Steel Plate with 70 ksi (485 MPa) Minimum Yield Strength to 4 in. (100 mm) Thick" were approved by ASTM in December 1985 and adopted by AASHTO in 1986.

Corrosion resistance data for this steel are not yet readily available but, based on the chemical composition requirements, the steel's corrosion resistance should be comparable to that of A588 and A709 Grade W steels.

4.5.1 Scope

The A852 specification covers quenched and tempered high-strength low-alloy structural steel plates for welded, riveted, or bolted construction. It is intended primarily for use in welded bridges and buildings where savings in weight, added durability, and good notch toughness are important. The atmospheric corrosion resistance of A852 steel is approximately two times that of carbon structural steel with copper, which is said to be equivalent to four times carbon structural steel without copper (max. 0.02 percent Cu). Welding technique is of fundamental importance, and it is presupposed that the welding procedure will be suitable for the steel and the intended service. This specification is limited to material up to 100 mm (4 in.) thick.

4.5.2 Chemical Requirements

A852 steel comes in one generic chemical composition given in Table 17. The contents of the alloying elements copper, nickel, chromium, and silicon, which give the steel atmospheric corrosion resistance, are similar to those of the A588 grades A, B, and C steels.

4.5.3 Tensile Requirements

Table 18 lists the tensile requirements for A852 steel. The 485-MPa (70 ksi) yield strength permits greater savings in weight than would be possible with A588 steel.

4.5.4 Impact Test Requirements

The results of longitudinal Charpy V-notch impact tests must meet an average minimum value of 27 J (20 ft-lbf) at 4°C (40°F). By agreement, a lower test temperature, greater energy level, or both, may be specified.

Table 17. Chemical requirements for A852 steel.

Composition, %								
C, max	Mn	P, max	S, max	Si	Ni, max	Cr	Cu	Va
0.19	0.80-1.35	0.04	0.05	0.20-0.65	0.50	0.40-0.70	0.25-0.40	0.02-0.10

Table 18. Tensile requirements for A852 steel.[a]

Tensile Strength, min, MPa (ksi)	Yield[b] Strength, min, MPa (ksi)	Elongation in 50mm (2 in.) min, %[c,d]
620-760 (90-100)	485 (70)	19

Notes:

a. For plates wider than 610 mm (24 in.), the test specimen is taken in the transverse direction. See 11.2 of Specification A6/A6M.

b. Measured at 0.2% offset or 0.5% extension underload.

c. For thicknesses of 19 mm (3/4 in.) and under, measured on 40 mm (1-1/2 in.) wide full thickness rectangular specimen as shown in Fig. 4 of Methods and Definitions A370, the elongation is measured in a 50-mm (2 in.) gage length that includes the fracture and shows the greatest elongation.

d. For plates wider than 610 mm (24 in.), the elongation requirement is reduced two percentage points.

ECONOMIC ANALYSIS

5.1 INTRODUCTION

The purpose of the analysis presented in this chapter is to provide a better understanding of the economic factors that influence the choice of type of steel for plate girder construction. The options examined are: painted A36, painted A572, bare A588, remedially painted A588, and periodically hosed A588 steel.

The analysis is performed for a simple-span bridge, a two-span continuous bridge, and a three-span continuous bridge. The bridges were designed with two optimum design programs for plate girders, one from Bethlehem Steel [Bethlehem Steel Corporation 1986] and the other from the University of Maryland [Schelling et al., 1986]. In these programs the initial cost, rather than weight, was optimized. Neither program calculates the life cycle cost.

The calculation of life cycle cost that was added in this study assumes an interest rate of 10 percent and an inflation rate of 6 percent. When the interest rate is higher than the inflation rate it is more advantageous, from a financial viewpoint, to minimize the initial investment and defer cost to later years.

5.2 UNIT COSTS

Both bridge analysis programs include the cost of fabricating the girders. The Bethlehem Steel program includes the initial painting cost of A36 steel, but not the cost of blast cleaning new A588 steel. The University of Maryland program does not provide the cost of painting or blast cleaning the steels, and these had to be calculated separately using the unit costs given in Table 19.

The $3.35/ft^2 unit cost of remedially blast cleaning and painting corroded weathering steel reflects the Michigan experience with rehabilitation of four bridges in Detroit (Table 38 in Chapter Twelve). This cost is 46 percent higher than the $2.29/ft^2 unit cost of repainting a painted bridge.

Table 19. Unit blast cleaning and painting costs. [Appleman 1985; Tinklenberg 1986a]

	A36 & A572 Steel ($/ft^2)	A588 Steel ($/ft^2)
Initial SP 10 near-white blast	. . .	0.32
Initial blast cleaning and painting SP 10 inorg. zinc/epoxy/urethane	1.15	. . .
Remedial blast cleaning and painting	. . .	3.35
Repainting	2.29	2.29

5.3 BRIDGES

Three bridges were analyzed in this study: a 30.5-m (100 ft) single span, a 30.5-30.5-m (100-100 ft) two-span continuous, and a 30.5-30.5-30.5-m (100-100-100 ft) three-span continuous bridge.

The bridges consisted of four plate girders acting compositely with a 20 cm (8 in.) thick by 10.16 m (33.31 ft) wide concrete deck. The girders were spaced 2.54 m (8.33 ft) apart, and the deck overhanged the facia girders by 1.27 m (4.17 ft).

The bridges were designed for HS-20 truck loading using the load-factor-design criteria of the AASHTO Standard Specifications for Highway Bridges. Load-factor-design provides bridges that are more economical than those provided by working stress design.

5.4 BETHLEHEM STEEL ANALYSIS

5.4.1 Basis of Analysis

Bethlehem Steel designed the three bridges (described in section 5.3) based on the assumption that the girders would be fabricated either from A36 steel painted with an average unspecified coating system or from bare A588 steel. The program provided the weight of the girders, weight index, and initial cost index relative to A588. It did not provide the cost of the girders. The calculated cost indices were based on the cost of fabrication of the steel girders. Other components, such as diaphragms, bearings, concrete deck, were not considered.

The program assumed that the bare A588 girders were not blast cleaned, and no special detailing was needed to improve the corrosion performance. The inclusion of such items would increase the cost of the A588 girders.

5.4.2 Initial Cost

According to the results of the comparative study (Table 20) the painted A36 steel girders initially cost 13 to 17 percent more than the bare A588 steel girders. The life cycle cost could not be calculated because the program did not output all cross sectional dimensions or any costs expressed in dollars. Only initial cost indices were provided.

Table 20. Initial cost of girders. [Bethlehem Steel Corporation 1986]

	Type of Bridge					
	One-Span		Two-Span		Three-Span	
	A36	A588	A36	A588	A36	588
Weight (tons)	40.9	30.6	76.6	63.5	107.4	97.1
Weight index	1.34	1.00	1.21	1.00	1.11	1.00
Initial cost index[a,b]	1.17	1.00	1.16	1.00	1.13	1.00

Notes:

a. Includes cost of initially painting A36 steel.

b. Does not include cost of blast cleaning A588 steel.

5.5 UNIVERSITY OF MARYLAND ANALYSIS

5.5.1 Basis of Analysis

The same three bridges were reanalyzed using the MERLIN program developed at the University of Maryland for optimizing bridge girder cost [Schelling et al., 1986]. The program provides all girder dimensions and the fabrication cost for A36 steel, excluding initial painting, and for bare A588 steel, excluding blast cleaning.

5.5.2 Initial Cost

Based on the output of the MERLIN program and the unit costs given in Table 19, the cost of blast cleaning and painting the girders was calculated (Table 21). Thereafter, the total initial cost and the initial cost index were determined for the three bridges having girders fabricated from painted A36, painted A572, and bare blast-cleaned A588 steel (Table 22).

It was assumed that the cost of fabricating A572 steel girders, including materials and labor, is 6 percent less than the cost of fabricating A588 steel girders. As in the Bethlehem Steel analysis, the cost of other components, such as diaphragms, bearings, and concrete deck, was not considered; nor were allowances made for special details that would improve the corrosion performance of A588 steel girders.

According to the results of the comparative study given in Table 22, the painted A36 steel girders initially cost 10 to 18 percent more than the bare, blast-cleaned A588 steel girders. The initial cost of painted A572 steel girders was 6 to 9 percent higher than for bare A588 steel.

As was found in the Bethlehem Steel study (Table 20), the University of Maryland analysis showed that, on an initial cost basis, bare weathering steel was the least expensive of the three alternatives (Table 22).

5.5.3 Life Cycle Cost

Alternate Systems. The initial cost analysis neglects important factors such as maintenance cost, repaint interval, and time value of money in determining the economics of corrosion protection systems. These factors must be included in the calculation of life cycle cost. This section examines the 60-year life cycle cost of five alternate systems, including estimates for the aforementioned factors. Two alternate systems involve painted steel, while the other three involve weathering steel with different levels of maintenance. The alternate systems are as follows:

Table 21. Cost of blast cleaning and painting girders.

	Type of Bridge								
	One-Span			Two-Span			Three-Span		
	A36	A572	A588	A36	A572	A588	A36	A572	A588
Surface area (ft^2)	6,080	5,540	5,540	11,420	10,340	10,340	16,930	15,450	15,450
Blast Cleaning ($)	. . .	. . .	1,770	. . .	. . .	3,310	. . .	. . .	4,940
Initial Painting ($)	6,990	6,370	. . .	13,130	11,890	. . .	19,470	17,770	. . .
Remedial Painting ($)	. . .	. . .	18,560	. . .	. . .	34,640	. . .	. . .	51,760
Repainting ($)	13,920	12,690	12,690	26,150	23,680	23,680	38,770	35,380	35,380

Table 22. Initial cost of girders.

	Type of Bridge								
	One-Span			Two-Span			Three-Span		
	A36	A572	A588	A36	A572	A588	A36	A572	A588
Weight (tons)	35.8	30.0	30.0	65.8	63.8	63.8	103.3	95.8	95.8
Weight index	1.19	1.00	1.00	1.03	1.00	1.00	1.08	1.00	1.00
Fabrication ($)	29,710	27,500	29,250	62,650	61,510	65,430	96,390	94,060	100,060
Blast Cleaning ($)	. . .	. . .	1,770	. . .	. . .	3,310	. . .	. . .	4,940
Initial painting ($)	6,990	6,370	. . .	13,130	11,890	. . .	19,470	17,770	. . .
Total initial cost ($)	36,700	33,870	31,020	75,780	73,400	68,740	115,860	111,830	105,010
Initial cost index	1.18	1.09	1.00	1.10	1.07	1.00	1.10	1.06	1.00

1. Painted A36 steel with 15-year repainting intervals.
2. Painted A572 steel with 15-year repainting intervals.
3. Bare A588 steel without any maintenance. This alternate is presented as a reference level for purposes of comparison. Experience with weathering steel bridges in service shows that some maintenance is always needed.
4. Bare A588 steel remedially painted after 15 years and repainted every 15 years thereafter. This case represents a bridge located in a severely corrosive environment, which must be protected to prevent excessive section losses.
5. Bare A588 steel washed every year by high pressure hosing. A few states and counties have begun washing their bridges periodically to extend the life of the corrosion protection system. Others in the snow belt states have suggested washing weathering steel bridges in the snow belt states at the end of each winter to remove deicing salts.

Present Cost. The first step in determining the life cycle cost is to calculate the present cost of all anticipated maintenance. The present cost of remedially painting the A588 steel bridges and repainting the A36, A572, and A588 bridges was obtained by multiplying the unit cost of remedially painting and repainting (Table 19) with the surface area of the girders. The results are given in Table 21.

The present cost of periodically washing the single span A588 steel bridge was estimated to be (4 workers) (4 hours) ($50/hr) = $800 per year. The hourly rate was based on a salary of $12.50 per hour plus 300 percent overhead for fringe benefits, materials, and equipment. This estimate was doubled to $1,600 per year for the two-span bridge and tripled to $2,400 per year for the three-span bridge.

The Pennsylvania Department of Transportation for the second time in 3 years had a contractor high-pressure hose the deck, piers, and structural steel, and clean the drains and downspouts of 46 bridges on Interstate 79 during the summer of 1986. The total cost was $371,000, or an average of $2,700 per year and per bridge.

Judging by the cost incurred in Pennsylvania, the actual cost of high-pressure hosing the structural steel on the three bridges being analyzed could be higher than was estimated above.

Present Worth. The second step in determining the life cycle cost is to calculate the present worth of the maintenance expenditures that will be incurred during the service life of the bridge. This is done by escalating the present cost of maintenance to the year of maintenance considering inflation and, then, calculating the present worth of the escalated cost considering the interest rate.

For a single maintenance expenditure after n years of service, the escalated cost is

$$EC = C(1 + e)^n \tag{4}$$

where: C = present cost of maintenance, e = inflation rate = 6 percent, and n = number of years till maintenance is needed. The present worth of the escalated cost is then calculated by inverting the compound interest formula (Eq. 4):

$$PW = \frac{EC}{(1 + i)^n} \tag{5}$$

where i = interest rate including inflation component = 10 percent.

Substituting EC from Eq. 4 gives:

$$PW = C\left[\frac{1 + e}{1 + i}\right]^n \tag{6}$$

For yearly maintenance expenditures the present worth is given by

$$PW = \sum_{n=1}^{N} C\left[\frac{1 + e}{1 + i}\right]^n \tag{7}$$

where N = number of years to last maintenance.

Examples. As an example, the present worth of the second repainting job of the two-span A36 steel bridge after 30 years of service is calculated as follows: Present cost of maintenance (from Table 21): C = $26,150. Escalated cost at 6 percent inflation rate (from Eq. 4): $EC = 26{,}150(1 + 0.06)^{30} =$ $150,170. Present worth at 10 percent interest rate (from Eq. 5): $PW = (150{,}170)/(1 + 0.10)^{30} =$ $8,610. The results are given in Table 23 in the column for "Painted A36, Year 30."

Similarly, the present worth of yearly hosing the two-span, bare A588 steel bridge for 60 years at a present hosing cost of $1,600 per year is (from Eq. 7):

$$PW = \sum_{n=1}^{60} 1{,}600\left[\frac{1 + 0.06}{1 + 0.10}\right]^n = \$37{,}810$$

The result is given in the last column of Table 23.

Life Cycle Cost. The life cycle cost is the sum of the initial cost plus the present worth of all maintenance costs. Table 23 provides calculations of life cycle costs for the two-span bridge. The initial cost (from Table 22) was entered on the 0-year line. The present cost, C, of the maintenance is for: repainting (from Table 21): $26,150 for A36 and $23,680 for A572 and A588 steel; remedially painting (from Table 21): $34,640 for A588 steel; and hose cleaning: $1,600 per year for A588 steel.

Using the present cost of maintenance, the escalated cost, EC, and the present worth, PW, of repainting and remedially painting were calculated with Eqs. 4 and 5. The present worth of hose cleaning was calculated with Eq. 7.

Evaluation. The cost index in Table 23 is the ratio of the life cycle cost of an alternate to the initial cost of bare A588 steel.

Table 24 ranks the five alternates for the one, two, and three-span bridges in order of increasing cost index. The bare maintenance-free A588 steel alternate (initial cost only) would be the most economical, but experience with bridges in service and current manufacturers' literature show that weathering steel is not a maintenance-free material. Of the remaining four alternates, painted A572 is the least expensive. Bare A588 steel would not be an economical choice if the exposure conditions required remedial painting or periodic washing.

By substituting the difference between the life cycle cost of painted A572 steel and the initial cost of bare A588 steel (for example, $PW = 99{,}250 - 68{,}740 =$ $30,510 for the two-span bridge) in Eq. 7 and solving for C, one finds that the life cycle costs of the painted A572 and bare A588 bridges would be equal if the yearly maintenance expenditures for bare A588 steel were $710/year, $1,300/year, and $1,940/year for the one-span, two-span, and three-span bridges. If the yearly maintenance cost could be kept below these expenditures, bare A588 steel with limited maintenance would be more economical than painted A572 steel.

Table 23. Life cycle cost of two-span bridge girders.

Year	Painted A36 Escalated Cost ($)	Painted A36 Present Worth ($)	Painted A572 Escalated Cost ($)	Painted A572 Present Worth ($)	Initial Cost of Bare A588 Escalated Cost ($)	Initial Cost of Bare A588 Present Worth ($)	Remedially Painted A588 Escalated Cost ($)	Remedially Painted A588 Present Worth ($)	Bare A588 Periodically Hosed Escalated Cost ($)	Bare A588 Periodically Hosed Present Worth ($)
0	. . .	75,780	. . .	73,400	. . .	68,740	. . .	68,740	. . .	68,740
15	62,680	15,010	56,760	13,590	. . .	. . .	91,410	21,890		
30	150,170	8,610	136,000	7,790	. . .	. . .	136,000	7,790		37,810[b]
45	359,930	4,940	325,980	4,470	. . .	. . .	325,980	4,470		
60	. . .	. . .	. . .	. . .	. . .	. . .	. . .	. . .		
Life cycle cost ($)[a]		104,340		99,250		68,740		102,890		106,550
Cost index		1.52		1.44		1.00		1.50		1.55

Notes:

a. Calculations are based on 6% inflation rate and 10% interest rate.

b. An annual, present cost of cleaning of $1,600/year for 60 years was assumed.

The foregoing conclusions are valid for the three bridges used in this particular example and the stated assumptions. Designers should perform similar cost analyses to determine the most economical alternate for a specific bridge, following the procedure outlined earlier.

Table 24. Summary of life-cycle cost indices.

Rank	Alternate	Life Cycle Cost Index: One-Span Bridge	Life Cycle Cost Index: Two-Span Bridge	Life Cycle Cost Index: Three-Span Bridge
1	Initial cost of bare A588 steel	1.00	1.00	1.00
2	Painted A572 steel	1.54	1.44	1.43
3	Remedially painted A588 steel	1.59	1.50	1.49
4	Painted A36 steel	1.67	1.52	1.51
5	Bare, periodically hosed A588 steel	1.61	1.55	1.54

CHAPTER SIX

STRUCTURAL DETAILS

6.1 INTRODUCTION

The design of details to enhance corrosion resistance is as important to the long-term performance of weathering steel bridges as are the conditions of exposure, environment, and general layout discussed in Chapters One and Two. Improperly designed details can cause future corrosion problems that require periodic maintenance. The potential savings that can be made by eliminating a future maintenance item greatly exceeds the added effort and cost of good detailing.

The type of details recommended below to reduce the corrosion of weathering steel in bridges will also prolong the service life of paint systems on ordinary steel bridges.

When unanticipated corrosion problems do arise, the required remedial actions are costly and not always effective. Therefore, every effort must be made at the design stage to avoid section geometries and design details that may result in poor corrosion performance. The designer should not count on remedially painting a weathering steel bridge during its service life if the steel does not perform as expected because, as the example in Chapter Five shows, the life cycle cost of a remedially painted

A588 steel bridge may be higher than the life cycle cost of a painted A572 bridge.

The most important considerations in designing a weathering steel bridge are preventing ponding, diverting the flow of runoff water away from the steel superstructure, preventing the accumulation of debris that traps moisture, and avoiding environments in which the bridge would be contaminated with salt. Under conditions of prolonged wetness or significant contamination with salt, weathering steel does not form a protective oxide coating, corrodes at about the same rate as ordinary steel, and offers no advantage in corrosion resistance.

In general, the detailing of members and assemblies should avoid pockets, crevices, recesses, reentrant corners, and locations that can collect and retain water, damp debris, and moisture. Smooth contours enhance self-cleaning and drainage, and facilitate maintenance.

6.2 DECK JOINTS

6.2.1 General Requirements

Leaking joints are the most serious and common cause of corrosion problems in weathering steel bridges. Runoff water leaking through the deck joints persistently wets the bearings, flanges, web, stiffeners, and diaphragms in the vicinity of the joint where it can migrate for long distances along the bottom flange and wick about 150 mm (6 in.) up the web. The resulting excessive corrosion of the superstructure and the freezing of expansion bearings create major bridge maintenance problems.

The superstructure must be allowed to expand and contract with changes in temperature. But these movements should not be accommodated with unnecessary bridge deck expansion joints and expansion bearings, which create corrosion problems when they leak. Instead, bridges should have a continuous superstructure, fixed or integral bearings at piers and abutments, and no bridge deck expansion joints unless absolutely necessary. When expansion joints cannot be avoided, they should be provided only at the abutments.

Tennessee [1982], for example, specifies that the superstructure should be continuous over the piers; have expansion joints at the abutments, only if the abutment is restrained against movement and rotation, and the total movement is greater than 6 mm ($\frac{1}{4}$ in.); and be built integrally with the abutment if the abutment is restrained against movement and the movement is less than 6 mm ($\frac{1}{4}$ in.), or the abutment is not restrained against movement and the movement is less than 50 mm (2 in.).

Weathering steel should only be used for jointless bridges or for bridges that have joints at the abutments alone, unless the joints can be reliably sealed. It should not be used for noncontinuous multispan bridges, cantilever bridges, and suspended span bridges.

6.2.2 Sealed Joints

Michigan has extensively evaluated sealing systems for bridge expansion joints [Bashore et al., 1979, 1980, 1984]. The systems were categorized into three general types: metal-reinforced polychloroprene pad systems, metal-supported and anchored modular polychloroprene compression seals, and metal-supported and anchored polychloroprene or EPDM continuous element extrusions. Joint seals were evaluated based on general appearance and condition, ride and noise qualities, movement, damage, and debris intrusion. Joints were also inspected during wet weather to determine the number of leaks and possible sources. The conclusions of the study are outlined below [Bashore et al., 1979].

The major advantage of the metal-reinforced polychloroprene pad systems was intended to be the ability to replace one or more damaged pads. However, this advantage cannot be gained in practice because the pads cannot be removed without damaging the blockout area as well as the adjacent pads. The systems are very prone to leakage through the joints between pads. Damage by snow removal equipment has been found on all pad-type systems, even when the pads were installed lower than the adjacent roadway surface. Pad systems do not meet the requirement to continuously extend the seal across the full width of the deck without field splices.

The metal-supported and anchored polychloroprene compression seals depend upon compression of the seal against the sidewall of the metal support to prevent leakage rather than upon a system of locking the seal into the support. In Michigan's experience, the system is incapable of providing a watertight joint. Snow removal equipment seriously damages the seals by frequently tearing the polychloroprene extrusion or removing it from the metal side channels, causing the system to leak. Compression seals are not recommended for use in deck joints of weathering steel bridges.

The remaining systems, classified as metal-supported and anchored polychloroprene or EPDM continuous element extrusions, incorporate an elastomeric strip as a continuous sealing element across both the expansion opening and the full deck width. Such systems effectively prevent leakage, are not susceptible to damage by snow removal equipment, provide a good and quiet riding surface, and experience no serious problems.

Bashore et al. [1979] rate the performance of various bridge deck joint sealing systems being used in Michigan's bridges. Bashore et al. [1980, 1984] rate the allowable movement of proprietary bridge deck expansion joint devices at various skew angles. Designers are advised to consult these performance data when searching for leak-proof sealed joints.

Recommendations for all systems include the following:

- The system must extend the full width of the deck, including any medians, sidewalks, and bridge rail areas. The single sealing element must be continuous across the expansion opening and along the full bridge width to effectively prevent leakage. If the walk area is covered with a sliding plate, the system should be placed beneath the plate. The ends of the system should be turned up to prevent water from dripping off the end [Bashore et al., 1979].
- The FHWA, in its Notice N5140.11 dated 27 October 1977, had recommended that the expansion dam sealing elements be continuous and extend the full width of the deck, thus preventing leakage of field joints of expansion dam segments.
- Field splices of sealing elements must not be permitted because they inevitably develop leaks.
- The seal element should be mechanically anchored to the metal supports in a way that maintains a compressive force upon the element other than that generated by simply inserting the element into a cavity in the metal support [Bashore et al., 1979].

- The installation should be carefully inspected to ensure that all materials and workmanship comply with the specifications.
- Seals should be regularly inspected and maintained. When leaks are found, the seal should be promptly repaired.
- The steel superstructure on either side of a sealed joint should be painted over a length equal to two beam depths or 3.0 m (10 ft), whichever is greater. This includes the girder ends, diaphragms, and lateral bracing within the specified distance. In addition painted deflector (drip) plates should be welded to the bottom flange at the end of each painted section. This preventive measure is recommended because the water tightness of sealed joints cannot be guaranteed, and the shortage of maintenance funds may not always permit immediate repair of the seal. The disadvantages of this measure are that (a) the paint itself becomes a maintenance item, and (b) the water runs more easily over the painted surfaces to the adjacent bare weathering steel surfaces.
- Ontario [1983] found that in a number of sites where joints were leaking badly, there was evidence of the benefits of good ventilation. Wide separations between the beams and the abutment wall lessened the severity of corrosion. The Ontario Highway Bridge Design Code now requires a 200 mm (8 in.) gap below deck joints in the following cases (Fig. 11): between the deck and the abutment wall in concrete slab bridges, between the ends of beams and the abutment wall in beam and slab bridges, and between the beam or deck slab ends in the case of deck joints over piers or at the ends of suspended spans. Such gaps should also be provided in weathering steel bridges.

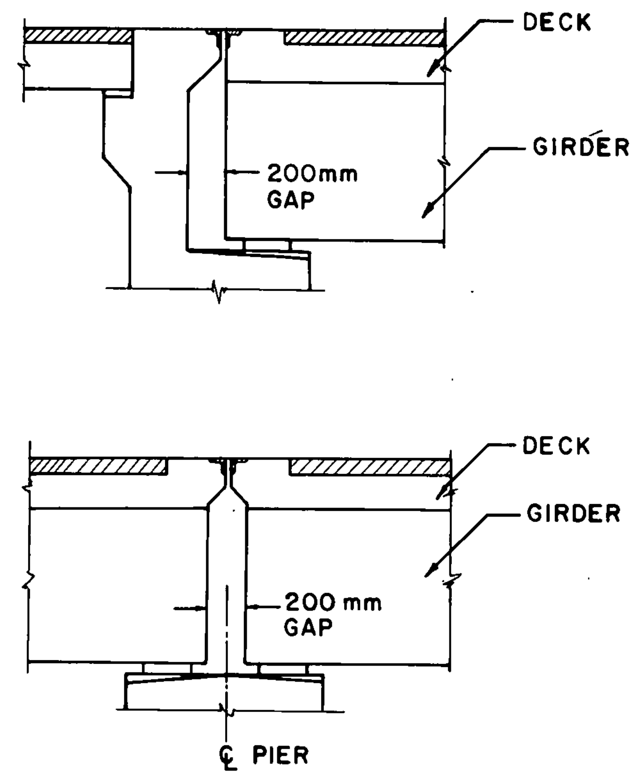

Figure 11. Gaps at abutment and pier. [From Ontario Ministry of Transportation and Communication, 1983]

6.2.3 Open Joints

To prevent leakage through open deck joints, the runoff water in some bridges is collected in troughs placed below open joints and discharged through drain pipes away from the bridge. This solution is only partly effective for two reasons. First, lacking periodic maintenance, accumulated road debris tends to clog the trough. Second, heavy water spray splashed into the joints by fast moving vehicles bypasses the troughs and runs down the steel structure.

The following recommendations should be considered with respect to open joints;

- Open expansion joints should not be used in weathering steel bridges except where unavoidable.
- In long-span bridges, when open joints cannot be avoided, a system of troughs and drain pipes must be installed below the joint to collect the runoff water and discharge it away from the bridge. Such systems must be maintained.
- Structural steel on either side of an open joint must be painted over a length of two beam depths or 3.0 m (10 ft), whichever is greater, and painted drip plates should be welded to the bottom flange at the end of each painted section.
- A minimum gap of 200 mm (8 in.) below open joints must be provided between a beam end and the abutment wall in beam-and-slab bridges, and between beam ends when the joint is located over the piers.

6.2.4 Jointless Bridges

Minimizing the number of joints in bridges reduces water leaks and lowers long-term maintenance and construction costs. The number of joints can be reduced to two, one at each abutment, by making the steel girders and deck continuous and jointless over the piers.

When soil conditions permit, joints should be eliminated altogether by making the deck continuous over the piers and integrating the girder ends with the abutment. In this case the abutments are built on flexible piles that bend as the steel girders expand and contract with changes in temperature (Fig. 12). While various states have set different limits on the overall length of bridges with integral abutments, steel bridges up to about 90 m (300 ft) long appear to perform satisfactorily. Table 25 summarizes the number of states in which integral abutment bridges were built to specific lengths.

The following measures are recommended:

- When movement due to changes in temperature can be accommodated, weather steel bridges should be built with continuous girders over the piers and jointless deck.
- When movement and soil conditions permit, the ends of the continuous girders should be built integrally with the abutments, thus eliminating all deck joints.
- The girder ends must be painted over a length equal to the embedment length plus 300 mm (1 ft) away from the abutment. The crevice between the embedded steel and the concrete must be sealed by caulking.

6.3 LINK PLATE AND PIN CONNECTIONS

Water leaking through open deck joints runs over the link plate-and-pin connection at cantilever expansion joints of steel

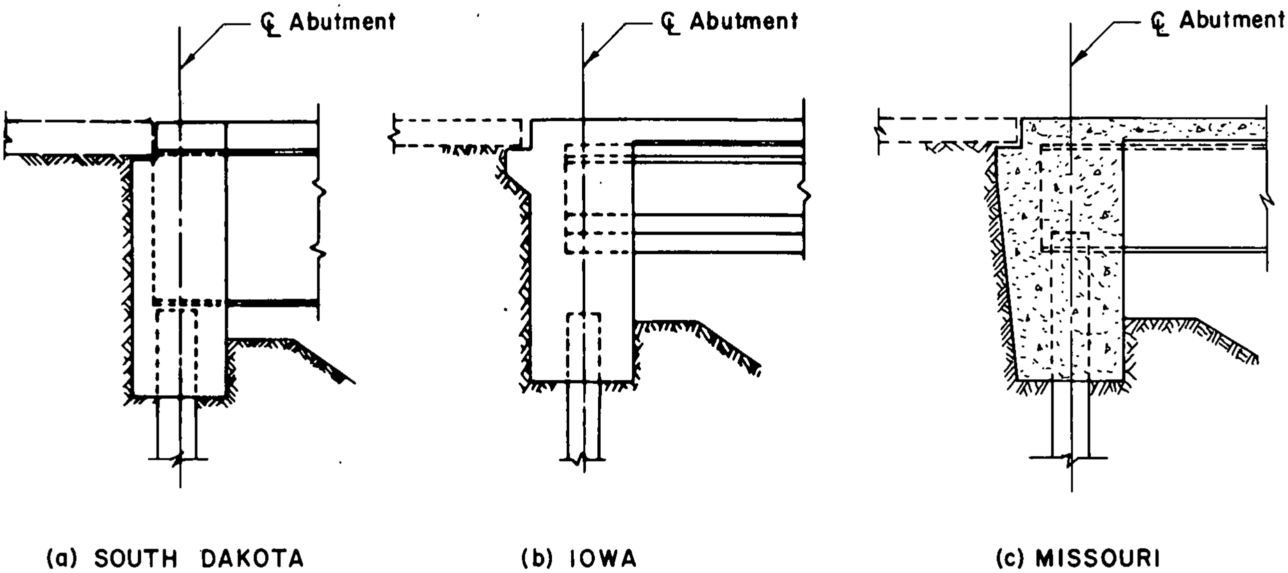

Figure 12. Integral abutments on flexible foundations. [Wolde-Tinsae et al., 1987]

girders. As a result the weathering steel is severely attacked by (1) crevice corrosion in the gap between the girder web and the link plates; and (2) galvanic corrosion of the steel girder with respect to the bronze washer installed between the girder web and the link plates. The crevices eventually become tightly packed with rust. The expansion joint then freezes, and the thermal expansion and contraction of the girders damage the fixed bearings and abutments (Fig. F-18 in Appen. F).

- Link plate-and-pin connections of girders at open deck joints must not be used in bare, exposed weathering steel bridges. They are susceptible to crevice corrosion and cannot be maintained.
- Severely corroded link plate-and-pin connections in existing weathering steel bridges should be rehabilitated as recommended in Chapter Twelve and Appendix B.

Table 25. Maximum length of integral abutment bridges as of 1981. [Wolde-Tinsae et al., 1987]

	Number of States		
Maximum Length	Steel	Concrete	Prestressed
800	. . .	1	1
500	. . .	1	2
450	. . .	1	3
400	2	3	4
350	1	3	1
300	8	8	8
250	2	1	. . .
200	5	1	2
150	1	. . .	. . .
100	. . .	1	. . .

6.4 BOX AND TUBULAR MEMBERS

6.4.1 Internal Corrosion

Box and tubular members for welded girders, columns, and trusses fabricated from any steel corrode internally if water or industrial pollutants can enter.

Water can enter box or tubular members by capillary action, inspiration, or condensation. Very narrow openings in partially closed or imperfectly sealed members permit water entry by capillary action. For example, capillary water can enter through a narrow bolt hole gap but not through a hole deliberately cut into the member, unless the hole diameter is extremely small.

Water can also enter by inspiration when the volume of air is rapidly reduced as the ambient temperature drops. The resulting partial vacuum sucks in water laying stagnant in the vicinity of a crevice. However, a hole so placed that water flowing under gravity cannot enter the member actually prevents the inspiration of water by equalizing pressure as the interior air cools.

A third possible way of water entry is by condensation. Observation of this phenomenon shows that, with normal changes in temperature between day and night, the amount of water vapor condensation is usually small because the difference in temperature between the member and the contained air is not great. The following sections examine various types of box and tubular members, and recommend ways of enhancing corrosion performance [Blodgett 1967; Manning 1984a; Manning et al., 1984b].

6.4.2 Sealed Box Members

When all the oxygen and water in a closed system are consumed, corrosion stops. As long as a hollow steel member remains hermetically sealed, the amount of rust that can be formed is slight enough to be almost imperceptible, irrespective of the relative humidity of the air at the time the welds were sealed. It is impossible to seal enough atmospheric oxygen into a hollow member to cause corrosion damage. The same applies to other corrosive agents that might be present in the atmosphere at the time a girder is weld-sealed.

Thus, any concern about corrosion damage in a hermetically sealed section is unwarranted. In such cases, the interior surfaces of the hollow member need no protective coating. It should be recognized, however, that to completely weld-seal boxes requires special care in fabrication and some means of testing the air tightness.

6.4.3 Nonsealed Box Members

Condensation within hollow members that have openings is very slight. When the temperature of the outside air is higher than that of the air in the box, the dew point cannot be reached at the interface. However, a sharp drop in outside temperature can chill the metal enough to cause internal condensation.

Whenever possible, box and tubular members should be constructed tight, if not airtight. Preventing ventilation does not lead to significant condensation. If tight construction is not feasible, a sufficient number of hatches, vents, or openings should be provided to create a draft. The inside surfaces should be painted and an extra heavy coating should be applied around the openings. Drain pipes should not be routed through box members.

6.4.4 Tubular Members

Tubular members are used for sign supports, pedestrian bridges, and railings. All tubes are considered to be closed if their extremities are welded onto plates, flanges, or other tubes. In this case the inside of the tubes are not susceptible to corrosion damage and need not be painted.

Partially closed tubes are those that have not been welded, but are flattened at the ends and drilled for bolts. The ends do not close perfectly and water can enter because the exterior painting does not seal interstitial spaces. Because internal corrosion may occur in service, the interior surfaces of partially closed tubes must be painted.

Open tubes have either open ends or they are provided with a hole or slit at one end to prevent the inspiration of water. The interior of open tubes must be painted.

6.4.5 Box Girders

Box girders cannot be sealed against moisture entry. Therefore, the box girder must be adequately drained and ventilated to reduce the potential for moisture entrapment and accelerated corrosion. Furthermore, if the box girder is inaccessible for inspection and maintenance, the interior surfaces must be painted. Accessible box girders may be left bare, but the inside must be periodically inspected for evidence of corrosion. Considering the many variables that influence the formation of a protective oxide, the designer should establish a schedule for inspection. Should excessive corrosion become apparent, the steel must be cleaned and painted.

Because of its smooth contour, the exterior of a box girder is less prone to corrosion than an I-girder on which debris, moisture, and salt spray accumulate more easily.

Implementing the following recommendations will enhance the corrosion performance of box girders:

- Water proof and pave the concrete deck to prevent runoff water from leaking through cracks in the deck and into the box girder.
- Be aware that radiant cooling of the deck during the night can cause humidity to condense on the steel members in contact with the deck.
- Close the end diaphragms to keep out vermin and moisture.
- Ventilate the box girder by drilling small holes in the bottom flange at the lowest point of each compartment. Insert a short tube in each hole. The top of the tube should be flush with the upper surface of the bottom flange and protrude below the flange so that any water that entered the box can easily drain and drip without clinging to the underside of the flange. Fit the tube with an insect screen.
- Locate the inspection hatches for easy access by inspection crews, but not by unauthorized persons. A suitable place is directly above the shoulders in grade separation structures.
- Extend the web plates about 20 mm (¾ in.) below the bottom flange to ensure that water running down the web drips off and does not continue to run on the underside of the flange (Fig. 13).

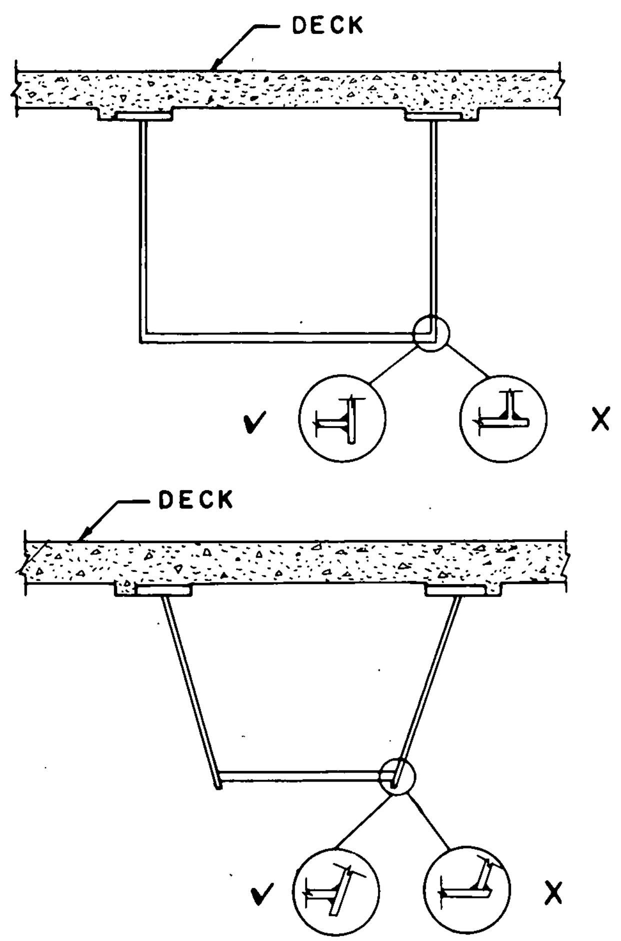

Figure 13. Web-to-flange weld detail for box girders.

- If a water line passes through a box girder, openings in the floor must be provided to drain the water should the line leak or break. The pipe system should be insulated to avoid condensation.
- Provide solid diaphragms.

6.5 I-GIRDER BRIDGE MEMBERS

6.5.1 Introduction

Water ponds and debris accumulate on horizontal surfaces and in corners formed by horizontal and vertical plates (reentrant corners), fostering excessive corrosion. In I-girder bridge members the most susceptible locations are bottom flanges, gusset plates for horizontal bracing, longitudinal stiffeners, bolted splices of horizontal and sloped members, and intersections of bearing and intermediate stiffeners with flanges and gusset plates. To avoid water ponding and debris accumulation, it is important to: (1) minimize the number of horizontal surfaces on which water can pond and debris can accumulate; (2) minimize the number of reentrant corners that prevent drainage and entrap debris and windblown dust; (3) design details for self-cleaning and easy discharge of water; and (4) avoid crevices.

Possible ways of implementing these measures are discussed in sections 6.5.2 through 6.5.8. The Ontario Ministry of Transportation and Communications, for example, has a policy of using weathering steel box girders instead of I-girders in grade separation structures because of the possible accumulation of salt spray on the I-girders, especially in areas of heavy salting [Manning 1984a].

6.5.2 Girders

- Slope girders longitudinally. If the alignment of the highway permits, the girders should be sloped downwards from midspan toward both supports.
- In welded flange splices, provide a transition in flange thickness rather than width. Experience has shown that uneven water flow at transitions in width cause nonuniform appearance and corrosion.
- In bolted flange splices, provide a 20-mm (0.8 in.) gap between the flange ends. Separate the splice plates on the underside of the bottom flange to break the water flow (Fig. 14).
- Note: Japanese engineers have tried to enhance the self-cleaning of welded plate girders by transversely sloping the bottom flange 5 percent away from the web [Yamada 1983]. This also requires machining the top surface of sole plates and cutting the ends of fitted or welded stiffeners to match the transverse slope of the bottom (Fig. 15). It would be impractical to transversely slope a rolled girder flange.

6.5.3 Floor Beams and Stringers

- Short floor beams and stringers need not be sloped. If these members are in the path of runoff water leaking through the joints, they must be painted or thermally sprayed and sealed.

6.5.4 Stiffeners

- Cut off intermediate stiffeners that are not fitted or welded to the bottom flange at least 30 mm (1¼ in.) above the bottom flange (Fig. 16(b)).
- Cope intermediate stiffeners and bearing stiffeners that are fitted or welded to the bottom flange with a 50 mm (2 in.) clearance (Fig. 16(a)).
- Note: Japanese engineers eliminated reentrant corners in one bridge by enclosing the space at the bearing stiffeners of plate girders with sloped, welded plates (Fig. 17). Sealed access holes were provided for maintenance, and the interior of the space was painted [Yamada 1983].

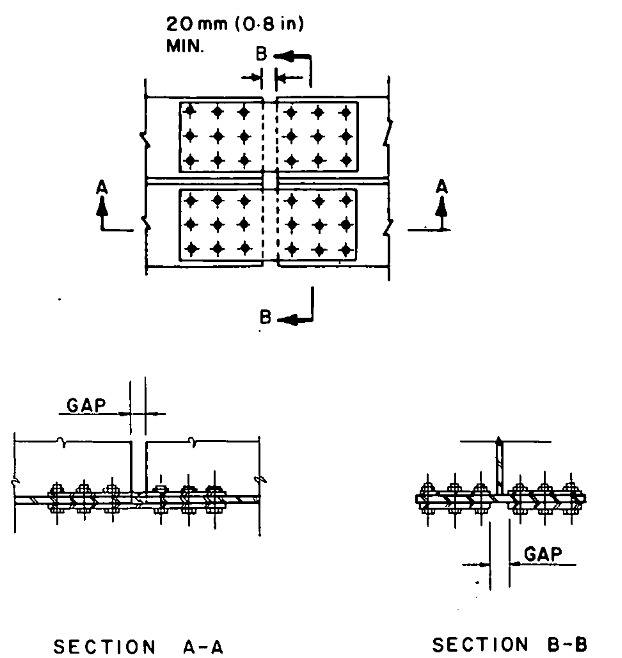

Figure 14. Bolted flange splice detail.

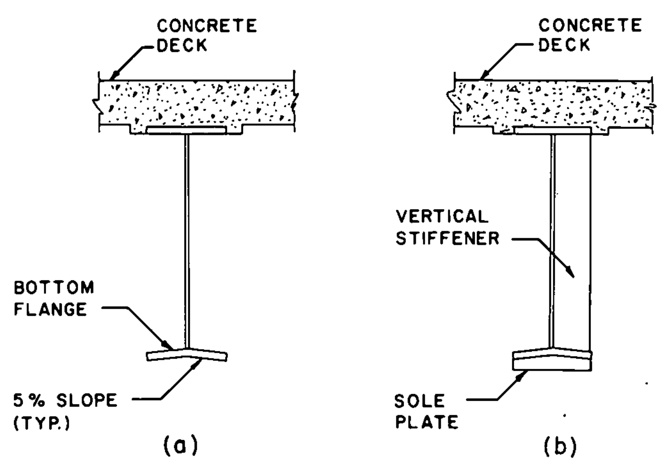

Figure 15. Transversely sloped bottom flange of plate girder.

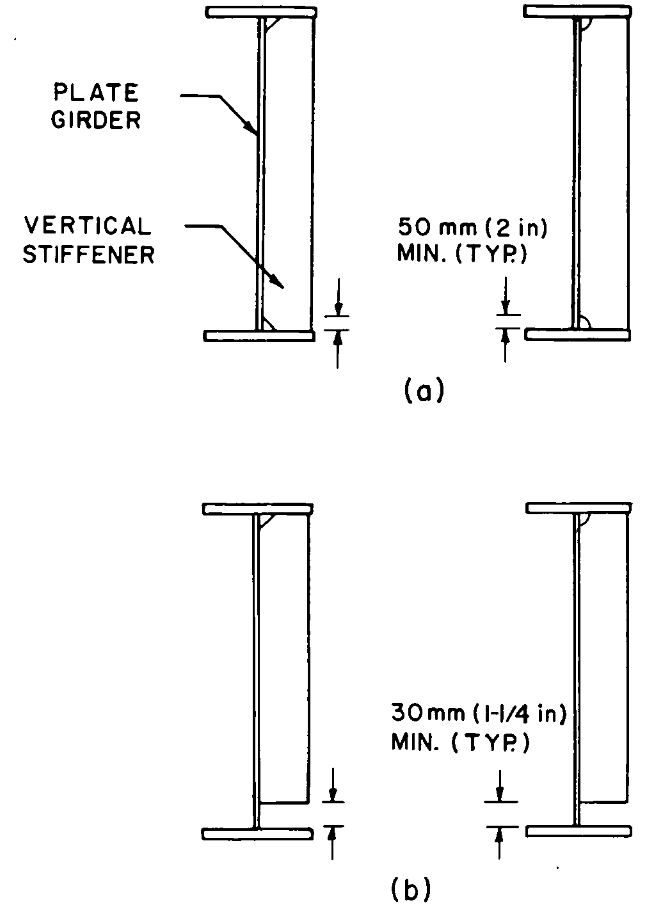

Figure 16. Stiffener details: (a) bearing or intermediate stiffeners fitted or welded to bottom flange; (b) intermediate stiffeners not fitted or welded to bottom flange.

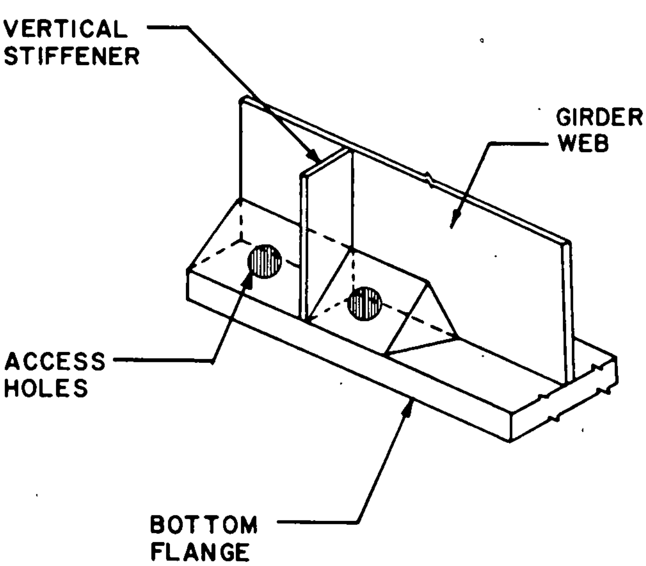

Figure 17. Closed box detail at bearing stiffener. [Yamada 1983]

6.5.5 Lateral Bracing and Diaphragms

- Use cross-bracing angles in the inverted position rather than the open-faced position (Fig. 18).
- K-bracing is preferable to X-bracing because the members do not intersect and crevices are thus avoided (Fig. 18).
- Connections for cross-bracing members may be welded all around or bolted, with bolt spacing and edge distance meeting the requirements outlined in Chapter Eight.
- Use Tee or angle sections for lateral bracing members in the inverted position and connect them to the underside of the gusset plate (Fig. 19).
- Cut a large opening in the gusset plate for the transverse stiffener to pass through (Fig. 19).
- Clip the gusset plate corner normal to the member center line to minimize the distance between the end bolts and the gusset plate edge (Fig. 19).

6.5.6 Utility Supports

- Install angle brackets in the open-faced position (Fig. 20). The horizontal leg of an inverted angle bracket under load rotates away from the connecting member, thus opening a crevice along the heel of the angle.

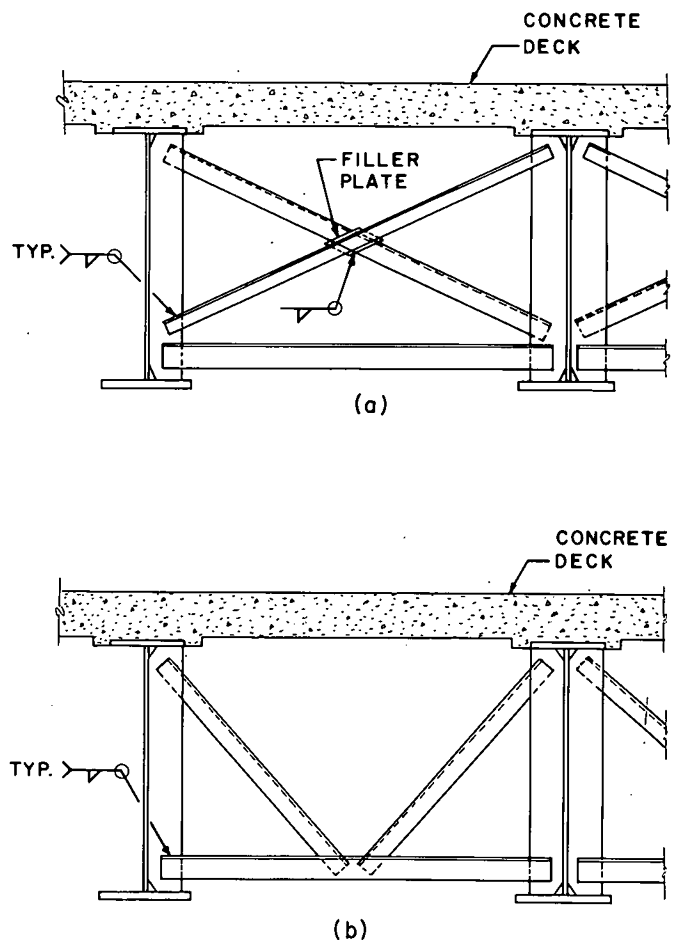

Figure 18. Cross bracing details: (a) X-bracing; (b) K-bracing.

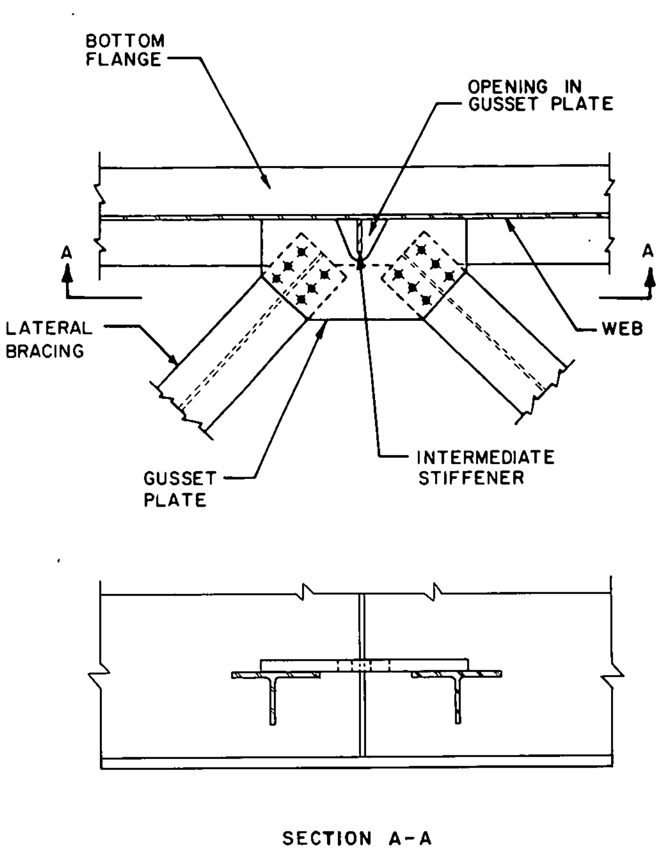

Figure 19. Lateral bracing detail.

6.5.7 Bearings

- In bridges whose ends are at different elevations, place the fixed support at the high end and the movable support at the low end. The fixed joints are less likely to leak and, therefore, the flow of runoff water along the girders is minimized.
- Provide ample space for better air circulation between the underside of the girder and the top of the abutment or pier cap.

6.5.8 Crevices

- Crevices must be avoided. When crevices are unavoidable the contact surfaces of the members forming the crevice must be painted and the edges of the crevices must be caulked to ensure proper sealing and to provide adequate protection against corrosion of the steel in the crevice.

6.6 TRUSS BRIDGE MEMBERS

6.6.1 Introduction

Pockets should be avoided and debris accumulation and water ponding on horizontal truss bridge members should be prevented. The recommendations outlined in section 6.5 for I-girder bridge members should be implemented for truss bridge members wherever applicable. In addition, the recommendations listed in the following sections 6.6.2 and 6.6.3 should be followed to ensure the formation of the protective oxide coating.

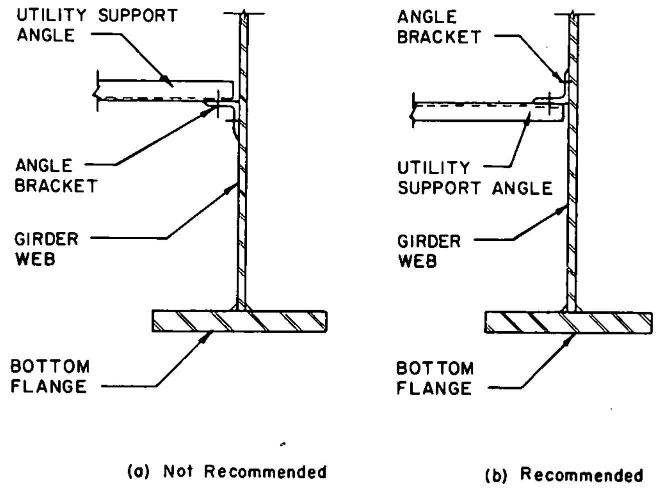

Figure 20. Utility support detail.

6.6.2 Open Sections

- Longitudinally slope trough-like truss chords that consist of a wide-flange section oriented so that the flanges are vertical and the web is horizontal. Drill an adequate number of 50-mm (2 in.) diameter drainage holes through the web.
- Connect the flanges of the diagonals and vertical members to the flanges of the chords, but do not connect the webs. This avoids pockets and prevents debris accumulation and water ponding.
- Arrange lateral bracing members in the inverted position and connect them to the underside of gusset plates (Fig. 19).

6.6.3 Closed Box Sections

- Slope the main trusses 5 percent out of plumb (Fig. 21).
- Seal the box members at both ends and provide drainage holes at the lowest point of the main truss diagonals (Fig. 22).
- Extend the top flange of the upper chord beyond the webs (Fig. 23(a)). Extend the webs of both chords below the underside of the bottom flange (Fig. 23(b)).

6.7 GUARDRAILS

Several factors play a part in the increased corrosion rate observed in lapped guardrail joints. Moisture is more likely to be retained in the joint area, creating conditions similar to submerging the weathering steel in water.

Furthermore, while the guardrail splice can initially have a fairly loose area of contact, the corrosion products building up within the splice eventually create the tight contact necessary for crevice corrosion. The corrosion products deposit outward from areas of first contact until eventually the whole joint forms

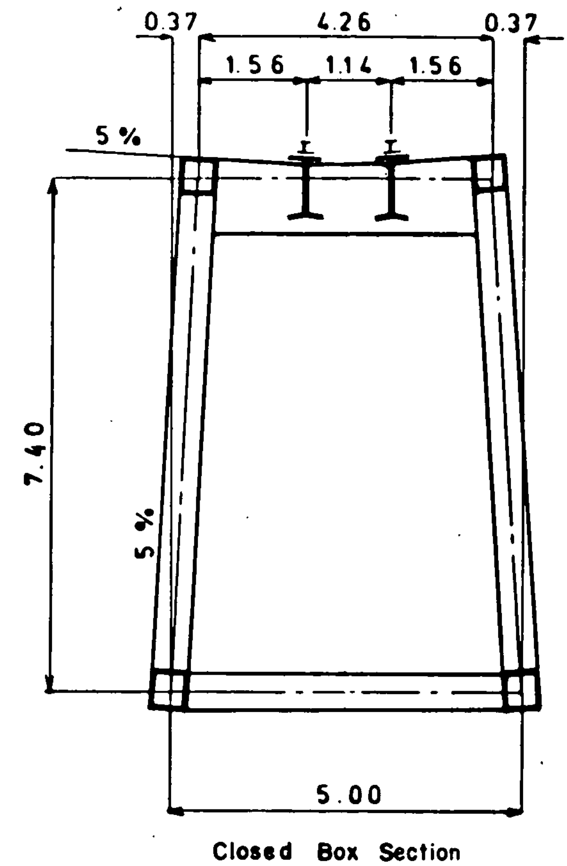

Figure 21. Cross section of railway truss bridge with box members built in Japan. [Yamada 1983]

a tight crevice. The crevices continuously build up salt concentration, soak up moisture by capillary wicking, and corrode at a much faster rate than the freely exposed faces of the guardrail that are rain-washed and sun-dried.

Guardrails may be built from weathering steel. But the contact surfaces at the lapped joints must be protected against crevice corrosion by painting or thermal spraying.

6.8 Drainage

- Water on the approach roadway must be intercepted by catch basins or other suitable means to prevent its flow onto the bridge deck.
- Downspouts for deck drains must be located such that runoff water is discharged away from any part of the bridge, assuming that the water spreads from the outlet at a 45-deg. angle from the vertical. Downspouts must extend below the adjacent members.
- Bridge decks should be drained with downpipes only when no acceptable alternative is available. Downpipes must be of rigid corrosion-resistant material, have an internal width or diameter of at least 200 mm (8 in.), and be at a slope of at least 8 percent. Elbows must have an angle not more than 45 deg, and changes of direction must have a radius of not less than 450 mm (18 in.). Cleanouts must be provided in sufficient numbers and at convenient locations to permit access to all parts of the downpipe system.

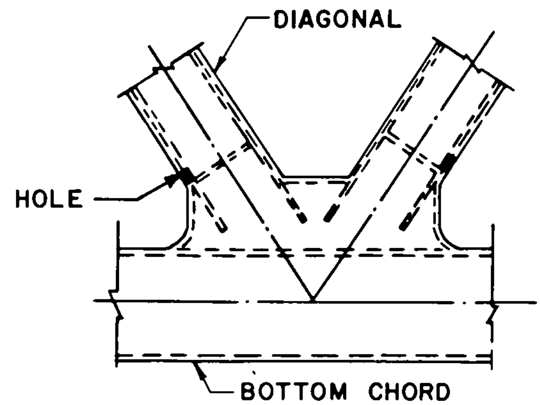

Figure 22. Bottom chord joint of main truss. [Yamada 1983]

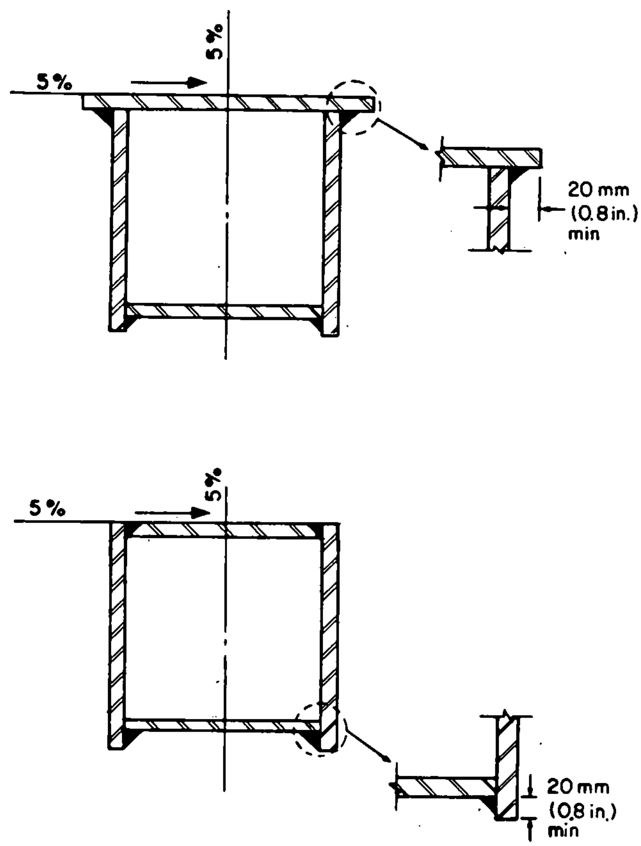

Figure 23. Cross section: (a) top chord and (b) bottom chord—Ohkawa Railroad Bridge. [Yamada 1983]

- Drain pipes must not be routed through box members.

6.9 ABUTMENTS AND PIERS

Runoff water carrying suspended particles of iron oxide released by weathering steel will stain concrete surfaces if allowed to drain over abutments and piers, particularly during the early years of exposure. For aesthetic reasons, every effort should be made to minimize unsightly rust staining of concrete supports visible to the public.

The best way to minimize staining is to prevent runoff water from draining over the weathering steel and onto the concrete. Continuous jointless decks and integral abutments (section 6.2.4) are most effective in preventing such contact. When joints cannot be avoided, they should be sealed and maintained properly.

The following features have been incorporated in weathering steel bridges, with mixed success, to help divert runoff water that might stain the concrete:

- Trough and down spout systems below open joints. The system eventually clogs and water overflows if no maintenance is provided.
- Stainless steel or fiberglass drip pans placed under bearings and cantilevered out from the pier. The pans may break and they are difficult to replace. At high piers the wind blows water dripping from the edge of the pan against the surface of the pier.
- Sloping abutment cap towards retaining wall and draining water into a dry well behind the wall. The drain pipes may eventually clog (Fig. 24).
- Drip plates welded to the bottom flange at a short distance from piers. Drip plates welded to the top of the bottom flange in the tension region of a plate girder must be cut short of the web-to-flange weld to prevent a triaxial state of tensile stress at intersecting welds (Fig. 25). Drip plates welded to the top and bottom of the bottom flange divert water flow but may not prevent migration of moisture past drip plate by capillary action and through breaks in discontinuous dam (Fig. 26).
- Parapet wall around the top of the abutment or pier. The water must be discharged through a downpipe embedded in the concrete or the overflow must be channeled along a V-groove down the surface of the abutment (Fig. 27). The parapet wall retains the debris.

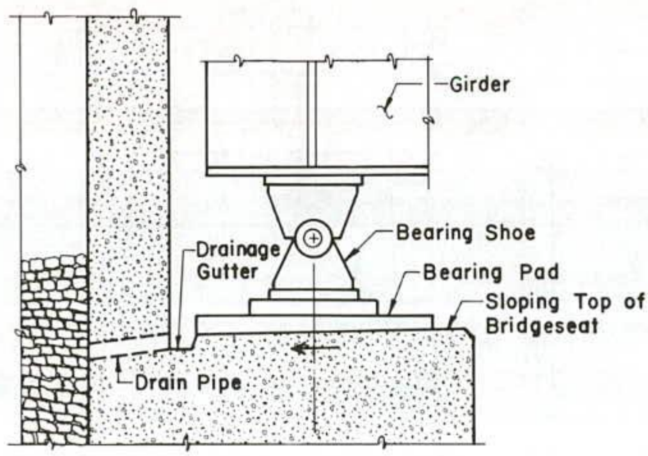

Figure 24. Sloped abutment cap and drain. [Bethlehem Steel Corporation 1983]

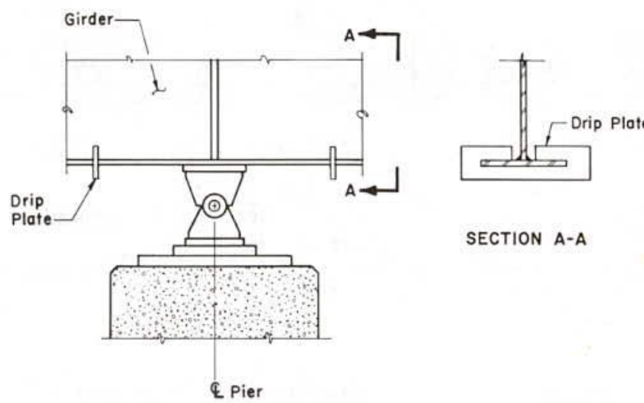

Figure 25. Drip plate attached to bottom flange.

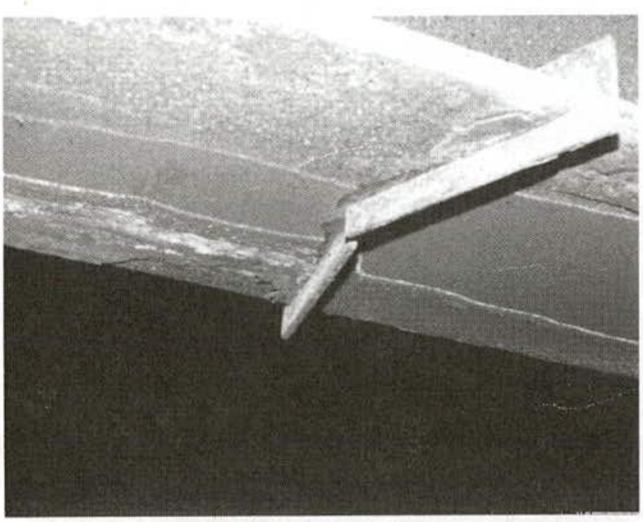

Figure 26. Water migration past drip plate by capillary action and through breaks in dicontinuous dam. [Michigan Department of Transportation]

To prevent water ponding and debris accumulation around weathering steel bearings, the abutment and pier caps must be sloped.

6.10 VERTICAL CLEARANCE

Combinations of high speed traffic, low clearance, and heavy use of deicing salt on the lower roadway pose a severe corrosion hazard as the spray kicked up by passing trucks settles on the overhead grade separation structure. The spray plume is about twice as high as the truck.

- Weathering steel bridges for grade separation structures must have adequate vertical clearance to keep most spray from reaching the bridge. The British specifications, for example, recommend a minimum of 7.5 m (24 ft).
- The pavement beneath a grade separation structure must be pitched to drain melted snow and rain water to curb side drainage.

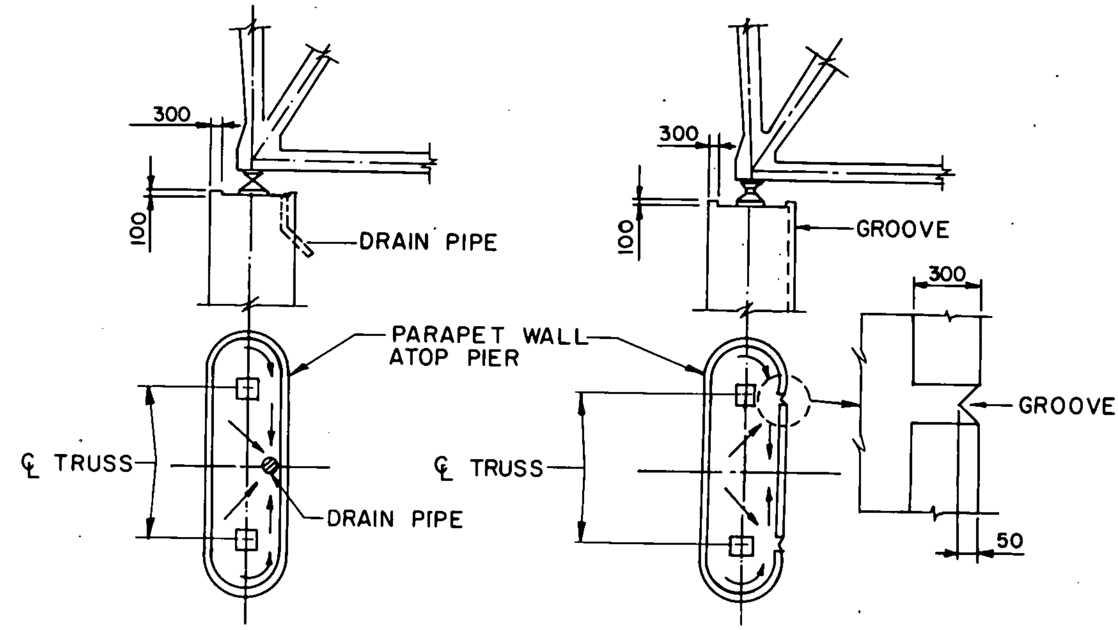

Figure 27. Parapet wall on top of pier with sloped cap and drainage—Ohkawa Railroad Bridge. [Yamada 1983]

- If there is no curb-side drainage beneath a grade separation structure, the snow should be removed.
- It is equally important to provide adequate vertical clearance over bodies of water so that vapor emanating from the water surface does not condense on the weathering steel. The required clearance depends on the degree of air circulation and the topography at the site.

6.11 CORROSION LOSSES

6.11.1 Definition of Corrosion Resistance

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

The ASTM requirement is based on analysis of pre-1968 data for which the corrosion resistance was calculated as the ratio of the corrosion rate of the reference steel (C or Cu) to that of weathering steel, over an increment of exposure time. The slopes of the shaded triangles in Figure 28 represent the corrosion rates. This implicitly accepted rating criterion was shown to be unreliable and misleading because it does not consistently discriminate between good and poor performance of weathering steel in a variety of environments, ranging from rural to moderate marine [Albrecht and Naeemi, 1984; Komp 1987]. The rating numbers do not reflect the actual corrosion resistance of a weathering steel in a given environment. Thus, statements of relative corrosion rates are not helpful for purposes of stress analysis of weathering steel members. Instead, members must be designed for average, uniform corrosion penetration per surface (loss of metal thickness per side) expected at the end of the service life.

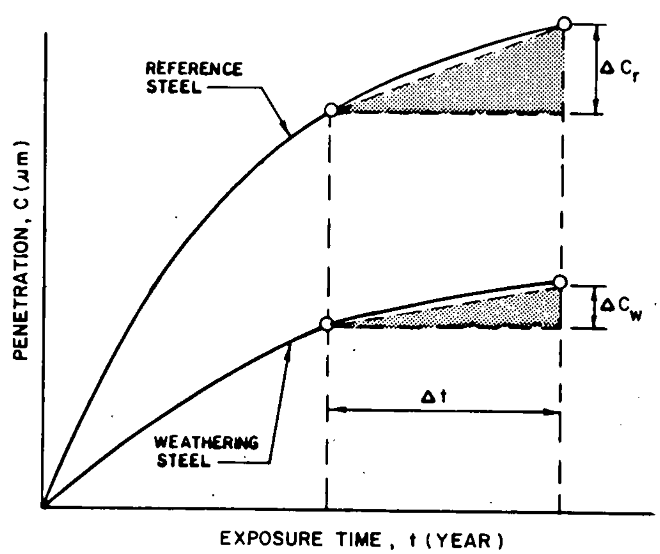

Figure 28. Basis for evaluating corrosion resistance of weathering steel relative to reference steel.

6.11.2 Corrosion Allowance

Designers should obtain realistic corrosion penetration data applicable to the weathering steel at the bridge site and extrapolate the data to the end of the service life. Some data are available from the open literature. For example, Albrecht and Naeemi [1984] summarized corrosion penetration data for ideal and service exposures. Tables 2, 3, and 4 show updated corrosion penetration data for ideal exposure of weathering steel in rural, industrial, and marine environments.

The ASTM specifications A242 and A588 state that, when required, the manufacturer shall supply evidence of corrosion resistance satisfactory to the purchaser. The purchaser should request such evidence from the manufacturer. Lacking suitable data, exposure tests should be performed at the site before weathering steel is specified.

The thickness of each component of a member with two exposed surfaces should be increased, as follows, to allow for corrosion losses of 7.5 μm/year (0.3 mil/year) over a 100-year service life (section 2.4.1):

- For weathering steel members less than 38 mm (1.5 in.) thick, the thickness should be increased by 0.8 mm/surface (64 mil/surface) above the thickness arrived at by stress calculation.
- For members 38 mm (1.5 in.) or greater in thickness, production tolerances will provide sufficient steel to compensate for these losses.

The addition of a corrosion allowance to member thickness is not intended to compensate for such corrosion losses as would occur in very high corrosivity environments where weathering steel cannot develop a protective oxide coating. This is shown in Figure 29 (see also Table 26), which compares the corrosion penetration band for expected performance of weathering steel in bridges (Eqs. 1 and 2) with the measured corrosion rates for various highway environments in Michigan [McCrum et al., 1985]. Curve 8, for urban and rural beams not exposed to traffic spray, and having an average corrosion rate of 8 μm/yr (0.32 mil/yr), falls within the band. But, curves 1 to 7, for exposures in which the protective oxide does not form, fall above the band, and the weathering steel should be painted accordingly.

Designers are cautioned that most published data on atmospheric corrosion were obtained from specimens ideally and boldly exposed at an angle of 30 deg from the horizontal, facing south, in accordance with ASTM Specification G50. In addition, most data listed in manufacturers' literature are for A242 Type 1 steel rather than for A588 steel. Sloping the test specimen at an angle about equal to the latitude of the test site provides ideal exposure and, hence, lowest corrosion rate. Portions of a weathering steel bridge in service may corrode at a higher rate than boldly exposed test specimens.

Other countries specify much greater increases in thickness than those recommended herein to allow for corrosion during the service life of the bridge. For example, the British Department of Transport calls for an increase of thickness per surface of 1 mm (40 mils) of steel per surface in rural atmospheres and 2 mm (80 mils) per surface in industrial atmospheres. The West-German specifications call for an increase of thickness per surface of 0.8 mm (32 mils) in rural, 1.2 mm (48 mils) in urban, and 1.5 mm (60 mils) in industrial and marine atmospheres, for a 60-year service life.

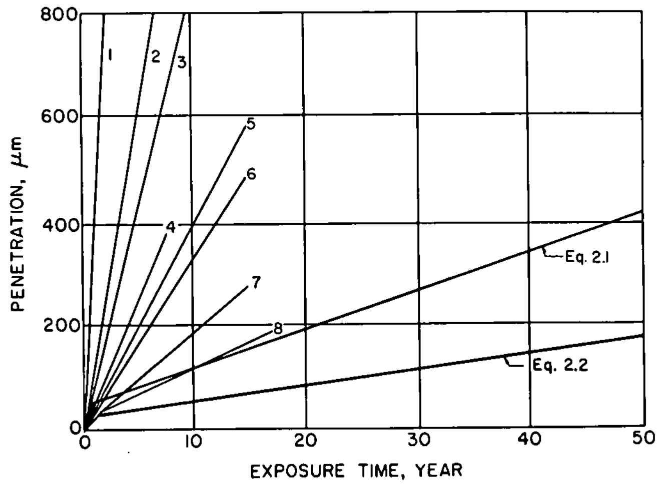

Figure 29. Comparison of expected performance of weathering steel bridges with average and worst-case corrosion rates for bridges in Michigan. See also Table 26.

Table 26. Average and worst-case corrosion rates of weathering steel bridges in Michigan (see also Figure 29). [McCrum et al., 1985]

Curve No.	Exposure	Corrosion
1	Inside gap between web and hanger plate at leaking expansion joint	Worst case pitting rate 400 µm/yr (16 mil/yr)
2	Inside gap between web and hanger plate at leaking expansion joint; lower web and flange of rural and urban beams exposed to traffic spray	Worst-case corrosion rate 125 to 160 µm/yr (5 to 6 mil/yr)
3	Outside gap between web and hanger plate at leaking expansion joint	Worst-case corrosion rate 75 to 100 µm/yr (3 to 4 mil/yr)
4	Flanges of all beams exposed to traffic spray less than 8 years	Average corrosion rate 38 to 64 µm/yr (1.5 to 2.5 mil/yr)
5	Flanges of first three beams exposed to traffic spray	Average corrosion rate 36 to 46 µm/yr (1.4 to 1.8 mil/yr)
6	Flanges of urban and rural beams exposed to traffic spray	Average corrosion rate 30 to 32 µm/yr (1.15 to 1.25 mil/yr)
7	Flanges of fifth and later beams exposed to traffic spray	Average corrosion rate 13 to 26 µm/yr (0.5 to 1.0 mil/yr)
8	Urban and rural beams not exposed to traffic spray (whole beam)	Average corrosion rate 5 to 15 µm/yr (0.2 to 0.6 mil/yr)

The need for corrosion allowances in Great Britain and northern Europe can be attributed to higher levels of atmospheric pollution and damper climates.

WELDING

7.1 INTRODUCTION

7.1.1 General

The welding procedure for weathering steel is similar to that for ordinary steel which has comparable strength but not atmospheric corrosion resistance. The main difference is that for bare, exposed applications of weathering steel, the weld metal must have similar corrosion behavior and coloring characteristics as the base metal. As with any other structural steel, good shop or field practice must be followed to obtain sound welds of desired strength and ductility.

7.1.2 Complementary Codes

The requirements summarized here are peculiar to welding weathering steel structures. They are to be used in conjunction with the latest edition of any complementary code or specification for the design and construction of steel bridges, such as: AWS D1.1 "Structural Welding Code—Steel" issued by the American Welding Society [1986]; and the AASHTO *Standard Specifications for Welding of Structural Steel Highway Bridges* [AASHTO 1981].

7.2 BASE METAL

7.2.1 Types of Steel

Weathering steel to be welded under these guidelines must conform to the requirements of the latest edition of the ASTM specifications A242, A588, A709 grades 50W and 100W, or A852.

Combinations of any approved weathering steel base may be welded together. Any weathering steel base may also be welded to a structural steel base that does not have atmospheric corrosion resistance such as the A36, A572, and A514 steels.

7.2.2 Weldability

A588 steel and A709 Grade W steel meeting the chemical and mechanical properties of A588 are weldable by the prequalified procedures of AWS D1.1.

When A242 steel is considered for use, its weldability must be investigated by the engineer who shall specify all pertinent material, design, and workmanship information not covered in AWS D1.1.

When an A709 Grade W steel is considered for use, its weldability must be established by the producer. The procedure for welding the steel must be established by qualification in accordance with the requirements of AWS D1.1 and other requirements that may be prescribed by the engineer.

Table 27. Filler metal requirements for bare, exposed applications of ASTM A242 and A588 steels.

Electrode Specification			
Shield Metal Arc	Submerged Arc	Gas Metal Arc	Flux Cored Arc
AWS A5.5.81	AWS A5.23-83[a,d]	AWS A5.28-70[d]	AWS A5.29-80
E7018-W	E7AX-EXXX-W	ER80S-Ni1	E8XT1-W
E8018-W	F7AX-EXXX-Ni1[b]	ER80S-Ni2	E8XTX-Ni1
E8016-C3 or E8018-C3	F7AX-EXXX-Ni4[b]	ER80S-Ni3	E8XTX-Ni2
E8016-C1 or E8018-C1	F7AX-EXXX-Ni2[b]	ER80S-B2L[a]	E80T5-Ni3
E8016-C2 or E8018-C2	F7AX-EXXX-Ni3[b]	ER80S-G[a,c]	E80T5-B2L[a]
E7016-C1L or E8018-C1L			E71T8-Ni1
E7016-C2L or E8018-C2L			E71T8-Ni2
E8018-B2L[a]			E7XTX-K2

Notes:

a. Deposited weld metal must have a minimum impact strength of Charpy V-notch 27.1 J (20 ft-lb) at-18°C (0°F). This requirement applies only to bridges.

b. The use of the same type of filler metal having next higher tensile strength as listed in AWS specification is permitted.

c. Deposited weld metal shall have a chemical composition the same as that for any one of the weld metals in this table.

d. Composite (metal cored) electrodes are designated as follows:

SAW: Insert letter "C" between the letters "E" and "XX"; e.g., F7AX-ECXXX-Ni1.

GMAW: Replace the letter "S" with the letter "C", and omit the letter "R"; e.g., E80C-Ni1.

7.3 WELDS

7.3.1 Filler Metal for A242 and A588 Steels

Bare, exposed A242 and A588 steel structures must be welded with the electrodes or electrode-flux combinations given in Table 27 to ensure that the weld matches the strength, corrosion resistance, and color of the base metal.

In multiple-pass welds, the weld metal may be deposited so that at least two layers are deposited on all exposed surfaces and edges with one of the filler metals specified in Table 27, provided the underlying layers are deposited with one of the filler metals specified in Table 28. The latter have the matching strength but not the corrosion resistance and coloring characteristics of weathering steel.

In single-pass welds, absorption of alloying elements from the weathering steel base may give the weld metal a corrosion resistance and coloring similar to that of the steel base. Accordingly, AWS D1.1 permits the following welds to be made with the filler metals given in Table 28.

1. *Shielded Metal Arc Welding*—Single-pass fillet welds up to 6.4 mm (¼ in.) maximum and 6.4-mm (¼ in.) groove welds made with a single pass or a single pass each side may be made by using an E70XX low hydrogen electrode.
2. *Submerged Arc Welding*—Single-pass fillet welds up to 8.0 mm (5/16 in.) maximum and groove welds made with a single pass or a single pass each side may be made by using an F7X-EXXX electrode-flux combination.
3. *Gas Metal Arc Welding*—Single-pass fillet welds up to 8.0 mm (5/16 in.) maximum and groove welds made with a single pass or a single pass each side may be made using an E70S-X electrode.

Table 28. Matching filler metal requirements. [American Welding Society 1986]

ASTM Steel Designations [a]	Electrode Specification[b,c] Shielded Metal Arc	Submerged Arc	Gas Metal Arc	Flux Cored Arc
A242[d], A588, and A709 Grade 50W	AWS A5.1 or A5.5 E7015 E7016 E7018 E7028	AWS A5.17 or A5.23 F7X-EXXX	AWS A5.18 ER70S-X	AWS A5.20 E7XT-X (Except -2, -3, -10, -GS)
A709 Grade 100W $t \leq 63$ mm ($t \leq 2.5$ in.)	AWS A5.5 E11015 E11016 E11018	AWS A5.23 F11X-EXXX[e]	AWS A5.28 ER110S[e]	AWS A5.29 E11XT[e]
A709 Grade 100W 63 mm $< t \leq$ 102 mm (2.5 in. $< t \leq$ 4 in.)	AWS A5.5 E10015 E10016 E10018	AWS A5.23 F10X-EXXX[e]	AWS A5.28 ER100S[e]	AWS A5.29 E10XT[e]

Notes:

a. In joints involving base metals of different groups, low-hydrogen filler metal requirements applicable to the lower strength group may be used. The low-hydrogen processes used will be subject to the technique requirements applicable to the higher strength group.

b. When welds are to be stress-relieved, the deposited weld metal shall not exceed 0.05% vanadium.

c. See Section 7.3.3 for electroslag and electrogas weld metal requirements.

d. Special welding materials and procedures may be required to match the notch toughness of base metal for applications involving impact loading or low temperature.

e. 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 applies only to bridges.

4. *Flux Cored Arc Welding*—Single-pass fillet welds up to 8.0 mm (5/16 in.) maximum and groove welds made with a single pass or a single pass each side may be made using an E70T-X electrode.

7.3.2 Filler Metal for A709 Grade 50W Steel

Bare, exposed A709 Grade 50W steels should be welded with the electrodes or electrode-flux combinations required for A242 and A588 steels (Table 27).

7.3.3 Filler Metal for A709 Grade 100W Steel

Bare exposed A709 Grade 100W steels may be welded with the electrodes or electrode-flux combinations given in Table 28. The filler metal must contain one or more of the elements Ni, Cr, Cu, and Si in quantities sufficient to match the corrosion resistance and color of the base metal.

7.3.4 Electroslag and Electrogas Welding

For electroslag and electrogas welding of bare, exposed A242 and A588 steels, the chemical composition of the electrode-flux combination must conform to one of the filler metals given in Table 27. In addition, the fabricator must demonstrate by qualification tests that each combination of shielding and filler metal produce weld metal that meets the mechanical properties (strength, ductility and toughness, and soundness requirements) given in the latest editions of the AWS filler metal specifications 5.25 and 5.26.

7.4 PREHEAT AND INTERPASS TEMPERATURES

The minimum preheat and interpass temperature requirements for welding weathering steel are specified in AWS D1.1. Care must be taken to ensure that preheat requirements are met because weathering steel is more susceptible to heat-affected-zone cracking than ordinary steel.

7.5 STRESS RELIEF HEAT TREATMENT

Stress relieving of weldments of A709 Grade 100W quenched and tempered steel is generally not recommended because this process may impair ductility and strength. The results of notch toughness tests have shown that postweld heat treatment sometimes impairs weld metal and heat-affected-zone toughness. Also, intergranular cracking sometimes occurs in the grain-coarsened region of the heat-affected zone of the weld. The stress relief treatment must conform to the requirements of section 4.4 of AWS D1.1.

7.6 OXYGEN OF PLASMA-ARC CUTTING

Weathering steel and its weldments may readily be oxygen or plasma-arc cut following the same general procedures used for cutting other kinds of structural steel. In so doing, the plates or shapes should be preheated to minimize the possibility of thermal cracking. The preheating temperature for cutting should be the same as that for welding.

7.7 DESIGN STRESSES

The allowable stresses for the static design of welds for weathering steel bridges are the same as those for ordinary steel bridges. Allowable stress ranges for the fatigue design of welded details are recommended in Chapter Nine.

7.8 PREPARATION

Welding flux, slag, and spatter on all exposed welds should be removed by power grinding according to SSPC-SP 3-63 "No. 3 Power Tool Cleaning" or by blast cleaning according to SSPC-SP 6-63 "No. 6 Commercial Blast Cleaning." Welds should be carefully ground to avoid the formation of a heat scale that retards the weathering process.

7.9 CONTINUOUS WELDING

Joints must be continuously welded on all sides to prevent moisture from entering the crevice formed by the contact surfaces and causing crevice corrosion. Fillet welds in built-up members, likewise, must be continuous.

CHAPTER EIGHT

MECHANICAL FASTENERS

8.1 INTRODUCTION

The design of mechanically fastened joints must comply with the requirements of the AASHTO specifications for highway bridges and the RCSC specification for structural joints using ASTM A325 or ASTM A490 for bolts [RCSC 1985]. The recommendations summarized in the following section pertain to only those provisions that are peculiar to weathering steel structures.

8.2 TYPES OF FASTENERS

8.2.1 Availability

Mechanical fasteners made of steel whose chemical compositions provide weathering characteristics comparable to those of the A242, A588, and A709 weathering steels are commonly available as high-strength bolts. Other types of fasteners such as regular bolts, unfinished bolts, and rivets may not be readily available.

8.2.2 High-Strength Bolts, Nuts, and Washers

High-strength bolts conforming to the requirements of the ASTM specifications A325 and A490 are available in Type 1, Type 2, and Type 3. Type 1 and Type 2 bolts, intended for use with painted steels, are made of medium-carbon steel and low-carbon martensitic steel. Type 3 bolts have atmospheric corrosion resistance and weathering characteristics comparable to those of the A242, A588, and A709 weathering steels.

Specifications A563 and F436, respectively, cover nuts and washers having atmospheric corrosion resistance and weathering characteristics comparable to those of the A242, A588, and A709 weathering steels. Grade C3 heavy hex nuts must be furnished for use with A325 Type 3 bolts, and Grade DH3 heavy hex nuts must be furnished for use with A490 Type 3 bolts. Grade DH3 nuts are acceptable alternatives to C3 nuts. Type 3 weathering steel washers must be furnished when Type 3 bolts are specified. Table 29 summarizes the types of bolts, nuts, and washers that must be used with bare weathering steel.

Bolts, nuts, and washers other than those given in Table 29 must not be used if they are less noble than the weathering steel. The galvanic couple that may be established between the small anodic area of a fastener and the large cathodic area of a weathering steel structure under conditions of prolonged wetness causes the fastener to corrode at an accelerated rate.

Table 29. Matching bolts, nuts, and washers for use with weathering steels.

Bolts	Nuts	Washers
A325, Type 3	A563, Grades C3 or DH3	F463, Type 3
A490, Type 3	A563, Grade DH3	F463, Type 3

Table 30. Producers of steel for A325 Type 3 high-strength bolts.

Class	Producer	Proprietary Name
A	U.S. Steel	Cor-Ten-X
	Stelco	Stelcoloy
B	Bethlehem Steel	Weath-R
C	Stelco	Stelcoloy
D	Armco	. . .
E	Republic Steel (LTV)	. . .
F	Bethlehem Steel	Weath R Type 3F

8.2.3 Other Types of Fasteners

Where lower strength fasteners are satisfactory, weathering steel bolts, unfinished bolts, and rivets made of steel conforming to the tensile requirements of A242 and A588 base metal are available in chemical compositions that provide weathering characteristics comparable to the aforementioned base metals.

8.2.4 Dissimilar Metal Bolts

Bolts made of dissimilar metals, such as stainless steel, may be used if the bolt metal is more noble than the weathering steel, and the bolted joint does not act as a crevice which may hold water and debris.

8.2.5 Coated Bolts

Zinc and cadmium coated bolts should not be used in weathering steel structures because in time the coating is sacrificed through cathodic protection, leaving an exposed carbon steel bolt less resistant to atmospheric corrosion than weathering steel.

8.3 CHEMICAL REQUIREMENTS

8.3.1 Corrosion Resistance

High strength bolts, nuts, and washers that meet the chemical requirements summarized in this chapter have atmospheric corrosion resistance and weathering characteristics comparable to those of the A242, A588, and A709 weathering steels.

8.3.2 High-Strength Bolts

Steel for A325 Type 3 bolts is supplied in classes of material A, B, C, D, E, and F. The producers are given in Table 30. The selection of the class is left to the discretion of the bolt manufacturer.

The chemical compositions of the proprietary classes of steel for A325 bolts were apparently patterned after the chemical compositions of the grades of A588 steel for plates and shapes. However, the producers of the different classes of bolt steel could not be identified. The newer edition of A325 specification is likely to combine all classes of A325 Type 3 bolts under one generic composition in a way similar to the approach already taken in the A490 specification.

8.3.3 Nuts

Steel for A563 Grade C3 nuts is supplied in classes of material A to F, and N. The selection of the class is left to the discretion of the bolt manufacturer.

Steel for A563 Grade DH3 nuts is generic supplied in one generic class of material. Different procedures can supply the steel from which the nuts are manufactured.

8.3.4 Washers

Steel for F436 Type 3 washers is supplied in one generic class of material. Weathering steel washers may also be manufactured from any of the steels that are used for making A325 Type 3 bolts of material classes A through F.

8.4 MECHANICAL REQUIREMENTS

8.4.1 High-Strength Bolts

The mechanical requirements for A325 and A490 Type 3 bolts are identical to the mechanical requirements for their Type 1 and Type 2 counterparts that do not have enhanced atmospheric corrosion resistance.

8.4.2 Nuts

A563 Grade C3 and Grade DH3 nuts must withstand a proof load stress of 993 and 1207 MPa (144 and 175 ksi) respectively, and meet the specified hardness requirement of 24 HRC min. to 38 HRC max.

8.4.3 Washers

F436 Type 3 through-hardened washers must have a hardness of 38 to 45 HRC. Hardness is the only mechanical requirement for washers.

8.5 TIGHTENING

8.5.1 Initial Bolt Tension

To provide a tight joint, all A325 and A490 bolts must be pretensioned to 70 percent of the tensile strength of the steel on the stress area of the bolt.

8.5.2 Tightening Methods

High-strength bolts must be tightened by (1) the turn-of-the-nut method, (2) with a calibrated wrench, or (3) with a wrench and load indicating device. Reference RCSC [1985] provides the criteria to be followed for turn-of-nut and calibrated wrench tightening. The suitability of the two commonly used load indicating devices—load indicator washer and tension-control bolt—for controlling the initial tension in Type 3 high-strength bolts is discussed below.

8.5.3 Load Indicator Washers

Load indicator washers with epoxy-coated and either mechanically galvanized or mechanically cadmium-coated surfaces are supplied for use with Type 3 high-strength bolts. The treatment appears to provide the washers with good corrosion protection. The epoxy-coated washers must be tightened to nil gap or zero gap to provide minimum tension in the bolt. The gap between the washer and the bolt head may act as a crevice that serves as a path for water ingression to the shank and threads of the bolt. This condition may lead to accelerated crevice corrosion of the weathering steel bolt and plates, particularly in the presence of salt water. To prevent this condition, the supplier recommends increasing the initial bolt tension until the protrusions are flattened and the gap is closed. According to the supplier, a 10 to 15 percent increase of the bolt tension over the specified minimum is needed to close the gap.

Because atmospheric exposure data in marine environments are not available for Type 3 high-strength bolted joints with load indicator washers, it is not possible to assess whether the gap will induce crevice corrosion in the presence of salt. The accelerated 1,000-hour tests that were performed in a salt fog cabinet have not been able to show the gap's susceptibility to crevice corrosion.

Load indicator washers are not recommended for use with bare weathering steel structures until long-term atmospheric exposure tests of bolted joints confirm the adequacy of their corrosion performance.

8.5.4 Tension-Control Bolts

Tension control bolts in diameters of 15.9 mm to 25.4 mm (⅝ in. to 1 in.) conform to the requirements of the A325 specification and are available in Type 1 alone. They do not have the atmospheric corrosion resistance and weathering characteristics of Type 3 bolts and must, therefore, be protected by painting.

8.6 SPACING

8.6.1 Corrosion Behavior

Bolted joints of exposed weathering steel members have shown good corrosion performance if the plate thickness, bolt spacing, and initial bolt tension are adequate. Under these conditions the joint remains tight, and the space between the contact surfaces of two weathering steel plates seals itself with corrosion products that form around the periphery of the joint.

However, if the plates are flexible and the bolts widely spaced or torqued to less than the minimum specified tension, the gap between the plates may foster crevice corrosion and induce pitting of the contact surfaces. The continuing formation of corrosion products within the joint leads to expansive forces which can: (1) deform the plates between adjacent bolts, (2) lift the plate edges, and (3) cause large tensile loads on the bolts. Under these unfavorable conditions plates have been deformed and bolts have, in some instances, failed in tension.

8.6.2 Corrosion Performance

The following guidelines should provide the stiffness and tightness needed for good corrosion performance [Brockenbrough 1983; Brockenbrough and Gallagher, 1985]:

1. The pitch on a line of fasteners adjacent to a free edge of plates or shapes in contact with one another should not exceed 14 times the thickness of the thinnest part joined and, in any event, not exceed 175 mm (7 in.).
2. The distance from the center of any bolt to the nearest free edge of plates or shapes in contact with one another should not exceed eight times the thickness of the thinnest part joined and, in any event, should not exceed 125 mm (5 in.). Edges of elements sandwiched between splice plates need not meet this requirement.
3. A325 and A490 Type 3 high-strength bolts must be used in exposed weathering steel joints, and the bolts must be torqued to the minimum specified bolt tension.

The contact surfaces of bolted joints that do not meet the aforementioned guidelines must be protected by painting.

8.7 DESIGN STRESSES

8.7.1 Tension

When the bolts are torqued and spaced in accordance with the guidelines mentioned in sections 8.5 and 8.6, and the joints do not trap debris and water, the expansive forces of the corrosion products that form on the contact surfaces are expected to be negligible. Under these conditions bolts in tension can be designed to the allowable stresses given in the AASHTO specifications.

8.7.2 Slip-Critical Connections

The mill scale is smoother and adheres more tightly to the underlying weathering steel base than to a carbon steel base. As a result, high-strength bolted connections of weathering steel members slip into bearing at lower shear stresses than those of carbon steel members when the mill scale is left intact [Yura et al. 1981].

Table 31 gives the allowable shear stresses for slip-critical, bolted connections with classes A, B, and C contact surfaces, as given in the new specification for structural joints using ASTM A325 and A490 bolts. It is recommended that a new Class D be added to cover contact surfaces with clean mill scale of high-strength low-alloy, and quenched and tempered steels.

Accordingly, slip-critical connections of members made of A242, A588, A709 grades 50W and 100W, and A852 weathering

Table 31. Recommended allowable shear stresses for slip-critical connections of bare and coated steel.

Contact Surface of Bolted Parts	Allowable Shear Stress for Standard Holes MPa(ksi)	
	A325	A490
Class A		
Coefficient of friction 0.33: Clean mill scale of carbon steel Blast-cleaned surfaces with Class A coatings	100 (14.5)	125 (18.0)
Class B		
Coefficient of friction 0.50: Blast-cleaned surfaces of all steels Blast-cleaned surfaces with Class B coatings	170 (25.0)	215 (31.0)
Class C		
Coefficient of friction 0.40: Hot-dip galvanized and roughened surfaces	135 (19.5)	170 (24.5)
Class D		
Coefficient of friction 0.23: Clean mill scale of low-alloy and quenched and tempered steels	75 (11.0)	100 (14.5)

steels must be designed to the allowable shear stresses of Class B for blast-cleaned surfaces, Class D for clean mill-scaled surfaces, and classes A and B for blast-cleaned and coated surfaces, whose coefficient of friction is measured by the "Testing Method to Determine the Slip Coefficient for Coatings Used in Bolted Connections" [Yura et al. 1985].

The weathering of bare steel surfaces for several months, while the members are being stored under bold exposure prior to erection, does not adversely affect the slip resistance of a bolted joint. However, any loose rust that may form on the contact surfaces under conditions of prolonged wetness must be removed before the connection is assembled.

8.7.3 Bearing-Type Connections

Bolts in bearing-type connections of weathering steel members that are torqued and spaced in accordance with the guidelines mentioned in sections 8.5 and 8.6 can be designed to the allowable stresses given in the AASHTO specifications.

8.8 BOLTED PARTS

The bolted steel parts must fit solidly throughout the joint after the bolts are tightened. They may be coated or uncoated. The surfaces of bolted parts in contact with the bolt head or nut must be normal to the bolt axis. Bolted parts that are sloped with respect to the bolt axis permit moisture to intrude beneath the tilted bolt head or nut. They must be protected by painting.

All joint surfaces to be assembled, including surfaces adjacent to the bolt head or nut, must be free of scale, loose rust, dirt or other foreign material, and burrs that could prevent solid seating of the connected parts.

Slotted holes in the outer ply of bolted parts must be completely covered with a plate washer or continuous bar of at least 8 mm ($\frac{5}{16}$ in.) thickness and a standard hole size. These washers or bars should be hardened.

Good bolted connection details are important. Any detail that traps water, damp debris, and moisture in crevices or pockets will lead to accelerated crevice or pitting corrosion. Under these conditions the protective oxide coating cannot develop and corrosion continues indefinitely. The designer must, therefore, detail such elements with extreme care to assure there is no possibility of moisture entrapment. If this condition cannot be avoided, all such unexposed surfaces, including the contact surfaces between the plies of the connection, are to be treated like ordinary steel and must be protected by painting. Any crevices should be sealed.

CHAPTER NINE

FATIGUE

9.1 INTRODUCTION

9.1.1 Environments

The aqueous environments of fresh water and salt water can reduce the fatigue strength of bare, exposed weathering steel bridges as compared to steel bridges that are protected with a well-maintained paint system.

The superstructure of a bridge may become wet in many ways—runoff water leaks through the expansion joints and seals, trucks passing under the bridge kick up a spray that settles uniformly on the superstructure, roadway debris and rust flakes accumulate on horizontal surfaces and hold moisture, water collects in poorly designed structural details, and lack of air circulation and low clearance over bodies of water facilitate the condensation of moisture on all steel surfaces when the nightly temperature drops below the dew point.

The conditions for wetting are worsened by contamination of the steel surface with salt from any source. On drying, salt crystals hygroscopically attract moisture from the air, thus increasing the time-of-wetness. Visible condensation in the form of droplets is not needed for corrosion to occur.

The chlorides from roadway deicing salt, marine breezes, marine fog, and seawater spray are the primary stimulants that accelerate pitting and general corrosion. Because the deck shelters the superstructure against rain washing, chlorides build up on the surfaces of sheltered members and create corrosive conditions similar to those found in severe marine environments.

During stress cycling a fatigue crack eventually creates its own environment irrespective of whether the steel member is wetted by or immersed in an aqueous solution. Because of the foregoing reasons corrosion fatigue tests of bridge steels are routinely performed with specimens that are immersed in fresh or salt water [Barsom and Novak, 1977; Roberts et al., 1986].

Depending on the quality of detailing, degree of maintenance, and type of environment to which the structure is subjected, the microenvironment at a bridge site can be characterized as being of medium, high, or very high corrosivity (Table 6). The corresponding exposure conditions are described as follows:

1. *Medium Corrosivity.* The steel structure is boldly exposed to rain and sun, the environment is free of salt, the structural details do not trap debris, the bridge is jointless or the joints do not leak, and the bridge is regularly maintained.
2. *High Corrosivity.* The steel structure is sheltered from the rain and sun, the weathering steel is contaminated with small amounts of salt, some details trap debris, joints may leak, or the bridge is not regularly maintained.
3. *Very High Corrosivity.* The steel structure is sheltered, the weathering steel is contaminated with significant amounts of salt, details trap debris, joints leak, the steel remains wet for long periods of time, or the bridge is not maintained.

9.1.2 Loading

Investigators have attempted to determine the fatigue strength of bare, exposed steel structures with tests that provided the following types of data: weathering fatigue S-N (WFSN) life, corrosion fatigue crack initiation (CFCI) life, corrosion fatigue crack propagation (CFCP) rate, corrosion fatigue S-N (CFSN) life, and weathering and corrosion fatigue S-N (W&CFSN) life.

In the "weathering fatigue" test the specimens were boldly exposed to the environment for many years and then stress cycled to failure in dry laboratory air. In the "corrosion fatigue tests" the nonweathered specimens were stress cycled in aqueous environments of fresh or salt water. In the "weathering and corrosion fatigue" tests the specimens were boldly exposed to the environment for many years and then stress cycled to failure in aqueous environments of fresh water or salt water.

Weathering steel bridges experience more complex combinations of loading and environmental exposure than the aforementioned tests can simulate. Before the bridge is opened to traffic, and during the initial years of service, weathering creates rust pits from which cracks may eventually initiate. During the service life, the aqueous environment enhances crack initiation and accelerates the rate of crack propagation.

The exposure conditions of a weathering steel bridge may, therefore, lead to a reduction in fatigue strength caused by the effect of weathering and corrosion fatigue on the crack initiation life plus the effect of corrosion fatigue on the crack propagation life. The effects are cumulative.

Sections 9.2 through 9.6 summarize the findings of numerous studies on which the recommendations of section 9.7 are based. The data cited herein were analyzed in detail in the following references: Albrecht [1982, 1983]; Albrecht and Naeemi [1984]; Albrecht and Sidani [1987]; and Albrecht [1988].

9.2 WEATHERING FATIGUE S-N (WFSN) LIFE

Weathering fatigue S-N (WFSN) data are available from 34 series of tests of 965 specimens fabricated from weathering steel and 11 series of tests of 600 specimens fabricated from ordinary steel, which are not atmospheric corrosion resistant [Albrecht 1982; Albrecht and Naeemi, 1984; Albrecht and Sidani, 1987]. The weathering steels were American ASTM A242 and A588, and Japanese JIS SMA 50 and SMA 58 steels. The ordinary steels were Japanese JIS SM 50 and SM 58 steels. The details tested were base metal, groove welds as welded and ground flush, bead welds, notched plates, transverse stiffeners, and attachments. The specimens were boldly weathered up to 11 years prior to stress cycling.

Figure 30 shows the loss in mean stress range due to weathering for the 29 sets of data for which the loss could be calculated. The loss in stress range is the vertical drop between the mean line of stress range versus cycles to failure (S-N line) for a series of nonweathered specimens and that of their weathered counterparts. It was calculated at 500,000 cycles of loading and plotted against the fatigue notch factor of the nonweathered specimens. The fatigue notch factor is the factor on stress range by which the mean S-N line for a set of nonweathered specimens falls below the mean S-N line for Category A base metal, both tested in dry air. The vertical grid lines locate the mean S-N lines for the following types of details that were fabricated from ordinary steel and tested in air: Category A rolled beam, Category B welded beam, Category C* transverse stiffener, Category C 50-mm (2 in.) attachment, Category D 100-mm (4 in.) attachment, and Category E and Category E′ cover plate. Those data were previously used to establish the AASHTO allowable S-N lines for Category A to Category E′ details [Fisher et al., 1970, 1974, 1979].

A comparison of all WFSN data shown in Figure 30 leads to the conclusion that the loss in stress range was highest for Category A base metal and continuously decreased with increasing fatigue notch factor of the detail. In other words, the higher the fatigue notch factor of a detail, the less rust pitting reduced the crack initiation life. Also, atmospheric exposure reduced alike the fatigue strength of weathering steel and ordinary steel specimens.

The WFSN data only model the effect of weathering on the crack initiation life. Because the test specimens were not stress cycled in an aqueous environment, as are weathering steel bridges in service, the obtained losses underestimate the loss in fatigue strength that weathering steel bridges may experience.

9.3 CORROSION FATIGUE CRACK INITIATION LIFE

Novak of USX Corporation determined the corrosion fatigue crack initiation (CFCI) behavior of A36, A588, and A517 steels by testing 48 notched specimens in a 3.5 percent sodium chloride solution at a constant-amplitude cyclic frequency of 0.2 Hz [Novak 1983]. The notched specimens had a theoretical stress concentration factor, $K_t = 3.42$. The CFCI life was defined as the number of cycles needed to initiate a crack from the notch and to grow the crack to a surface length of 0.75 to 1.75 mm (0.030 and 0.070 in.).

The fatigue crack initiation (FCI) threshold in air was f_r = 147, 182, and 250 MPa (21, 26, and 36 ksi) for A36, A588, and A517 steels, respectively, where f_r is the nominal stress range at the notch.

No CFCI thresholds were found for any of the steels tested in a sodium chloride solution despite strong attempts to char-

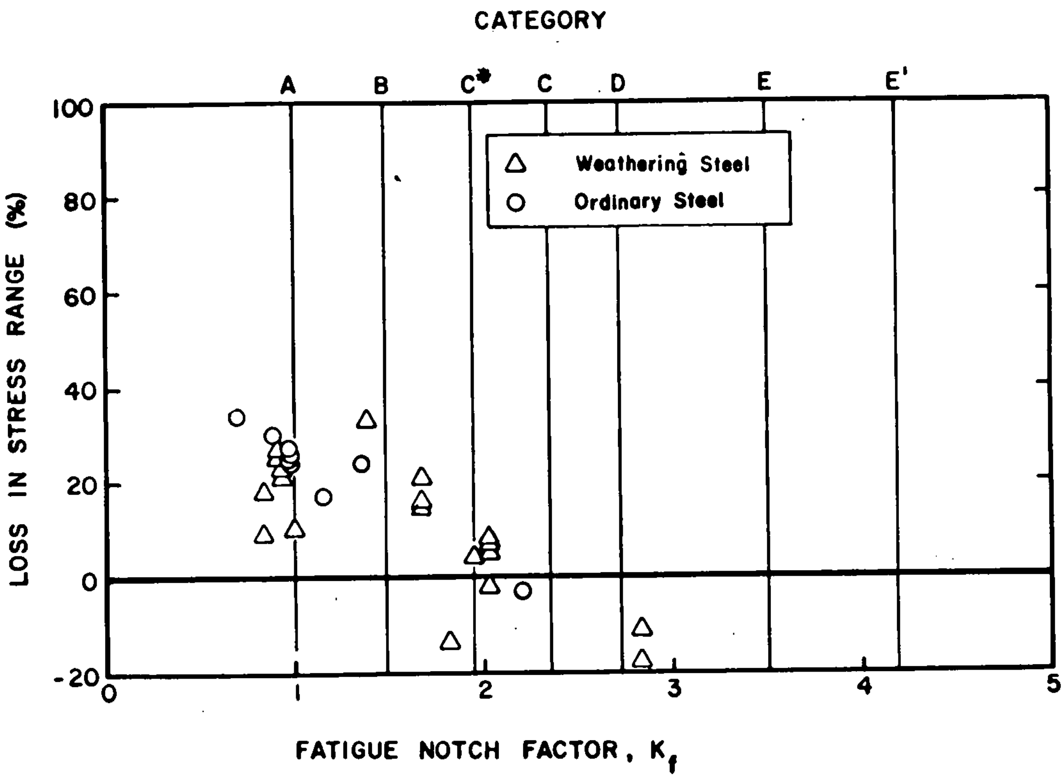

Figure 30. Loss in stress range of weathered specimens tested in air.

acterize the long-life behavior of primary interest for structural applications such as bridges.

The long-life (3,000,000 cycles) fatigue strength for CFCI behavior was f_r = 68 MPa (10 ksi) for all steels. The loss in stress range, determined from a comparison of 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 for the A36, A588, and A514 steels, respectively, at 3,000,000 cycles.

Taylor and Barsom [1981] of USX Corporation reported similar findings for the CFCI life of A517 Grade F Steel specimens.

The data show that the CFCI life of weathering steel details in the salt water environment was much shorter than the FCI life of the same details in clean air. There was no fatigue limit in this aqueous environment. Therefore, the long-life fatigue strength of bare, exposed weathering steel bridges contaminated with salt should be expected to be much lower than the AASHTO fatigue limits for "over 2,000,000 cycles" of loading, which were based on the results of fatigue tests performed in clean laboratory air and are intended for painted steel bridges.

9.4 CORROSION FATIGUE CRACK PROPAGATION RATE

Measurements of corrosion fatigue crack propagation (CFCP) rate in accordance with ASTM Specification E647 show what effect an aqueous environment has on the rate of crack growth during the propagation phase of the total fatigue life of a detail.

Yazdani and Albrecht [1983] collected and analyzed a total of 3,254 data points for crack growth rate in A36 mild steel, A588 and X-52 high-strength low-alloy (HSLA) steels, and A514 quenched and tempered steel specimens tested in air and aqueous environments. The aqueous environments consisted of distilled water and a solution of sodium chloride in distilled water. Most CFCP data came from an NCHRP project funded at USX Corporation [Barsom and Novak, 1977] and an FHWA project funded at Lehigh University [Roberts et al., 1986]. The remainder of the data came from Klingermann and Fisher [1973] and Mayfield and Maxey [1982].

After an extensive statistical analysis of the effects of type of steel, loading, environment, and testing laboratory, Yazdani and Albrecht eliminated the insignificant variables. For example, the difference in crack growth rate was found to be statistically insignificant between mild and HSLA steels, and between fresh and salt environments. This left type of steel (mild and HSLA steels versus quenched and tempered steels) and environment (air versus aqueous environments) as the only significant variables. Accordingly, the following equations of crack growth rate, da/dN versus range of stress intensity factor, ΔK, were obtained. For mild and HSLA steels (A242, A588, and A709 Grade 50W) in air: $(da/dN) = 1.54 \times 10^{-12} (\Delta K)^{3.344}$. For mild and HSLA steels (A242, A588, and A709 Grade 50W) in aqueous environments: $(da/dN) = 4.16 \times 10^{-12} (\Delta K)^{3.279}$.

For quenched and tempered steels (A709 Grade 100W) in air: $(da/dN) = 2.27 \times 10^{-11} (\Delta K)^{2.534}$. For quenched and tempered steels (A709 Grade 100W) in aqueous environments: $(da/dN) = 6.00 \times 10^{-11} (\Delta K)^{2.420}$.

In the foregoing equations da/dN and ΔK have units of m/cycle and $MPa\sqrt{m}$.

The air and aqueous lines for each group of steels, plotted in Figure 31, are nearly parallel. At the lowest stress intensity factor range, $\Delta K = 14.8$ MPa$\sqrt{m}$ (13.5 ksi $\sqrt{in.}$), at which CFCP rates were measured, cracks grew faster in aqueous environments than in air by a factor of 2.3 for mild and HSLA steels, and by a factor of 1.9 for quenched and tempered steels. Both sets of equations indicate that structural details on bare weathering steel bridges have shorter crack propagation lives than those on painted steel bridges. This conclusion applies to all types of details.

9.5 CORROSION FATIGUE S-N LIFE

A better measure of how a corrosive environment affects the total fatigue life (crack initiation plus propagation) of weathering steel details is obtained by comparing the fatigue lives of specimens cycled to failure in air with those of the same specimens cycled in an aqueous environment. Such corrosion fatigue S-N (CFSN) tests involve crack initiation and crack propagation at all values of ΔK, from near threshold to failure.

Albrecht and Sidani [1987] collected and analyzed 49 sets of CFSN data from 705 specimen tests that were reported in the literature. The specimens consisted of the following types of details: smooth plate, as-rolled base metal, groove weld, welded T-joint, welded oblique joint, welded cruciform joint, welded longitudinal joint, and notched plate. The specimens were stress cycled in air and in aqueous environments of fresh or salt water. They were tested under moist or immersed conditions. The stress ranges were applied in tension or stress reversal at frequencies of 0.1 to 50 Hz.

The loss in stress range was determined for 46 sets of CFSN data at 2,000,000 cycles of loading. The results are plotted in Figure 32 against the fatigue notch factor of the specimens tested

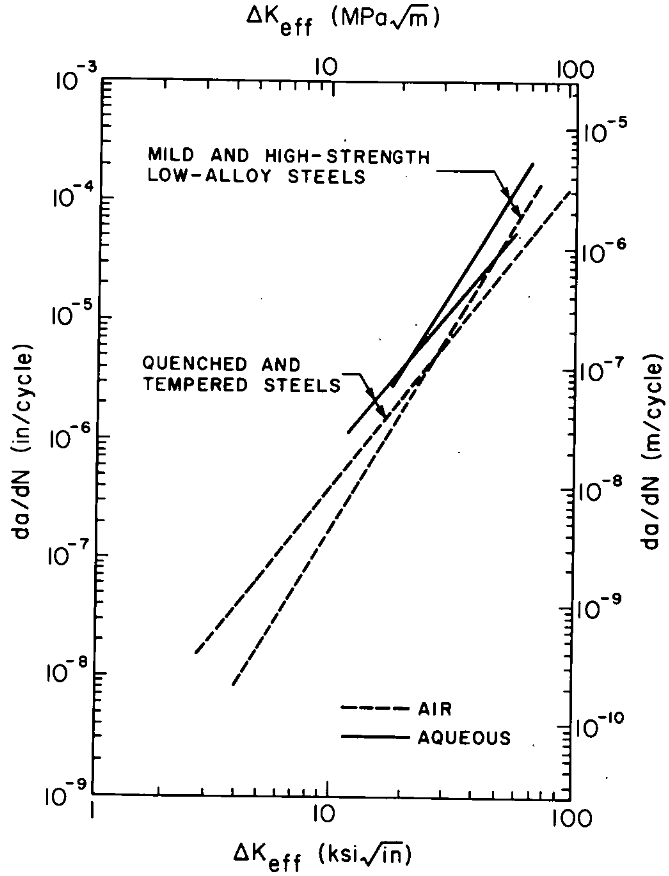

Figure 31. Crack propagation rates for structural steels tested in air and aqueous environments.

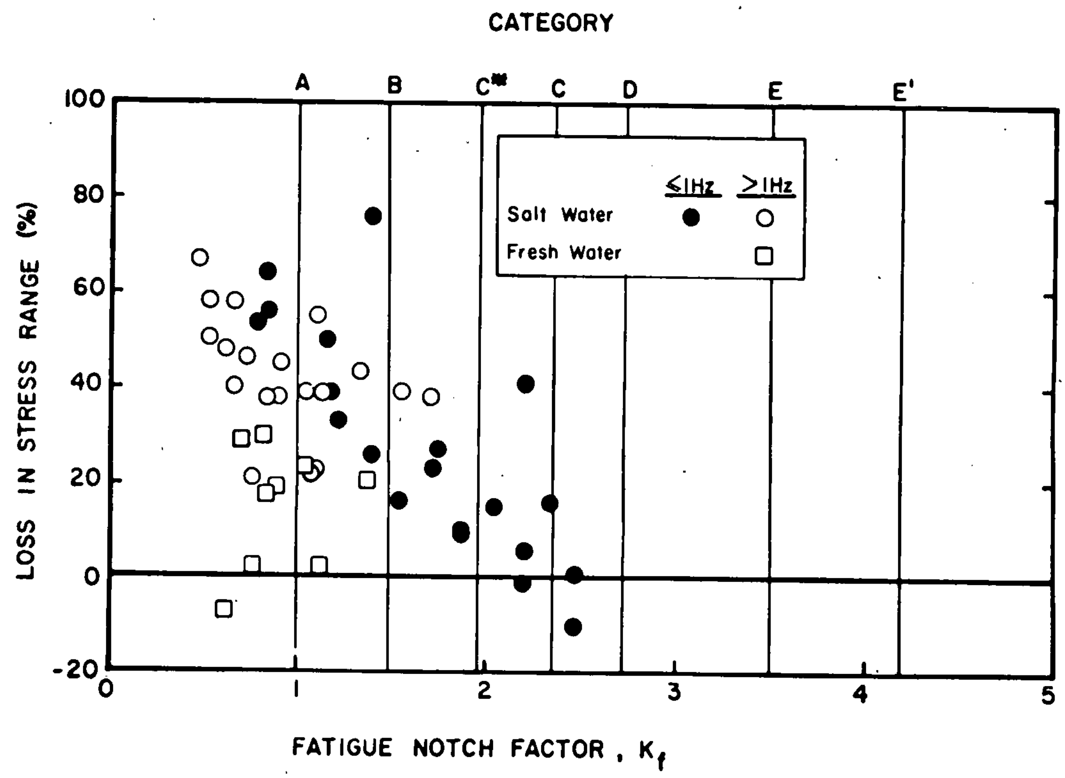

Figure 32. Loss in stress range of specimens tested in aqueous environments.

in air. A comparison of all corrosion fatigue data shows that salt water environments induced greater losses in stress range than fresh water environments.

The loss in stress range in aqueous environments was greater at low cyclic frequencies of 0.1 Hz to 1.0 Hz, typical of highway bridge loading, than at high cyclic frequencies. The loss in stress range was highest for details having the fatigue strength of Category A base metal. The loss diminished, going from Category A to Category C in the direction of increasing fatigue notch factor.

Several details exhibited no fatigue limit in aqueous environments. For example, details having the fatigue strength of Category A in air were failing in an aqueous environment at or below the fatigue limit for Category D after over 20 million cycles of loading.

No CFSN data were found in the literature for details having the fatigue strength of categories D, E, and E′.

9.6 WEATHERING AND CORROSION FATIGUE S-N LIFE

Weathering and corrosion fatigue S-N (W&CFSN) data are available from three series of tests of 51 specimens fabricated from A588 steel. In one series, specimens with transverse stiffeners were ideally weathered 8 years and then stress cycled at 0.75 Hz in a 3 percent sodium chloride solution [Albrecht and Sidani, 1987].

In the other two series, W 14×30 rolled beams, as well as welded beams of same cross section as the rolled beams, were weathered 5.5 to 7 years [Albrecht 1988]. The beams were lightly sprayed with a 3 percent sodium chloride solution three times per week during three winter months of every year. They were then stress cycled at 0.75 Hz in a moist salt water environment. The beam tests represent highly corrosive exposures.

Relative to their nonweathered counterparts cycled in air, the loss in stress range was 71 percent for the Category A rolled beams, 56 percent for the Category B welded beams, and 23 percent for the Category C* transverse stiffeners. The data points are plotted in Figure 33 with solid rectangular symbols. Because the beams had corroded severely during the weathering time and were stress cycled in a moist environment, the losses in stress range were greater than those found in the WFSN and CFSN tests.

9.7 RECOMMENDED ALLOWABLE STRESS RANGES

The vast amount of available fatigue test data of various types consistently shows that weathering prior to stress cycling reduces the crack initiation life. In addition, stress cycling in a corrosive environment reduces the crack initiation life, crack propagation life, and the total fatigue life. Accordingly, bare exposed weathering steel bridges must be expected to have lower fatigue strength than painted steel bridges.

Based on a careful analysis of all data, the following reductions in allowable stress range are recommended in this study for bare, exposed weathering steel bridges, depending on the type of detail and the type of environment to which a bridge is subjected.

For environments of medium corrosivity, categories A, B, and C details: 34, 24, and 13 percent, respectively, are recommended; categories D, E, and E′ details, 10 percent.

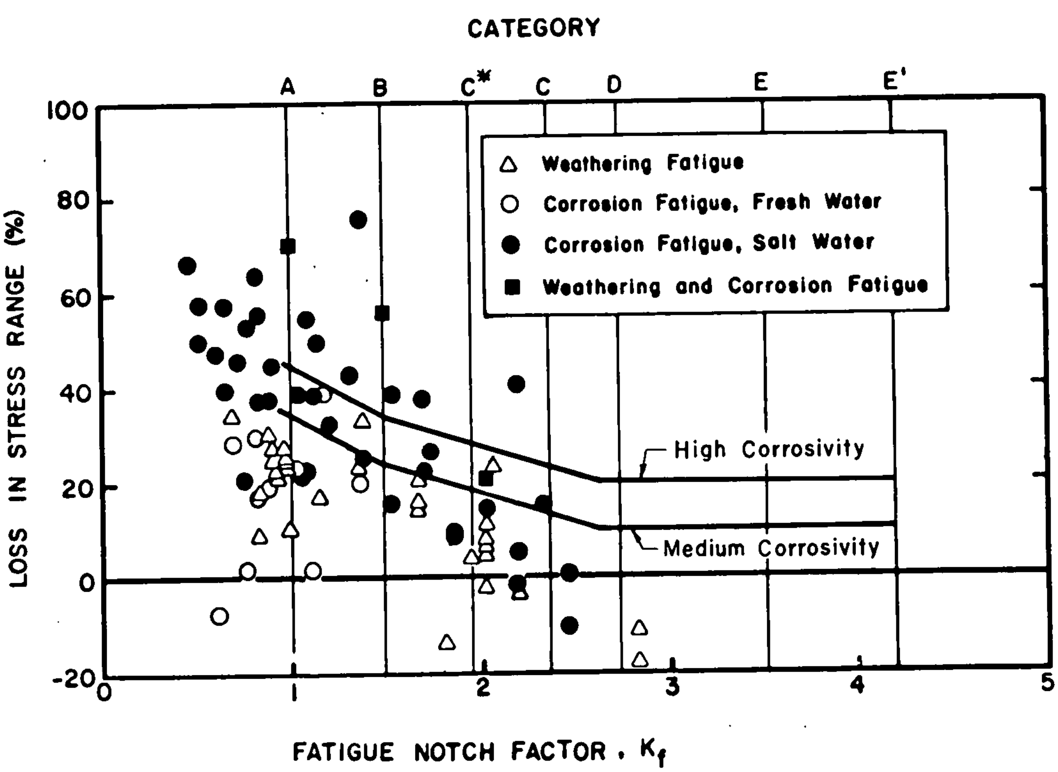

Figure 33. Comparison of loss in stress range with recommended reductions in allowable stress range for bare, exposed weathering steel structures.

For environments of high corrosivity, categories A, B, and C details: 44, 34, and 23 percent, respectively, are recommended; categories D, E, and E′ details: 20 percent.

The recommended reductions are also given in Table 32 and plotted as solid curves in Figure 33. Section 9.1.1 describes the exposure conditions in medium and high corrosivity environments. Applying these reductions to the allowable stress ranges given in Table 10.3.1A of the AASHTO specifications, which were intended for painted steel structures, give the allowable stress range for the fatigue design of bare, exposed weathering steel bridges. The stress ranges need not be reduced when the weathering steel is painted and the paint system is properly maintained.

The curve for medium-corrosivity environments, shown in Figure 33, was taken from Albrecht [1982, 1983] and Albrecht and Naeemi [1984]. It was based on the following observations: (1) reduction of the crack initiation portion of the fatigue life due to the effect of rust pitting, as determined from the 1,565 WFSN tests whose results are represented by the triangular symbols in Figure 33; and (2) reduction of the crack propagation portion of the fatigue life due to the more rapid growth of cracks in aqueous environments than in dry laboratory air, as determined from the 3,254 measurements of CFCP summarized in Figure 31.

The curve for high-corrosivity environments, shown in Figure 33, was obtained by increasing the loss in stress range for medium-corrosivity environments by 10 percent. This increase was based on the following observations: (1) large reduction in the CFCI life in salt water—62 percent loss in stress range for A588 steel specimens—and absence of a CFCI fatigue limit despite strong attempts to characterize the long-life behavior of primary interest for bridges; and (2) much larger loss in stress range for specimens stress cycled in aqueous environments than in dry laboratory air, as determined from the 705 CFSN tests whose results are represented by the circular symbols in Figure 33.

Table 32. Recommended reductions in allowable stress ranges for fatigue design of bare, exposed weathering steel bridges.

Corrosivity of Environment	Reduction in Allowable Stress Range (%)					
	A	B	C	D	E	E'
Medium	34	24	13	10	10	10
High	44	34	23	20	20	20

The recommended reductions in allowable stress range are not applicable to very-high-corrosivity environments (Table 6) in which the long time-of-wetness or high contamination with salt produce severe corrosion that deeply pits the steel and significantly reduces the net section. Ongoing research at the University of Maryland is showing that exposure to very-high-corrosivity environments is reducing the fatigue strength of Category A rolled beams and Category B welded beams fabricated from A588 steel to that of Category E [Albrecht 1988]. These data are represented by the square symbols in Figure 33.

An alternative view of the significance of the data on which these recommendations were based has been presented in Fisher [1988] and Barsom et al. [1988].

CHAPTER TEN

CONSTRUCTION

10.1 HANDLING

Exposed weathering steel must be handled carefully to keep it clean and must be treated with the amount of care required by a finished architectural product. Fabrication, erection, and construction crews must be cautioned that they are handling a finished architectural metal.

The steel members must not be gouged, scratched, or dented during handling. To prevent such damage from occurring, members must be padded in appropriate places, blocked in transit, and handled with slings instead of chains.

Exposed weathering steel must be kept free and clean of all foreign substances, such as grease, oil, mortar and concrete splatter, chalk marks, crayon marks, paint, dirt, both in the shop and on the job site. Natural oxidation of the steel is not considered foreign matter. Cover cloths may be necessary for protection of the material.

The alternative to careful handling, i.e., extensive recleaning of the steel by abrasive blasting or other means after the bridge has been built, can be costly.

10.2 STORAGE

Weathering steel members should be stored under bold exposure or under conditions similar to those that are expected in service. To prevent unsightly, uneven weathering and excessive corrosion, the following conditions should be avoided: storage in transit, open cars, or trucks for an extended period of time; standing water on material in storage; entrapment of moisture; contact between members; contact between members and the ground; contact with chemically treated lumber used for blocking; and contact with any type of foreign matter that can soil the steel surface.

Storage of members in yards or at the job site awaiting erection should be minimized. For short-term storage of 3 to 6 months, all members must be raised off the ground, placed on metal supports, and sloped to allow free drainage of melted snow, rainwater, and dew condensate. Plate girders and rolled beams should be stored with the web in an upright position. When storage space is limited, members may be stacked, provided metal supports separate individual members. Members should not be nestled together nor be bundled.

If it becomes necessary to store the steel outdoors for longer than 6 months, the material must be stored under cover. Individual members should not be nestled or bundled. Instead, they should be separated to prevent trapping water at the contact points.

Bolts and small items such as splice plates should not be stored in drums unless large drain holes in the bottom of the drum prevent water from collecting and keeping the steel wet for long periods. Small items, particularly bolts, stored longer than 6 months should be kept dry.

Plastic sheeting generally is not recommended for covering weathering steel members stored close to the soil because ground moisture may condense on the sheet and drip onto the steel. Furthermore, covers hinder inspection.

Lumber treated to prevent decay and retard fire should not be used in contact with weathering steel unless the lumber is covered with heavy-gage polyethylene sheeting or the lumber and the contacting steel surfaces are painted. Otherwise, the combination of water and salts used in the lumber treatment chemically attacks and consumes the steel. Treated lumber suppliers' recommendations for painting treated lumber must be followed.

Stored weathering steel members must be inspected periodically to ensure that no unsightly, loose, nonprotective oxide is forming on the surface.

10.3 CLEANING STEEL DURING FABRICATION

10.3.1 Purpose of Cleaning

A uniform weathering of steel is desirable in most applications. It is, therefore, necessary at the outset to provide a sound, uniform surface for the foundation of the protective oxide.

The mill scale—a surface oxide that forms on all hot-rolled steel products cooled in air—does not develop the same color and texture as the weathered steel surface. Unless the mill scale is removed, the steel surface will appear mottled, flaky, and nonuniform for several years, depending on the degree of exposure and the aggressiveness of the local environment. Therefore, for aesthetic reasons all surfaces visible to the public should be blast-cleaned.

The mill scale should also be removed when it is possible that parts of or the entire structure may be subjected to lengthy periods of wetness or contaminated with deicing salts. Under these conditions, the less noble weathering steel corrodes galvanically along the numerous cracks in the mill scale and where flecks of scale are missing. This results in corrosion pits that may reduce the fatigue strength of the structural member.

Oil, grease, and other detrimental foreign matter must also be removed because a clean surface is necessary for uniform weathering.

If aesthetic considerations are not of prime importance, such as might be the case for interior members or in rural areas away from public view, and the exposure conditions do not promote galvanic corrosion, weathering steel may be specified with mill scale intact. Of course, in that case the time needed for the steel to develop a uniform appearance will increase from months to several years. The less aggressive the environment, the longer it will take for the mill scale to weather away, and the longer it will take for the steel to develop a uniform oxide coating.

In nonaggressive environments the long-term corrosion properties and the ultimate appearance of weathering steel are not affected by surface cleanliness. Mill scale and certain forms of minor soilage will ultimately weather off naturally on exposed surfaces. However, cleanliness and surface preparation are important where early uniform appearance is desirable.

10.3.2 Blast Cleaning

All surfaces exposed to public view should be blast cleaned to remove the mill scale. Blast cleaning must be performed in accordance with specifications for surface preparation described by the Steel Structures Painting Council, SSPC-SP.

The mill scale on weathering steel is more adherent than that on ordinary steel and its removal will require a greater effort. In trying to remove residues of mill scale the steel surface must not be overblasted.

"Blast-Cleaning to White Metal" (SSPC-SP 5) provides a surface that, when viewed without magnification, is free of all oil, grease, dirt, visible mill scale, corrosion products, paint, or any other foreign matter. It is not generally recommended for cleaning new weathering steel.

"Near-White Blast Cleaning" (SSPC-SP 10) provides a surface that is free of all oil, grease, mill scale, corrosion products, paint, or other foreign matter from the surface except for very light shadows, very light streaks, or slight discoloration caused by rust stain, mill scale oxides, or slight residues of paint or coating that remain. At least 95 percent of each square inch (650 mm^2) of surface area must be free of all visible residues, and the remainder must be limited to the light discoloration mentioned previously. Near-white blast cleaning provides for an intermediate level of surface preparation which is adequate for nearly uniform weathering of surfaces exposed to public view. It costs less than blast cleaning to white metal.

"Commercial Blast Cleaning" (SSPC-SP 6) provides for a surface from which all oil, grease, dirt, rust-scale, and foreign matter have been completely removed from the surface; and all rust, mill scale, and old paint have been completely removed except for slight shadows, streaks, discolorations caused by rust stain, mill scale oxides, or slight residues of paint or coating. If the surface is pitted, slight residues of rust or paint may be found in the bottom of the pits. At least two-thirds of each square inch (650 mm^2) of surface area must be free of all visible residues and the remainder must be limited to the light discoloration, slight staining, or slight residues mentioned above.

Commercial blast cleaning removes practically all rust, mill scale, and other detrimental matter from the surface. The surface will not necessarily be uniform in color, nor will all surfaces be uniformly clean. It is recommended for surfaces not visible to the public, such as those of the interior members of a bridge for which a somewhat uniform weathering of the fully exposed steel is sufficient. Commercial blast cleaning costs only about one-third of blast cleaning to white metal.

In some instances it may be necessary to remove heavy coatings of oil or grease with a suitable solvent prior to blast cleaning, in accordance with the specification SSPC-SP 1 "No. 1 Solvent Cleaning." Removal of oil and grease permits more efficient blast cleaning.

10.3.3 Pickling

Pickling (SSPC-SP 8) is used to remove mill scale from cold-rolled products (sheet and strip) during the normal sequence of production at the steel mill. It is often preferred for cleaning slender, light weight structural elements. It is not recommended nor is it considered practical for bridge members. When used, however, overpickling should be avoided, and the pickled material must be thoroughly rinsed to remove all traces of pickling acid.

10.3.4 Welded Areas

Welding flux, slag, and spatter should be removed from all exposed welds by power grinding to SSPC-SP3 "No. 3 Power Tool Cleaning" or by commercial blast cleaning to SSPC-SP6. Care should be taken when grinding to avoid the formation of a heat-scale which retards the weathering process. Discoloration of the surface due to additional heating and welding after blast cleaning should be removed if uniform weathering is desired.

10.3.5 Effectiveness of Cleaning Procedure

The effectiveness of the selected cleaning procedure may be evaluated by wetting the surface of the steel. In approximately 24 hours after wetting, the steel will show a uniform, light-yellow oxide on all areas that are free of mill scale and other foreign matter. Those areas covered with mill scale will retain the hard bluish-gray appearance of the mill scale. Areas that are still covered with residual oil retain the appearance of the original steel or they may appear somewhat mottled.

10.4 FINAL CLEANING OF STEEL

10.4.1 General Remarks

Any foreign materials that adhere to the steel and inhibit the formation of the protective oxide must be removed soon after erection. Some materials, such as mud and concrete dust, will normally be displaced by the corrosion products during the natural weathering process.

Guidelines for removing identification marks, soluble soilage, and nonsoluble soilage are presented in the following sections. The specifications for surface preparation issued by the Steel Structures Painting Council should be followed when cleaning steel.

10.4.2 Identification Marks

Purchase orders for weathering steel must specify that the mill and the fabricator mark or identify structural members with metal tags, soapstone (tailor's chalk), or some other readily removable material.

Paint or wax-based crayons should not be used for marking because these materials do not wash or weather away even after many years of exposure. If they are used, these markings must be removed during the final cleaning operation or should be located where they are not visible to the public.

The familiar wax-based carpenter's crayon, for example, permanently marks the steel. Any attempt to remove such markings by grinding or power wire brushing only softens and spreads the wax over a wider area. Instead, the crayon marking should be removed by applying some household abrasive cleaner and the surface wiped with a clean rag or scrubbed with a stiff bristle brush wetted in 82°C (180°F) water. The hot water softens the wax, enabling the abrasive to remove the pigment and the wax.

10.4.3 Soluble Deposits

Oil, grease, cutting compounds, salts, and other soluble contaminants can be removed from steel surfaces by solvent cleaning, according to the requirements of the SSPC-SP specification "No. 1 Solvent Cleaning."

Prior to solvent cleaning, foreign matter (other than grease, oil and salts) must be removed by one or a combination of the following: abrading, scraping, brushing with a stiff fiber or wire brush, or cleaning with solutions of appropriate cleaners, provided such cleaning is followed by a fresh water rinse.

First, heavy oil or grease is removed with a scraper or a clean rag. Care should be taken not to spread the deposit over a wider area. The remaining oil and grease are then removed by wiping or scrubbing the surface with rags or brushes soaked in solvent. Clean solvent and clean rags or brushes should be used for the final wiping. Emulsion or alkaline cleaners may also be used. In the latter case, after treatment, the surface should be washed with fresh water or steam to remove detrimental residues. A third method is to steam-detergent-clean the soiled surface followed by steam or fresh water rinsing to remove detrimental residues.

Petroleum-base mineral spirits with a minimum flash point of 38°C (100°F) or Stoddard Solvent is recommended as the general purpose solvent for cleaning when the ambient temperature is lower than 26°C (80°F). High-flash mineral spirits with a minimum flash point of 49°C (120°F) should be used at temperatures between 26 and 35°C (80 to 95°F). Paint thinner may be used when greater solvency is required. These materials may have a low flash point and, therefore, workers must protect themselves against incidence of fire.

An effective alkaline cleaner can be prepared with about 56 g (2 oz.) of trisodiumphosphate and some soap or household detergent dissolved in 3.8 liters (1 gallon) of water. After removing the deposit, the surface should be thoroughly rinsed to prevent white staining from the phosphate or any residual alkalinity.

Minor contamination of the steel surface with salt and other soluble compounds during the early stages of exposure can be removed by fresh water hosing. More severe contamination requires steam cleaning, using detergents and cleaners, followed by a steam or fresh water rinse to remove detrimental residues.

10.4.4 Nonsoluble Deposits

Loose mill scale, loose rust, loose paint (markings), rust scale, weld slag, and other loose detrimental foreign matter can be removed from steel surfaces by hand cleaning in accordance with the requirements of the SSPC-SP specification "No. 2 Hand Tool Cleaning." This process is not intended for removal of adherent mill scale, rust, and paint. Mill scale, rust, and paint are considered adherent if they cannot be removed with a dull putty knife.

Suitable methods of hand cleaning consist of using impact hand tools to remove rust scale and weld slag; and hand wire brushing, hand abrading, hand scraping, or other similar nonimpact methods to remove loose mill scale, nonadherent rust, and loose paint.

Where the deposit is too difficult to be removed by hand cleaning, the soiled areas should be cleaned in accordance with the requirements of the SSPC-SP specification "No. 3 Power Tool Cleaning" or "No. 7 Brush-off Blast Cleaning."

Suitable methods of power cleaning consist of using rotary or impact power tools to remove rust scale and weld slag; and power wire brushing, power abrading, power impact, or other power rotary tools to remove loose mill scale, nonadherent rust, and loose paint. Power tools must be operated in a manner that prevents the formation of burrs, sharp ridges, sharp cuts, or heat scale that retards the weathering process.

Brush-off blast cleaning is a method of preparing steel surfaces which, when viewed without magnification, must be free of all visible oil, grease, dirt, dust, loose mill scale, loose rust, and loose paint. Tightly adherent scale, rust, and paint may remain on the surface.

All visible oil, grease, soluble welding residues, and salts must be removed by the methods outlined in the SSPC-SP "No. 1 Solvent Cleaning" before the surface is cleaned with a hand tool, power tool, or brush-off blast.

10.4.5 Acid Cleaning

The use of acids to remove scale and stains in the field must not be permitted because acids are difficult to neutralize and their residues will severely pit the steel in joints and crevices.

10.5 CONCRETE PROTECTION

10.5.1 Rust Staining

Water draining over the steel and moisture dripping from the steel contain suspended particles of insoluble iron oxide. As this water runs over concrete piers and abutments and then evaporates, the residual oxide stains and streaks the concrete.

Rust staining is most severe during the initial years of exposure. Although the rate at which weathering steel releases oxide particles tends to decrease with the length of exposure, the rust particles accumulate on the concrete surfaces and the rust staining gets worse with time. The problem is aggravated by frequent rainfall during the early months of exposure and also by windy exposures in which the runoff water dripping from the structure is blown against the concrete surfaces.

10.5.2 During Construction

During construction, so long as the deck is not in place, water running over the weathering steel superstructure loosens oxide particles onto the piers and abutments where it stains the concrete. To prevent staining during this period, all affected concrete surfaces should be draped, wrapped, or otherwise sheltered with a heavy-gage polyethylene sheeting capable of resisting tearing by wind gusts and construction operations. Once the deck is in place and a system is installed to carry away the rust-laden water, the plastic sheets can be removed.

10.5.3 After Construction

After construction and during the service life of the bridge, the concrete surfaces continue to be rust stained for an indefinite period because of water running or dripping from the girders to the piers and abutments. To reduce the penetration by rust stain, the concrete can be coated with liquid silicone-based sealers or other proven proprietary formulations. Although the coating reduces penetration of the rust particles, the concrete surfaces still stain but to a lesser degree.

Designers must understand that the treatment in some cases may discolor the concrete surface. Furthermore, such coatings may break down with time and have to be reapplied to continue protection.

10.5.4 Stain Removal from Concrete

If the protective measures do not prove adequate, the rust stains can be removed from the concrete surfaces by using proprietary chemical stain removers or, if the stained areas are large, by abrasive blast cleaning.

Mild abrasive cleaners are applied to the stained surface for longer periods of time. They react with the iron by complexing it and are then washed or scrubbed away without dissolving any concrete.

Stain removers based on hydrochloric acid or phosphoric acid are applied to the concrete surface for 10 to 20 minutes and then scrubbed off with a bristle brush. The acid attacks the concrete by destroying a thin layer together with the deposited rust particles. Acid solutions should never be allowed to drain over the steel because the drainage products can attack not only the protective oxide but the steel itself.

Chemical stain removers must be handled with proper safety precautions. The manufacturer's recommendations should be followed.

CHAPTER ELEVEN

INSPECTION AND MAINTENANCE

11.1 METHOD OF INSPECTION

An effective inspection and maintenance program is essential to ensuring that a weathering steel bridge reaches its design life. The "Bridge Inspector's Training Manual" [Federal Highway Administration, 1979], which offers a uniform set of instructions for the training of bridge inspectors, can serve as a guide for inspecting weathering steel bridges. Although it is not comprehensive, it nevertheless outlines the principal parts of the bridge that require inspection. This chapter is intended to supplement Manual 70 with additional recommendations for inspecting corrosion damage on weathering steel bridges.

The inspection of weathering steel bridges differs from and is more difficult than the inspection of painted ordinary steel bridges. Unlike painted structures where rust is undesirable and its appearance serves as a warning of incipient paint failure, in weathering steel bridges the entire structure is covered with rust. In the latter case the inspector must learn to distinguish between a protective and nonprotective oxide coating. The inspection is also more difficult because in a painted steel bridge the rust stands out from a distance against the color of the paint, whereas in a weathering steel bridge the nonprotective rust can often be detected only at close range, say, 1-m (3 ft) distance. The need to get within reach increases the cost of inspecting a weathering steel bridge.

Before the inspection the inspector should have access to the as-built drawings or, in their absence, to the design drawings. A review of the drawings helps to identify areas that may corrode excessively.

At present, numerous types of other structures are being erected from both A242 and A588 steels in various parts of the country. They are in the form of guardrails, light posts, buildings, and transmission towers. As part of his training an inspector should visit some of these structures before embarking on a bridge inspection, particularly when these structures are located near the site of a weathering steel bridge.

Weathering steel bridges must be inspected every 2 years as is required for painted bridges.

11.2 CONDITION OF OXIDE FILM

The appearance of the oxide film (color and texture) may indicate the degree to which the oxide that forms on weathering steel is protective. However, the visual appearance alone can be deceptive. Therefore, the oxide film must be tested by tapping with a hammer and by vigorously wire brushing to determine whether it still adheres to the substrate or has debonded in the form of granules, flakes, or laminar sheets. When its exact condition cannot be ascertained, the oxide film should be removed by blast cleaning to determine the extent of pitting and to measure section loss with a caliper. Alternatively, the corroded surface can be partially ground to measure the section loss with an ultrasonic thickness gage.

Tables 33 and 34 summarize the characteristic colors and textures of the oxide film as well as the condition of which the appearance is indicative. Because, for corrosion protection, the weathering steel bridge depends on the uniformity and continuity of the oxide film, the inspector must be familiar with the various colors, textures, and general appearance that the oxide film assumes when exposed to different macro- and microenvironments. The relationship between appearance and condition is described in the following sections.

11.2.1 Protective Oxide Film

The protective oxide film on a weathering steel structure requires from 3 to 5 years to stabilize in an urban environment where there is some industrial activity.

Because of the progressive change in the oxide film, the appearance at the first biannual inspection can vary, depending on the location of the structure. Optimum film appearance—color and texture—develops most rapidly in industrial-urban locations where plants emit sulfur oxides into the atmosphere. The color changes from an early yellow-orange to a light brown. The texture at this point can be termed "dusty" as loose oxide particles can be easily rubbed off by hand. By the second biannual inspection the dusty character changes to where little can be rubbed off and a smoother surface is becoming apparent.

Eventually the color approaches a chocolate brown and under certain lighting conditions appears metallic gray to purple. This condition is typical of a structural member boldly exposed to the weather, such as the outside of a facia girder.

The interior structural members of the bridge are likely to exhibit a color similar to that of the exposed surface. However, when the steel is sheltered from rain and wind, the initial corrosion products do not readily dust away or become dislodged. Hence, the texture of the oxide remains somewhat granular, suggesting a coarse finish.

In rural areas where the sulfur oxide levels for the most part are relatively low, the color of the oxide remains light and the texture stays dusty for a longer period of time. However, eventually, the more tightly adherent chocolate brown oxide begins to form.

In all instances the greater the extent of alloying, the darker is the color. The absence of nickel, and varying levels of nickel and chromium, can lighten the color of the oxide film. However, unless the respective weathering steels are adjacent to one another, it is difficult to state with certainty as to what degree a particular grade of steel is alloyed. The A588 and copper-bearing steels greatly differ in color and texture.

Vigorous wire brushing of a protective oxide film merely scratches the surface, exposing a light colored substrate that rapidly darkens again as the steel reoxidizes. Wire brushing does not damage the weathering steel.

Table 33. Color of oxide.

Color	Condition
Yellow orange	Initial stage of exposure
Light brown	Early stage of exposure
Chocolate brown to purple brown	Development of protective oxide
Black	Nonprotective oxide

Table 34. Texture of oxide.

Texture	Condition
Tightly adherent, capable of withstanding hammering or vigorous wire brushing	Protective oxide
Dusty	Early stages of exposure; should change after a few years
Granular	Possible indication of problem, depending on length of exposure and location of structure
Small flakes, 6 mm (1/4 in.) diameter	Initial indication of nonprotective oxide
Large flakes, 12 mm (1/2 in.) diameter or greater	Nonprotective oxide
Laminar sheets or nodules	Nonprotective oxide, severe corrosion

11.2.2 Nonprotective Oxide Film

Weathering steel does not form a protective oxide film when the time-of-wetness is long or when the steel is contaminated with salts. The texture of the nonprotective oxide is typically laminar or granular.

Slab Rust. A laminar oxide forms when the steel is repeatedly subjected to a long period of rain or dampness followed by a dry season. A lamina of nonprotective oxide grows during the wet season, dehydrates and shrinks from the base metal during the dry season, and leaves a minute invisible space adjacent to the base metal that permits the steel to oxidize again during the next wet season. As these long cycles of wetting and drying are repeated and the laminae build up, a slab of up to 12 mm (½ in.) thickness forms, with each lamina corresponding to one long wet-dry cycle.

Assuming laminar rust slabs did not fall off or were not removed, the actual member section loss can range from one-fifth to one-twentieth of the slab thickness because the rust is more voluminous than the steel from which it was converted. This condition generally occurs on horizontal surfaces where the pitch is such that any moisture or water flow that reaches the area is retained until it dries.

Another form of slab rust consisting of thin and fragile sheets is found on vertical surfaces. It generally occurs on the webs of girders that, likewise, experience periods of dampness either from water flow or nightly dew formation, and are so sheltered that little ventilation or breeze is present to dry the surface or dislodge the resulting oxide. The web can also become wet from moisture that is retained on top of the bottom flange, because of an accumulation of dust and dirt, and by capillary action wicks up the web a distance of several inches, carrying with it any salts which the water may have leached from the accumulated debris. This results in a light colored streak at the top of the wick area. Because the presence of salt and moisture causes severe scaling, the oxide in this area is loosely attached to the base metal and is nonuniform in color relative to the texture and appearance of the oxide film higher up on the web.

Granular and Flaky Rust. When the structure is frequently dampened by salt fog, exposed to sea breezes, or subjected to a fine mist created by high-speed truck traffic on highways where deicing salts are used, the texture of the oxide film is very coarse. In these cases the steel surface is covered with large granules of about 3 mm (⅛ in.) diameter and flakes of up to 12 mm (½ in.) diameter, depending on the severity of the environment. The granules and flakes fall off in a shower of rust particles when the steel surface is wire brushed or even hand rubbed. Removal of the rust may reveal a black substrate of magnetite or magnetic oxide—an unstable condition indicative of an environment in which the steel cannot form a protective oxide film.

The steel surface below a granular and flaky oxide film is typically pitted. Large oxide nodules are indicative of severe pitting.

Under the aforementioned conditions of granular and flaky rust, the color of the oxide slowly changes from yellow-orange to maroon, retaining a reddish-brown hue even after being exposed for many years. Again, the structural members not boldly exposed to the weather exhibit a coarser texture and take longer to develop a darker color.

11.2.3 Analysis of Contaminants

Atmospheric contaminants and salts are trapped in the oxide film and can foster corrosion of steel, depending on their level of concentration. Of particular concern are the salts, from any source, whose concentration is highest on the metal-oxide interface. Samples of debris lying on the surface and samples of nonprotective oxide taken as close to the steel surface as is practical should be chemically analyzed for type and concentration of contaminants. With regard to salts, in several states analysis of rust samples taken from poorly performing weathering steel bridges have shown salt concentrations of 0.10 percent or even greater.

11.3 CONDITION OF STEEL

When the appearance (color and texture) suggests that the oxide film is nonprotective, the condition of the underlying steel base should be checked by blast cleaning the surface to bare metal at selected locations so as to qualitatively determine the extent of corrosion and pitting. The uniform corrosion penetration and the pit depth should then be measured as needed to determine the degree of section loss.

11.3.1 Corrosion Penetration

An ultrasonic thickness gage and an electrically powered disk grinder are needed to measure the corrosion penetration. Rechargeable grinders do not have sufficient power and are not recommended. The corrosion penetration per side is measured as follows [McCrum et al., 1985]:

1. Grind the oxide off a small area on one side of the plate until the bare metal is exposed only on the highest points of the corroded surface, leaving any depressions filled with oxide. Approximately 30 percent of the ground surface will have metallic appearance.
2. Move the probe of the ultrasonic thickness gage around a ground area of 19 mm (¾ in.) diameter and retain the smallest reading. This peak-to-valley reading is a reasonable estimate of the plate thickness.
3. Subtract the measured plate thickness from the original plate thickness specified on the as-built drawings and divide the difference by two to obtain the mean corrosion penetration per side.

The ultrasonic thickness gage measures the distance between the peaks on the probe side and the valleys on the opposite side of the plate. This peak-to-valley measurement provides a good estimate of the effective plate thickness when both sides are equally pitted. The probe side must be ground so that the probe makes good contact with the steel surface. The dense oxide has about the same wave transmission speed as the steel and, hence, does not give the erroneous high readings that can result when coupling agents of low wave transmission speed fill the pits of a blast-cleaned surface.

Two uncertainties may affect the accuracy of the peak-to-valley reading. On the probe side of the plate a small amount of the peak is ground off. On the opposite side of the plate the ultrasound waves are reflected by the metal-oxide interface. But, if the oxide film is dense, the waves may penetrate the inner layers of the film, in which case part of the film is included in the reading. These inaccuracies tend to cancel each other out. Indeed, the peak-to-valley reading was found to be a good measure of the plate thickness [McCrum et al., 1985].

Ultrasonic thickness gages should be accurate to within ± 0.025 mm (±2.5 mil). Their accuracy should be periodically verified by measuring the known thickness of a test block.

Users wishing to monitor the corrosion penetration of weathering steel bridges during their service life should measure the initial thickness of the critical members after fabrication. This is advisable because a 50-mm (2 in.) thick flange plate, for example, could be as much as 2.3 mm (90 mil) over and 0.25 mm (10 mil) under thickness, for a total possible variation of 2.5 mm (100 mil). For web plates the variation could be 1.0 mm (40 mil). Variations in thickness are typically smaller for shapes than for plates because steel producers sell shapes by linear foot and plates by weight.

11.3.2 Pit Depth

The pit depth can be measured with an ultrasonic thickness gage as described below. The first two steps are the same as those given in the section on measuring corrosion penetration.

1. Grind the oxide off a small area on one side of the plate until the bare metal just shows on the highest peaks.
2. Measure the thickness with the ultrasonic thickness gage. Move the probe around and retain the smallest reading. This represents the peak-to-valley thickness.
3. Continue to grind the area until only traces of oxide remain.
4. Measure again the thickness with the ultrasonic gage, retaining the smallest reading. This represents the valley-to-valley thickness.
5. Subtract the valley-to-valley thickness (step 4) from the peak-to-valley thickness (step 2) to obtain the pit depth.

Alternatively, pit depths can be measured with a depth gage after blast cleaning the surface to bare metal. In addition to measuring the pit depth, record the estimated frequency of occurrence and the diameter of the pits.

11.4 CONDITION OF STRUCTURE

In addition to inspecting the oxide and the steel, the inspector should also examine the joints, deck, abutments, and substructure. The condition of these elements can provide information helpful in determining the causes and severity of the corrosion problem.

11.5 AREAS OF INSPECTION

For the principal signs of existing or impending distress, the inspector should take note of the appearance of a nonprotective oxide film as described in section 11.2; accumulation on horizontal surfaces and sheltered corners of windblown dust, debris, and oxide particles shed mainly during the early years of weathering; water streaks and drain patterns on vertical, sloped, and any other surfaces that conduct drain water; leaking expansion joints; and rust packout in crevices. Table 35 gives examples of

Table 35. Visual signs of distress on weathering steel bridges and their probable causes.

Structural Member	Nature of Distress	Probable Cause
1. Abutments and piers	Rust stains on walls and piers.	Leaking expansion joints.
	Rust stains on grade slabs.	Dew droplets falling from superstructure. Inadequate ventilation. Clogged and overflowing drains.
	Accumulation of dust and debris on abutments and pier caps.	Windblown dust, and bird nests and excrement.
	Cracks in abutments.	Frozen hanger and pin connection.
	Crevice corrosion where girder penetrates concrete abutment.	Shrinkage of concrete and formation of crevices.
2. Bearings	Accumulation of moisture, dust and debris on masonry plate; development of lamellar rust slab.	Confined area, inadequate ventilation, leaking expansion joint.
3. Girders	General and lamellar scaling above traffic lanes.	Traffic spray.
	Accumulation on bottom flange of rust particles falling from web and development of lamellar sheets of rust.	Damp for extended periods; salt contamination; water draining through leaking joint and flowing along flange.
	Scale on lower 150 mm (6 in.) of web.	Wicking of moisture that accumulated on top of bottom flange.
	Accumulation of windblown debris at joint formed by stiffener, flange and web.	Entrapment and retention at re-entrant corners.
4. Floor beams and stringers	Granular oxide film.	Traffic spray often containing deicing salts.
	Lamellar corrosion near expansion joints.	Drainage from leaking expansion joints.
5. Box girders	Accumulation of water inside box girder.	End diaphragms not completely sealed. Hatch doors not well sealed with gasket. Failure to seal small openings by caulking.
	Presence of birds nests.	Missing wire screen over vent holes; open hatches.
6. Trusses	Accumulation of debris and moisture at truss joints.	Entrapment at pockets formed by gusset plates and truss members.
7. Lateral bracing	Accumulation of debris and moisture on horizontal gusset plate and open faced angle and tee-braces.	Entrapment and retention at re-entrant corners.
	Crevice corrosion between intersecting braces and gusset plate, and gusset plate and web.	Existence of crevices; excessive bolt spacings; non-continuous weld.
8. Diaphragms	May experience same problems as lateral bracing.	Same as for lateral bracing.
9. Bolts, welds, and hangers	Crevice corrosion and packout at bolted joints.	Widely spaced bolts and large edge distances.
	Accelerated corrosion of bolts and welds.	Galvanic corrosion between weathering steel member and carbon steel welds and bolts.
	Crevice corrosion and packout at welded joints.	Member not welded all around, thus permitting entry of moisture; intermittent welds.
	Corrosion between hanger and washer in hanger plate and pin connection assembly.	Crevice and galvanic corrosion enhanced by salt-laden water runoff through leaking expansion joint.
10. Metal accessories	General corrosion and rust pack out at support brackets for utility services, sign structures and railings.	Galvanic and crevice corrosion; inadvertent use of unpainted carbon steel.
11. Sealed joints	Leaking joints, torn seals, missing seals, failure of seal bond at steel seat angle.	Dirt intrusion, deterioration of adhesives, failed field splice of seal, snow plow damage.
12. Concrete substructure and pavement	Rust stains.	Water runoff and drippage from overhead members.
13. Mill scale	Pitting along breaks in the mill scale.	Galvanic corrosion, mainly in presence of salt.

Table 36. Inspection form for corrosion damage of weathering steel girder bridges.

STRUCTURE: .... DATE ....

LOCATION .... TYPE .... SPANS .... LENGTH ....

YEAR BUILT .... CLEARANCE .... URBAN .... RURAL ....

TRAFFIC OVER (...........mph) UNDER (..........mph)

OXIDE CONDITION

COMPONENT		RUST COLOR	OXIDE TEXTURE	COMMENTS	DEBRIS TYPE	DEBRIS AMOUNT
FACIA GIRDER	B OF T FL					
	WEB					
	T OF B FL					
	B OF B FL					
INT. GIRDER TRAFFIC SIDE	B OF T FL					
	WEB					
	T OF B FL					
	B OF B FL					
INT. GIRDER OPPS. SIDE	B OF T FL					
	WEB					
	T OF B FL					
	B OF B FL					
SPECIAL AREAS	JOINTS					
	DIAPHRAGM					
	ABUTMENTS					
	OTHER					

STEEL CONDITION

COMPONENT		PITTING DEPTH	PITTING FREQUENCY	MILL SCALE	SECTION LOSS	
FACIA	B OF T FL					
	WEB					
	T OF B FL					
	B OF B FL					
INT.	B OF T FL					
	WEB					
	T OF B FL					
	B OF B FL					
SPECI AREAS	JOINTS					
	DIAPHRAGM					
	ABUTMENTS					
	OTHER					

STRUCTURE CONDITION

COMPONENT	CONDITION	COMMENTS	
JOINTS			
DECK			
ABUTMENTS			

B OF T FL = BOTTOM OF TOP FLANGE, T OF B FL = TOP OF BOTTOM FLANGE,
B OF B FL = BOTTOM OF BOTTOM FLANGE, INT. = INTERIOR, OPPS = OPPOSING

Notes:

visual signs of distress found on weathering steel bridges and their probable causes. It serves as an inspection aid.

Table 36 is a sample form for inspection of corrosion damage on a weathering steel girder bridge. Similar forms that enumerate the major problem areas can be prepared for truss and box girder bridges. The form consists of three parts in which the condition of the oxide, steel, and structure are recorded as discussed in sections 11.2 to 11.4. The condition of the oxide and steel should be determined separately for the facia girder, the sides of interior girders facing the oncoming traffic and those opposite to the oncoming traffic, and other specific areas. In each case the inspector should record the condition of the bottom of the top flange, the web, the top of the bottom flange, and the bottom of the bottom flange, as each of these areas is exposed differently. Special attention should be given to the first interior girder above oncoming traffic.

As the weathering process, as well as the process of deterioration, gradually change with time, a file of color photographs should be kept so that changes in the appearance of the rust can be noted between subsequent biannual inspections. The record-keeping suggested in Chapter III of Manual 70 should be followed for this purpose.

11.6 MAINTENANCE

Weathering steel is not a maintenance-free material. Experience has shown that highway bridges by their nature and use accumulate much debris, become wet from condensation, leaky joints and traffic spray, and are exposed to salts and atmospheric pollutants. Different combinations of these factors may create exposure conditions under which the weathering steels cannot form a protective oxide coating. Therefore, the bridges must be maintained properly.

The following examples illustrate the type of periodic maintenance that may be needed:

- Remove loose debris with a jet of compressed air or vacuum cleaning equipment.
- Scrape off sheets of rust.
- High-pressure hose wet debris and aggressive agents from the steel surfaces, particularly where the surfaces are contaminated with salt.
- Trace leaks to their sources by inspecting the bridge on rainy days or by hosing the top of the deck near expansion joints and observing drainage lines. Repair all leaky joints.
- Install drainage systems, drip plates, and deflector plates that divert runoff water away from the superstructure and abutments.
- Clean drains and downspouts.
- Epoxy inject or seal weld all crevices such as those occurring at widely spaced bolt patterns and discontinuous welds.
- Remedially paint areas of excessive corrosion as recommended in Chapter Twelve.

Some maintenance procedures may only be partially effective. For example, plates welded to the bottom flange may cause runoff water to drip before it can run down bearings and concrete piers, but it does not stop all moisture from migrating past the drip plate (Fig. 26). High-pressure hosing removes debris and washes contaminants from the surface, but it cannot remove salt trapped at the steel-oxide interface and in the rust pits.

11.7 CRACK DETECTION

The visual detection of fatigue cracks in painted steel structures is made easier by the color contrast between the rust stains and the paint along the crack as well as the streaks of moisture-laden rust oozing from the crack. This advantage is absent in weathering steel structures. Observations of crack growth in fatigue tests of weathered steel beams stress cycled in air or in a moist environment showed that fatigue cracks less than, say, 150 mm (6 in.) long are very difficult to find on visual inspection [Albrecht 1988]. The shortest crack length that can be visually detected is likely to be even greater in bridges because the crack forms a crevice that completely fills with rust during the long service exposure.

At present other methods for detecting cracks in weathering steel bridges, such as ultrasonic testing, acoustic emission, and radiography, have not been studied.

CHAPTER TWELVE

REHABILITATION

12.1 INTRODUCTION

Weathering steel bridges that are undergoing a high degree of uniform or local corrosion must be remedially painted to protect the affected areas against further section loss.

Remedial painting of weathering steel bridges, like repainting of ordinary steel bridges, involves the following steps: (1) selecting a good coating system, (2) developing sound and practical specifications, (3) estimating the cost of painting, (4) inspection during painting by an accredited inspector, and (5) proper maintenance after the painting is completed.

The following sections discuss each of the aforementioned steps. Emphasis has been laid on differences between remedial painting of a weathered A588 steel bridge and that of repainting an ordinary steel bridge whose paint system has failed. Recommendations for the rehabilitation of crevices are summarized in section 12.7.

12.2 COATING SYSTEM

12.2.1 Selection of Coating Systems

The process of selecting a coating system is the same for all structures whether fabricated from ordinary or weathering steels. This section outlines the specific selection process for severely corroded weathering steel bridges that typically have rough and deeply pitted surfaces, making it impossible to get rid of all visible rust and contaminants from the pits.

For effective and cost-efficient corrosion control, it is important to carefully select a coating system whose properties meet the special requirements of a weathering steel bridge.

The first step is to evaluate the structure and its environment. This includes gathering information on age, type, and length of the bridge; traffic characteristics; factors that lead to corrosion; environmental conditions that contribute to the corrosion process such as time-of-wetness, salt contamination, proximity to a body of water; and expected service life of the coating system.

The second step is to match the properties of a coating system—known from field experience, laboratory testing, and theory—to the condition of the bridge. Among the most desirable properties are: (1) the paint should be able to tolerate large dry film thickness variations caused by the rough surface of the steel substrate; (2) be insensitive to residues of rust and chemical contaminants that are practically and economically impossible to remove from the numerous pits; and (3) have a low water vapor transmission rate to prevent osmotic blistering of the film.

Some systems can be eliminated from the onset, especially if they are sensitive to surface contaminants and require a very clean, rust-free substrate below the film. Also, if they have a high water vapor transmission rate, they may cause the paint film to blister rapidly.

12.2.2 Surface Preparation

While preparing weathering steel bridges for remedial painting, contractors have found surface roughness of typically 100 μm to 200 μm (4 to 8 mil) or even higher. The surface is also covered with numerous pits of small diameter and depths varying from 125 μm to 375 μm (5 to 15 mil). The pits usually contain chemical contaminants such as salt from various sources.

The roughness of the surface and the presence of numerous pits make it practically impossible to remove all visible rust products. Some rust products often remain in the bottom of the pits even after the surface has been thoroughly cleaned. Because specifications for painting steel structures usually require near-white blast cleaning of surfaces (SSPC-10), meaning removal of all visible rust products, the difficulty in achieving this condition must be addressed in the specification. One method is to specify that the surface meet the requirements of the SSPC visual standards at a viewing distance of 0.6 m (2 ft), deleting the verbal description of a near-white blast.

The roughness of the surface makes it difficult to determine the quantity and thickness of the primer. A great deal of primer is required to fill the profile. Experience with remedially painted weathering steel bridges has shown that one gallon of primer covers only about one-fourth of the area given in the manufacturer's product data sheet.

To ensure that the primer has the desired thickness, the dry film thickness gage should be calibrated on the surface to be coated, or the contribution of the surface roughness should be determined and subtracted from the reading. (See SSPC-PA-2, Guide to Calibration of Dry Film Thickness Gauges.)

12.2.3 Performance

Little information is available on the performance of coating systems over weathered A588 steel substrates. In the absence of such information, coating systems are often recommended based on their performance over new weathering steel, new carbon steel, or old carbon steel. While this information is helpful, good performance over these substrates does not necessarily ensure good performance over weathered A588.

Only three publications were found on remedial painting of weathered A588 steel [Tinklenberg 1982; Raska 1986; Keane et al., 1983]. These studies are briefly reviewed below.

The work at the Michigan Department of State Highways and Transportation was done with panels cut from hanger plates that had been removed from severely corroded joints of weathering steel bridges in Metropolitan Detroit [Tinklenberg 1982]. In addition to these panels, sets of new A588 and A36 steel panels were also tested for comparison. The surfaces were prepared in five different ways, two of which involved a water wash. One of the five was a gradient blast, with the panels divided into one-quarter areas and the separate areas blasted to the requirements of specifications SSPC-5 (white metal), SSPC-10 (near-white), SSPC-6 (commercial), and SSPC-7 (brush-off). One of the other two dry-blasted panels was exposed to 100 percent humidity for 10 minutes during which time a "green mold" formed on the surface. This blue-green discoloration after blasting had been observed in the field and was described in Culp and Tinklenberg [1980].

The panels were then coated with ten paint systems of seven generic types. All panels were exposed 3,000 hours in a salt fog cabinet, according to the requirement of ASTM specification B-117. The following observations were made:

- Washing had little effect on the performance of paint systems.
- The "green mold phenomenon" had little effect on paint performance.
- Blistering was most severe on the brush-off (SSPC-7) and commercial (SSPC-6) grade blasted areas.
- The generic paint systems that were applied to the panels fabricated from the bridge hanger plates ranked as follows:

Ranking	Point System
Best	Multi-component organic zinc-rich
•	Single-component organic zinc-rich
•	Single-component inorganic zinc-rich
•	Moisture-cured urethane
•	Epoxy primer
•	Chlorinated rubber
Poorest	Alkyd systems

Similar tests described in Culp and Tinklenberg [1980] showed that:

- Shop-coated A588 and A36 steels performed alike in a moist chloride environment (salt fog).
- Paint systems had shorter lives on remedially painted A588 steel than on shop-coated steel, even when the panels were ideally exposed after painting.
- It is possible to get good corrosion protection on A588 steel by careful preparation and application. This was also found for steel exposed to a leaking joint environment.
- The surface roughness of naturally weathered A588 steel cannot be reliably duplicated in the laboratory in accelerated salt exposure tests.

As part of ongoing work at the Michigan Department of State Highways and Transportation, the performance of many coating systems is being evaluated in a battery of accelerated tests. The initial results are confirming the findings of previous studies [Culp and Tinklenberg, 1980; Tinklenberg 1982].

In the second study on the performance of weathered and remedially painted A588 steel, 150 mm by 300 mm (6 in. by 12 in.) panels were exposed for 6 months atop a bridge pier near the Gulf Coast in Southern Texas [Raska 1986]. Thereafter, the panels were cleaned by four different methods. One method consisted only of blast cleaning, while the other three involved the following combinations of washing and blast cleaning: (1) brush-off blast cleaning, followed by water blasting, and then near-white blast cleaning; (2) water blasting, followed by near-white blast cleaning; and (3) brush-off blast cleaning, followed by thorough flushing with fresh water under low pressure, and then near-white blast cleaning.

In addition to these surface cleaning methods, one-half of the panels cleaned by each method were allowed to flash rust prior to painting, which is a common problem in Texas.

The panels were coated with nine paint systems that included alkyds, urethanes, epoxies, and organic and inorganic zinc-rich primers. All painted panels were exposed at a bridge site for 2 years and then evaluated. The results of this evaluation showed that:

- Overall, the organic zinc-rich systems (epoxy primer, epoxy intermediate coat, and vinyl topcoat) were the best.
- The barrier type systems (e.g., those that did not contain zinc) were the best over the flash-rusted panels.
- Flushing with water was better than blasting with water.
- Flushing with water improved overall performance.
- Only the zinc-rich systems were not undercut.
- It is more difficult to clean weathered A588 steel than A36 steel.

In the third study, the Steel Structures Painting Council surveyed the literature for information on the remedial painting of weathered A588 steels [Keane et al., 1983]. The report on this survey is a good source of background information. The appendixes provide bibliographic information accompanied by a summary of each article. The report also found that existing information on the remedial painting of weathered A588 steel was insufficient.

12.2.4 Recommendation

The three studies described above showed that there are coating systems with demonstrated good performance over weathered A588 steel. They consist of an epoxy zinc-rich primer, an epoxy polyamide intermediate coat, and either a urethane or vinyl topcoat. This hybrid system of galvanic and barrier protection has excellent tolerance to dry film thickness variation, and good tolerance to surface contaminants and application errors.

On a note of caution, inorganic zinc-rich systems are known to perform poorly to fair on weathered A588 steel. The trapping of chlorides, the porosity of many inorganic zinc-rich primers, and their sensitivity to large variations in dry film thickness contribute to a short service life. Michigan coated, with poor results, some weathered A588 steel with inorganic zinc-rich systems.

12.3 SPECIFICATIONS

The most important requirement of a specification is that it is understood by the bidder and inspector, and enforced by the bridge owner. During the last 3 years the Michigan Department of Transportation has developed a specification for the remedial painting of weathered A588 steel bridges (Appendix A). The specification reflects the findings of the studies discussed in section 12.2, and the field experience in remedially painting A588 steel bridges in Michigan.

With FHWA support, the Michigan Department of Transportation in 1984 began to evaluate the performance of six remedially painted weathering steel bridges in the Detroit area. The variables of particular interest in this study were: the age and local environment of the structures, the effectiveness of different blasting media, the comparison of generically equivalent paints from different manufacturers, and the cost of the coating system.

The information gathered to date has been most valuable in updating the specification for remedially painting weathered A588 steel bridges. The current version is provided in Appendix A.

12.4 COST

Only limited cost data are available because to date only four weathering steel bridges have been remedially painted in Michigan and 11 in other states. Furthermore, most contractors had bid for such jobs for the first time, and various "extras" were subsequently added to the initial bid. This resulted in data reflecting a large range of costs.

Table 37 summarizes the bid prices, per square foot of surface area, of remedially painting four weathering steel bridges in Detroit, Michigan, between 1984 and 1986. All contracts included the cost of painting the bridge in addition to other costs. The bids are ranked in Table 37 in the order of lowest (No. 1) to highest (No. 5) total cost. The contract was always awarded to the bidder with the lowest total cost (No. 1). Because the contract included several items, the ranking of the bidders by lowest total cost does not necessarily correspond to the ranking by lowest "expected" cost of remedially painting the bridge.

Table 38 compares the unit costs of repainting an A36 steel bridge and remedially painting an A588 steel bridge [Appleman 1985]. The latter is 46 percent higher for the reasons cited below.

Field experience and investigations undertaken so far show that the following conditions affect the cost of remedially painting weathered A588 steel as compared to that of repainting A36 steel:

Table 37. Bid prices for coating weathered weathering steel structures in Detroit, Michigan.

Structure	Area (ft^2)	Cost ($/$ft^2$)				
		Bid #1	Bid #2	Bid #3	Bid #4	Engineer's Estimate
8 Mile @ U.S. 10	36,000	2.03	2.88	4.36	...	3.20
Grand River over I-96	83,000	3.27	2.40	2.65	2.65	...
Ecorse Road over I-275	43,000	2.50	3.26	3.61	3.15	3.50
I-275 over North Line	27,600	2.40	3.91	4.50	3.87	3.38

Table 38. Cost comparison of painting existing A36 and A588 bridges.

Item	Cost of Repainting A36 Steel[a] ($/$ft^2$)	Cost of Remedially Painting A588 Steel ($/$ft^2$)
Near-White Blast		
SSPC-10	.86	1.29[c]
Material[b]		
Primer	.138	.276[d]
Intermediate coat	.105	.136[e]
Topcoat coat	.079	.103[e]
Application		
Primer	.25	.375[f]
Intermediate coat	.20	.25[g]
Topcoat	.20	.25[g]
	1.83	2.68
Multiply by 1.25[h]		
Total Cost	2.29	3.35

Notes:

a. 1984 prices as listed in March 1985 issue of The Journal of Protective Coatings and Linings.

b. This system was chosen due to its similarity to both the Texas and Michigan systems.

c. Assume 50% more difficult to blast.

d. Assume 1/4 of theoretical coverage instead of the more typical 1/2.

e. Assume 30% more paint due to surface roughness.

f. Assume 30% increase in time required. (It does not take twice as long to apply twice the amount of paint.)

g. Assume 25% increase in time required.

h. Since the structure is a simple steel structure in the air, the above ground prices must be multiplied by 125%.

- The entire weathering steel bridge is corroded, whereas the paint on A36 steel bridges rarely fails on more than 30 percent of the surface area. It is easier to blast clean new steel covered with mill scale or the painted portions of a "failed" paint system than to blast clean weathered A588 steel. Thus, 30 percent or less of the total surface area of a painted A36 steel bridge is as difficult to blast as the total surface of a weathered A588 steel bridge.
- The pits on weathered A588 steel tend to be smaller but deeper than those on corroded A36 steel. The difficulty of removing the rust from the pits reduces the blasting productivity.
- The volume of paint required to meet the specified dry film thickness is higher in corroded areas than in noncorroded areas. Past experience in painting new steel or partially corroded steel has led contractors and owners to grossly underestimate the volume of paint needed to remedially paint weathering steel bridges.
- The number of passes and the time needed to apply the paint is higher for weathered A588 steel than for A36 steel.

The limited, available data on the projected life of coating systems indicate that the life expectancy of a system is shorter for weathered A588 steel than for new steel.

Most painting contractors lack experience in remedially painting weathering steel bridges and tend to bid too low. State officials should hold a pre-bid conference to ensure that the interested contractors understand the scope of work.

12.5 INSPECTION

The process of inspecting the remedial painting of weathered A588 steel bridges is similar to that of inspecting other painting jobs. The use of knowledgeable inspectors is critical to the success of the project. Yet, there are few qualified inspectors with experience in remedial painting of weathered A588 steel structures because to date few A588 bridges have been remedially painted. These problems are compounded by the contractor's lack of experience in remedially painting A588 steel structures.

12.6 MAINTENANCE OF COATING SYSTEMS

Coating systems must be maintained. In the projected life of a coating system some areas fail relatively early, typically within 5 years. To extend the life of the coating and reduce local corrosion losses, paint failures must be repaired. In general, regular maintenance has the added benefit of reducing the life-cycle cost of a coating system.

12.7 REHABILITATION OF CREVICES

The major structural problem related to section loss has been found to result from accelerated corrosion in crevices. This problem is compounded by chloride contamination. The crevices that are formed between back-to-back angles, intermittently welded members, and between link plates and the web at some expansion joints are particularly vulnerable. The corrosion rate inside these crevices is much higher than that on exposed surfaces of the structure. Crevices must be rehabilitated before the structure is remedially painted.

Crevices in connections of members whose contact surfaces are excessively corroding can be treated by one of the following methods, depending on the type of detail and degree of corrosion: (1) disassemble, blast clean, paint the contact surfaces, and reassemble; (2) epoxy inject the crevice and epoxy caulk all edges; and (3) seal weld all edges.

Some crevices, like the gap between the link plates and the web at expansion joints, cannot be eliminated. To rehabilitate this type of detail prior to remedial painting of the bridge, the link plate and pin assemblies must be removed and replaced at a considerable cost. Appendixes B through E give sample specifications for pins and link plates for nonredundant bridges, pins and link plates for redundant bridges, temporary support of suspended span girder end, and removal and erection of hanger assembly.

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APPENDIX A

MICHIGAN
DEPARTMENT OF TRANSPORTATION
BUREAU OF HIGHWAYS

SPECIAL PROVISION
FOR
CLEANING AND COATING EXISTING STEEL STRUCTURES
TYPE 4

a. Description.-This work shall consist of the complete blast cleaning and coating of the metal surfaces of existing steel structures, including downspouts and all brackets. When the entire deck is to be removed, then the top and sides of all the top flanges shall also be blast cleaned and prime coated according to this specification. Utility conduits, including all brackets and hangers, shall also be cleaned and coated according to this specification but shall be done only when called for on the plans. This work excludes hand railings and chain link fence enclosures.

The coating system shall consist of a coat of an epoxy organic zinc-rich primer, a coat of high-build epoxy, and a urethane protective coat.

Terminology used herein is in accordance with the definitions used in Volume 2, Systems and Specifications of the SSPC Steel Structures Painting Manual (1982 Edition).

The work shall be done in accordance with the 1984 Standard Specifications except as otherwise specified herein.

b. Seasonal Limitations on Field Painting.-Except as otherwise authorized by the Engineer, no field coating shall be allowed between October 1 and May 1 for Districts 1, 2, 3, and 4, and between October 15 and April 15 for Districts 5, 6, 7, 8, and Metro.

c. Coating System.-The Contractor shall select a complete coating system from one of the approved coating systems listed in the attached Qualified Products List (QPL).

The color for the urethane protective coat shall match color number _____ (light _____) of Federal Standards Number 595a, dated January 2, 1968. The Contractor shall supply the Engineer with the product data sheets before any coating is done. The product data sheets shall indicate the mixing and thinning directions, and the recommended spray nozzles and pressures.

d. Cleaning of Structures.-All areas of oil and grease on surfaces to be coated shall be cleaned with clean petroleum solvents and then all the surfaces to be coated shall be blast cleaned to a near-white finish as defined in SSPC-SP10 (page 47, Volume 2). See SSPC Visual Standards. (See Note 1.)

Prior to blast cleaning a beam, the top of the bottom flange shall be scraped (with a garden hoe, for example) to remove the accumulated dust and dirt.

All fins, tears, slivers, and burred or sharp edges that are present on any steel member, or that appear during the blasting operation, shall be removed by grinding and the area reblasted to give a 1 to 2.5-mil surface profile. Scaling hammers may be used to remove heavy scale but heavier type chipping hammers which would excessively scar the metal shall not be used.

The abrasive used for blast cleaning shall be an approved low dusting abrasive and shall have a gradation such that the abrasive will produce a uniform profile of 1 to 2.5 mils, as measured with extra coarse Testex Replica Tape. (The Contractor shall select an abrasive from one of the approved abrasives listed in the Prequalified Materials List of the Department's Materials Sampling Guide under 5.04 1. Low-Dusting Abrasives-Blast Cleaning of Bridge Structures.) Due to surface roughness from corrosion, this method will not work on A-588 structures; thus for each lot of abrasive, the Contractor shall supply an unblasted piece of steel at least one foot square and 1/4 inch thick and blast it on site with their standard procedures. The Engineer will determine the profile on this piece.

All abrasive and coating residue shall be removed from steel surfaces with a good commercial grade vacuum cleaner equipped with a brush-type cleaning tool, or by double blowing. If the double blowing method is used, the exposed top surfaces of all structural steel, including flanges, longitudinal stiffeners,

splice plates, hangers, etc., shall be vacuumed after the double blowing operations are completed. The air line used for blowing the steel clean shall have an in-line water trap and the air shall be free of oil and water as it leaves the air line. The steel shall then be kept dust free and primed within 8 hours after blast cleaning.

Care shall be taken to protect freshly coated surfaces, bridge bearing components, hand railings, galvanized fence enclosures, all appurtenances, and any adjacent concrete from blast cleaning operations. These areas shall be protected from blast cleaning operations by shielding or masking. Blast-damaged primed surfaces shall be thoroughly wire brushed or if visible rust occurs, reblasted to a near-white condition. The wire-brushed or blast-cleaned surfaces shall be vacuumed and reprimed.

For structures with piers, a minimum of 5 feet on each side of the piers shall be blast cleaned on the same day and primed as a unit to prevent damage to previously primed surfaces.

e. Mixing the Coating.-The coating shall be mixed with a high shear mixer (such as Jiffy Mixer) in accordance with the manufacturer's directions, to a smooth, lump-free consistency. Paddle mixers or paint shakers are not permitted. Mixing shall be done, as far as possible, in the original containers and shall be continued until all of the metallic powder or pigment is in suspension.

Care shall be taken to ensure that all of the coating solids that may have settled to the bottom of the container are thoroughly dispersed. The coating shall then be strained through a screen having openings no larger than those specified for a No. 50 sieve in ASTM E 11. After straining, the mixed primer shall be kept under continuous agitation up to and during the time of application.

f. Thinning the Coating.-In general the coatings are supplied for normal use without thinning. If it is necessary to thin the coating for proper application in cool weather or to obtain better coverage of the urethane protective coat, the thinning shall be done in accordance with the manufacturer's recommendations.

g. Conditions for Coating.-Coating shall be applied only when the following conditions have been met:

1. Temperature.-The temperature of the air and the steel shall be above 50 F for coatings other than the topcoat. This 50 F minimum temperature shall be maintained throughout the minimum cure times as listed in the QPL. For the urethane topcoat the temperature of the air and steel shall be above 40 F. Coatings shall not be applied if the temperature is high enough to cause blistering. The surface temperature of the steel shall be at least 5 F higher than the dew point. (This requires the steel to be dry and free of any condensation regardless of the actual temperature of the steel.)
2. Humidity.-The coating shall not be applied when the relative humidity is greater than 90 percent nor when a combination of temperature and humidity conditions are such that moisture condenses on the surface being coated.

h. Applying the Coating.-After the surface to be coated has been cleaned and approved by the Engineer, the coatings shall be applied with the spray nozzles and pressures recommended by the producer of the coating system, so as to attain the film thicknesses specified. The minimum dry film thickness for the primer shall be 4.0 mils, and all areas not having the required minimum dry film thickness of primer shall be recoated. For the intermediate coat and for the urethane topcoat, the dry film thickness shall be sufficient to provide complete coverage and a uniform color and appearance but in no case shall the intermediate coat be less than 3.5 mils or the urethane topcoat be less than 1.0 mil. (See Note 2). The dry film thickness will be determined by use of a magnetic film thickness gage. The gage shall be calibrated on the blasted steel with plastic shims approximately the same thickness as the minimum dry film thickness. A Tooke film thickness gage may be used to verify the coating thickness when requested by the Engineer (See Note 3). If the Tooke gage shows the primer coat to be less than the specified minimum thickness, the total coating system will be rejected even if the total dry film thickness exceeds the minimum.

If the application of coating at the required thickness in one pass produces runs, bubbles, or sags, the coating shall be applied in multiple passes of the spray gun, the passes separated by several minutes. Where excessive coating thickness produces "mud-cracking," such coating shall be scraped back to soundly bonded coating and the area recoated to the required thickness.

All dry spray shall be removed, by sanding if necessary. In areas of deficient primer thickness, the areas shall be thoroughly cleaned with power washing equipment, as necessary to remove all dirt; the areas shall then be wire brushed, vacuumed, and recoated.

Proper curing conditions will be required between the application of all coats. The minimum curing time between coats is listed in the attached QPL. The maximum time between coats shall be in accordance with the manufacturer's recommendation except that no more than 60 calendar days will be permitted between coats. If the maximum time between coats is exceeded, all newly coated surfaces shall be completely blast cleaned again to a near-white finish (SSPC-SP10) and recoated and shall be at the Contractor's expense.

After the steel is primed, it shall be vacuumed again before subsequent coating. If for any reason this vacuuming does not remove all the accumulated dust and/or dirt, or if more than 3 weeks has elapsed since the steel was primed, or if in the opinion of the Engineer the surface is unfit for topcoating, the surface shall be scrubbed with a mild detergent solution (any commercial laundry detergent) and thoroughly rinsed with water and allowed to dry for 24 hours before the surface is coated.

All metal coated with impure, unsatisfactory, or unauthorized coating material, or coated in an unworkmanlike or objectionable manner, shall be thoroughly cleaned and recoated or otherwise corrected as directed by the Engineer.

i. Provisions for Field Inspection.-The Contractor shall furnish and erect scaffolding meeting the approval of the Engineer to permit inspection of the steel prior to and after coating.

Rubber rollers, or other protective devices meeting the approval of the Engineer, shall be used on scaffold fastenings. Metal rollers or clamps and other types fastenings which will mar or damage freshly coated surfaces shall not be used.

j. Protection of the Work.- All portions of the structure (superstructure, substructure, slope protection, and highway appurtenances) shall be protected against splatter, splashes, and smirches of coating material by means of protective covering suitable for the purpose. Similar protection shall be afforded any highway appurtenances that could be damaged by blast cleaning operations.

Pedestrian, vehicular, and other traffic upon or underneath the structure shall be protected as provided under Subsection 1.05.13 of the 1984 Standard Specifications. The Contractor shall be responsible for any damage caused by his operations to vehicles, persons, or property.

During blast cleaning operations, the Contractor shall make provisions for protecting existing traffic from any hazards resulting from the blast cleaning operations. These provisions shall include a type of barrier system which would protect against direct blasting of vehicles or pedestrians, eliminate abrasive materials and debris from falling on the traveled portions of the pavement, and prevent the spreading of abrasive materials and debris into an area which would create a traffic hazard.

Whenever the intended purposes of the protective devices are not being accomplished, work shall be suspended until corrections are made. In addition, any abrasive material and debris deposited on the pavement, shoulders, or slope paving in the working area shall be removed before those areas are reopened to traffic.

During the coating of the structure, the Contractor shall take whatever measures are necessary to prevent any coating spray from reaching vehicles (including water vessels) or other private property. The required measures may include tarping the area being coated or coating during time periods of low traffic volumes. If the wind velocity and direction are such that no measures are effective in controlling the coating spray, the Engineer may temporarily suspend the coating.

k. Stenciling Requirement.-At the completion of the coating, the completion date (month and year) and the number of the type of coating system used shall be stenciled on the structure in 4-inch numbers; for example: 6/85-4. The paint used for this marking shall be any urethane spray paint and the color shall be black.

The numbers shall be stenciled on the inside of each fascia beam at the approaching traffic end of the structure. The two required markings shall be located at least 10 feet above ground level or the fill slope elevation and at least 10 feet from the abutment. If these locations are not applicable to the structure, the locations of the two markings will be designated by the Engineer.

l. Measurement and Payment.-The completed work as measured for CLEANING AND COATING EXISTING STEEL STRUCTURES will be paid for at the contract unit prices for the following contract items (pay items).

Pay Item	Pay Unit
Cleaning Existing Steel Structure, Type 4 (Structure Number)......................	Lump Sum
Coating Existing Steel Structure, Type 4 (Structure Number)........................	Lump Sum

Stenciling is considered a part of the work of Coating Existing Steel Structure, Type 4 and will not be paid for separately.

Cleaning and coating existing utility conduits including all brackets and hangers, when called for on the plans, is considered a part of the work of Cleaning and Coating Existing Steel Structures, Type 4 and will not be paid for separately.

APPENDIX

The following notes are listed only to be a help to the bidder in determining the bid. They are not contract provisions, but point out some of the not so obvious problems we have encountered during our blasting and coating of weathered A-588 Steel and heavily corroded structural steel.

Note 1. In many areas, especially under joints, the steel is heavily pitted. The complete removal of the last remaining trace of visible rust products is practically impossible. This being the case the definition of a near-white blast cannot be achieved. To solve this problem in these areas the appearance of a near-white blast is required, i.e. when compared to the visual standard the surface shall look the same. Even this is difficult but it does allow for very very small rust deposits at the base of a pit.

Note 2. Once again the pitting in the blasted surface causes a problem. The dry film thickness of the primer varies greatly, typically between 4 and 12 mils. The specification calls for a minimum of 4.0 mils; to achieve this, much more coating than normal is required in a pitted area. The Engineer is instructed to look for the low areas and the Engineer will require the Contractor to recoat all areas that are below the required minimum of 4.0 mils.

There are some spray techniques and equipment that greatly affect the amount of urethane that is required for complete coverage and a uniform appearance. These include the application technique of both the primer and the intermediate coat.

Note 3. All dry film thickness gages shall be calibrated on a relatively smooth section of the blasted web, not in a heavily pitted area.

MICHIGAN DEPARTMENT OF TRANSPORTATION
Qualified Product List
Systems Listed in Alphabetical Order by Producer
Use: Complete Shop Coating
Type 4

Producer	Represented By:	Coats	Products	Minimum Dry Film Thickness Mils(a)	Color	Min.Time Between Coats Hrs.
Ameron Protective	Lance Schmidt	1st	Amercoat 68A	4.0	Tinted	12
Coatings Division	22811 Mack Ave, Suite 108	2nd	Amercoat 383HS	3.5	White	12
201 North Berry Street	St. Clair Shores, MI 48080	3rd	Amercoat 450GL	1.0	(b)	
Brea, CA 92621	313-776-5755					
Carboline	Bob Marshall	1st	Carboline 658	4.0	Tinted	12
320 Hanley Industrial Ct.	1554 Harthorn	2nd	Carboline 190HB	3.5	White	12
St. Louis, MO 63144	Grosse Pt. Woods,MI 48236	3rd	Carboline.134	1.0	(b)	
	313-886-5555					
Devoe-Napko	Ron Hayden	1st	Zinc Prime 115	4.0	Tinted	12
P.O. Box 7600	414-646-8359	2nd	547 Chemfast Epoxy	3.5	White	12
Louisville, KY 40207		3rd	369 Prufthane	1.0	(b)	
DuPont DeNemours	Stephen L. Cluff	1st	62-Y-001 Zinc Filled	4.0	Tinted	24
Clayton Building	Same	2nd	Corlar 823 Epoxy Enamel	3.5	White	12
Concord Plaza	1-800-346-4748	3rd	Imron 326 Polyurethane	1.0	(b)	
Wilmington, DE 19898						
Glidden Coatings & Resins	William F. Ashmore	1st	Glid-Zinc Organic Coating	4.0	Tinted	24
16451 Sprague Road	Same	2nd	5431/5434 Hi Solids Epoxy	3.5	White	12
Strongsville, OH 44136	216-826-5342	3rd	Glid-Thane II	1.0	(b)	
Hempel Marine Paints, Inc.	David Miller	1st	Hempadur Zinc 1536	4.0	Tinted	12
P.O. Box 3279	Same	2nd	Hempadur Hi-Build 4523	3.5	White	12
Wallington, NJ 07057	1-800-222-0907	3rd	Hempathane 5528	1.0	(b)	
P.P.G. Industries	Same	1st	Aquapon Zinc Rich	4.0	Tinted	16
9933 Lawler Ave.	312-677-1771	2nd	Aquapon 97-3	3.5	White	16
Suite 260		3rd	Pitthane CP3685	1.0	(b)	
Skokie, IL 60077						
Porter-International	Pontiac Paint Co.	1st	Zinc Lock 308	4.0	Tinted	24
400 South 13 Street	Doug Winfield	2nd	MCR 43 Epoxy HB 4361	3.5	White	24
Louisville, KY 40201	1310 West Wide Track Dr.	3rd	Hythane	1.0	(b)	
502-588-9200	Pontiac,MI48058					
	313-335-3175					
Sherwin Williams	Bill Allman	1st	Zinc Clad 7	4.0	Tinted	12
	1137 Haco Drive	2nd	Tile Clad II Enamel	3.5	White	12
	Lansing, MI 48912	3rd	Hi-Bild Alphatic			
	517-482-5587		Polyurethane Enamel	1.0	(b)	
Tnemec Company, Inc	Mich Protective Coatings	1st	90-94 Tneme-Zinc	4.0	Tinted	16
North Kansas City,	Brad Brown	2nd	Series 66 Epoxoline	3.5	White	12
Missouri 64116	P.O. Box 39287	3rd	Series 72 Endura Shield II	1.0	(b)	
	Detroit,MI48239					
	313-538-7878					
Valspar Corporation	Bill Slabinski	1st	MZ-4	4.0	Tinted	16
901 N. Greenwood Ave.	1401 Severn Street	2nd	Val-Chem 89 Epoxy	3.5	White	16
Kankakee, IL 60901	Baltimore, MD 21230	3rd	40 Series Urethane	1.0	(b)	
	1-800-638-7756					

(a) The intermediate coat and the urethane topcoat shall be of sufficient dry film thickness to completely cover the prime coat and the intermediate coat respectively and produce a uniform color and appearance.
(b) The color number for the urethane topcoat shall match the color number shown on page 1 of this specification.

APPENDIX B

MICHIGAN
DEPARTMENT OF TRANSPORTATION
BUREAU OF HIGHWAYS

SUPPLEMENTAL SPECIFICATION
FOR
PINS AND LINK PLATES
FOR NON-REDUNDANT BRIDGES

FHWA Approval 04-18-88

a. Description.-This work shall consist of furnishing, fabricating, and delivering to the specified delivery point all structural steel required for the pins and link plates; and furnishing the necessary materials, labor, and equipment to take field measurements of the existing hanger assemblies when the plans so specify. Structural steel required for replacing existing pins and link plates will be paid for independently of the steel required for pins and link plates in new construction.

b. Materials.-The material used for link plates shall meet the requirements of ASTM A-572, Grade 50, or ASTM A-588 and for pins shall meet the requirements of ASTM A-588 or ASTM A-668, Class F, or Glass G, except as modified herein.

Longitudinal Charpy V-Notch impact values for both pin and link plate materials shall meet an average of 30 ft.-lbf. when tested at the Lowest Anticipated Service Temperature (LAST) specified for the MDOT District in which the structure is located.

District	LAST
1,2	-25 F
3,4	-20 F
5,6	-15 F
7,8, Metro	-10 F

In order to meet the Charpy V-Notch impact requirements, the steel may need to be heat treated.

Notch toughness tests on specimens shall be performed in accordance with Test Frequency P (Piece Testing) of ASTM A-673.

c. Furnishing and Fabricating.-Furnishing and fabricating the pins and link plates shall be in accordance with Section 5.04 of the 1984 Standard Specifications.

The longitudinal axis of the link plates and pins shall be oriented in the direction of rolling or forging of the plates or bars.

Pins shall be completely hard chrome plated (including cotter pin holes) to a minimum thickness of 3 mils. The surface finish on the chromed pins shall be less than 20 micro inches per inch (rms) on the bearing surface and less than 125 micro inches per inch (rms) on the ends.

Link plates shall be shop coated with the Type 4S Coating System as specified in the current specification for Complete Shop Coating of Structural Steel and Field Repair of Damaged Coating Type 4S, numbered 5.04 (22 series). This shall include applying the urethane protective coating to all surfaces.

Surface finishes on the link plates shall be less than 125 micro inches per inch (rms) on all cut edges and bored holes.

No welding repairs will be permitted on the pins or link plates.

When the plans so specify, the Contractor shall take field measurements of all hanger assemblies to be replaced and submit these "as built dimensions" in the existing structure, along with a drawing showing the span and girder end where the measurements were taken, to the Engineer. The measurements required on the existing hanger assemblies include the pin diameter, the center to center distance between pins (measured on each side) in each assembly, and the length, width and thickness of the link plates. Girder web alignment shall be checked by laying a straight edge across the pin plate gap at both the top and bottom of the girder. Any girder offset shall be measured and reported.

Any lane or shoulder closure required to permit the field measurements of the hanger assemblies to be taken shall be in conformance with the Michigan Manual of Uniform Traffic Control Devices and Special Provision for Maintaining Traffic, and shall be approved by the Engineer. The Contractor shall not leave a lane or shoulder closed overnight when such lane or shoulder closure is required to take the field measurements.

The center to center of pins dimension of the replacement link plates shall be built to the existing dimensions if they differ by more than ± 1/8 inch from the plan dimensions. Other plan dimensions for replacement may be changed to accommodate the existing dimensions as directed by the Engineer.

Shop drawings of the link plates and pins shall be submitted for approval prior to fabrication of the assemblies. The fabricator shall submit a final report on the pin and hanger assemblies that lists the as built dimensions of the new link plates and pins. This report shall clearly show that the specified tolerances between the pins and the inside diameter of the installed link plate bushings have been met. Pin and link plate pairs shall either be assembled and shipped as a unit or match marked to insure the proper matching in the field.

d. Measurement and Payment.-The completed work as measured for PINS AND LINK PLATES FOR NON-REDUNDANT BRIDGES will be paid for at the contract unit prices for the following contract items (pay items):

Pay Item	Pay Unit
Structural Steel, Furnishing and Fabricating (Pin & Hanger)	Pound
Field Measurement of Hanger Assemblies	Lump Sum

Payment for Structural Steel, Furnishing and Fabricating (Pin & Hanger) includes all the structural steel required for replacing existing pins and link plates.

Payment for Field Measurement of Hanger Assemblies includes payment for all work, materials and equipment necessary to take and record the measurements, to maintain traffic while the measurements are being taken, and to provide the Engineer with a location drawing showing the span and girder end where the measurements were taken.

Structural steel required for pins and link plates in new construction will be measured and paid for as specified in Subsection 5.04.37 of the 1984 Standard Specifications for Structural Steel, Furnishing and Fabricating (Mixed), (Rolled, or (Plate).

APPENDIX C

MICHIGAN
DEPARTMENT OF TRANSPORTATION
BUREAU OF HIGHWAYS

SUPPLEMENTAL SPECIFICATION FOR PINS AND LINK PLATES FOR REDUNDANT BRIDGES

FHWA Approval 04-18-88

a. Description.-This work shall consist of furnishing, fabricating, and delivering to the specified delivery point all structural steel required for the pins and link plates; and furnishing the necessary materials, labor, and equipment to take field measurements of the existing hanger assemblies when the plans so specify. Structural steel required for replacing existing pins and link plates will be paid for independently of the steel required for pins and link plates in new construction.

b. Materials.-The material used for link plates shall meet the requirements of ASTM A-572, Grade 50, or ASTM A-588 and for pins shall meet the requirements of ASTM A-588 or ASTM A-668, Class F, or Class G, except as modified herein.

Longitudinal Charpy V-Notch impact values for both pin and link plate materials shall meet the requirements specified for High Strength Structural Steel in Subsection 8.06.04 of the 1984 Standard Specifications. The steel yield point stress used to determine the testing temperature shall be the value given in the Certified Mill Test Report. In order to meet the Charpy V-Notch impact requirements, the steel may need to be heat treated.

Notch toughness tests on specimens shall be performed in accordance with Test Frequency P (Piece Testing) of ASTM A-673.

c. Furnishing and Fabricating.-Furnishing and fabricating the pins and link plates shall be in accordance with Section 5.04 of the 1984 Standard Specifications.

The longitudinal axis of the link plates and pins shall be oriented in the direction of rolling or forging of the plates or bars.

Pins shall be completely hard chrome plated (including the cotter pin holes) to a minimum thickness of 3 mils. The surface finish on the chromed pins shall be less than 20 micro inches per inch (rms) on the bearing surface and less than 125 micro inches per inch (rms) on the ends.

Link plates shall be shop coated with the Type 4S Coating System as specified in the current specification for Complete Shop Coating of Structural Steel and Field Repair of Damaged Coating Type 4S, numbered 5.04 (22 series). This shall include applying the urethane protective coating to all surfaces.

Surface finishes on the link plates shall be less than 125 micro inches per inch (rms) on all cut edges and bored holes.

No welding repairs will be permitted on the pins or link plates.

When the plans so specify, the Contractor shall take field measurements of all hanger assemblies to be replaced and submit these "as built dimensions" in the existing structure, along with a drawing showing the span and girder end where the measurements were taken, to the Engineer. The measurements required on the existing hanger assemblies include the pin diameter, the center to center distance between pins (measured on each side) in each assembly, and the length, width, and thickness of the link plates. Girder web alignment shall be checked by laying a straight edge across the pin plate gap at both the top and bottom of the girder. Any girder offset shall be measured and reported.

Any lane or shoulder closure required to permit the field measurements of the hanger assemblies to be taken shall be in conformance with the Michigan Manual of Uniform Traffic Control Devices and the Special Provision for Maintaining Traffic, and shall be approved by the Engineer. The Contractor shall not leave a lane or shoulder closed overnight when such lane or shoulder closure is required to take the field measurements.

The center to center of pins dimension of the replacement link plates shall be built to the existing dimensions if they differ by more than ± 1/16 inch from the plan dimensions. Other plan dimensions for replacement may be changed to accommodate the existing dimensions as directed by the Engineer.

Shop drawings of the link plates and pins shall be submitted for approval prior to fabrication of the assemblies. The fabricator shall submit a final report on the pin and hanger assemblies that lists the as built dimensions of the new link plates and pins. This report shall clearly show that the specified tolerances between the pins and the inside diameter of the installed link plate bushings have been met. Pin and link plate pairs shall either be assembled and shipped as a unit or match marked to insure the proper matching in the field.

d. Measurement and Payment.-The completed work as measured for PINS AND LINK PLATES FOR REDUNDANT BRIDGES will be paid for at the contract unit prices for the following contract items (pay items):

Pay Item	Pay Item
Structural Steel, Furnishing and Fabricating (Pin & Hanger)	Pound
Field Measurement of Hanger Assemblies	Lump Sum

Payment for Structural Steel, Furnishing and Fabricating (Pin & Hanger) includes all the structural steel required for replacing existing pins and link plates.

Payment for Field Measurement of Hanger Assemblies includes payment for all work, materials and equipment necessary to take and record the measurements, to maintain traffic while the measurements are being taken, and to provide the Engineer with a location drawing showing the span and girder end where the measurements were taken.

Structural steel required for pins and link plates in new construction will be measured and paid for as specified in Subsection 5.04.37 of the 1984 Standard Specifications for Structural Steel, Furnishing and Fabricating (Mixed), (Rolled), or (Plate).

APPENDIX D

MICHIGAN
DEPARTMENT OF TRANSPORTATION
BUREAU OF HIGHWAYS

SPECIAL PROVISION
FOR
TEMPORARY SUPPORT OF
SUSPENDED SPAN GIRDER END

a) DESCRIPTION

This work shall consist of furnishing and placing the necessary materials, labor and equipment to construct and maintain a temporary support for the end of one girder of a suspended span while the hanger assembly is being replaced, and to remove and dispose of the temporary support after the new hanger assembly has been installed. The work shall be done as specified herein and on the plans, and other special provisions.

The contractor may submit an alternate design for the temporary support to the Engineer for approval. The alternate design shall be based on loads and allowable soil pressures as noted on the plans; the calculations used to arrive at the alternate design shall be included in the submittal.

b) MATERIALS AND EQUIPMENT

The materials shall meet the requirements specified in the section of the 1984 Standard Specifications designated, as follows:

Concrete, Grade 35S............................7.01
Steel Reinforcement, Grade 60 ....8.05
Structural Steel, ASTM A36......................8.06
Structural Timber and Lumber....................8.12

Hydraulic jacks shall have a minimum capacity, as stated on the plans, and shall have a minimum stroke capable of accommodating three inches of settlement that may occur during the period of time the temporary support is required. The hydraulic system shall be equipped with a dual gage that enables determination of the external load. Hydraulic jacks shall remain in place continuously, with the hydraulic lines and pump attached, until the new link plates and pins are installed and fully operational.

Hydraulic jacks shall have locking rings or some other positive locking device to prevent settlement in case of hydraulic failure. The locking devices shall be used during and after jack load changes until stable shims are in place and all loads have been removed from the jacks.

c) FABRICATION AND ERECTION

Fabrication and erection of structural steel shall be in accordance with Section 5.04 of the Standard Specifications and Supplemental Specification for Fabricating

Structural Steel and Aluminum, 5.04 (1 series). Shop drawings shall be submitted for approval prior to fabrication of the temporary supports.

If the Contractor elects to use a temporary support that is already fabricated, the Department reserves the right to verify the structural adequacy of the entire system. This verification may include, but is not limited to, visual inspection and nondestructive testing by Department personnel; and requiring the Contractor to furnish to the Department Mill Certification of the material used and shop drawings of the original fabrication. If the Department determines that the temporary support is not structurally adequate, the Contractor shall make the required corrections, as deemed necessary by the Department, prior to using the temporary support.

After erection and prior to loading of the temporary support, the horizontal offset of the top of the column from the bottom of the column shall be determined by the use of a plumb line. The horizontal offset of the hydraulic jack from the column centerline shall also be determined. Column and hydraulic jack offsets shall be measured in the directions parallel and perpendicular to the column web. At each temporary support the individual offsets and the sum, in each measured direction, of the column and jack offsets shall not exceed one inch maximum. Jack offsets are considered positive regardless of the direction of the column offsets.

Where Structure Embankment (CIP) is not called for, natural ground shall be compacted for a depth of 9 inches, as shown on the plans, to not less than 95 percent of its maximum unit weight before the footings are placed.

When the temporary support is placed on a paved shoulder or roadway the leveling course shall be 21AA aggregate, asphaltic cold-patch material, or approved equal. The material used for leveling shall be compacted to 95 percent of its maximum unit weight before the footings are placed.

Bracing of the temporary supports, as directed by the Engineer, may be required depending on the method selected by the Contractor for removing the link plates and pins.

d) MAINTENANCE

The Contractor shall check for settlement of the temporary support hourly during the first four hours after loading. Subsequent settlement checks shall be made daily. Corrective action shall be taken by the Contractor, by adding additional shims to the temporary support, to prevent the girder end from subsiding more than 1/16 inch from its original position.

The maintenance of the temporary support shall include replacement in case of partial or complete failure. The Department reserves the right, in case of delay or inadequate progress in making repairs and replacement, to furnish such labor, materials, and supervision of work as may be necessary to restore the movement of traffic.

e) MEASUREMENT AND PAYMENT

The completed work, as measured for Temporary Support of Suspended Span Girder End, will be paid for at the contract unit price for the following contract item (pay item):

Pay Item	Pay Unit
Temporary Support, Type _____	Each

Payment for Temporary Supports, of the type specified, includes payment for furnishing, placing, maintaining, and removing the necessary materials and equipment as described herein and as shown on the plans.

The quantity for Temporary Support, of the type specified, indicates the number of girder ends to be supported and not the number of devices required. However, sufficient number of support devices shall be furnished and used to ensure completion of the project within the contract time.

s224c

MICHIGAN
DEPARTMENT OF TRANSPORTATION
BUREAU OF HIGHWAYS

SPECIAL PROVISION
FOR
REMOVAL AND ERECTION OF HANGER ASSEMBLY
REDUNDANT BRIDGES

a) DESCRIPTION

This work shall consist of furnishing the necessary materials, labor, and equipment to remove two pins, two link plates, and shear locks; to blast clean and apply and cure the paint in the joint area; to install the new hanger assembly; to protect the completed joint area by enclosure; and to protect the newly painted area adjacent to the joint area. The work shall be done as specified herein and in accordance with the plans.

b) REMOVAL AND ERECTION OF HANGER ASSEMBLY

Removal methods shall be in accordance with the requirements of Subsection 2.06.02 of the 1984 Standard Specifications and as specified herein and as shown on the plans.

If the contractor elects to remove and replace the link plates and pins of more than one girder at one time, the work shall not be done on the same end of any adjacent girder. The suspender at the opposite end of a girder where the link plates and pins will be removed shall be completely assembled and operational.

The two pins and two link plates shall be removed in each assembly. No component shall be removed until the girder end is completely supported on stable shims, as shown on the plans, and all loads have been removed from the hydraulic jacks. When it is necessary to flame cut the link plates and/or pins for removal, the following procedures shall be followed:

1. The link plates may be flame cut in two pieces by making a sloping transverse cut that coincides with the joint opening between the girder ends. If the link plates are cut at the pin a sheet metal heat shield shall be positioned behind the link plate to protect the end of the girder from the cutting process.

2. The pins may be flame cut for removal after a metal heat shield is placed around the pin hole so that the pin plate is protected from the cutting process. The pin ends may be trimmed to within 1 inch minimum of the girder pin plate. The pin may then be split longitudinally in two pieces by burning a hole through the pin at the center and then making a vertical cut through the pin, working upward and downward from this hole. If the hole in the girder pin plate is gouged by this removal it shall be ground smooth to remove the gouge before blast cleaning and painting. No welding repair of the girder pin plate hole shall be allowed except when authorized in writing by the Engineer.

s181a

When the end diaphragms prevent the installation of the new pin (may occur on sharply skewed bridges at the top pin) an oblong hole may be flame cut in the web of one of the end diaphragms using a 1/8 inch thick (min.) steel hole template, clamped to the channel section, as a cutting guide. After flame cutting, the hole edges shall be ground smooth to remove any gouges or irregularities to a maximum surface roughness of 125 micro inches per inch (rms). This hole shall be blast cleaned and painted and left in the finished structure, unless otherwise noted on the plans. At no time shall the end diaphragm be loosened or removed as this would require the removal of traffic loading from the deck area above.

After the pins have been removed, all notches and deep pits that exist in the girder pin plate around the periphery of the hole shall be ground smooth to a maximum surface roughness of 125 micro inches per inch (rms). The girder ends, within two feet each side of the centerline of the pin holes, shall then be blast cleaned and painted in accordance with the Type 4 Coating System, specified in the special provision, before the new hanger assembly is installed.

The following modifications to the blast cleaning and paint cure requirements shall apply to the four foot joint painting limits only. The other portions of the bridge shall be blast cleaned and coated in strict conformance with the requirements of the Type 4 Coating System, specified in the special provision.

1. The joint area shall be enclosed and heated, in a manner acceptable to the Engineer, to maintain the temperature of the steel and the air at 50°F or above. If the ambient air temperature is at or above 50°F at the time of blast cleaning and application and curing of the paint, the enclosure of the joint will not be necessary. When enclosure of the joint area is required, all coats of paint shall be applied prior to removing the enclosure. The relative humidity shall not exceed the 90 percent maximum requirement as specified in the Type 4 specification.

2. The joint area shall be blast cleaned to a white metal finish, as defined in SSPC-SP5 (see SSPC Visual Standards), with a surface profile between 1 to 2 mils.

3. The primer coat shall be applied with spray equipment and cured at or above 50°F for a minimum of 12 hours. The dry film thickness shall then be measured and shall be 4.0 mils minimum.

4. The epoxy intermediate coat shall be applied with spray equipment and shall be allowed to cure at or above 50°F for a minimum of 1 hour, or more if requireduntil dry to the touch.

5. The new pins and link plates may be installed when the epoxy intermediate coat has cured for a **minimum of 1 hour** and is dry to the touch. The epoxy intermediate coat cure shall be continued at or above 50°F for a minimum total **time period of 12 hours**. The dry film thickness shall then be measured and shall be 3.5 mils minimum.

6. The urethane top coat shall be applied with spray equipment to the assembled joint areas with the temperature at or above 50°F. It shall be applied in sufficient thickness to provide complete coverage and a uniform appearance. The areas behind the assembled link plates shall be coated with urethane to the extent possible. This final coat shall be applied as soon as possible after the epoxy intermediate coat has cured. No payment shall be made for the item Hanger Assembly Removal and Erection until the final top coat has been applied.

7. The Type 4 Coating System applied to the joint area shall be selected from the restricted Qualified Products List (QPL) attached to this specification. The system selected does not have to be by the same manufacturer selected for painting the remainder of the bridge.

The new hanger assembly shall be completely installed and operational before the falsework shims are removed.

The area of the girder that was coated after removal of the hanger assembly, including the new pins and link plates, shall be boxed in or otherwise securely covered prior to blast cleaning and prime coating of the girders. The box or covering shall be removed prior to topcoating the girders.

Girder areas, which have been painted with the Type 4 Coating System prior to blast cleaning the hanger areas, shall be protected as approved by the Engineer. The protection shall prevent damage to the Type 4 Coating System during the blast cleaning and painting of the four foot joint area.

c) MEASUREMENT AND PAYMENT

The completed work, as measured for Removal and Erection of Hanger Assembly will be paid for at the contract unit price for the following contract item (pay item):

Pay Item	Pay Unit
Hanger Assembly Removal and Erection	Each

Payment for Hanger Assembly Removal and Erection includes payment for all work, materials, and equipment necessary for removing two pins, two link plates, and shear locks; for blast cleaning and applying and curing the paint in the joint area; for installing the new link plates and pins; for protecting the completed joint area by enclosure; and for protecting the newly painted area adjacent to the joint area.

MICHIGAN DEPARTMENT OF TRANSPORTATION

QUALIFIED PRODUCT LIST
Systems Listed in Alphabetical Order by Producer
Use: Coating Joint Areas During Pin and Link Plate Replacement
Type 4

Producer	Represented By	Coats	Products	Dry Film Thickness Mils Min.	Color	Min. Time Between Coats Hrs.
Ameron Protective	B. Marshall	1st	Dimetcoat 68	4.0	-	12
Coating Division	Ameron Protective	2nd	Amercoat 383HS	3.5	White	12
201 N. Berry St.	Coating Division	3rd	Amercoat 450GL	*	**15488 or	
Brea, CA	(313) 886-5555				16440	
P.P.G. Industries	John Felice	1st	Aquapon Zinc Rich	4.0	-	12
9933 Lawler Ave.	3928 W. Saginaw	2nd	Aquapon 97-3	3.5	White	12
Suite 260	Lansing, MI 48917	3rd	Pitthane CP3685	*	**15488 or	
Skokie, IL 60077	(517) 323-9144				16440	
(312) 677-0560						
Porter Paint Co.	Pontiac Paint Co.	1st	Zinc Lock 308	4.0	-	12
400 S. 13 St.	1310 W. Wide Track	2nd	MCR 43	3.5	White	12
Louisville, KY 40201	Pontiac, MI 48058	3rd	Hythane	*	**15488 or	
(502) 588-9200				16440		
Tnemec Co., Inc.	MI Protective	1st	90-94 Tnemec-Zinc	4.0	-	12
North Kansas City,	Coatings	2nd	Series 66	3.5	White	12
MO 64116	PO Box 39287		Epoxoline			
	Detroit, MI 48239	3rd	Series 72 Endura	*	**15488 or	
	(313) 538-7878		Shield II		16440	

* The urethane topcoat shall be of sufficient dry film thickness to completely cover the intermediate coat and produce a uniform appearance.

APPENDIX E - Section 2

MICHIGAN
DEPARTMENT OF TRANSPORTATION
BUREAU OF HIGHWAYS

SPECIAL PROVISION
FOR
REMOVAL AND ERECTION OF HANGER ASSEMBLY
NON-REDUNDANT BRIDGES

a) DESCRIPTION

This work shall consist of furnishing the necessary materials, labor, and equipment to remove two pins and two link plates; to blast clean and apply and cure the paint in the joint area; to install the new hanger assembly; to protect the completed joint area by enclosure; and to protect the newly painted area adjacent to the joint area. The work shall be done as specified herein and in accordance with the plans.

b) REMOVAL AND ERECTION OF HANGER ASSEMBLY

Removal methods shall be in accordance with the requirements of Subsection 2.06.02 of the 1984 Standard Specifications and as specified herein and as shown on the plans.

If the Contractor elects to remove and replace the link plates and pins of more than one girder at one time, the work shall not be done on the same end of any adjacent girder. The suspender at the opposite end of a girder where the link plates and pins will be removed shall be completely assembled and operational.

The two pins and two link plates shall be removed in each assembly. No component shall be removed until the girder end is completely supported on stable shims, as shown on the plans, and all loads have been removed from the hydraulic jacks. When it is necessary to flame cut the link plates and/or pins for removal, the following procedures shall be followed:

1. The link plates may be flame cut in two pieces by making a sloping transverse cut that coincides with the joint opening between the girder ends. If the link plates are cut at the pin a sheet metal heat shield shall be positioned behind the link plate to protect the end of the girder from the cutting process.

2. The pins may be flame cut for removal after a metal heat shield is placed around the pin hole so that the pin plate is protected from the cutting process. The pin ends may be trimmed to within 1 inch minimum of the girder pin plate. The pin may then be split longitudinally in two pieces by burning a hole through the pin at the center and then making a vertical cut through the pin, working upward and downward from this hole. If the hole in the girder pin plate is gouged by this removal it shall be ground smooth to remove the gouge before blast cleaning and painting. No welding repair of the girder pin plate hole shall be allowed except when authorized in writing by the Engineer.

s181b

When the end diaphragms prevent the installation of the new pin (may occur on sharply skewed bridges at the top pin) an oblong hole may be flame cut in the web of one of the end diaphragms using a 1/8 inch thick (min.) steel hole template, clamped to the channel section, as a cutting guide. After flame cutting, the hole edges shall be ground smooth to remove any gouges or irregularities to a maximum surface roughness of 125 micro inches per inch (rms). This hole shall be blast cleaned and painted and left in the finished structure, unless otherwise noted on the plans. At no time shall the end diaphragm be loosened or removed except when authorized in writing by the Engineer.

After the pins have been removed, all notches and deep pits that exist in the girder pin plate around the periphery of the hole shall be ground smooth to a maximum surface roughness of 125 micro inches per inch (rms). The girder ends, within two feet each side of the centerline of the pin holes, shall then be blast cleaned and painted in accordance with the Type 4 Coating System, specified in the Special Provision for Cleaning and Coating Existing Steel Structures, before the new hanger assembly is installed.

The following modifications to the blast cleaning and paint cure requirements shall apply to the four foot joint painting limits only. The other portions of the bridge shall be blast cleaned and coated in strict conformance with the requirements of the Type 4 Coating System, specified in the Special Provision for Cleaning and Coating Existing Steel Structures.

1. The joint area shall be enclosed and heated, in a manner acceptable to the Engineer, to maintain the temperature of the steel and the air at 50°F or above. If the ambient air temperature is at or above 50°F at the time of blast cleaning and application and curing of the paint, the enclosure of the joint will not be necessary. When enclosure of the joint area is required, all coats of paint shall be applied prior to removing the enclosure. The relative humidity shall not exceed the 90 percent maximum requirement as specified in the Type 4 specification.

2. The joint area shall be blast cleaned to a white metal finish, as defined in SSPC-SP5 (see SSPC Visual Standards), with a surface profile between 1 to 2 mils.

3. The primer coat shall be applied with spray equipment and cured at or above 50°F for a minimum of 12 hours. The dry film thickness shall then be measured and shall be 4.0 mils minimum.

4. The epoxy intermediate coat shall be applied with spray equipment and shall be allowed to cure at or above 50°F for a minimum of 1 hour, or more if required until dry to the touch.

5. The new pins and link plates may be installed when the epoxy intermediate coat has cured for a minimum of 1 hour and is dry to the touch. The epoxy intermediate coat cure shall be continued at or above 50°F for a minimum total time period of 12 hours. The dry film thickness shall then be measured and shall be 3.5 mils minimum.

6. The urethane top coat shall be applied with spray equipment to the assembled joint areas with the temperature at or above 50°F. It shall be applied in sufficient thickness to provide complete coverage and a uniform appearance. The areas behind the assembled link plates shall be coated with urethane to the extent possible. This final coat shall be applied as soon as possible after the epoxy intermediate coat has cured. No payment shall be made for the item Hanger Assembly Removal and Erection until the final top coat has been applied.

7. The Type 4 Coating System applied to the joint area shall be selected from the restricted Qualified Products List (QPL) attached to this specification. The system selected does not have to be by the same manufacturer selected for painting other portions of the bridge.

The new hanger assembly shall be completely installed and operational before the falsework shims are removed.

The area of the girder that was coated after removal of the hanger assembly, including the new pins and link plates, shall be boxed in or otherwise securely covered prior to blast cleaning and prime coating of the adjacent areas. The box or covering shall be removed prior to topcoating the girders.

All girder areas, which have been painted with the Type 4 Coating System prior to blast cleaning the hanger areas, shall be protected as approved by the Engineer. The protection shall prevent damage to the Type 4 Coating System during the blast cleaning and painting of the four foot joint area.

c) <u>MEASUREMENT AND PAYMENT</u>

The completed work, as measured for Removal and Erection of Hanger Assembly Non-Redundant Bridges will be paid for at the contract unit price for the following contract item (pay item):

Pay Item	Pay Unit
Hanger Assembly Removal and Erection	Each

Payment for Hanger Assembly Removal and Erection includes payment for all work, materials, and equipment necessary for removing two pins and two link plates; for blast cleaning and applying and curing the paint in the joint area; for installing the new link plates and pins; for protecting the completed joint area by enclosure; and for protecting the newly painted area adjacent to the joint area.

MICHIGAN DEPARTMENT OF TRANSPORTATION

QUALIFIED PRODUCT LIST
Systems Listed in Alphabetical Order by Producer
Use: Coating Joint Areas During Pin and Link Plate Replacement
Type 4

Producer	Represented By	Coats	Products	Dry Film Thickness Mils Min.	Color	Min. Time Between Coats Hrs.
Ameron Protective Coating Division 201 N. Berry St. Brea, CA	B. Marshall Ameron Protective Coating Division (313) 886-5555	1st	Dimetcoat 68	4.0	-	12
		2nd	Amercoat 383HS	3.5	White	12
		3rd	Amercoat 450GL	*	**15488 or 16440	
P.P.G. Industries 9933 Lawler Ave. Suite 260 Skokie, IL 60077 (312) 677-0560	John Felice 3928 W. Saginaw Lansing, MI 48917 (517) 323-9144	1st	Aquapon Zinc Rich	4.0	-	12
		2nd	Aquapon 97-3	3.5	White	12
		3rd	Pitthane CP3685	*	**15488 or 16440	
Porter Paint Co. 400 S. 13 St. Louisville, KY 40201 (502) 588-9200	Pontiac Paint Co. 1310 W. Wide Track Pontiac, MI 48058	1st	Zinc Lock 308	4.0	-	12
		2nd	MCR 43	3.5	White	12
		3rd	Hythane	*	**15488 or 16440	
Tnemec Co., Inc. North Kansas City, MO 64116	MI Protective Coatings PO Box 39287 Detroit, MI 48239 (313) 538-7878	1st	90-94 Tnemec-Zinc	4.0	-	12
		2nd	Series 66 Epoxoline	3.5	White	12
		3rd	Series 72 Endura Shield II	*	**15488 or 16440	

* The urethane topcoat shall be of sufficient dry film thickness to completely cover the intermediate coat and produce a uniform appearance.

APPENDIX F

CASE STUDIES OF WEATHERING STEEL BRIDGES

F.1 INTRODUCTION

The purpose of the following case studies is to illustrate the effect that continuous exposure to moisture, marine environment, deicing salt, and debris can have on the corrosion performance of weathering steel bridges.

F.2 CONTINUOUS MOISTURE

The results of experiments show that weathering steel does not form a protective oxide coating if it remains continuously moist [Larrabee 1953]. Under such conditions weathering steel was found to corrode at the same rate as ordinary steel, reaching in one year or less the corrosion penetration that ideally and boldly exposed weathering steel would reach in 20 years (see Figure 9 of this report and Figure 61 of *NCHRP Report 272*).

Direct precipitation of rain or snow is not needed for a steel surface to become wet. Moisture can be deposited by (1) nightly condensation when the surface temperature of the steel falls below the dew point; (2) radiant cooling of the skyward deck surface in a clear night, causing moisture to condense on the upper portions of the steel girder and run down the web along drip lines; (3) capillary action of the porous oxide coating and crevices in structural details; (4) adsorption by corrosion products particularly in the presence of salt; (5) leaking bridge deck joints; and (6) traffic spray kicked up in the wake of high-speed traffic settling on the members of a grade separation structure. The deposition of moisture is enhanced by high relative humidity, nightly fog, moisture evaporating from bodies of water, and poor air circulation.

The mechanisms of wetting by traffic spray and leakage through expansion joints are particularly severe in the presence of deicing salts. Further discussion is given in section F.4.2.

F.2.1 Ohio

Several weathering steel bridges crossing streams have performed poorly in rural areas of Tuscarawas, Franklin, and Butler Counties, Ohio.

Tuscarawas County. The bridges on County Road 37 over Little Still Water Creek (Fig. F-1) and on County Road 99 over Sugar Creek, Tuscarawas County, consist of 36.6 and 37.8 m (120 and 124 ft) long trusses with wide flange members fabricated from A588 Grade A steel. They have about 3.7 to 4.6 m (12 to 15 ft) clearance above the creek. The County Road 37 Bridge, built in 1973, had corroded severely. The worst corrosion was observed along the bottom chord as well as the bottom flange and lower web of the floor beams and stringers (Fig. F-2 and Fig. F-3). Rust slabs 6.4 mm (¼ in.) thick came off the bottom flanges. Some members lost 15 percent of their section. The truss members above the deck, however, corroded less. This pattern of corrosion is typically observed when water ponds and rust and other debris accumulate on top of the bottom flange. The trapped moisture then wicks up the lower portion of the web and migrates around the bottom flange edges, clinging to the lower surface. As a result the lower part of the beam dries slowly and corrodes more than the upper part.

The low-clearance bridge crosses the shallow and slow-moving Little Still Water Creek. There is little wind in the valley and a high incidence of fog in the fall. Trees grow along the creek bed. The environment and the terrain combine to keep the steel wet for long periods of time. The problem of long time-of-wetness is aggravated by the deck which consists of corrugated, galvanized steel sheets covered with a 150-mm (6 in.) thick asphalt-cement mix. Water penetrates the driving course through flexural cracks, corrodes the metal deck, and leaks through drain holes in the bottom of the corrugations onto the weathering steel members.

In 1979, the County Road 37 Bridge was sandblasted and remedially spray painted with one coat of zinc-rich primer and one top coat of rubberized paint. In 1983, the peeling top coat was scraped off. Rust spots on the primer were hand wire brushed, and the bridge was brush coated with a modified alkyd resin paint. Figures F-2 and F-3 show the conditions of the paint system in 1985, 2 years after the repainting.

The County Road 99 Bridge, built in 1979, was painted 2 months after it had been opened to service as a precautionary measure to avoid the corrosion problems found in the County Road Bridge. Because the bridge was new, the steel surface was not blast cleaned. The steel was painted with one coat of zinc-rich primer and one top coat of modified alkyd resin paint. It was repainted in 1983.

Franklin County. The three-span, multigirder bridge on Brand Road over the North Fork of Indian Run, Franklin County, was fabricated from A588 Grade A steel (Fig. F-4). Built in 1979, the bridge was severely corroding within 1 year, and large sheets of rust were separating from the lower one-third of the beam webs and lower flanges within 4 years. The interior beams were more severely affected than the exterior beams. When the bridge was inspected in mid-morning, after the fog had dissipated, heavy condensation was observed on all surfaces as well as beneath the rust sheets.

Chemical analysis of rust 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. Instead, the rapid rate of corrosion was caused by prolonged periods of wetness from condensation of moisture constantly evaporating from the stream, high incidence of fog, and runoff water leaking through the thin asphalt pavement on the corrugated metal decking. The bridge has only a 2.4-m (8 ft) clearance over the stream, and the wooded area shelters the bridge against the drying effects of sunlight and air circulation.

In 1983, the bridge was carefully sandblasted to bare metal and then remedially coated with a maintenance painting system used by the Ohio DOT, consisting of a semiquick drying red lead primer and an oil alkyd finish coat. Within 1 year of painting rust spots began to break through the coating (Fig. F-5). In 1987, the paint system had completely failed and the bridge was remedially painted for the second time.

Butler County. Six of ten weathering steel bridges in Butler County were remedially painted beginning in 1983. The Howard Road Bridge over Howards Creek, the second oldest weathering steel bridge built in 1971 with a 3-m (10 ft) clearance over a stream in a forested area, was corroding the most. The corrosion was particularly severe along the lower portion of the interior beams (Fig. F-6). After blast cleaning the steel for remedial painting, it was found that corrosion had perforated the 9.9-mm (0.39 in.) thick web of the W18 × 55 beams at four locations along the lower 50 mm (2 in.) of the web, mostly within 600 mm (2 ft) of the supports. The corresponding pitting corrosion rate was at least 410 μm (16 mil) per year. The corrosion penetration of the bottom flange was 130 μm (5.2 mil) per year, or a 3.2 mm (⅛ in.) flange thickness loss after 12 years of service. The underside of the bottom flange was deeply pitted.

Asphalt pavement fills the corrugations of the metal deck, exceeding the top of the deck by 25 mm (1 in.) at the sides to 50 to 62 mm (2 to 2½ in.) at the center. Water leaks onto the beams through flexural cracks in the pavement and drainage holes in the corrugated, galvanized metal deck. It also leaks through the deck joints at both ends. Bridges with a concrete slab were found to perform better than bridges with an asphalt pavement on a corrugated metal deck.

The beams of the Howard Road Bridge were strengthened by welding 6.4-mm (¼ in.) thick plates on each side of the web and 75 × 75 × 10-mm (3 × 3 × ⅜ in.) angles on each side of the bottom flange to web junction.

The steel was blast cleaned to a near-white condition and heavily coated with an asphalt-based paint. County authorities now periodically wash the bridges by high-pressure hosing.

The case studies from Ohio show that the use of unpainted weathering steel in bridges spanning slow flowing streams with clearances of no more than 2.4 m to 5 m (8 ft to 15 ft), sheltered from any breeze or wind, and exposed to a humid environment is akin to placing the structure in a steam room. Under such conditions of persistent dampness, the steel does not dry and cannot consolidate the initial rust film into a protective oxide.

F.2.2 California

Five A588 Grade A and Grade H steel bridges built in Redwood National Park, California, have a superstructure consisting of rolled beams and 50-mm × 200-mm (2 in. × 8 in.) or 50-mm × 250-mm (2 in. × 10 in.) nail-laminated timber decks. One bridge was built in 1973 and four in 1975. The first (Prairie

Creek Bridge) is located in the open, where it appears to be 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 like that of a rain forest. These environmental conditions are similar to those found in Southeast Alaska, except that the Redwood National Park bridges are 16 km to 24 km (10 miles to 15 miles) from the ocean, whereas the Alaska bridges are within sight of the ocean (see section F.3.).

An inspection of the bridges in 1981 revealed that the interior girders and the interior of the facia girders of two bridges on Lost Man Creek were severely corroding. Large slabs of rust separated from the bottom of the top flange, and large rust flakes fell from the webs. This corrosion pattern suggests that moisture is leaking through the timber deck. The rust debris accumulated 12 mm to 18 mm (½ in. to ¾ in.) deep on the top of the bottom flange along the full length of the girders. Retention of moisture prevented the formulation of the protective oxide coating. The corrosion penetration was estimated to be 60 μm (2.5 mils) per year per surface.

Again, frequent rainfall and high relative humidity combined to prevent the formation of the protective oxide. No evidence was found of significant concentrations of chlorides or sulfates in the rust scale.

To compound the problem the web of the facia girders was clad with redwood half-logs to give the weathering steel bridge the appearance of a log bridge. The crevice between the logs and the web fosters constant dampness. The resulting leachings from the wood attack the steel. Similar conditions exist in the crevice between the timber deck and the top flange of the girders.

F.3 MARINE ENVIRONMENT

F.3.1 Salt Deposition

Salt particles (nuclei) found in oceanic air are the residual salt from evaporated droplets formed by bubbles bursting at the sea surface [Woodcock and Gifford, 1949; Blanchard and Woodcock, 1957]. The numbers and sizes of these particles in marine air are related to the numbers and sizes of the bursting bubbles producing the droplets, and both are related to the wind force. Generally, the stronger the wind and the greater the distance it has traveled over the sea, the larger is the number of bubbles from breaking waves (white caps) on the sea surface, and the greater the salt particle concentration in the atmosphere [Kientzler et al., 1954; Woodcock 1953]. Thus, the sea surface becomes a source of salt particles in the overlying air. These particles are mixed upwards 2 km (6,500 ft) into the marine atmosphere over the ocean.

Onshore winds carry the salt-laden air inland where the salt particles can be deposited by wind impingement, gravitational fallout, and diffusion, or the salt can be washed out by precipitation. As a result, the salt concentration in the air and the rate of deposition is reduced with distance from the sea.

Waves breaking along the beach also release many bubbles. In this case the salt from the larger bubbles is rapidly deposited by gravitational fallout, making the marine environment less severe as the distance from the shore line increases. Indeed, steel samples exposed 24 m (80 ft) from the beach in Kure Beach, North Carolina, corroded much more than samples exposed 240 m (800 ft) from the beach. The former lot is said to represent a severe marine environment; and the latter, a moderate marine environment. Case studies of weathering steel bridges exposed to marine environments are described below.

F.3.2 Louisiana

Sixteen weathering steel bridges were erected in Louisiana since 1975. Following annual inspections for 8 years, it was found that 13 were performing satisfactorily while two were not. One of these structures is the five-span, A588 Grade A steel, plate girder bridge on Louisiana Route 23 over the Doullut Canal in Empirc, Louisiana. The bridge is located in the lower Mississippi River delta 64 km (40 miles) S-SE of New Orleans. It has a clearance of 16.8 m (55 ft) over open marsh country only a few miles from the Gulf Coast (Fig. F-7).

The boldly exposed surface of the northeast exterior girder exhibited a tightly adherent oxide of normal appearance. This surface is sheltered from the southerly, on-shore, 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 3 mm × 3 mm (⅛ in. × ⅛ in.) to 6 mm × 12 mm (¼ in. × ½ in.) (Fig. F-8). The steel surface beneath the rust is severely pitted (Fig. F-9). 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 with accumulated rust flakes were scaling. The threads and the bolt ends protruding from some nuts had corroded away.

Analysis of the rust particles indicated chloride levels of 0.09 percent and 0.22 percent. A poultice sample contained 0.11 percent chloride. These levels of chloride also have been found on test specimens exposed at the 24-m (80 ft) test lot in Kure Beach, North Carolina, which is characteristic of a severe marine environment. The contamination of the steel with airborne salt carried by the frequent breezes blowing from the Gulf together with the high relative humidity and high incidence of fog prevented the steel from developing the protective oxide.

The second bridge that was excessively corroding is the Larose Bridge on Louisiana 308 over the Intracoastal Waterway. This bridge, located 48 km (30 miles) S-SW of New Orleans, has corroded somewhat less than the Doullut Canal Bridge because of its greater distance from the Gulf. Both were erected in 1975.

F.3.3 Texas

The High Island Bridge on State Route 124 over the Intracoastal Waterway northeast of Galveston is exposed to conditions similar to those previously described for the Louisiana bridges. This bridge is located 8 km (5 miles) from salt water in the path of the prevailing onshore winds and has a 26-m (85 ft) clearance (Fig. F-10 and Fig. F-11). The steel surface is covered with rust flakes 1.6 mm to 3.2 mm ($\frac{1}{16}$ in. to ⅛ in.) in diameter. The oxide beneath the flakes is porous, and it appears that pitting of the surfaces is general. Highway officials who have regularly inspected the bridge report that the steel tends to rust to a point and, then, sheds the oxide coating and begins to rust again with the cycle repeating itself on an indeterminate period.

A number of tensile specimens were exposed among these structural members for 30 months before being tested (Fig. F-

12). It is evident that rust flakes have also formed on the tensile specimens. On the basis of the loss in load carrying capacity, as compared to that of nonexposed tensile specimens, the exposed specimens were corroding at a rate of 142 μm (5.6 mil) per year (see Figure 66 of *NCHRP Report 272*). Texas officials used this information to help support the decision to paint the bridge.

F.3.4 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. The bridges span across Little Salt Creek, Duke Creek, Black Bear Creek, and Steelhead River. They consist of two longitudinal plate girders, transverse floor beams resting on the top girder flanges, and a longitudinally laminated and treated timber deck. The four bridges are located within 400 m (¼ mile) of salt water. The area receives 3.8 m to 4.6 m (150 in. to 180 in.) of rain per year.

In 1981, the bridges were found to be severely corroding. The girders corroded most on top of the bottom flanges and along the web to flange junction. This area was covered with a 6 to 10 mm (¼ in. to ⅜ in.) thick layer of debris consisting mainly of rust scale that had fallen off the lower 600 mm to 800 mm (2 ft to 3 ft) portion of the web. Thick slabs of rust separated from the flange and weld. Although all steel was affected, the exterior facia of the seaward girder had corroded most. Because no significant concentrations of chloride were found in rust samples, it is believed that the heavy rainfall tended to wash off the chlorides. The steel was corroding at an estimated rate of 50 μm/year (2 mil/year).

The steel composition met the chemical requirements for A588 Grade A, with the exception of the 0.25 percent silicon content which is lower than the minimum specified since 1977 (Table 12 in Chap. Four of main text) for enhanced corrosion resistance.

The high annual rainfall and the proximity to the ocean create a humid environment along the southeastern coast of Alaska. Wind-driven rain, fog, mist, and water leaking through the timber deck frequently wet the steel. In addition, moisture condenses on the steel surfaces when the nightly temperature drops below the dew point. The leaking water and condensed moisture slowly drain down the web. As the web dries from the top down, the lower web portion remains wet longer and corrodes more. The accumulated rust debris on the lower flange retains moisture and aggravates flange corrosion. It is possible that water leachings from the treated timber deck also may be attacking the steel. State officials will decide in the near future whether to remedially paint the bridges.

As the examples from Louisiana, Texas, and Alaska show, salt-contaminated weathering steel along humid coastal areas cannot develop a protective oxide coating and, therefore, corrodes at an unacceptably high rate.

F.3.5 Contamination During Shipping

In contrast to the bridges located near the Gulf Coast, the cable-stayed Luling Bridge over the Mississippi River 29 km (18 miles) west of New Orleans, Louisiana, presented a different problem. The steel sections were fabricated in Japan and transported by ship to Louisiana. Upon arrival the sections were hosed with water to remove the sea salt residue that may have accumulated. Heavy rust scaling began to show up after erection had commenced. Not all sections were equally affected because some were transported in the hold of the ship while others were on the deck and received more salt. After the rust scaling became evident, the contact surfaces of the bolted joints of sections that had not yet been erected and all surfaces exposed to view were sandblasted. In addition the steel deck was sandblasted to a near-white condition before the wearing surface was applied, but it was not always possible to remove all rust from the pits in the deck. There is no indication at the present time that the initial rust scaling is reoccurring.

Walk-through inspection of the box girder revealed no surface texture problems as the interior was dry and appeared to be free of rain water intrusion. In some locations the edges of the bolted tower splice plates are bulging slightly as rainwater draining down the tower and capillary intrusion of moisture condensate are corroding the steel in the crevice (Fig. F-13). To contain the corrosion process, the edges of the splice may have to be caulked with a silicone caulking composition.

Similar initial rust scaling of salt-contaminated sections shipped from Korea and Japan has occurred on weathering steel bridges in Texas and California.

F.4 DEICING SALT

F.4.1 Salt Deposition

Deicing salt is deposited on weathering steel bridges by salt-laden runoff water leaking through the deck joints and by salt-laden traffic spray being kicked up behind trucks and settling on an overhead grade separation structure.

The most serious and common cause of corrosion problems in weathering steel bridges is that caused by runoff water leaking through deck joints and wetting for long periods of time the diaphragms, stiffeners, webs, flanges, and bearings in the vicinity of the joint. Such runoff water can migrate by wicking a long distance on the bottom flange as well as a short distance (about 150 mm (6 in.)) up the web of girders. The resulting severe corrosion of the steel and the locking of expansion bearings create major bridge maintenance problems. In their response to the survey (section 1.2 in Chap. One of main text), officials in 17 states cited problems caused by severe corrosion at leaking joints.

Although water leaking through deck joints wets a relatively small part of the total steel surface area, it is nevertheless a significant problem. Judging from past service experience, joints notoriously leak. As a result, the dampened weathering steel will corrode severely.

Salt deposition by traffic spray poses a severe corrosion hazard under combinations of low overhead clearance, high traffic speed, and heavy use of deicing salt. It is particularly severe in depressed and half-tunneled highways where the lateral confinement helps to direct the spray onto the overpassing bridge and prevents winds from clearing the spray (Fig. F-14).

While water runoff generally contaminates the steel in the vicinity of a leaking joint, traffic spray deposits deicing salts on all steel members over the traffic lanes. The deposition is greatest on the first few beams in the traffic path.

F.4.2 Michigan

Severe problems with corrosion of salt-contaminated weathering steel have led Michigan officials in June 1979 to institute a partial moratorium on the use of bare A588 steel for bridges in the following situations: (1) depressed roadway sections where low underclearance—less than 6.1 m (20 ft)—and vertical retaining walls trap salt sprays and other highway pollutants; and (2) urban and industrial areas where heavy roadway salting and automotive pollution create an aggressively corrosive environment. The moratorium was extended in March 1980 to all bare A588 steel applications for bridges on the state highway system. The unsatisfactory performance of weathering steel that brought about the complete ban was thoroughly documented and explained [Culp and Tinklenberg, 1980; Arnold et al., 1981; Allemeier 1981; McCrum et al., 1985]. The following description of the corrosion problems is based on the above documentation and two site visits.

Leaking Joints. The most readily apparent corrosion problem is caused by salt-contaminated runoff water draining onto the steel structure through expansion joints and leaking seals. Figure F-15 shows a hanger plate-and-pin connection of the I75 bridge over Fort Street, Detroit. The corrosion damage is most severe beneath the leaking expansion joint where it affects the web, flange, and cross bracing. The runoff water then advances for long distances along the bottom flange both upgrade and downgrade from the source and wicks up the web by capillary action (Fig. F-16).

Figure F-17 shows the corroded web behind the hanger plate of an expansion joint before and after blast cleaning. A circular bronze washer separated the hanger plate from the web. The ring around the pin hole indicates (Fig. F-17, right photo) the surface of the web matching the size of the bronze washer. The web surface was severely corroded and pitted by galvanic corrosion around the outside diameter of the bronze washer and by corrosion in the crevice between the hanger and web away from the bronze washer. The following worst-case penetrations of the web were measured [McCrum et al., 1985]: 400 μm/year (16 mil/year) pitting rate inside the gap; 125 to 150 μm/year (5 to 6 mil/year) corrosion rate inside the gap; and 75 to 100 μm/year (3 to 4 mil/year) corrosion rate along the lower web and flange (see curves 1, 2, and 3 in Figure 29, Chap. Six). The average corrosion rate was less than 50 μm/year (2 mil/year) a short distance from the worst corrosion areas.

The rust products that filled the crevice between the web and hanger plates in some cases locked the expansion joint in the cantilever span and resulted in damage to the headwall (Fig. F-18).

The problems caused by leaking joints appear to be worse for, but are not limited to, hanger plate-and-pin connections. They can occur wherever joints leak, including at fixed and movable bearings at abutments and piers (Fig. F-19).

Traffic Spray. Spray from traffic passing under a grade separation structure appears to be creating the same degree of corrosion damage as are leaking expansion joints. Regardless of source—leakage from above or spray from below—salt is the major contributing factor in accelerating the corrosion of weathering steel bridges [McCrum et al., 1985].

Measurements of section loss of weathering steel bridges in Michigan show that the first few beams subjected to traffic spray, especially the facia beam, are corroding almost twice as fast as those following later in the traffic path. This was observed in urban and rural bridges alike. The average corrosion rate of the flanges was 36 to 46 μm/year (1.4 to 1.8 mil/year) for the first three beams and 13 to 26 μm/year (0.5 to 1.0 mil/year) for the fifth and later beams in the traffic path.

In comparison, beams on urban and rural bridges not exposed to traffic spray were corroding at a lower rate of 5 to 15 μm/year (0.2 to 0.6 mil/year) [McCrum et al., 1985] (see curves 5, 7, and 8 in Figure 29, Chap. Six).

Figures F-20 and F-21 show typical corrosion patterns of beams contaminated with salt from traffic spray. Thick rust scales form under these conditions and eventually separate from the underlying steel (Fig. F-22). Rust debris accumulates on the top of the bottom flange, creating a wet poultice that continuously fosters corrosion.

The chlorides migrate to the rust-steel interface where they are trapped on the steel surface and in the pits. They are found to a much lesser degree in the rust scale. Therefore, most salt does not fall off with the scale. Also, hosing bridges with water can clear the debris and wash off surface contaminants, but it cannot remove salt trapped below the rust scale.

Mill Scale. The measured depth of pits on the surface of weathering steel bridge members that still retained about 90 percent of the mill scale usually did not exceed 250 μm (10 mil), which corresponds to a pitting rate of 50 to 75 μm/year (2 to 3 mil/year) [McCrum et al., 1985]. These pits are likely formed by galvanic corrosion along breaks in the mill scale in the presence of salt. The rate of pitting corrosion should diminish with time as the rust undercuts the mill scale and the ratio of mill scale to steel area continuously decreases.

Impurities. Weathering steel surfaces sometime develop pits that are much deeper than the average corrosion penetration. Preferential corrosion at impurities and rolled-in imperfections may be the cause of such deep pitting that has been observed in bridges in service and in beam specimens exposed outdoors for many years [McCrum et al., 1985; Albrecht 1988].

Remedial Painting. Thus far, the Michigan Department of Transportation has remedially painted 34 A588 steel bridges. It is planned that 36 additional A588 steel bridges will be remedially painted in 1989.

F.4.4 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 a tightly adherent oxide coating of a dark brown color with no evidence of unusual corrosion. 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 the Racoon River, West Des Moines, in a semirural environment. It has been subjected to heavy salting. The steel under the expansion joints was shedding rust flakes and was pitting. An area of flaky rust also extended along the outside web 50 mm to 75 mm (2 in. to 3 in.) above the bottom flange, and ran the full length of the outside stringers. There were minor areas of chloride drainage through the deck. Analysis of rust samples indicated 0.56 percent chloride content near the abutment and 0.52 percent at the flange weld. The Racoon River Bridge was remedially painted in 1984.

Leakage of salt-contaminated runoff water through deck joints is a common problem in the snowbelt states.

F.5 DEBRIS

Debris of flaky, granular, or fibrous material that holds water in place on horizontal surfaces of weathering steel bridges can lead to conditions resembling immersion and result in corrosion rates much higher than are found in atmospheric corrosion. Common forms of debris are accumulations of granules and flakes of rust that weathering steel continuously sheds, wind-blown dust and roadway debris at bearings, pigeon excrement, construction materials left behind in the interior of box girders, and bird nests.

To avoid horizontal surfaces upon which debris and traffic spray can accumulate, the Ontario Ministry of Transportation and Communication uses box girders instead of I-girders for grade separation structures. However, box girders are not free of problems and must be carefully designed so that water will not intrude (Fig. F-23) or cling to the underside of the bottom flange (Fig. F-24) [Manning 1984a; Manning et al., 1984b].

Figure F-1. Truss bridge over slow moving, shallow stream surrounded by trees (County Road 37 Bridge over Little Still Water Creek, Tuscarawas County, Ohio).

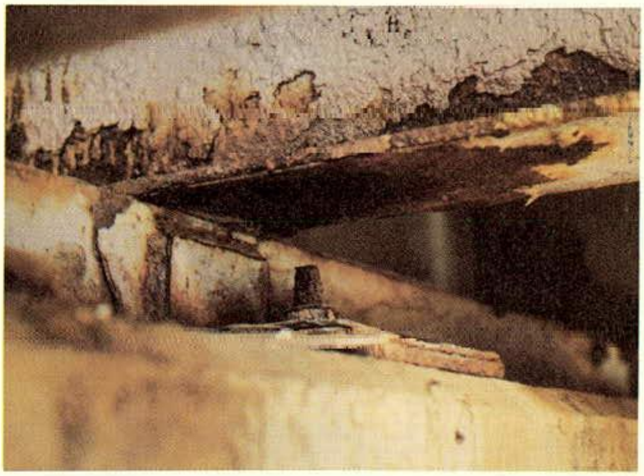

Figure F-3. Paint failure and severe corrosion of weathering steel stringers near abutment (County Road 37 Bridge).

Figure F-2. Paint failure and severe corrosion of weathering steel floor beam and stringers of remedially painted truss bridge (County Road 37 Bridge).

Figure F-4. Three-span continuous bridge with low clearance over creek in forested rural areas (Brand Road Bridge over North Fork of Indian Run, Franklin County, Ohio).

Figure F-5. Paint failure of beams one year after remedial painting (Brand Road Bridge).

Figure F-6. Simple-span bridge with clearance over creek in forested rural area (Howard Road Bridge, Butler County, Ohio).

Figure F-7. Five-span, continuous plate girder bridge over open marsh country in Mississippi Delta (Doullut Canal Bridge, Empire, Louisiana).

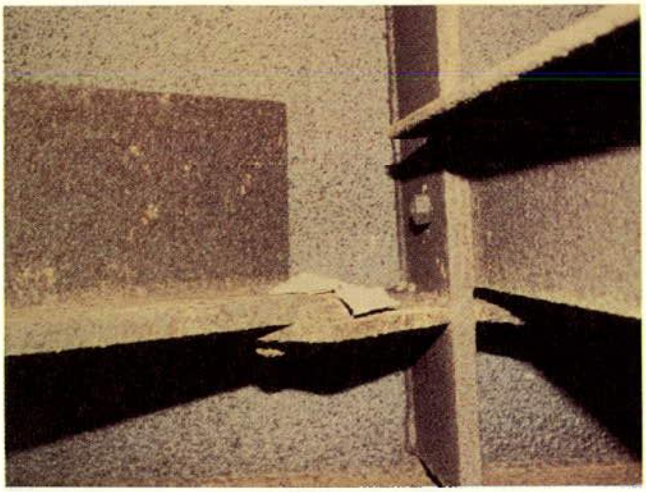

Figure F-8. Flaky texture of oxide film caused by contamination with airborne salt from on-shore fog and wind (Doullut Canal Bridge).

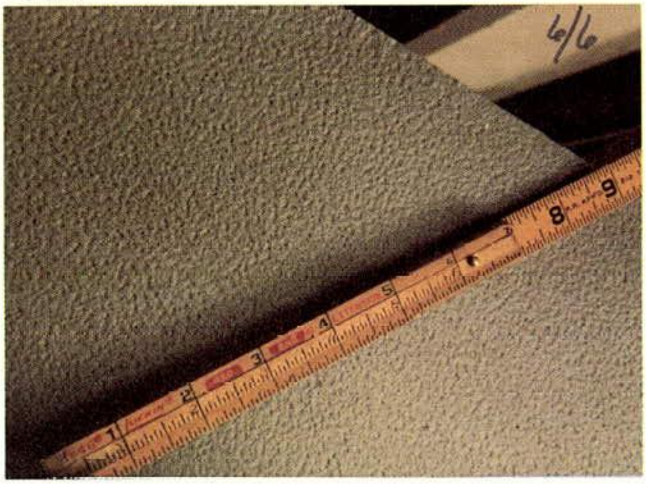

Figure F-9. Severely pitted surface of cross-bracing member after blast cleaning (Doullut Canal Bridge). [Louisiana Department of Transportation]

Figure F-10. Plate girder bridge located 8 km (5 miles) from Gulf Coast (Route I27 Bridge over Intercoastal Waterway, High Island, Texas).

Figure F-11. Substructure of High Island bridge

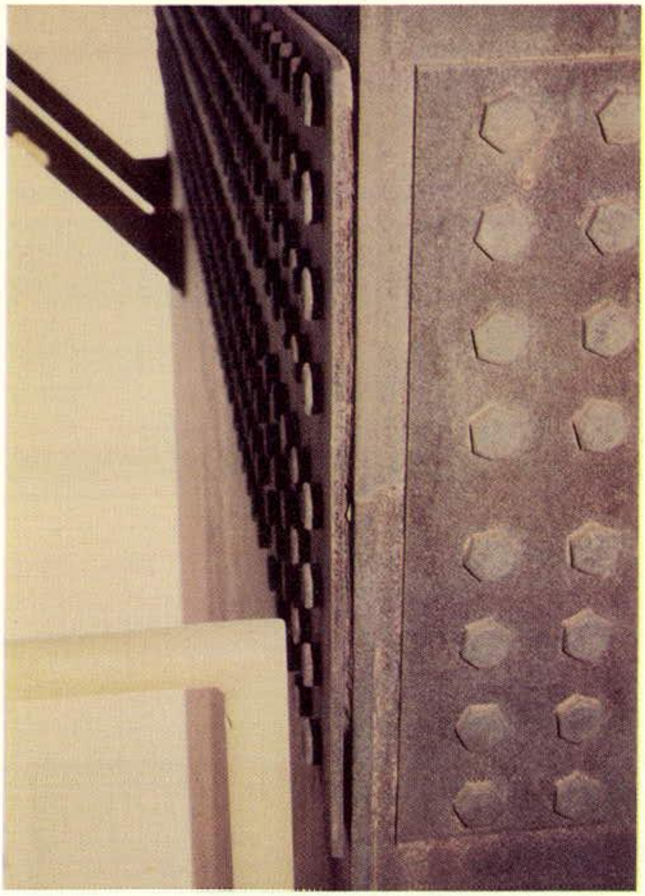

Figure F-13. Corrosion at edges of tower splice plates (Luling Bridge over Mississippi River, Louisiana).

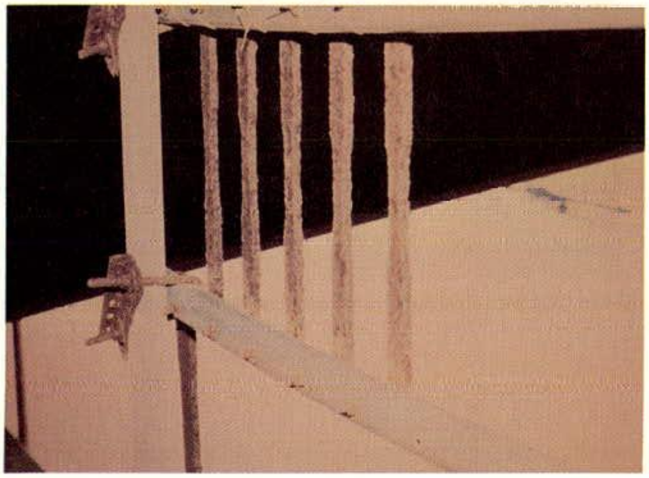

Figure F-12. Rust flakes forming on tensile specimens mounted on rack attached to substructure of High Island bridge.

Figure F-14. Grade separation structure with low clearance, lateral retaining walls, and salt-laden melting snow draining across roadway (8 Mile Road Bridge over US 10, Detroit, Michigan).

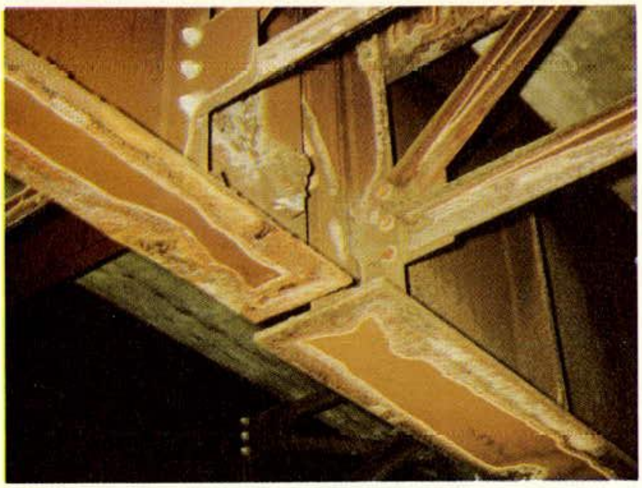

Figure F-15. Salt residue from leakage through expansion joint (I75 Bridge over Fort Street, Detroit, Michigan). [Culp 1980].

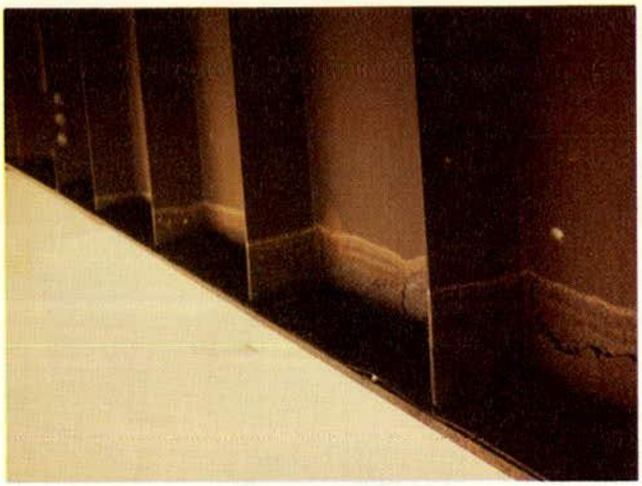

Figure F-16. Water migrating along bottom flange and wicking up web by capillary action (8 Mile Road Bridge over US 10, Detroit, Michigan).

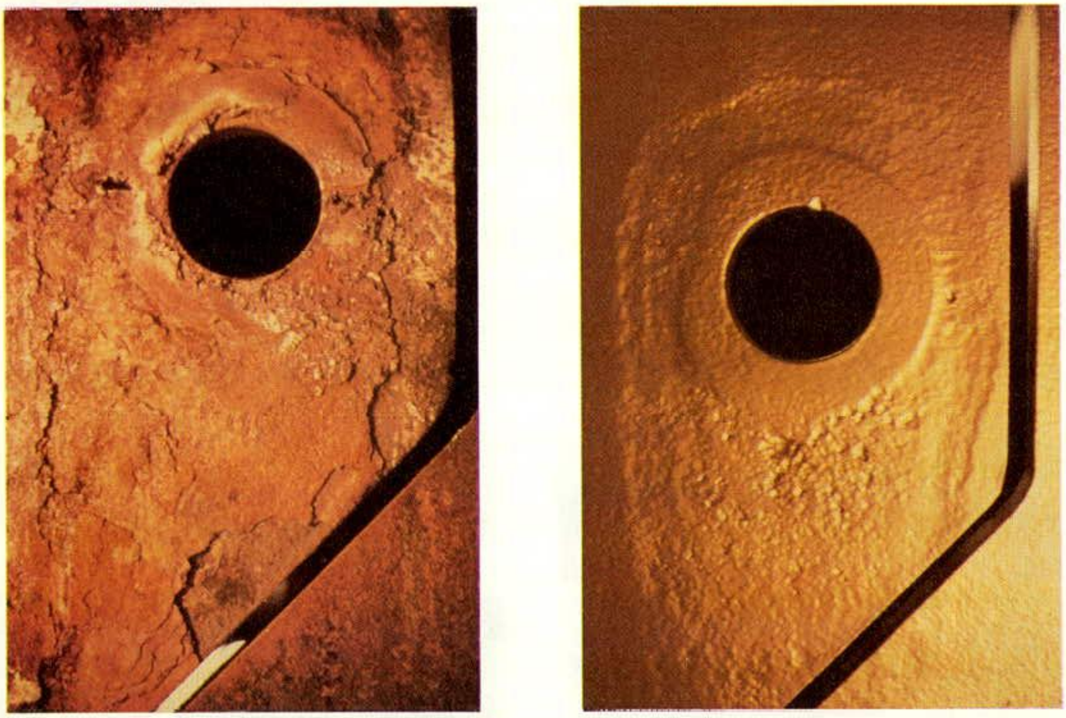

Figure F-17. Girder web behind hanger plate-and-pin connection: left photo—before blast cleaning; and right photo—after blast cleaning (I75 Bridge over 8 Mile Road, Detroit, Michigan). [Culp 1980]

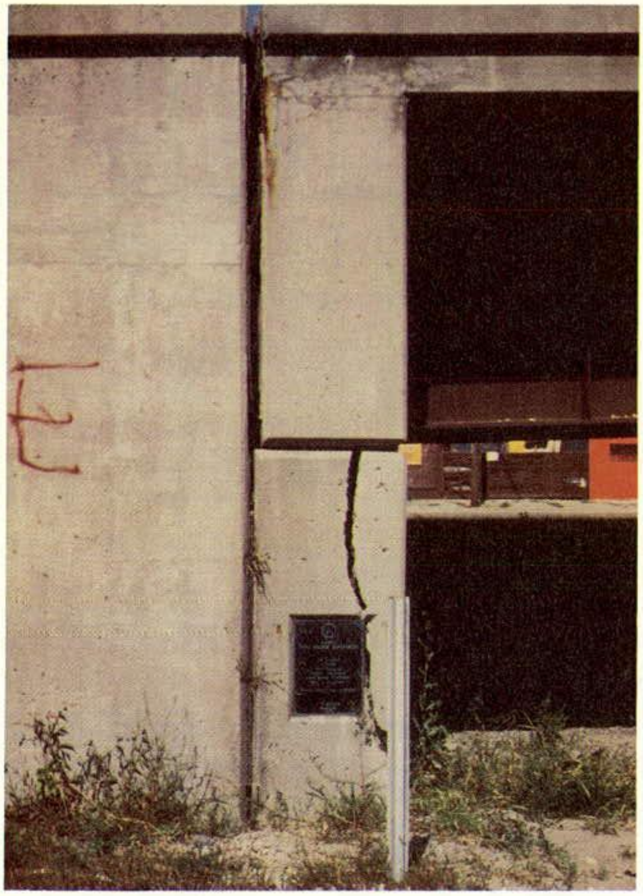

Figure F-18. Headwall separation and concrete fracture resulting from rust-frozen, centilevered expansion joint (8 Mile Road Bridge over US 10, Detroit, Michigan). [Michigan Department of Transportation]

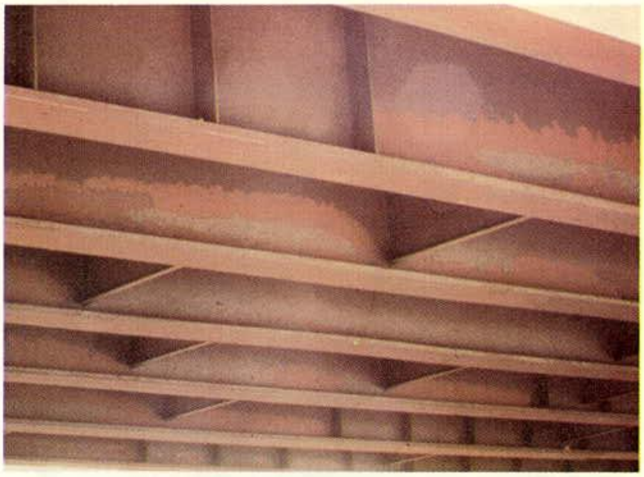

Figure F-20. Corrosion of beams subjected to traffic spray. [Michigan Department of Transportation]

Figure F-21. Accumulation of salt from traffic spray.

Figure F-19. Corrosion of steel below leaking joint at bearing (I75 Bridge over Fort Street, Detroit, Michigan).

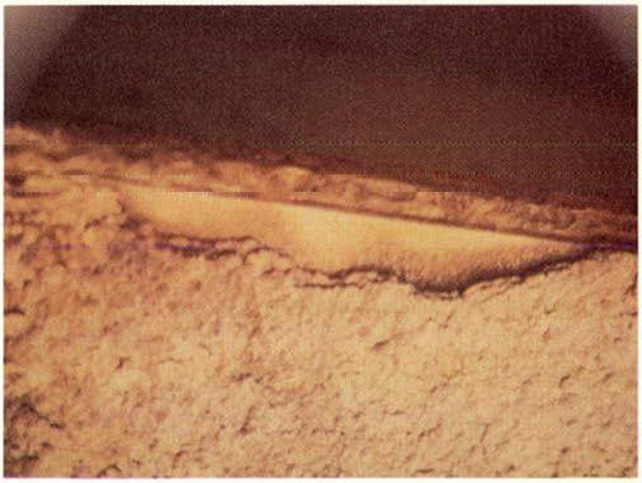

Figure F-22. Rust scaling of cover plate welded to tension flange (8 Mile Road over US 10, Detroit, Michigan). [Michigan Department of Transportation]

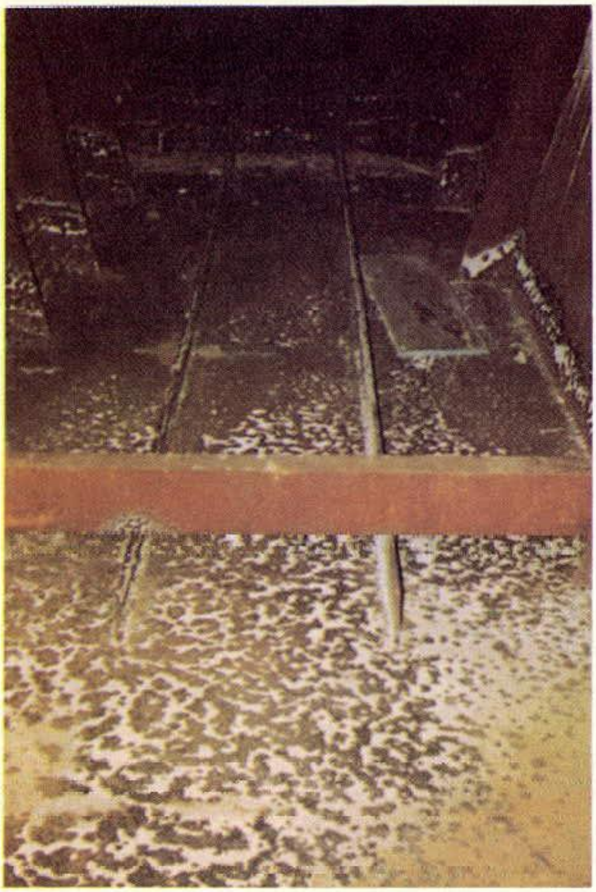

Figure F-23. Seepage at top and bottom of end diaphragm and formwork left behind after erection (Highway Bridge over Rideau River, Kemptville, Ontario). [Manning et al., 1984b]

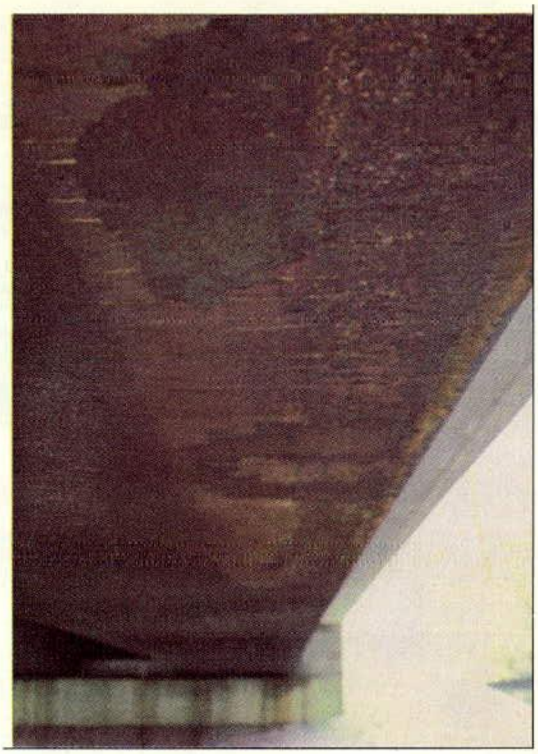

Figure F-24. Moisture clinging to underside of box girder. [Manning 1984a]

THE TRANSPORTATION RESEARCH BOARD is a unit of the National Research Council, which serves the National Academy of Sciences and the National Academy of Engineering. It evolved in 1974 from the Highway Research Board which was established in 1920. The TRB incorporates all former HRB activities and also performs additional functions under a broader scope involving all modes of transportation and the interactions of transportation with society. The Board's purpose is to stimulate research concerning the nature and performance of transportation systems, to disseminate information that the research produces, and to encourage the application of appropriate research findings. The Board's program is carried out by more than 270 committees, task forces, and panels composed of more than 3,300 administrators, engineers, social scientists, attorneys, educators, and others concerned with transportation; they serve without compensation. The program is supported by state transportation and highway departments, the modal administrations of the U.S. Department of Transportation, the Association of American Railroads, the National Highway Traffic Safety Administration, and other organizations and individuals interested in the development of transportation.

The National Academy of Sciences is a private, nonprofit, self-perpetuating society of distinguished scholars engaged in scientific and engineering research, dedicated to the furtherance of science and technology and to their use for the general welfare. Upon the authority of the charter granted to it by the Congress in 1863, the Academy has a mandate that requires it to advise the federal government on scientific and technical matters. Dr. Frank Press is president of the National Academy of Sciences.

The National Academy of Engineering was established in 1964, under the charter of the National Academy of Sciences, as a parallel organization of outstanding engineers. It is autonomous in its administration and in the selection of its members, sharing with the National Academy of Sciences the responsibility for advising the federal government. The National Academy of Engineering also sponsors engineering programs aimed at meeting national needs, encourages education and research, and recognizes the superior achievements of engineers. Dr. Robert M. White is president of the National Academy of Engineering.

The Institute of Medicine was established in 1970 by the National Academy of Sciences to secure the services of eminent members of appropriate professions in the examination of policy matters pertaining to the health of the public. The Institute acts under the responsibility given to the National Academy of Sciences by its congressional charter to be an adviser to the federal government and, upon its own initiative, to identify issues of medical care, research, and education. Dr. Samuel O. Thier is president of the Institute of Medicine.

The National Research Council was organized by the National Academy of Sciences in 1916 to associate the broad community of science and technology with the Academy's purpose of furthering knowledge and advising the federal government. Functioning in accordance with general policies determined by the Academy, the Council has become the principal operating agency of both the National Academy of Sciences and the National Academy of Engineering in providing services to the government, the public, and the scientific and engineering communities. The Council is administered jointly by both Academies and the Institute of Medicine. Dr. Frank Press and Dr. Robert M. White are chairman and vice chairman, respectively, of the National Research Council.

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