FORUM ON WEATHERING STEEL FOR HIGHWAY STRUCTURES
SUMMARY REPORT

Submitted to:

U.S. Department of Transportation
Federal Highway Administration
Turner-Fairbank Highway Research Center
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16. Abstract

On July 12-13, 1988, the Office of Research, Development and Technology of the Federal Highway Administration (FHWA) hosted a forum on Weathering Steel for Highway Structures. This conference took place at the Ramada Hotel in Alexandria, Virginia. The major objectives of this forum were to examine the state of the art in weathering steel use and maintenance, to develop rules for its use in new construction, and for maintenance of existing structures.

The forum brought together 131 participants representing Federal and State governments and industry.

The forum was organized into four main sessions, including a panel discussion and several individual presentations. This report summarizes the forum.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

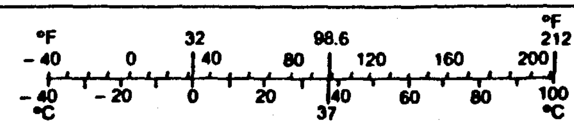
Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimetres	mm
ft	feet	0.305	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
		AREA		
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.093	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometres squared	km^2
		VOLUME		
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.028	metres cubed	m^3
yd^3	cubic yards	0.765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9	Celcius temperature	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
		AREA		
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
ha	hectares	2.47	acres	ac
km^2	kilometres squared	0.386	square miles	mi^2
		VOLUME		
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F



* SI is the symbol for the International System of Measurement

These factors conform to the requirement of FHWA Order 5190.1A.

(Revised April 1989)

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LIST OF ACRONYMS

AAPT	Association of Asphalt Paving Technologists
AASHTO	American Association of State Highway and Transportation Officials
AISC	American Institute of Steel Construction
AISI	American Iron and Steel Institute
ASCE	American Society for Civil Engineers
ASM	American Society for Metals
ASTM	American Society for Testing and Materials
DOT	Department of Transportation
FHWA	Federal Highway Administration
IBTT	International Bridge Tunnel and Turnpike
LRFD	Load Resistance Factor Design
NACE	National Association of Corrosion Engineers
NAPM	National Association of Purchasing Managers
NCHRP	National Cooperative Highway Research Program
R&D	Research and Development
SSPC	Steel Structures Painting Council
TRB	Transportation Research Board

I. INTRODUCTION

Forum Purpose

Weathering steel--a high strength, low alloy, corrosion-resistant material--was first used in bridge building in the 1960's by the Michigan Department of Transportation. Since that time, more than 2,000 bridge structures have been built across the country using weathering steel. Although the material has been successfully used in other applications--including guardrails, transmission towers, buildings, sign poles, etc.--it has been used most pervasively in bridge building.

From the beginning of its use, it was known that there were certain conditions of both design and location which would adversely affect its corrosion-resistant properties. However, through inexperience or overconfidence, weathering steel was frequently misused as a building material. Furthermore, there remained--and still remains--much to learn in terms of fully understanding how, where, and why it works (or fails to do so).

Over the years, these examples of misapplication and poor understanding have become increasingly evident as the structures require more and more maintenance--and ultimate replacement. Finally, in 1980, Michigan issued a unilateral moratorium on the use of weathering steel in its bridge structures. This epitomized Michigan's disillusion with the material.

Correctly designed and located, weathering steel is an extremely viable option. The purpose of this forum was to examine the state of the art in weathering steel's use and maintenance and, based on this, to develop rules for its use in new construction and for maintaining existing structures. Consequently, the Federal Highway Administration (FHWA) assembled experts in the field to speak to a group of concerned users on the various aspects of the weathering steel problem.

Forum Procedure

The FHWA Forum on Weathering Steel for Highway Structures was held on July 12 and 13, 1988, at the Ramada Hotel in Old Town Alexandria, Virginia. There were 131 participants drawn from 20 States and Ontario, Canada. These people represented a variety of organizations: FHWA; the Transportation Research Board (TRB); various State highway departments; universities, including Lehigh University and the University of Maryland; technical journals, including Civil Engineering and ENR magazines; trade associations, including the American Iron and Steel Institute (AISI), the American Institute of Steel Construction (AISC), and the Steel Structures Painting Council (SSPC); steel manufacturers, including Bethlehem Steel and U.S. Steel; paint manufacturers, including Porter Paint Company; power companies; and consulting firms.

The forum was organized into four main sessions, the first of which was an introduction to the forum and an overview of weathering steel. Besides the keynote speeches--which presented the Federal, State, and private sector viewpoints--this session's presentations covered the overall performance of weathering steel, design issues, and maintenance considerations.

Session 2 went into more depth regarding the uses of weathering steel. Topics presented included the mechanics of corrosion, the cost effectiveness of using weathering steel, and users' experiences with the material.

The second day of the forum opened with a split session, in which the topics of design and maintenance were separately considered. The majority of the participants attended the design session, which consisted of technical presentations on fatigue resistance, location of weathering steel, and jointless bridges, as well as a panel discussion/question and answer session on fatigue resistance. The maintenance session comprised two presentations on field experiences and a lengthy panel discussion which--through considerable participant feedback and interaction--resulted in several recommendations for future actions.

The fourth session consisted of a brief wrap-up of the key issues raised during the two morning sessions.

Most of the presentations given were based on formal technical papers and were accompanied by slides and/or vugraphs. These visual aids illustrated the broad range of applications for weathering steel, and showed both success stories and failures--and the reasons for success or failure. The speakers and session moderators represented a wide variety of perspectives, and drew upon their respective years of professional experience with weathering steel in either an academic, consulting, or highway department setting. The presentations and overall conference were well received by the attendees, as exemplified by their extensive participation in the interactive sessions.

Report Organization

This summary of proceedings focuses on the technical presentations and the question-and-answer sessions at the forum.

The findings and conclusions of the forum, together with the findings from other research, development projects, and field experience, form the basis for guidelines on use of weathering steel. As Section II explains, FHWA intends to publish these guidelines following the American Association of State Highway and Transportation Officials (AASHTO) and industry review.

Section III is a series of synopses and transcripts of each of the forum's 25 speeches, presentations, and panel discussions. Each synopsis includes a brief biography of the presenter/moderator, highlights of the presentation, and a detailed summary of the key points made by the speaker.

Finally, Section IV presents a list of attendees, complete with addresses and phone numbers.

II. FORUM RECOMMENDATIONS

One major objective of the forum was to evaluate and document a broad range of experience with uncoated weathering grade steel in highway structures and to identify locations, conditions, and details which have resulted in either successful or unsuccessful performance of the material. The findings and conclusions of the forum, in addition to other documented findings of research and development and field experience, were used to develop guidelines for application of uncoated weathering steel in highway structures. These guidelines have been formulated to assist bridge and highway owners in selecting locations and details appropriate for use of uncoated weathering steel.

The FHWA intends to publish these guidelines, after review by representatives of AASHTO and the industry, as part of a technical advisory on use of uncoated weathering steel in highway structures.

III. SYNOPSES OF FORUM PRESENTATIONS

OPENING INTRODUCTIONS

Moderated by D.K. Phillips

[David K. Phillips is the Associate Administrator for Research, Development (R&D), and Technology Transfer at the FHWA Turner-Fairbank Highway Research Center.]

Summary:

Mr. Phillips pointed out that as a result of this forum, an interim guideline on the use of unpainted weathering steel for future bridges and the protection, maintenance, and rehabilitation of existing bridges will be produced. The American Association of State Highway and Transportion Officials (AASHTO), particularly the AASHTO Subcommittee on Bridges, and the private sector will be asked for input into this development. The guideline will be an interim one, since this is a very active subject. In fact, there are 15 research studies currently in progress in the States, primarily under the Federal-Aid State Highway Planning and Research Program (HP&R) addressing weathering steel and structures. Mr. Phillips asked the participants to provide input and comments as appropriate for inclusion in the guideline and/or as suggestions for future studies.

Mr. Phillips then introduced the three keynote speakers:

- Lowell Jackson, Deputy Administrator, FHWA, representing the Federal perspective;

- Clellon Loveall, Assistant Executive Director, Tennessee Department of Transportation, and Chairman of the AASHTO Subcommittee on Bridges, representing the State and AASHTO viewpoint; and

- William Mathay, Consultant, representing the private sector.

KEYNOTE SPEECH #1

Presentation by L. Jackson

[Lowell Jackson was the Deputy Administrator of the Federal Highway Administration. Before his appointment, Mr. Jackson served as Director of the Colorado Department of Highways; earlier, he was Secretary of the Wisconsin Department of Transportation and, prior to that, Secretary of the Wisconsin Department of Labor. Mr. Jackson spent 21 years in academia at Purdue University and the University of Wisconsin. He is a Professional Engineer, with a master's degree in civil engineering from Purdue University. Mr. Jackson is the immediate Past Chairman of the Executive Committee of the Transportation Research Board. In 1986, he was appointed by President Reagan to serve on a five-person national council on public works improvements.]

Highlights:

Key points covered by Mr. Jackson included:

- Weathering steel appears to be the perfect construction material; when properly used, it can eliminate many of the costs and problems associated with steels that require periodic painting.

- We have gotten off track in terms of our effective use of weathering steel and need to review our approach, particularly with regard to design details.

Summary:

Mr. Jackson described the FHWA perspective on the use of weathering steel. FHWA is responsible for a multi-billion dollar stewardship and directs public funds for the design, construction, rehabilitation, and replacement of various elements of the country's highway systems. Increasingly, many of these dollars are used to replace highway bridges.

As part of its mission, FHWA sponsors and supports private and public efforts to develop and implement new materials, design methods, and construction techniques for lowering the cost of construction, extending a facility's useful life, and reducing maintenance costs. One such private sector initiative was the development of unpainted weathering steel (ASTM A588). When properly used, A588 steel can eliminate many of the costs, disruptions, and environmental problems of steels that require periodic painting.

Industry competition is a way of keeping costs down and fosters innovative use of materials. Like "the black and white wars" between asphalt concrete and portland cement concrete, there is a competition between steel and concrete as the primary bridge construction material. In deciding between the two, consider both the initial cost and the subsequent maintenance costs associated with a structure. Such a comparison is easier to do on an individual--rather than collective--basis.

The Nation's first weathering steel structures are now about 20 years old. Many have performed very well and have met or exceeded expectations. In fact, weathering steel appears to be the perfect material--it is both an "honest" building material and it affords reduced maintenance costs.

However, several States recently have discovered that the steel's protective corrosion mechanism has not resulted in a stable oxide layer, and corrosions continue to progress. There are many cases of moderate to severe corrosion, pitting of steel, and even significant section loss. This has raised serious concerns over the condition of existing bridges and the future use of unpainted weathering steel. Some States have stopped using weathering steel altogether.

Much research has been done on the problem and much is currently under way. The real challenge lies ahead. Through the collective wisdom of the forum, we need to put weathering steel--wherever it has gone off the track in terms of design detail--back <u>on</u> track so that it can again meet its highest design purpose of form following function, and providing a natural use of materials to cut down the life cycle cost of the material.

KEYNOTE SPEECH #2

WEATHERING STEELS--A PERSPECTIVE FROM THE BRIDGE COMMITTEE

Presentation by C.L. Loveall

[Clellon L. Loveall is the Assistant Executive Director, Bureau of Planning and Development, Tennessee Department of Transportation. Additionally, he chairs the AASHTO Subcommittee on Bridges, and is the former Chairman of the AASHTO Technical Committee for Structural Steel Design. Earlier, Mr. Loveall spent 10 years as Chief Bridge Engineer for the Tennessee Department of Transportation.]

Highlights:

Mr. Loveall presented a brief history of the development of weathering steel. Other key points covered by Mr. Loveall included:

- Certain locations are inappropriate for the use of weathering steel (e.g., areas with heavy use of deicing salts, coastal areas).
- Restrictions against field blast-cleaning of lead-based paints are making weathering steels attractive.
- In general, weathering steel performs very well and provides good aesthetics.

Summary:

Development of weathering steels began in the 1930's, when it was determined that by using small amounts of chromium and nickel combined with copper, a material could be produced to successfully resist corrosion. The first commercial application of weathering steel came in 1933 in the construction of railroad hopper cars. These cars hauled sulfur-bearing coal, and the use of the new steel doubled and tripled their lives. About a million such cars have since been constructed and put in use. Weathering steel was first used in building construction in the mid-sixties.

In 1964-65, the first seven weathering steel highway bridges were built. Since then, over 2,000 bridges have been built by State highway departments across the Nation. Most have served well, but some problems have arisen, illustrating that there are certain conditions under which unpainted weathering steel is not appropriate, and that care must be exercised in the use of details.

Michigan particularly has had problems: although it built more such bridges than any other State, in 1980, Michigan put a moratorium on any further use of weathering steel in bridges. As a northern State with heavy ice and snow, Michigan uses a lot of deicing salts on its highways. Consequently, many of the problem bridges were subjected to extensive salt spray at vulnerable construction points. We've learned from Michigan's experience that these are not conducive conditions for unpainted weathering steel. Other locations, such as coastal regions and those with commercial corrosion atmospheres, also may not be suited for the use of unpainted weathering steel.

Most weathering steel bridges are functioning well and provide the economy of an unpainted bridge. Engineers are generally satisfied with the steel's use, since they are looking for bridges with long maintenance-free service at the least overall cost and--in most instances--which provide good aesthetics. While some people may not particularly like the look of weathering steel, it is generally well received.

Field blast-cleaning of lead-based paints and other paints that might harm the environment is fast becoming a key issue. Because of limitations placed on blast-cleaning and paint overspray, the cost of painting over waterways, roads, and railroads will increase. One result of this situation is the use of less effective paints. Weathering steel provides a way around this problem.

Mr. Loveall pointed out that Tennessee has approached the use of weathering steel somewhat cautiously. In its weathering steel bridge projects, Tennessee tries to minimize the use of joints and does not use design details that would encourage salt to seek out and damage the A588 detail.

The bridges constructed from the steel have performed very well, and the State plans to make a thorough inspection to ensure that the bridges are in fact "doing what we think they're doing."

This raises the issue of how to determine when one of these bridges is performing well. The State doesn't have a great deal of expertise in this area. Furthermore, appearance alone is not helpful, since all weathering steel bridges look rusted. Mr. Loveall concluded, however, that although there are places where its use isn't appropriate, weathering steel is a very viable product when used in correct locations with proper design details.

Slides/Visual Aids:
Mr. Loveall showed several examples of what Tennessee is doing with unpainted and painted steels in bridge construction. He emphasized the aesthetics--blending and appearance--of weathering steel in construction.

KEYNOTE SPEECH #3

DEVELOPMENT AND APPLICATION OF WEATHERING STEELS

Presentation by W. Mathay

[William Mathay is a corrosion and marketing consultant. Previously, he worked for U.S. Steel in several capacities, including Chief Staff Engineer, Marketing Manager and Senior Planning Specialist.]

Highlights:

Mr. Mathay presented a paper written by Robert Schmidt. Key points covered by Mr. Mathay included:

- Weathering steels, because of their corrosion resistance, higher strength (and corresponding lower weight), and low maintenance requirements, have been used in numerous applications.

- Most such applications have been successful; where they have failed, the architect or designer has usually misapplied or misused the product.

- Weathering steel applications fail when the steels are not allowed the wetting/drying cycle that enables them to form their protective rust patina.

Summary:

One of the most important material advances in the past half-century has been the development of weathering steels. Research on the effect of composition on the corrosion resistance of copper and steel began in the early 1800's. Pioneering work by D.M. Buck (1912) on copper-bearing steels led to subsequent studies of the effects of phosphorus, chromium, copper, and nickel in imparting corrosion resistance to carbon steel. The first high strength, low alloy weathering steel was patented by U.S. Steel in 1935; it was also licensed in Europe, Japan, and Canada.

Characteristics of Weathering Steel

Weathering steel contains 2 to 3 percent chromium, copper, silicon, nickel, and phosphorus. (Certain weathering steels also contain relatively large amounts of manganese and vanadium to enhance their strength.) Weathering steels exhibit yield strengths about 1 1/2 times those of carbon steel and atmospheric corrosion rates several times greater.

When exposed boldly to the atmosphere, weathering steels develop tightly adherent protective oxide films: these seal the surface from further corrosion. The film's texture and protective nature are determined by (1) the degree of atmospheric contamination and (2) the frequency of surface wetting (by dew and rainfall) and drying (by the wind and sun). The protective film will <u>not</u> form if weathering steels are used in the submerged condition or remain wet for long periods of time.

Early Applications of Weathering Steel

The first application of weathering steel was in railroad hopper cars (1934). This was an extremely good application as compared with carbon steel, which corroded quickly. Weathering steel provided not only corrosion resistance, but also enabled lighter-weight cars to be built. In the early years, these cars were painted.

New markets for weathering steels started to develop in the 1930's. In the fifties, they were fully developed and produced as structural steels.

Unpainted Applications

The first buildings made of unpainted weathering steel were the John Deere and Co. buildings in Moline, Illinois. The first weathering steel highrise (the Chicago Civic Center) was constructed in 1963; soon after, the USX Building (one of the world's tallest buildings), the Ford Foundation Building, and hundreds of other buildings were also constructed. Besides architects, sculptors (including Picasso) began to use weathering steel.

Structural failures have occurred when builders and architects have not adhered to published guidance. Specifically, problems have occurred when moisture comes into contact with panel backsides, consequently keeping the steel wet for long periods of time and prohibiting development of the protective film. This problem could have been eliminated by either painting the backsides or providing for adequate drainage.

Electrical Transmission Towers

The market for weathering steels began to expand rapidly in the 1960's as the unique properties of corrosion resistance and strength--combined with the steels' potential for low maintenance and weight saving--caught the eye of the design engineer. Weathering steel was tested for use in electrical transmission towers to avoid the expense and inconvenience of painting. The application was successful: for example, the Virginia Power and Light Company has used more than 8,000 weathering steel towers over the past 25 years.

There were some problems reported with this application, however. In the mid-seventies, the Savannah Power and Light Company experienced extensive thickness loss around the base of many of its towers, requiring tower replacement. The cause of this problem was vegetation which had been allowed to grow around the base of the towers. This, combined with high humidity, meant that the wetting/drying cycle could not occur and the patina did not develop.

When the towers were removed, some of the bolted joints had corrosion product accumulations so large that they actually caused distortion and bowing of sections ("pack-out"). Guidelines were subsequently developed on proper bolt spacing, and towers are now constructed in accordance with these guidelines.

Bridges

The largest market for weathering steels to date was initiated in 1964 when Michigan built its first weathering steel bridge. Iowa, Ohio, and the New Jersey Turnpike Authority quickly followed suit. Since then, over 2,000 bridges have been constructed of weathering steel in the U.S. To further reduce highway maintenance, some States also have employed weathering steel for light standards and guardrails. This has been done mostly in rural areas where the steel blends well with the surroundings. (Another use of weathering steel first pursued in the 1960's was in chemical and bulk paper mill plants; these structures were usually painted.)

In Europe, weathering steel--although used somewhat in buildings--has mostly been used for bridges. There are about 100 weathering steel bridges in Great Britain; the material is also used in Finland, Germany, Holland, Italy, Switzerland, and Spain, and, elsewhere, in Japan, Brazil, and the Arab countries.

While most of the weathering steel bridges in this country are performing satisfactorily, bridges in several States are experiencing higher than normal corrosion because of contact with deicing salts and prolonged periods of wetness. This problem was first recognized in 1974 in Michigan. Followup Statewide inspections revealed that areas on structures with moderate to heavy corrosion were (1) near leaky expansion joints, or (2) the bridge decks where deicing salts drained onto girder flanges. Also, in a tunnel-like situation where traffic sprayed deicing salt on the steel, a nonprotective oxide developed. These problems caused Michigan to order a complete moratorium in 1980 on the use of weathering steel for bridges, guardrails, sign posts, and light standards. Other States also stopped using the steel or restricted its use.

Bridge design engineers need to provide detailing to divert the flow of runoff water from the structure and to prevent ponding and avoid ledges and crevices which accumulate debris and hold water and chlorides. Another important factor is site location. Weathering steel should not be used where recurrent wetting by salt spray can occur, in coastal and saltmarsh areas, or where highly corrosive industrial contaminants are known to exist.

In conclusion, over the past 30 years, most weathering steel structures have performed satisfactorily. As with all new products, problems have occurred: manufacturers' guidelines have not always been followed and design details on many occasions have been inadequate. Through careful design and appropriate site selection, weathering steel can provide limited maintenance and economic advantages over other steel structure materials.

PERFORMANCE OF WEATHERING STEEL BRIDGES
Presentation by R.S. Fountain

[Richard S. Fountain is a Professional Engineer with a varied career in civil engineering. He was educated at Georgia Tech, and worked for several years with the Georgia Department of Transportation in the Bridge Department. Mr. Fountain worked 8 years with the Portland Cement Association, 20 years with the United States Steel Corporation, and 5 years with Parsons Brinckerhoff.]

Highlights:

Mr. Fountain presented the August 1982 (First Phase) report of the American Iron and Steel Institute's Task Group on Weathering Steel. Key points of the study discussed by Mr. Fountain included:

- Bridge areas which exhibited moderate to heavy corrosion were exposed to continuous wetness and deicing salts through either leaky seals, open expansion dams in bridge joints, or other sources.

- Deicing salts are the major contributor to corrosion.

- Mill scale has little effect on weathering steel's long-term performance and should only be removed for aesthetics.

- The accumulation of chlorides on steel surfaces aggravates corrosion by providing a poultice environment.

Summary:

The AISI Task Group on Weathering Steel was formed primarily in response to Michigan's 1980 moratorium on the use of weathering steel in its highway program. The task group--whose members included steel industry corrosion and metallurgical representatives; State bridge engineers from Michigan, Illinois, Maryland, New York, North Carolina, and Wisconsin; the Chief Engineer of the

New Jersey Turnpike Commission; and representatives from the Federal Highway Administration, the West Virginia Materials Division, and the American Institute of Steel Construction--set the following agenda:

- Inspect existing weathering steel bridges;
- Study the use of deicing salts;
- Determine the effect of mill scale;
- Study the effects of corrosive deposits;
- Review studies on the effect of fatigue life; and
- Develop a cleaning and painting specification.

The group's final goal was to issue a report to help States evaluate the use of weathering steel.

Inspection Program

The first task was to prepare an inspection program and develop a uniform inspection procedure and reporting form. The task group selected the following bridge site geometric configurations for inclusion in a cross-section investigation:

- grade separation, urban;
- grade separation, rural;
- stream crossing; and
- bridges with the "tunnel effect."

The effects of light and heavy salt use and various climates were investigated in combination with these geometric conditions. The sample comprised 49 bridges drawn from the 900+ bridge combined inventory of the participating States.

Bridges Inspected. Mr. Fountain showed a series of slides of structures in Wisconsin, Maryland, New York, North Carolina, Illinois, and Michigan, and along the New Jersey Turnpike to describe the findings of the task group inspections.

Summary of Inspection Findings. Most of the steel inspected had developed the expected oxide coating. Depending on exposure type and environment, the texture of this coating varied from fine granular to flaky laminar. Of the 49 bridges inspected, 30 percent showed good performance in all areas; 58 percent showed good overall performance with moderate corrosion in some areas; 12 percent showed good overall performance with heavy local corrosion in some areas. One or more of the following factors were responsible for the formation of nonadherent flaky rust:

- Water runoff, contaminated with deicing salts, draining through leaky seals and open joints or expansion dams. This was by far the most prevalent and important factor leading to the flaky rust formation.

- Tunnel-like conditions concentrating salt-laden road sprays from traffic passing under the bridge; this causes accumulation of water, dirt, and salt on the superstructure.

- Water and deicing salts leaking through cracks in the concrete deck.

- Salt-laden water runoff draining directly over the bridge edge onto the steel superstructure.

Effects of Deicing Salts

Deicing salts are the single most active factor influencing corrosion rates. The rate of corrosion, as measured by the quantity of nonadherent rust, was greater in areas where the steel was directly exposed to deicing salts at leaky joints or from traffic spray.

Effects of Mill Scale

There is no evidence that weathering steel with intact mill scale performed less well than sand-blasted steel. Tight mill scale takes a long time to weather off and leaves the patina characteristic of weathering steel. Thus, mill scale only needs to be blasted off for aesthetic considerations.

Effects of Corrosive Deposits

Bridge rust samples were analyzed by wet chemistry, spectrographic, and X-ray diffraction techniques. The results indicate that nonadherent deposits in localized areas are caused by deicing salts and considerable moisture. Sulfates do not appear to be a factor in the formation of the nonadherent deposits. The accumulation of these deposits on horizontal surfaces further aggravates corrosion by providing a poultice. Bird excrement on the steel surface has no apparent effect on corrosion.

Effects of Corrosion on Fatigue Life

The task group concluded that it did not have enough information to assess the effects on fatigue life of surface roughness due to corrosion. No evidence of fatigue problems was observed on any of the relatively young bridges inspected.

Painting

The task group contacted the Steel Structures Painting Council to study the cleaning and painting of weathering steel after exposure to aggressive environments.

Conclusions

The use of weathering steel for bridges depends on individual engineering judgment. To best make this determination, consider the following factors:

- Site--evaluating the bridge location is essential. Consider overall bridge conditions; avoid any geometric or natural conditions creating continuous wetting and very high chloride concentrations.

- Economics--compare the cost of initial painting of other steels to the cost of weathering steel.

- Safety.

- Aesthetics.

Design and detail are important in avoiding possible structural problems. This illustrates the need for thorough quality control and assurance for fabricating and installing bridge joint systems, including a continuing field inspection program and a remedial painting capability.

The task group did not produce any evidence supporting major changes in how decisions are made on the use of weathering steels. The majority of the weathering steel actually installed is performing satisfactorily. However, there are notable exceptions in Michigan where local conditions indicated the need to re-evaluate the use of weathering steel in bridges. These local environmental considerations include the unusually heavy use of deicing salts, and design details such as pin/hanger connections for cantilevered/suspended spans. Michigan has revised its detailing practice to avoid the use of suspended spans when possible, but has not changed its rate of salt application. Consequently, when weathering steel is used in Michigan, a protective coating will be applied.

DESIGN RAMIFICATIONS

Presentation by R.L. Nickerson

[Robert L. Nickerson has a bachelor's degree in civil engineering from Bucknell University, and a master's in civil engineering from the University of Illinois. He worked for the former U.S. Bureau of Public Roads, and in the 1970's, was employed by a consulting engineering firm. Mr. Nickerson is currently Chief of the Design and Review Branch at FHWA Headquarters. He has extensive bridge field experience, both at the regional and division office levels.]

Highlights:

Key points covered by Mr. Nickerson included:

- Design is the key factor determining a structure's cost and long-term performance.
- Unnecessarily complicated design creates problems which ultimately shorten a structure's life.
- Joints and scuppers are structural details which can frequently be eliminated to simplify bridge design and enhance bridge performance.

Summary:

The ramifications of the decisions made by designers are critical in the performance of weathering steel. The designer holds the key to the cost of a structure and to its long-term performance. Cost is a very important parameter these days; consequently, that material--whether it be concrete or steel--which offers the most cost-effective solution must be selected. The design engineer determines:

- What will be built and where;
- What material will be used; and, probably most importantly,
- What shape those details and materials will take.

"What will be built" here refers primarily to highway bridges; "what material will be used" refers to weathering steel--i.e., ASTM A588 or the AASHTO equivalent, M222. (These designations were recently changed to ASTM A709 and AASHTO M270; also all structural steels for highway bridges are now categorized under a single specification. This means that the designer only needs to specify AASHTO M270 and the appropriate grade, e.g., 36,000, 50,000, or 100,000 yield steel. This may result in more of these higher strength steels being used in weathering applications, since their availability was previously not well known.)

The shape of the end product--or the details that the engineer picks--many times determines not only construction cost, but also product performance. There are many examples of both success and failure in the use of weathering steel in bridges: to a great extent, this success or failure was influenced by engineers' decisions. While success can be accidental, failure is almost exclusively the result of poor environmental location or detail selection.

Where to use weathering steel is a factor that has to be considered at the earliest stage of development. In 1982, the American Iron and Steel Institute in its first phase report, "Performance of Weathering Steel in Highway Bridges," concluded that "consideration of overall bridge site conditions which create continuous wetting and very high concentration of chlorides must be avoided." This conclusion is still true today, yet designers persist in specifying unpainted steel in these environments. To assist designers, the steel industry provides advice on proper location of steel structures, including A588 weathering steel. In general, though, environmental considerations are fairly simple to define. More difficult to define are the proper welding details to be used on a given structure.

Unnecessarily complicated details have many shortcomings:

- They are difficult to fabricate properly and may entrap discontinuities which may reduce the life of the structure.

- They are usually fatigue-sensitive, thereby potentially reducing the life of the structure.

- They make structure inspection more difficult.

- They may trap moisture or provide locations for pigeons to nest; these will affect bridge performance.

- Last, but not least, more complicated details are more costly to construct.

The proper approach to design is thus to keep it simple.

Two structural details are of equal concern: bridge joints and scuppers. Although bridge joints are almost universally used, not too many of them have long-term performance records. The simplest joint available for larger movements is the finger joint. Although far from perfect, if detailed properly, it will outperform any other joint and be easier to maintain. Jointless bridges are potentially an even better solution.

Scuppers--or bridge deck drains--are another issue. An FHWA research report on bridge deck drainage recommends leaving 1,400 ft (426.7 m) between scuppers for a typical bridge. This would eliminate bridge deck drains from most of our highway bridges--and, consequently, many of the associated performance problems. Is the ultimate solution then the design of jointless and scupperless bridges?

The goal of this conference is to identify areas, details, or locations where designers have to be particularly careful when utilizing weathering steel. This same goal, however, has to apply to <u>all</u> structures--concrete, painted steel, and weathering steel. Weathering steel has a proper place in our highway environment, provided its limitations are kept in mind and proper details and location are specified so as to make maximum, cost-effective use of this material.

<u>Slides/Visual Aids</u>:

Mr. Nickerson showed several slides illustrating (1) good bridge construction and (2) problems arising from improper design details, notably joints and scupper systems. Points made during the slide presentation included:

- The use of joints can lead to rusting, splitting, and leaking. In particular, many modular type joints haven't worked well, and can cause rough riding and leaking. Therefore, be aware of long-term performance when specifying joints in bridge design.

- When using finger joints, be sure to include a properly designed trough underneath to collect/control water.

- As a safeguard measure, weathering steel has been partially painted under joints, at abutments, pier joints, fascia side of fascia girder, etc. Note, however, that this painting system can also fail.

- Scupper systems do not have a good success rate: problems can include clogged pipes and flat run drainage systems.

- In box girder bridges, watch for openings that can be used for nest building. This is detrimental for both performance and inspection.

MAINTENANCE CONSIDERATIONS

Presentation by B.R. Appleman

[Bernard R. Appleman has a Ph.D. in chemistry from Ohio State University, and did post-doctoral research at Columbia University. He worked for the Navy in the area of marine coatings for 3 years; he then spent 5 years in R&D with the FHWA, and was responsible for FHWA's R&D program in coatings. Subsequently, he spent 3 years with Exxon. Since 1984, Mr. Appleman has been Executive Director of the Steel Structures Painting Council (SSPC).]

Highlights:

Key points covered by Mr. Appleman included:

- o A maintenance program must reflect a unified bridge management policy in terms of authority and objectives.
- o Many aspects of bridge inspection, evaluation, and analysis need to be quantified, specified, or otherwise standardized to ensure consistent application of techniques and interpretation of results.
- o Many States and other entities have guidelines available on various aspects of maintenance; these need to be compiled.

Summary:

Mr. Appleman delineated the five basic steps entailed in developing a general maintenance program for weathering steel bridges.

1. Recognize the Need for Maintenance

Getting to this point is in itself a major milestone, as the previous guiding philosophy was "erect and neglect." As this forum demonstrates, however, recognition of the significance of maintenance is increasing.

2. Plan a Maintenance Program

Bridge maintenance must be planned in accordance with a unified policy to ensure that it is conducted in a regular and continuing fashion.

Identify Support and Authority. The first step in developing this program is for officials to make a commitment in terms of program authority, budget, and resources.

Identify Objectives. Frequently, there are different sets of maintenance objectives (e.g., preventing corrosion, maintaining appearance, performing lowest cost maintenance only, etc.). Examine these objectives and ensure that they are consistent with the desired goals.

Identify Resources. Identify all available funds, personnel--e.g., specialists (individuals, groups, agencies) in corrosion, design, painting, etc.--and technical reports or guidelines.

Review Scope. Determine the number, type, and size of bridges to be maintained. Set any relevant limitations on the maintenance "inventory" (e.g., inspect/maintain only those bridges over 10 years old, those in high-corrosion-inducing environments, etc.).

Estimate Budgets. Estimate both a budget for what is needed to do the job properly, and a budget which addresses minimum maintenance requirements only. In preparing these budgets, consider the following points:

- All budgets are tight.
- Corrosion is gaining acceptance as a budget item.
- Just as there are consequences and costs associated with not establishing a regular maintenance program, there are specific benefits for doing so, including avoidance of future repairs, ability to make most efficient use of funds, and the potential for design improvements in the future.

- Generate or obtain definitive data on the costs for performing versus neglecting maintenance.

3. Assess the Condition of Structures

Inspection Procedures. Since the States cannot initially perform an extensive inspection for corrosion on all weathering steel structures, they must prioritize their inspections. The States should perform a preliminary screening to determine what structures would benefit most from in-depth inspection. Next, establish standardized inspection procedures as per the following guidelines:

- Locations on bridge;
- Inspection equipment, including inspection standards and criteria;
- Inspecting corrosion, including inspection techniques and results for evaluating oxide appearance, scaling, pitting, metal thickness, and oxide sampling;
- Environmental factors, including chloride detection, leaky joints, and moisture pockets; and
- Bridge factors, including pins, bolts, rivets; roadway-bridge configuration; traffic volume; and salt usage.

Evaluation and Analysis. The next step is to evaluate and analyze the data collected in the inspection. Is there chloride in the corrosion products? What types/forms of oxide are present? How are defects distributed? (Distribution is important in determining a method for how much and where to paint.)

Develop condition ratings for both worst case (those areas subject to the highest amounts of salt or moisture collection) and general case (to show whether there are things occurring away from the worst case areas). These ratings should be used to determine appropriate action.

In evaluating and analyzing the data, additional data may be required if the inspection was not detailed enough or an insufficient number of points on the structure were examined.

4. Determine Actions and Procedures for Conducting Maintenance

At this point, maintenance options are identified. One such option is no action, i.e., defer maintenance or ignore the problem altogether. Other options include: additional inspection; partial or complete painting of the structure; and other maintenance actions--e.g., install new drains, replace/add components, clean the structure (periodically wash the surface with high pressure water), and reduce or replace salts.

While much remains to be developed regarding how to determine the appropriate option, some guidelines--particularly in the area of painting--have already been established, and simply require compilation.

Painting Procedures. In establishing any painting program, certain general procedures need to be established addressing surface preparation, coating materials, application/film thickness, quality assurance, specifications, and contracts. The most significant of these areas, for both carbon steel and--especially--weathering steel is the surface preparation requirements. The surface must be:

- Rough, so paint can bond to the steel; blast-cleaning is usually used to achieve this effect.

- Visually clean--i.e., free of rust, mill scale, oil, or other visible contaminants.

- Chemically clean--i.e., free of nonvisible contaminants, primarily chlorides which will become imbedded in the steel and which are difficult to remove.

Surface Preparation Methods. The most commonly used method for cleaning the surface is dry abrasive blasting. This method, however, will generally not remove most of the chlorides. Furthermore, weathering steel usually takes more effort to dry blast than do other steels. Other proposed cleaning methods, in descending order of effectiveness, include wet abrasive blasting, steam cleaning, high pressure water jetting, low pressure water flushing, hand/power tool cleaning, and chemical/solvent cleaning.

Although many typical highway bridge coatings have been proposed for use with weathering steel, there are some environmental restrictions on these coatings, such as limitations on use of lead paint and the amount of solvent in the paint. Many evaluations are under way to determine the most effective coatings for weathering steel; the data from these studies need to be compiled. One important project, sponsored by FHWA and performed by SSPC, is examining several coatings and using actual bridge steel parts in various stages of chloride contamination to conduct the test.

5. Implement the Maintenance Program

The last step is to implement the maintenance program. This comprises the following:

- Recognition/commitment;
- Compilation of current technology;
- Attention to critical needs;
- Identification of gaps in knowledge;
- Development of new technology to meet information gaps; and
- Utilization of best technology.

Mr. Appleman concluded by summarizing our major gaps in knowledge:

- Need a definitive, standard technique to examine/evaluate corrosion;

- Need to know how much of the surface needs to be painted (all or limited areas);

- Need to determine the acceptable level of chloride that can be left on the surface prior to painting;

- Need to develop cost-effective procedures for cleaning contaminated surfaces;

- Need to know the long-term consequences of ignoring corrosion;

- Need to know what the best system is in terms of cost-effective coating materials for contaminated surfaces; and

- Need to know the merits of periodic cleanings.

While it may seem as if there are more problems than answers, note that the issue of weathering steel maintenance has only been considered for a very short time--5 or 6 years. We have made some progress: we now need to come up with definitive programs for both the long and short term.

CORROSION MECHANICS

Presentation by M.E. Komp

[M. Edward Komp holds a bachelor's degree in chemical engineering from Youngstown University and a master's degree in metal engineering from the University of Pittsburgh. As a Professional Engineer, he worked for 26 years with the USX Corporation before retiring in 1986.]

Highlights:

Mr. Komp focused his discussion in two areas:

- The mechanics of atmospheric corrosion as they apply to carbon and low alloy weathering steel; and

- Some of the problems that exist in the rating systems used to measure the corrosion resistance of these alloys.

Summary:

Corrosion in the atmosphere basically follows the same mechanics as under immersed conditions: both are electrochemical in nature. Anode and cathode areas are formed on the surface of the steel. Oxidation occurs at the anode, breaking down the iron to form ferrous ions and electrons. The electrons flow through the metal to the cathodic areas, where they are used in oxygen reduction, producing hydroxyl ions.

The basic difference between atmospheric corrosion and immersed corrosion is that the former takes place in a thin layer of moisture on the steel surface. Reactions in this moisture layer form ferrous hydroxide. Oxidation of the ferrous hydroxide produces magnetite (Fe_3O_4) and ferroxyhydroxide (FeOOH); this last is the main ingredient of rust.

The corrosion film produced through these reactions acts as a barrier that prevents oxygen, water, and other corrosive elements from diffusing into the steel, as well as preventing the outward diffusion of the ferrous ions. Because this diffusion barrier builds up over time, the corrosion rate for all steels tends to decrease.

Most atmospheric corrosion reaction rates follow the exponential equation:

$$C = At^B \qquad (1)$$

where:

C = corrosion loss
A & B = constants
t = time

Plotting this equation on a log-log graph usually yields a straight line.

Many investigators have shown that the corrosion film that covers the steel surface is composed of two layers. The outer layer contains many dust particles which are absent in the inner layer. The two-layer structure was explained on the basis of a reduction-oxidation (redox) front where FeOOH is produced from Fe_3O_4 in the outer layer is then electrochemically reduced to magnetite (Fe_3O_4) in the inner layer.

Atmospheric pollutants increase the rate of corrosion. The most important pollutants, as far as the corrosion of steel is concerned, are sulfur oxides and mineral salts (particularly chlorides).

Sulfur oxides can react directly with water to produce corrosive sulfuric acid, H_2SO_4. SO_2 can also react with water, producing another acid. A major problem with the sulfur oxides is the "acid regeneration cycle" that is produced. This cycle occurs when the acid reacts with the steel to form ferrosulfate. This compound is further oxidized to form ferric sulfate and FeOOH. The ferric sulfate can hydrolyze and reform sulfuric acid and more FeOOH, allowing the cycle to begin again.

Generally, mineral salts increase the corrosion rates because:

- o They tend to absorb moisture and thus increase the time of wetness.
- o They increase the conductivity of the electrolyte film on the steel surface, speeding up the electrochemical reactions.
- o Acid salts, such as ferric chloride, can form.

Gaseous pollutants such as nitrogen oxide, hydrogen sulfide, ozone, and carbon dioxide seem to make little difference with respect to steel corrosion. Dusts, on the other hand, do have a significant effect on steel. For example, some dusts can absorb SO_2 and water from the atmosphere at low relative humidities, causing corrosion in otherwise non-corrosive surroundings.

The alloying elements of weathering steel have no effect on the initial rate of attack on a clean steel surface. Only after a corrosive layer is formed do the improvements of weathering steel start to show. There are theories on the effects of alloying elements dating back to Buck in 1919. While none of the theories have produced definite answers as to why the alloys protect, most theories agree that rust is a two-layer structure, and alloying improves the protective quality of the inner layer.

Problems exist with today's corrosion rating system, including:

- o Methods of calculating corrosion rates are not standardized. There are several methods used to make these calculations including time averaging, linear slope, and linear regression. These different methods yield highly different numbers that cannot be compared.

- o The common belief that copper steel is twice as resistant to corrosion as carbon steel is untrue. While the <u>average</u> corrosive resistance of these types of steel is twice that of carbon steel, the actual ratios vary from 1:1 to 4:1.

- o There is a lack of significance to corrosion rate numbers.

Because of these problems, Mr. Komp has been working on a new method for estimating the corrosion performance of weathering steel. The new method involves a log-log linear extrapolation of exposure time (in years) versus penetration depth. Additionally, a "corrosion index" based on the chemistry or composition of steel has been developed.

PERFORMANCE OF WEATHERING STEEL IN THE HIGHWAY ENVIRONMENT--MICHIGAN

Presentation by C.J. Arnold

[Charles Arnold has been with the Michigan Department of Transportation since 1954. He has both a bachelor's and a master's degree in engineering from Michigan State University. He is presently the Engineer of Research with Michigan DOT.]

<u>Highlights</u>:

Key points covered by Mr. Arnold included:

- o Michigan has a major investment in weathering steel to protect.
- o Significant corrosion rates have been observed in Michigan which substantially reduce the life of its unpainted weathering steel structures.

<u>Summary</u>:

Mr. Arnold made the point that frequently more can be learned "from our warts than from our beauty spots," an analogy to the difficulties Michigan has encountered in the use of weathering steel. Michigan built a total of 513 weathering steel bridges during the time between 1964-65 and 1980; Michigan counties built approximately another 100. Some of these bridges were painted only near the joints; more recently, the A588 bridges were completely painted. Over time, Michigan discovered a number of problems, as follows, affecting its weathering steel structures:

- o Salt contamination
- o Accumulation of debris
- o Capillarity of corrosion products
- o Crevice corrosion
- o Pitting
- o Mill scale

While the correct priority of these problems remains uncertain, salt contamination is believed to be the most significant threat to A588. Salt solutions contaminate the steel primarily through (1) leaky joints, and (2) spray from underneath. This salt contamination destroys the protective oxide that should form to slow the rate of corrosion. Mr. Arnold showed several slides of bridges corroded by both types of salt contamination.

Sand-blasting must be used to obtain an accurate account of the level of corrosion. Before blasting, the metal often showed no significant deterioration. Removing the layer of rust to expose the bare metal, however, showed pitting of as much as 0.1 in (2.5 mm).

Corrosion problems were also found in the linkages. Corrosion between the link plate and bridge eventually reaches a point where the connection ceases. This raises concern over the loss of movement that should occur at this point and the pressures on other parts of the bridge as a result of this immobility. Corrosion rates of up to 16 mils per surface per year were found at these links.

Another concern is the ability to inspect bridges. On certain types of structures, cracks in the metal are often difficult to locate. Mr. Arnold showed slides of a structure where cracks were not visible until after the area around the cracks had been painted. Very close inspection of other areas of the same bridge indicated potential cracks. Not until the areas were cleaned did the cracks become obvious. Mr. Arnold recommended that attention be drawn to the detection of cracks in unpainted structures that develop a rust coating of significant thickness.

In Michigan, it was found that the areas that benefit most by using weathering steel (i.e., heavily traveled metropolitan areas) are also the areas with the most problems associated with salt contamination, due to heavy use of deicing salts.

Michigan has gone over to total shop painting; this costs $28 - $40 per ton, which is within the range of what Michigan pays for A588 steel.

MICHIGAN'S EXPERIENCE WITH WEATHERING STEEL BRIDGES

Presentation by S.K. Coburn

[Seymour K. Coburn was educated at the University of Chicago and the Illinois Institute of Technology, and has been employed as a corrosion engineer for both the Association of American Railroads and U.S. Steel. He is a member of American Society for Testing and Materials (ASTM), American Society for Metals (ASM), American Society for Civil Engineers (ASCE), and the National Association of Corrosion Engineers, (NACE) and has published in the journals of each of those organizations. Mr. Coburn now operates his own consulting firm, Corrosion Consultants, Inc.]

Highlights:

Key points covered by Mr. Coburn included:

- Michigan's problems with its weathering steel bridges are related in part to environmental factors; some of these factors could be addressed through better road maintenance activities (e.g., hosing off winter salt deposits, removing salt-containing slush from roadways, etc.).

- An industrial environment does not affect corrosion rates.

- Small test panels do not--and should not be made to--represent actual structures.

Summary:

The problems facing the Michigan Department of Transportation are cold winters, frequent heavy and long-lasting snows, and high traffic density of trucks traveling throughout the State. Since Detroit has its own saltmines, there are always clear roads in the winter, but this has been found to be very harsh on the steel structures with which the chloride solutions come into contact.

In 1968, Michigan was facing major bridge maintenance problems caused by a dramatic increase in the number of bridges, larger traffic volume, and higher traffic speeds. Weathering steel, which had been recently--and successfully--used in the construction of the John Deere Building, seemed a viable solution in terms of eliminating the need for painting and reducing bridge maintenance needs and costs.

Michigan was advised that the corrosion-resistant properties of weathering steel did not necessarily cover the effects of snow removal chemicals. To cope with this problem, deflector plates were devised to prevent the drips from open bridge joints from falling on the steel beneath.

In 1968, Mr. Coburn recommended to Michigan DOT that the salt-contaminated weathering steel structures be hosed down after the winter season to prevent salt deposits from being sealed by future deposits of salt, oil, exhaust, and soot. This maintenance procedure has never been performed.

A 1980 report of the Michigan DOT laboratory stated that the successful use of weathering steel required the following:

- Alternate wetting and drying without prolonged wetness;
- Absence of salt-bearing water drainage;
- Absence of detailed geometry retaining moisture and debris;
- Washing of exposed surfaces by rain.

The report also stated that in the field "nearly all of the highway bridges contain departures from these requirements, and are experiencing various degrees of corrosion." Problems of "major proportions" were found in bridges and structures with clearances of 14 to 15 ft (4.3 to 4.6 m) with vertical retaining walls, often referred to as "tunnel freeways."

Further studies showed that scaling and leakage problems were common in structures 7 to 8 years old. Immediate action was taken by painting the bridges 5 ft (2 m) on either side of expansion joints. Laboratory experiments were conducted to search for a suitable protective coating, and investigate

methods of removing salt residues using a combination of blast cleaning and high pressure water spray. Some of these tests are ongoing.
Mr. Coburn showed several slides to illustrate key problems, particularly that of inadequate drainage. Poorly designed drainage systems result in a large backup of snow and slush on the deck surface; this brine solution is then continuously sprayed on the metal surfaces by passing cars. Another significant problem was the selection of shallow, 15-ft (4.6 m) clearances--these resulted in heavy salt build-up on the bottom of flanges and webs.

One of the concerns that led to Michigan's weathering steel moratorium was the industrial environment surrounding the Detroit area. Mr. Coburn, however, disagrees with the idea that industrial atmosphere significantly affects steel corrosion, and points out that the primary industrial pollutant, sulfur dioxide, is required by weathering steels to form the rust coating that protects the metal. Furthermore, when compared with several other industrial cities, Detroit was found to be among the cleanest.

Another argument Mr. Coburn raises is with the 4 x 6 in (100 x 150 mm) test panels used to study the steels. These small plates cannot be extrapolated to represent an entire structure. The reason for this is simple: because of their size and isolation, dew condenses on the test panels when the sun goes down, leaving them wet for up to twelve hours. Large structures, however, are integrated with concrete and retain more heat, resulting in little or no condensation forming on the metal.

Mr. Coburn also discussed the effect of salt concentration on oxygen availability. Oxygen is necessary in the corrosion process; when the salt content of water is increased, the oxygen level decreases, thereby increasing the length of time needed for corrosion. Thus, much of the corrosion now seen in Michigan is the result, not of ongoing actions, but of interactions that occurred during the structures' first, second, and third years. After that point, the salt caking tends to act somewhat like a protective coating.

PERFORMANCE OF WEATHERING STEEL IN THE HIGHWAY ENVIRONMENT -- MARYLAND

Presentation by J. Weisner

[John Weisner is a graduate of Johns Hopkins University, and has been with the Maryland State Highway Administration for 29 years. He is currently assigned to the Bureau of Research.]

Highlights:

Key points covered by Mr. Weisner included:

- o Cracking has been discovered in the joints of some weathering steel high-mast light poles; these probably result from capillary action interfering with the wetting/drying cycle required for patina formation.

- o The anchoring method used in securing the poles needs to be examined, as salt build-up and oxidation have been discovered at the base of the poles.

Summary:

Mr. Weisner began his presentation by summarizing the use of A588 steel in Maryland. The State has 190 weathering steel bridge structures, some unpainted, others painted 10 ft (3.0 m) on either side of the expansion joint. To date, none of these has required remedial action. When maintenance is required, the structures will simply be blasted and the corroded area painted. One problem with weathering steel bridges is that it is much more difficult to detect cracks on them than on painted structures.

Mr. Weisner then introduced his main topic: the problems associated with the use of weathering steel high-mast light poles. Maryland started installing these structures on the Interstate highway system in 1972. Over 600 telescoping-type poles have been erected, ranging from 70 to 125 ft (21 to 38.1 m), with 30-ft- (9.1 m) long sections tapering at a rate of 0.16 in (4.1 mm) per ft. All poles were fabricated to the AASHTO requirements, using either ASTM A595 (a weathering steel tapered pole), or A588 steel. The overlap areas at the joints were required to be at least twice the pole diameter; this requirement was exceeded on most of the poles inspected: some overlapped by as much as three times the pole diameter.

In August 1987, cracks were detected in the joints of 63 of the weathering steel poles. Two sample poles containing a total of 10 joints were brought into the lab, cut open, and studied. Cores were removed for chemical analysis, and the welds were radiographed. Upon examination, considerably more oxidation was found at the crack locations.

The oxidation between the two pole sections was caused by rainwater running down the outside of the poles, rolling around the corner at the joint, and being drawn up between the two sections by capillary action. This water then sat between the sections, causing oxidation. The oxidation acted as a sponge to hold even more water. This constant wetness prevented the protective patina from forming.

A second source of oxidation was located at a void between the male and female ends of the pole; furthermore, a crack always appeared when a void existed at the longitudinal weld. This condition was probably caused by a packout effect which overstressed the longitudinal weld. Since this weld turns out to be the weaker plane, cracking occurs. Today, AASHTO requires that the longitudinal weld be a full penetration weld; these were not, as shown by radiographs.

A third problem was that encountered in the method used to anchor the poles. This method basically consisted of setting the poles on leveling nuts and then grouting around them. This appears to be a poor method, as there was a tremendous amount of oxidation found at the base of the pole. On the poles that were below grade, where the salt gets knocked down the side, there was little adherence between the grout and the base plate. As a result, salt builds up on the base plate and corrodes.

NEW JERSEY TURNPIKE'S 20 YEARS' EXPERIENCE WITH A588 WEATHERING STEEL

Presentation by B. Noel

[Bruce Noel is a Supervising Construction Engineer from the New Jersey Turnpike Authority, in charge of bridge management systems and all construction. He has been with the Authority since 1956, and is a member of AASHTO, the Transportation Research Board, International Bridge Tunnel and Turnpike Authority, and Association of Asphalt Paving Technologists.]

Highlights:

Key points covered by Mr. Noel included:

- o Bridge construction with weathering steel can work if the "seven commandments" are followed.
- o The New Jersey Turnpike has had a high rate of success with weathering steel, and has designed and implemented several innovative details and mechanisms to ensure its success.

Summary:

This year, an estimated 200 million cars will travel the 36-year-old New Jersey Turnpike. To keep up with this rapid traffic growth, aggressive expansion programs have taken place. The most recent of these is a $2-billion widening program which began in July 1987.

Since 1964, 197 weathering steel bridges have been built as part of the Turnpike; an additional 22 bridges are now under construction. The current widening program contains plans for another 175 bridges; this includes widening and replacing existing structures as well as new construction.
A 1963 meeting on weathering steel presented guidelines for its use. These included seven "commandments" to be followed. Experience has since shown that

bridges not designed in accordance with these commandments have developed problems which now require corrective action. The seven commandments are:

1. Avoid environments which interfere with a tight protective oxide coating.

2. Avoid retention of water and debris on the steel surfaces.

3. Protect steel joints.

4. Consider the use of continuous spans.

5. Prevent any surface runoff from the deck to the steel below such as steel flooring systems or bridge scuppers.

6. Flush bridges at areas which accumulate debris (including salt) on a regular basis.

7. Prevent weathering steel from contact with vegetation, masonry, or other materials so that the weathering process can proceed on a natural basis.

Major differences in working with A588 steel are using closed joints and keeping drainage systems clean. The lessons the Turnpike Authority learned about joints include the fact that transflex joints do not work, the tooth dam is invaluable in its ability to accommodate large openings, and compression seals can be successfully used on smaller joints. However, newer and more effective methods are needed for building good joints.

In general, the Turnpike Authority has had very good success with weathering steel structures. Where there have been problems, these can be traced to failure to follow the above guidelines. Additionally, the Authority has devised methods--particularly in the area of drainage control and joints--to deal with the most serious deterioration problems.

Slides/Visual Aids:

Mr. Noel showed several slides to illustrate the variety and number of weathering steel Turnpike structures; problems encountered with weathering steel by the Authority; and methods, techniques, and devices for counteracting these problems. Points made by Mr. Noel included the following:

- Certain turnpike bridges are exposed to extensive deicing salts. To protect against the salt's effects, tight galvanized joints and appropriate maintenance are used.

- Like Maryland, the Turnpike Authority has erected a number of weathering steel light poles. These have posed no problem to date.

- Roadway slopes used on the Turnpike are designed to avoid the tunneling effect; there is usually a 16- to 17-ft (4.9 to 5.2 m) clearance.

- An easily maintained, effective gutter for use with a toothed dam is currently being used in three locations; more are planned for future construction.

- A drip plate that prevents water from entering the bearing is another example of simple, but effective, design.

- The Authority has replaced all A558 pins and hinges with stainless steel; other engineers should do the same.

- Mr. Noel contended that cracking on A588 can be detected through differences in color and patina appearance.

- The Turnpike Authority had lot of trouble with aluminum structures because of their poor fatigue life; these are being replaced with weathering steel structures.

- The Turnpike Authority uses reinforced continuous belts under finger joints to carry debris rather than splash plates; Mr. Noel feels splash plates prevent gutter maintenance.

- One of the reasons for the use of weathering steel is its elimination of major bridge painting; given the traffic volume, it is very difficult to schedule a lane closing so bridges can be painted.

LOUISIANA'S WEATHERING STEEL EXPERIENCE AND PRACTICE

Presentation by A. Dunn

[Al Dunn graduated from Louisiana Tech with a bachelor's degree in civil engineering. He then worked for 2 years as a field engineer for J. Ray McDermott, a Louisiana construction company. He joined the Louisiana Department of Transportation and Development (LaDOTD), and has been with LaDOTD for more than 30 years. Mr. Dunn has extensive experience as a project engineer; 15 years as a structure and facilities inspection and maintenance engineer; and recently spent 3 years as a district engineer at Lake Charles. He presently works in the LaDOTD Design Section.]

Highlights:

Key points covered by Mr. Dunn included:

- o LaDOTD uses a detailed bridge inspection form to capture the specific data needed to assess A588 performance.

- o Louisiana has experienced some problems with its weathering steel bridges, and has conducted--and continues to conduct--various studies to determine the cause of these problems and possible solutions to them.

Summary:

Louisiana's experience with weathering steel began around 1970 with small railroad bridges. The good results achieved by these structures prompted the use of weathering steel in more and larger railway and highway bridges, including the State's largest bridge which was built in Japan and contains about 20,000 tons (18,144 metric tons) of A588 steel.

To ensure proper maintenance of its bridges, Louisiana has developed an in-depth bridge inspection/report form covering every bridge component. A special form used for A588 structures lists all parts of the girders, bearing areas, flanges, etc., so as to collect the specific data needed. One such report submitted for a weathering steel bridge gave the first indication of

A588 performance problems. On this bridge, a flaky rust was building up and falling off, thereby preventing the protective patina from developing. As this cycle continued, the rust flakes and scales became larger. Sand-blasting showed that there was pitting ranging from 16 to 40 mils deep.

A similar type of flaking or scaling was found on other bridges as well. This scaling is not only a problem to the surface on which it forms, but also to other areas of the bridge: when the scales eventually fall off the metal surfaces, they frequently collect on the flat surfaces where they retain moisture, creating pits up to 60 mils deep.

Mr. Dunn brought up the question of acid rain and its effect on weathering steel. Presently, little or no information is available on this topic, which is perhaps most relevant in areas of high humidity.

Several bridges in Louisiana are located in areas where moisture remains on the bridge for up to half a day. The condensation builds up to a point where it begins to run down the metal surfaces, covering a significant part of the bridge. Even on areas that appear to be dry, moisture has collected beneath the flaky surface, creating a muddy substance that is the source of corrosion.

In conjunction with Lousiana State University, LaDOTD studied seven bridges in Louisiana and Texas. Various solutions of phosphoric, benzoic, and tannic acids were applied to small sections of a steel surface in order to develop a method of determining section loss due to this scaling-type rust. Study results showed that the underside of bridges near the coastal area are losing--on average--5 to 7 mils per year on each surface. On the outside surfaces, the loss is approximately 1 mil per year.

LaDOTD contracted for one of its flaking/pitting bridges to be painted. Problems arose, however, with the contractor performing the work, and the painting was never completed. Other problems with painting and painters include nonstandard paint practices, paint inspectors who are not sufficiently thorough, failure to clean the bridge (especially at corners and in areas with debris build-up), improper application (overspray and/or underspray), and sloppy work.

Minor problems have also been experienced with deck construction. Decks were sand-blasted and painted as required, but rust spots began to appear at the bottom of the pits on these surfaces. Initially, this was thought to be rust coming through the paint. Further investigation showed that this was not the case. Sand-blasting below the deck was generating rust and corrosion dust which was blown around and settling on the deck surface. These dust particles would then settle in the paint and begin to rust again.

PERFORMANCE OF WEATHERING STEEL BRIDGES IN FHWA REGION 10

Presentation by G. Kasza

[Gary Kasza is the Director of the Office of Structures with FHWA Region 10. He has a bachelor's degree in civil engineering from Colorado State University, and is a registered Professional Engineer in all the West Coast States. He has been with FHWA since the early sixties, and has worked as a project engineer on Federal construction projects and as a design engineer in FHWA's western bridge design offices. Mr. Kasza has also served as a bridge engineer at the division level.)

Highlights:

Key points covered by Mr. Kasza included:

- o Region 10 has areas of rainfall varying from less than 20 in (500 mm) to more than 200 in (5.08 m) annually. The performance of weathering steel has a direct relationship to these climate variations as exhibited by its corrosion reactions.

Summary:

The Federal Highway Administration Region 10 is comprised of Alaska, Idaho, Oregon, and Washington. The region varies from damp coastal areas to arid inland regions. Rainfall and precipitation throughout the region ranges from less than 20 in (500 mm) to over 200 in (5.08 m) per year.

In 1981, four 9-year-old weathering steel bridges located in southeast Alaska (average annual precipitation between 150 and 200 in [3.81 m and 5.08 m]) per year were discovered to have significant corrosion with rust scale and laminar rust. In contrast, 10-year-old bridges located in Idaho that are subjected to about 30 in (762 mm) per year of rain had a very tight patina.

A 1986 evaluation of the region's weathering steel bridges examined 11 bridges in the western part (i.e., west of the Cascades) of Washington State. Bridge age varied from 2- to 20-years-old; the study included the area's oldest weathering steel bridge, which was built in 1966. In conducting the study, the inspectors categorized metal surfaces as either (1) gritty (sandy material which is easily removed by running one's fingers across the surface); (2) flaky (contains up to 1/8 inch in easily removed flakes); or (3) laminar (not easy to remove). In addition, some bridges contained combinations of these (e.g., flaking laminar).

Mr. Kasza discussed each bridge assessment in detail, showing slides to illustrate the findings at each site.

The conclusions of the study were that in the wetter areas of Region 10 (southeast Alaska, and west of the Cascades in Washington and Oregon), a sandy, gritty surface can be expected within 5 years, flaking within 10 years, and--probably--laminar rust within 15 years. It is thus recommended not to use weathering steel in areas with high rainfall. On the other hand, weathering steel can be expected to perform--and has performed--very well in the other, dryer parts of the region.

ECONOMICS OF WEATHERING STEEL IN HIGHWAY STRUCTURES

Presentation by F.D. Aulthouse

[F. Dale Aulthouse is Vice President for Purchasing with High Steel Structures, Inc., and has been with the firm for the past 30 years. He attended Franklin and Marshall College, and has been certified as a Purchasing Manager by the National Association of Purchasing Managers.)

Highlights:

Key points covered by Mr. Aulthouse included:

- Weathering steel can represent considerable cost savings over a structure's life cycle. These savings primarily derive from the decreased maintenance costs afforded by A588 steel.

Summary:

Mr. Aulthouse presented a paper which compared life cycle costs of an A588 weathering steel bridge to that of the same structure had it been fabricated with nonweathering steel and painted. The bridge used for the cost comparison--the Lewisburg Bridge in Northumberland County, Pennsylvania--is comprised of 886 tons (804 metric tons) of A588 weathering steel, spans 1,223 ft (372.8 m), and represents 128,678 ft^2 (11,954.6 m^2) of surface area to be painted.

Assumptions

Steels. The weathering steel bridge consisted entirely of A588 steel. For comparison, lower cost nonweathering materials were substituted, i.e., A572 grade 50 and A36.

Painted Surfaces. Because Pennsylvania requires that weathering steel structures be painted 10 ft (3.0 m) from the joints, this analysis includes the cost of painting and maintaining the bridge's two finger expansion dams (4,013 ft^2 [372.8 m^2]). Also, since some States require that the fascia surface (23,960 ft^2 [2,225.9 m^2]) be painted, this cost is also included here.

Paints. The coating system selected, which seems to be the least controversial system, consists of shop-applied inorganic zinc primer at 3 mils, and field-applied intermediate and top coats of epoxy (5 mils) and urethane (2 mils), respectively.

Life Cycle. The analysis was extended to predict life cycle cost over a 50-year period. It was assumed that the inorganic zinc primer and epoxy intermediate coat would last 50 years; the urethane top coat would last approximately 15 years; and areas around the joints would require blasting and a complete re-painting every 20 years.

Costs

Steel Costs. Depending on plate thickness, A572 grade 50 steel costs either \$55/ton or \$29/ton less than does A588. A36 steel ranged from \$105/ton to \$99/ton to \$73/ton less. Converting all the shapes from A588 to A36 steel costs \$134/ton less. In all, the total savings of using A572 grade 50 and A36 steels over A588 is \$57,850.

Painting Costs. The painting costs were developed from a study by A.H. Roebuck and G.H. Brevoort. Using a 5-percent inflation factor to bring the 1983 figures up to date, the shop coating of the primer is \$0.77 per square foot; the intermediate and top coats cost is \$1.08 per square foot. The total initial painting cost comes to \$240,000.

Top Coat Maintenance Costs. Repair of the urethane top coat will include a high pressure water blast to prepare it for a new coat. Maintenance cost of the top coat is estimated to be \$0.52 per square foot. Over 50 years, the total cost to maintain the top coat comes to \$1,030,000.

Joint Maintenance Costs. Joint maintenance every 20 years at \$2.35 per square foot will total \$91,500 over the 50-year cycle. This cost, of course, would apply to the weathering steel structure as designed because of the joint painting required by Pennsylvania.

Life Cycle Cost Comparisons

Life Cycle Costs

Painted Bridge:

Initial painting	$ 240,000
Top coat maintenance	$1,030,000
Joint maintenance	$ 91,500
TOTAL COST	$1,361,500

A588 Steel Bridge:

Additional steel cost	$ 58,000
Initial cost	$ 7,500
Joint maintenance	$ 91,500
Fascia painting/maintenance	$ 236,000
TOTAL COST	$ 393,000

Comparison #1: Best case scenario--joints do not leak; fascia does not require painting. Total cost savings of A588 steel = $1,200,000.

Comparison #2: Joints leak, painting required 10 feet in, fascia does not require painting. Total cost savings of A588 steel = $1,200,000.

Comparison #3: Worst case scenario--fascia and joints need painting. Total cost savings of A588 steel = $968,500.

Comparison #4: Annual washing down of bridge (estimated at $0.10 per square foot, including the cost of traffic control, per year, for a 50-year life cycle cost of $113,708), joints leak, fascia does not require painting. Total cost savings of A588 steel = $1,090,000.

From these figures, it is obvious that weathering steel should be seriously considered for locations that support it. The chief benefit is cost effectiveness. Other benefits include the elimination of the hydrocarbons (as per the Clean Air Act) in paint which react with nitrogen oxides to form ozone, the long-term durability of A588, and the economy of maintenance.

FATIGUE RESISTANCE OF WEATHERING STEEL

Presentation by P. Albrecht was based on the following paper

[Dr. Pedro Albrecht is professor of civil engineering at the University of Maryland.]

Introduction

This paper reviews the previous test data relevant to the fatigue strength of weathering steel details, presents new data from ongoing research projects, and recommends allowable stress ranges for the fatigue design of bare, exposed weathering steel structures. The review of the previous data and the recommendations are based on two reports prepared under the National Cooperative Highway Research Program (NCHRP), Project 10-22/1 (Albrecht and Sidani, 1987, Albrecht, et al., 1988b). Some of the present data come from NCHRP Project 10-22/1 (Albrecht and Sidani, 1987), and the remainder from two studies sponsored by the Maryland State Highway Administration in cooperation with the Federal Highway Administration (Albrecht and Xu, 1988a; Albrecht, 1988c).

The opinions and conclusions expressed in this paper are those of the author. They are not necessarily those of the agencies that have sponsored the research.

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 uniformly settles 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 falls 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 spray from breaking ocean waves 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 salt-contaminated members and can create corrosive conditions similar to those found in severe marine environments.

During stress cycling, the fatigue crack eventually creates its own environment irrespective of whether the steel member is wetted by or immersed in an aqueous solution. For the above 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, Fisher, and Irwin, 1986; Albrecht and Sidani, 1987).

Depending on the type of details, degree of maintenance, and severity of the 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. The corresponding exposure conditions can be described as follows:

- <u>Medium Corrosivity</u>: The steel structure is boldly exposed to rain washing and sun drying, 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.

- 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.

- 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.

Typical values of corrosion penetration per surface for the above corrosivity categories are as follows: medium - 1 to 5 μm/y (0.04 to 0.2 mil/y), high - 5 to 10 μm/y (0.2 to 0.4 mil/y), and very high - greater than 10 μm/y) (0.4 mil/y) (ISO, 1988).

Testing

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.

- Weathering and corrosion fatigue S-N (W&CFSN) life.

In the "weathering fatigue" tests, 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 an aqueous environment 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 an aqueous environment of fresh or salt water.

Weathering steel bridges experience more complex combinations of loading and environmental exposure than the above mentioned tests can simulate because, in service, corrosion and stress cycling occur concurrently. 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. The data are summarized in the following by type of test.

<u>Weathering Fatigue S-N (WFSN) Life</u>

Weathering fatigue 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 steels which were 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 weld as welded and ground flush, bead weld, notched plate, welded transverse stiffener, and welded short attachment. The specimens were boldly weathered up to 11 years prior to stress cycling.

Fig. 1 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 the mean line for their weathered counterparts. See fig. 2. 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.

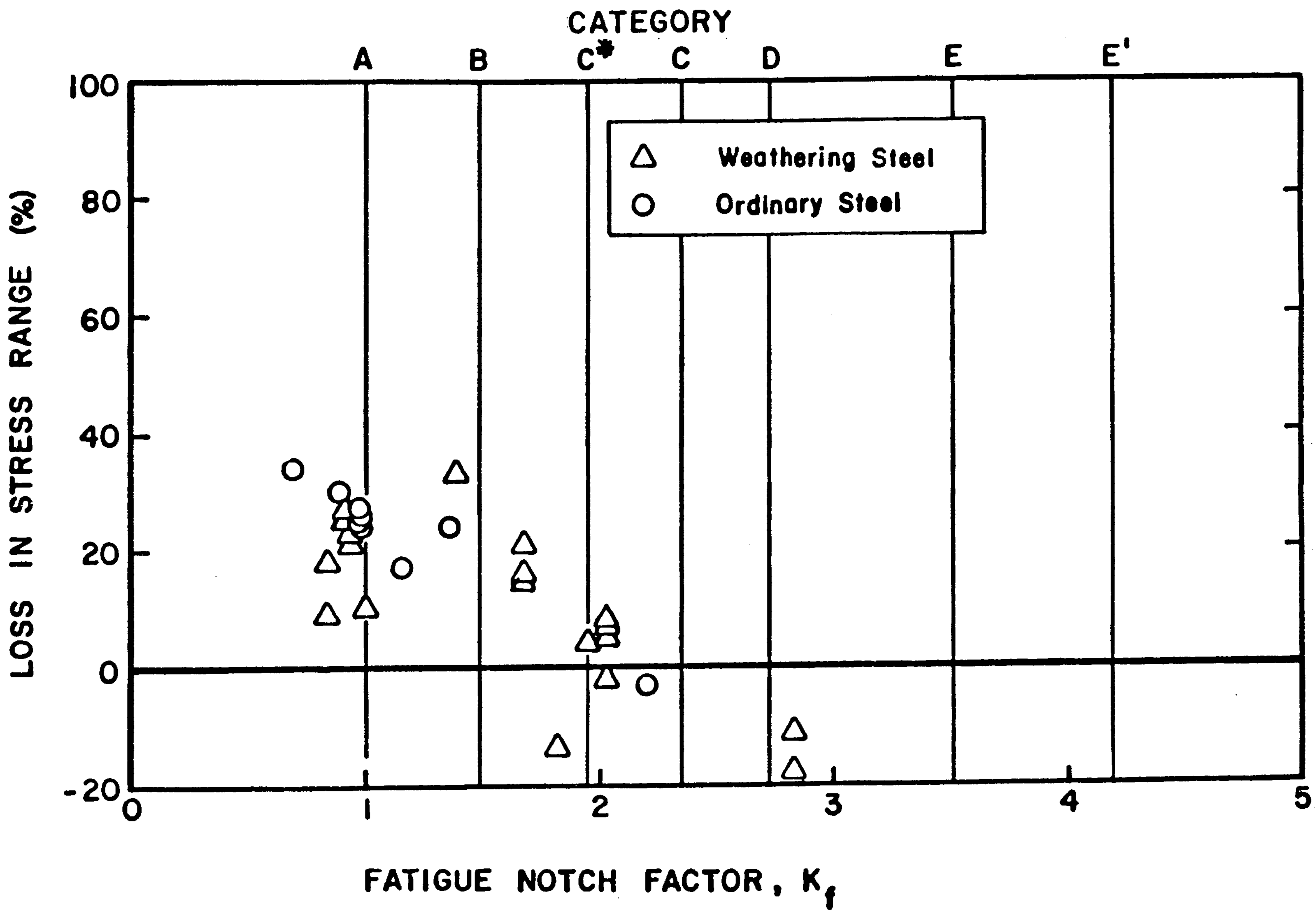

Figure 1. Loss in stress range of weathered specimens tested in air.

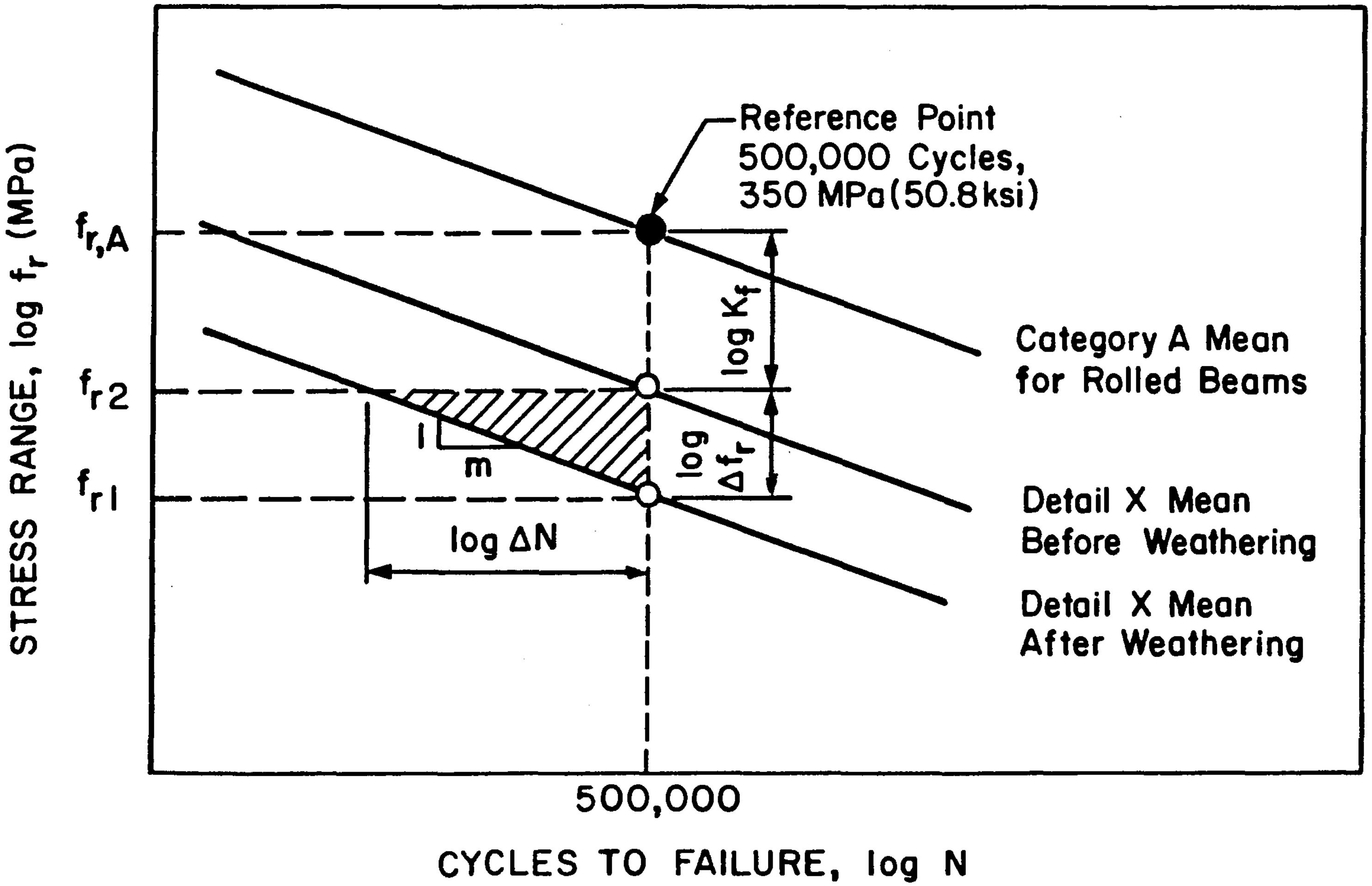

Figure 2. Illustration of loss in stress range and fatigue notch factor.

See fig. 2. The vertical grid lines in fig. 1 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 E′ cover plate (Fisher et al. 1970; Fisher, Albrecht, et al. 1974; Fisher et al. 1979). Those data were previously used to establish the AASHTO allowable S-N lines for category A to E′ details (AASHTO, 1983).

A comparison of all WFSN data shown in fig. 1 leads to the following conclusions:

- 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.

- Atmospheric exposure reduced alike the fatigue strength of weathering steel and ordinary steel specimens.

The WSFN data only modeled 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.

Corrosion Fatigue Crack Initiation (CFCI) Life

Novak (1983), formerly of USX Corporation, determined the 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. The notched specimens had a theoretical stress concentration factor, K_t = 3.42, similar to that of category C to D details. 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 findings were as follows:

- o The fatigue crack initiation (FCI) threshold in air was f_r = 147, 182, and 250 MPa (21, 26, and 38 ksi) for A36, A588, and A517 steels, respectively, where f_r is the nominal stress range at the notch.

- o No CFCI thresholds were found for any of the steels tested in a sodium chloride solution despite strong attempts to characterize the long-life behavior of primary interest for structural applications such as bridges.

- o The long-life (3,000,000 cycles) fatigue strength for CFCI behavior was f_r = 68 MPa (10 ksi) for all steels.

- o 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 showed 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 dry 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 were intended for painted steel bridges.

<u>Corrosion Fatigue Crack Propagation (CFCP) Rate</u>
Measurements of 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) steel, 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 a NCHRP project funded at USX Corporation (Barsom and Novak, 1977) and a FHWA project funded at Lehigh University (Roberts, Irwin, and Fisher, 1986). The remainder of the data came from Refs. (Klingerman and Fisher, 1973; Mayfield and Maxey, 1982).

Based on an extensive statistical analysis of the effects of type of steel, loading, environment, and testing laboratory, Yazdani and Albrecht (1983) found that the difference in crack growth rate was 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 steel) 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 in air:

$$\frac{da}{dN} = 1.54 \times 10^{-12} (\Delta K)^{3.344} \qquad (1)$$

For mild and HSLA steels in aqueous environments:

$$\frac{da}{dN} = 4.16 \times 10^{-12} (\Delta K)^{3.279} \qquad (2)$$

For quenched and tempered steel in air:

$$\frac{da}{dN} = 2.27 \times 10^{-11} (\Delta K)^{2.534} \qquad (3)$$

For quenched and tempered steel in aqueous environments:

$$\frac{da}{dN} = 6.00 \times 10^{-11} (\Delta K)^{2.420} \qquad (4)$$

In the above equations, da/dN and ΔK have units of m/cycle and $MPa\sqrt{m}$ Eqs. 1 and 2 apply to A242, A588, and A709 Grade 50W steels, whereas eqs. 3 and 4 apply to A709 Grade 100W steel.

The air and aqueous lines for each group of steel, plotted in fig. 3, 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 steel. 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.

Corrosion Fatigue S-N (CFSN) Life

A better measure of how a corrosive environment affects both the crack initiation and propagation phases of the total fatigue life 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 CSFN 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 others had reported in the literature. The specimens consisted of the following types of details: smooth place, 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. The aqueous environment tests were performed 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 were plotted in fig. 4 against the fatigue notch factor of the specimens tested in air. A comparison of all corrosion fatigue data shows that:

- o Salt water environments induced greater losses in stress range than fresh water environments.

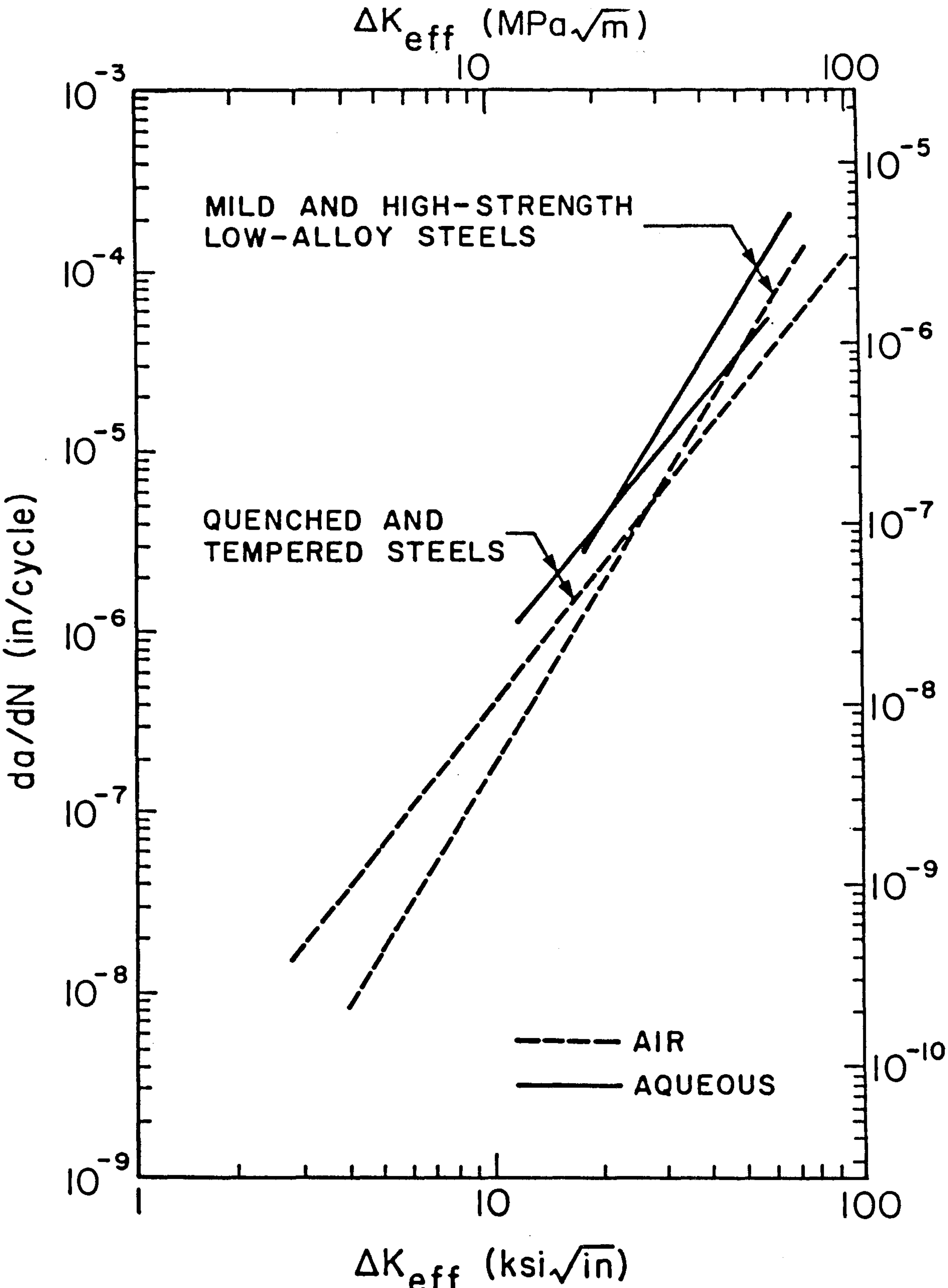

Figure 3. Crack propagation rates for structural steels tested in air and aqueous environments.

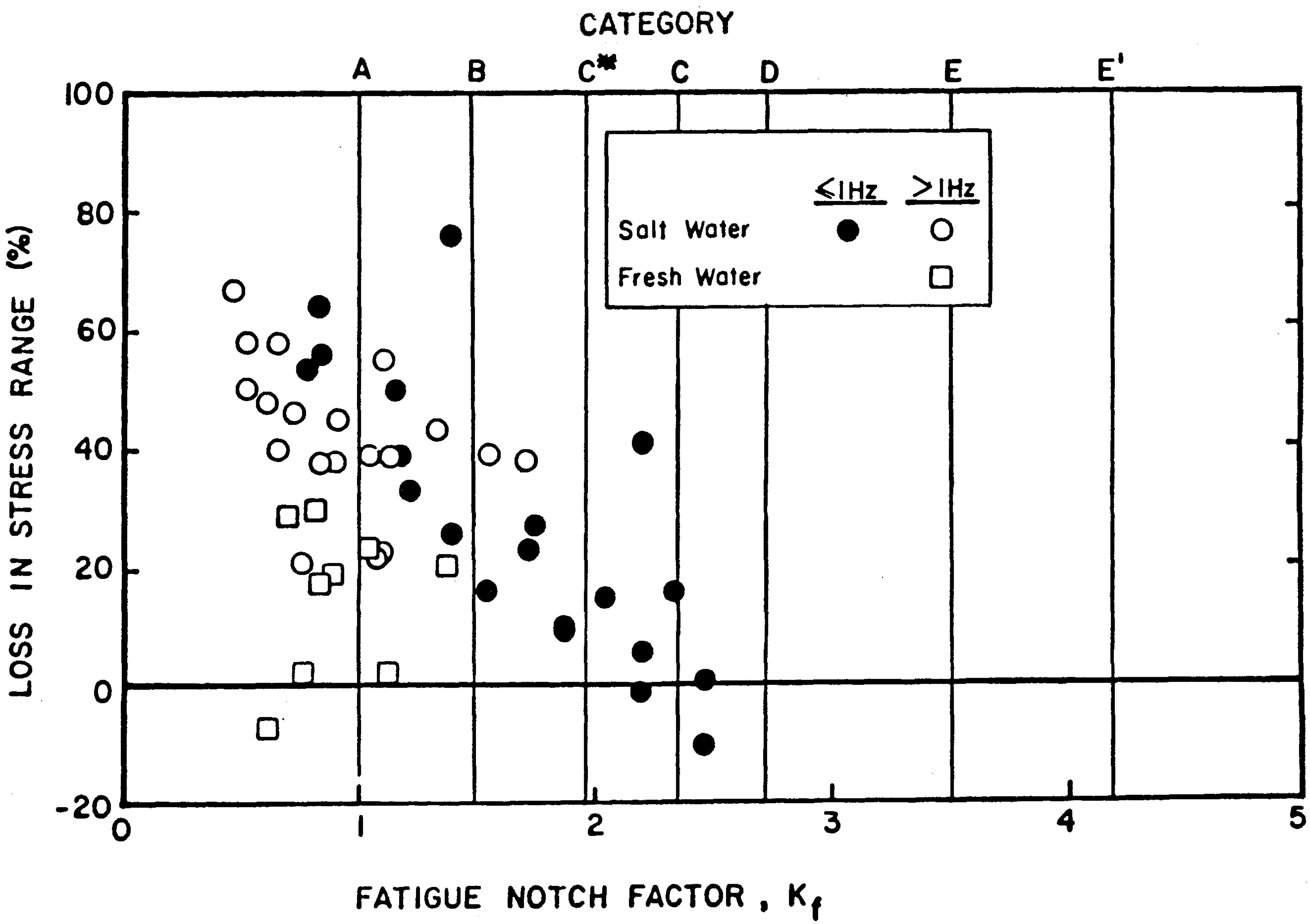

Figure 4. Loss in stress range of specimens tested in aqueous environments.

- o The loss in stress range in aqueous environments was greater at low cyclic frequencies of 0.1 to 1.0 Hz typical of highway bridge loading than at high cyclic frequencies.

- o 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 C, in the direction of increasing fatigue notch factor.

- o 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′.

<u>Weathering and Corrosion Fatigue S-N (W&CFSN) Life</u>

Weathering and corrosion fatigue S-N data are available from three series of tests of 51 specimens fabricated from A88 steel. In one series, tensile 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 14x30 rolled beams as well as welded beams of same cross section as the rolled beams were weathered 5.5 to 7 years (Albrecht 1988c). The beams were lightly sprayed with a 3 percent sodium chloride solution 3 times per week during 3 winter months of every year. They were then stress cycled at 0.75 Hz in a moist salt water environment. The exposure and test environments for the beams were of very high corrosivity. 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 were plotted in fig. 5 with solid rectangular symbols. Because

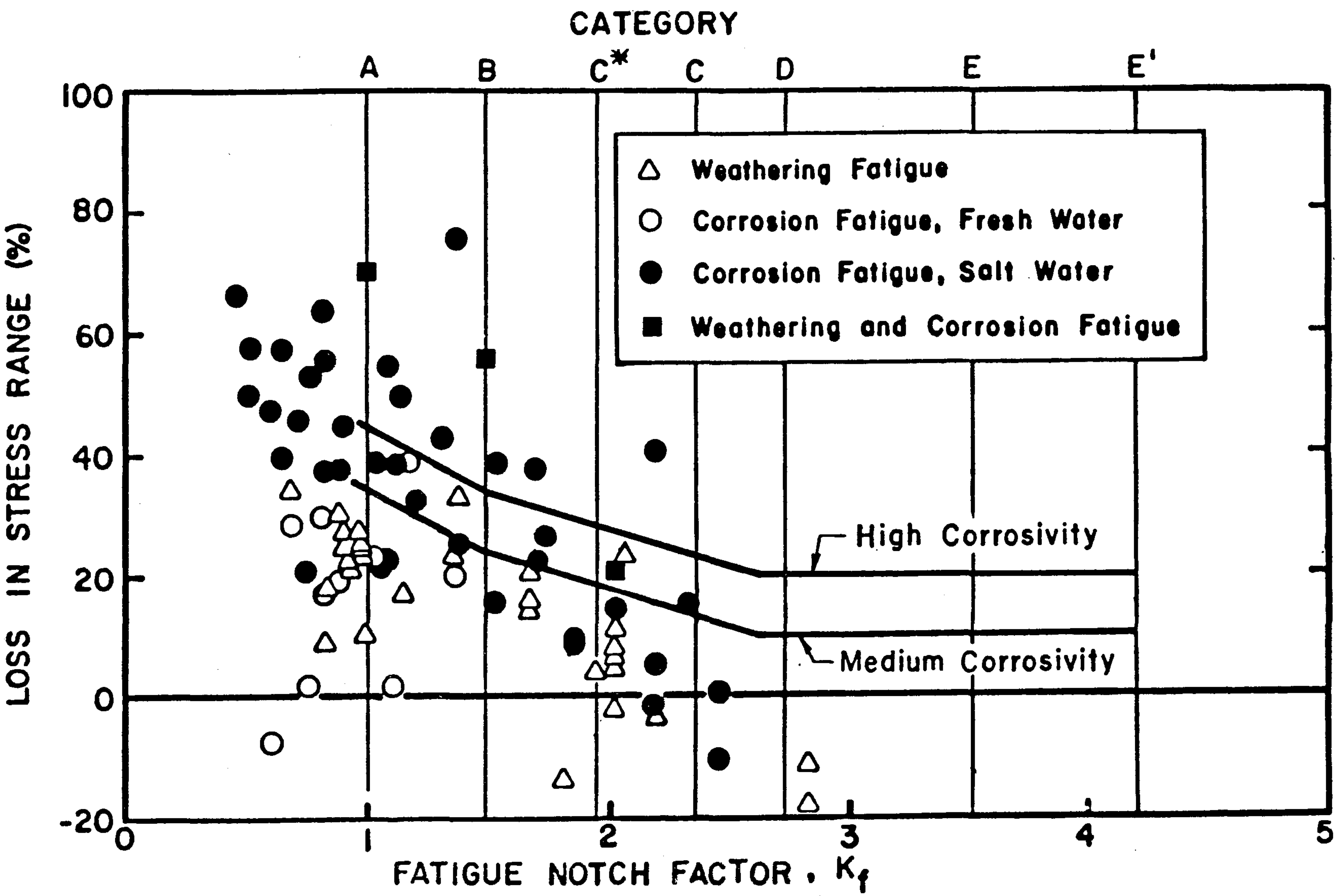

Figure 5. Comparison of loss in stress range with recommended reductions in allowable stress range for bare, exposed weathering steel structures.

the beams had severely corroded 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.

Recommended Allowable Stress Ranges

The vast amount of available fatigue test data of various types consistently show 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 therefore recommended for bare, exposed weathering steel bridges, depending on the type of detail and the type of environment to which a bridge is subjected.

Environments of medium corrosivity:

- o Category A, B, and C details: 34, 24, and 13 percent, respectively.
- o Category D, E, and E′ details: 10 percent.

Environments of high corrosivity:

- o Category A, B, and C details: 44, 34, and 23 percent, respectively.
- o Category D, E, and E′ details: 20 percent.

The recommended reductions are also given in table 1 and plotted as solid curves in fig. 5. The exposure conditions in medium and high corrosivity environments are described at the beginning of the paper. Applying these reductions to the allowable stress ranges listed in table 10.3.1A of the AASHTO specifications, which were intended for painted steel structures, gives the allowable stress range for the fatigue design of bare, exposed weathering

Table 1. 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

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 fig. 5, was taken from Refs. (Albrecht 1982, 1983; Albrecht and Naeemi, 1984). It was based on the following observations: (1) reduction of the crack initiation phase 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 fig. 5; and (2) reduction of the crack propagation phase 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 fig. 3.

The curve for high-corrosivity environments, also shown in fig. 5, 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 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 fig. 5.

The recommended reductions in allowable stress range are not applicable to very-high-corrosivity environments 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 1988c). These data are represented by the square symbols lying on the category A and B grid lines in fig. 5.

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FATIGUE RESISTANCE OF WEATHERING STEEL COMPONENTS

Presentation by J. Fisher was based on the following paper*

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Introduction

The AASHTO Specifications (1983) contain provisions for the fatigue design of steel bridge details. These provisions are based on a set of fatigue resistance curves which define the strength of different classes of details. The curves were developed from an extensive research program sponsored by the National Cooperative Highway Research Program (NCHRP) under the direction of the Transportation Research Board. The program, conducted over a period of 6 years from 1966 to 1972, involved the fatigue testing of 800 full-sized, welded steel bridge details (Fisher et al, 1970 and 1974). The statistically-designed experimental program was conducted under controlled conditions so that analysis of the test data would reveal the parameters that were significant in describing fatigue behavior. The result was the quantification of the fatigue strength of welded bridge details and the development of comprehensive design and specification provisions.

Since the adoption of the AASHTO fatigue specifications in 1974 (Fisher, 1977), several major fatigue studies have been carried out on similar beam-type specimens. Tests were conducted in East Germany, Japan, Switzerland, Office of Research and Experiments of the International Union of Railways - ORE (West Germany, Poland, England, and Holland), as well as here in the United States. The additional studies evaluated the applicability of the findings of the NCHRP test program to fabrication conditions elsewhere in the world and were used to develop similar fatigue codes. The additional tests

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augmented the NCHRP findings and often defined the fatigue strength of details that were not tested under the NCHRP program. For example, many of the Japanese data stem from research performed to develop fatigue specifications for the design of long-span bridges for the Honshu-Shikoku Bridge Authority. Many of the simulated details are typical of those found in welded box members.

A detailed review of all of the test data on welded details was reported in NCHRP Report 286 (Keating and Fisher, 1986). Only slight modifications were made to the original fatigue resistance curves as a result of this review. The curves were all set to a common constant slope of -3, as this was more compatible with large samples of test data for a given detail, as can be seen in fig. 6 which shows experimental data on longitudinal welded joints.

The adjustment in slope to -3 is also compatible with crack propagation concepts and cumulative damage theories. The fatigue resistance in the finite life region is compatible with the crack growth rate for bridge steels. Under constant cycle loading, a fatigue limit is approached which is identified in fig. 6 by the dashed horizontal line. This is also compatible with the crack growth threshold observed in crack growth studies. Under variable loading, the extension of the S-N curve below the constant cycle fatigue limit provided an accurate estimate of fatigue resistance when some of the variable stress cycles exceed the fatigue limit.

Fig. 7 shows the fatigue design curves adopted for the AASHTO specifications in 1986 (AASHTO, 1987). In general, these resistance curves are based on a lower bound to the experimental fatigue resistance of prototype details. Cracks that form at the weld toe, which is represented by AASHTO Cats. C, D, E, and E′, represent detail which usually have pre-existing defects at the weld toe where cracks initiate (AASHTO, 1983; Fisher et al, 1970). Hence, there is no significant crack initiation time period experienced if the cyclic stress is above the fatigue limit.

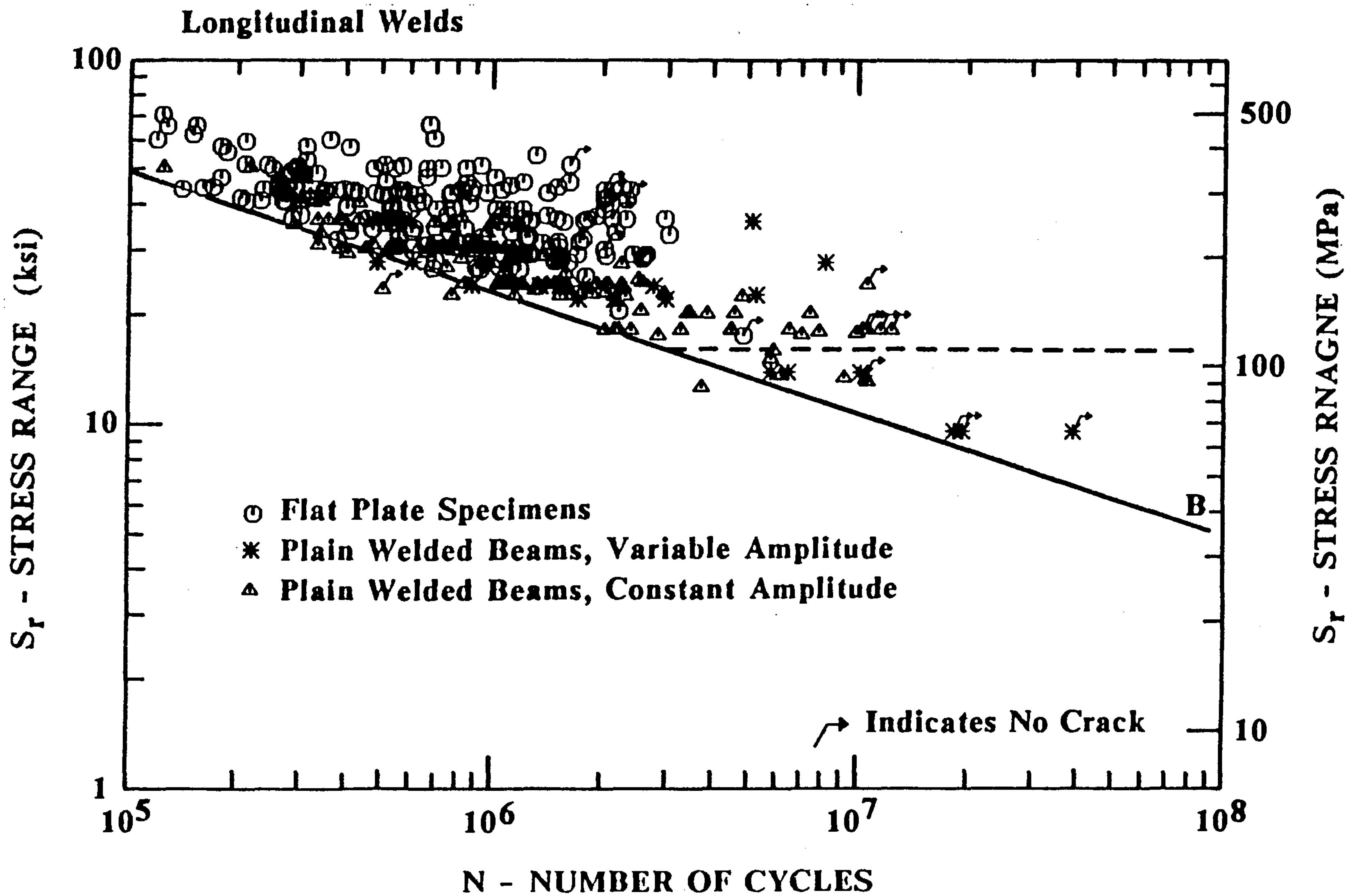

Figure 6. Experimental data on longitudinal fillet and groove welds.

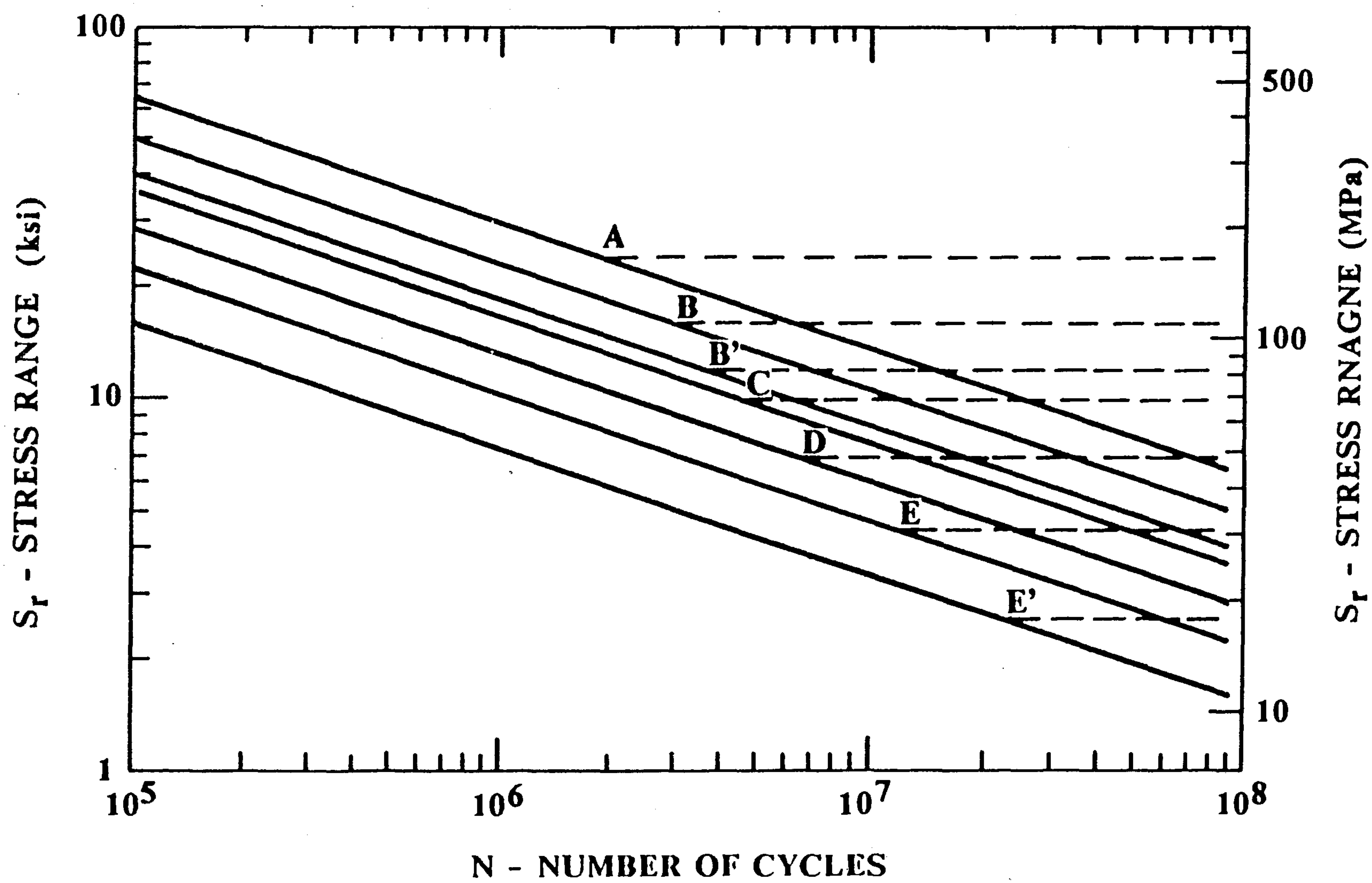

Figure 7. AASHTO fatigue design curves.

Experiments on Weathering Steel

General Air Exposure

Two separate studies investigated the fatigue characteristics of weathering steel--one by Albrecht (Albrecht and Naeemi, 1984) and the other by Yamada (Yamada and Kikuchi, 1984). Both test programs used plate specimens fabricated with automatic submerged arc welds which resulted in very good profiles. The specimens were tested under constant amplitude load conditions. Each program examined unweathered and weathered specimens. The weathered specimens were subjected to varying degrees of atmospheric exposure prior to testing. The tests did not simulate actual field conditions, because the weathering process was not continued during the actual fatigue testing.

The Albrecht study involved the fatigue testing of specimens that either simulated a transverse stiffener detail (category C) or an attachment plate (category D). The stiffener type specimens were 1.0 in (25 mm) and 0.4 in (10 mm) thick plates, smaller than similar cruciform specimens used in other studies (Yamada and Kikuchi, 1984) in the as-received condition and after 2 and 4 years exposure or 3 and 8 years exposure. The test results all exceeded the AASHTO category C resistance curve, as can be seen in fig. 8. The test results at and below the category C curve are from English data in the Maddox study (1982). The attachment specimens in this study consisted of a 4 in-(100 mm) long plate welded around the entire perimeter to a base plate, similar to the NCHRP Report 188 test specimens (Schilling et al, 1978). This would normally correspond to a category D type detail. A schematic of this specimen may be seen in fig. 9. The specimens were fatigue tested as-fabricated (unweathered), after 2 years of exposure and after they were weathered for 4 years.

The results from this study are plotted in fig. 9 and compared with the AASHTO category D resistance curve. All data plot significantly above this curve, just as observed with the small scale simulated tests in the NCHRP Report 188 (Schilling, et al, 1978).

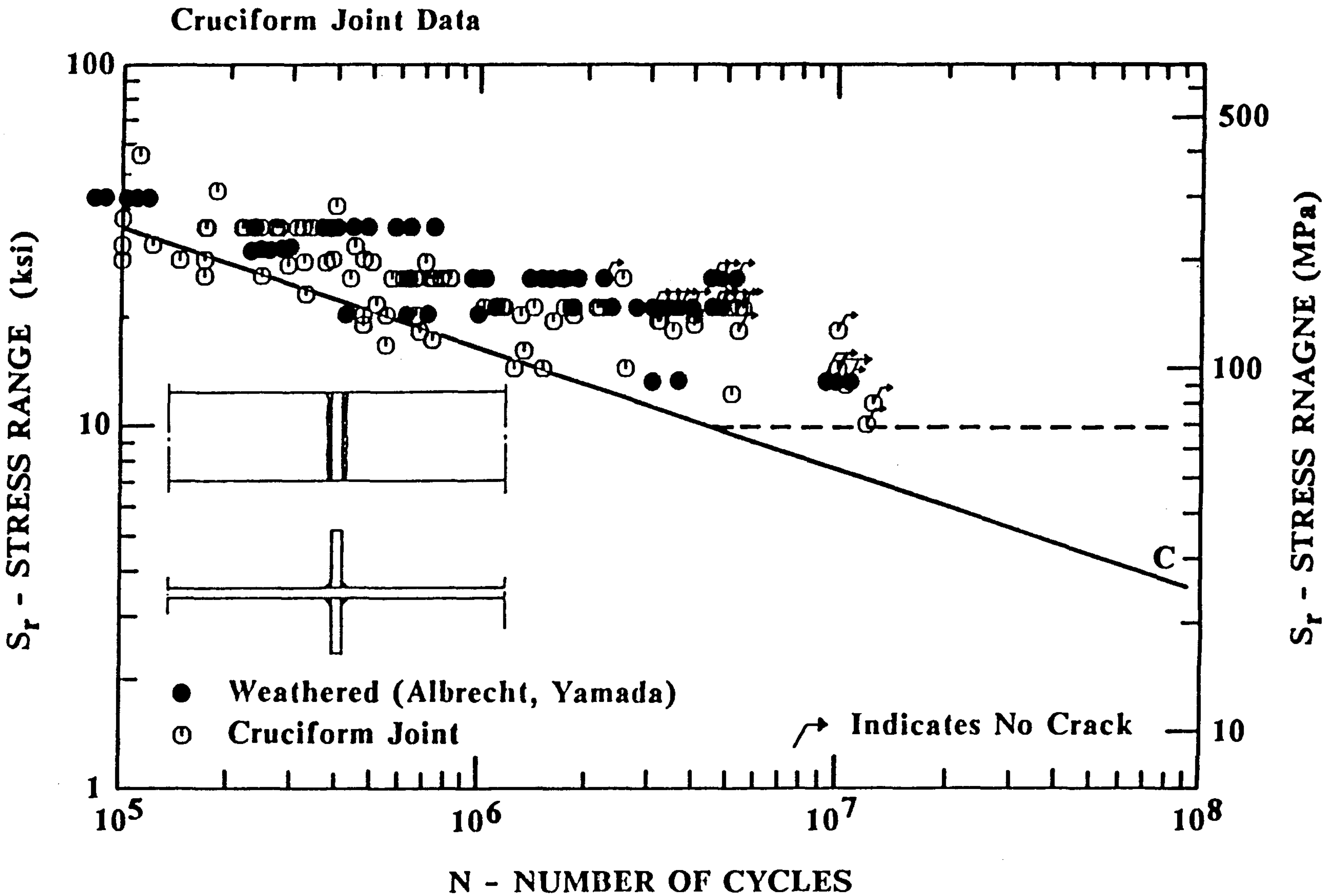

Figure 8. Comparison of weathered and unweathered cruciform joint test data.

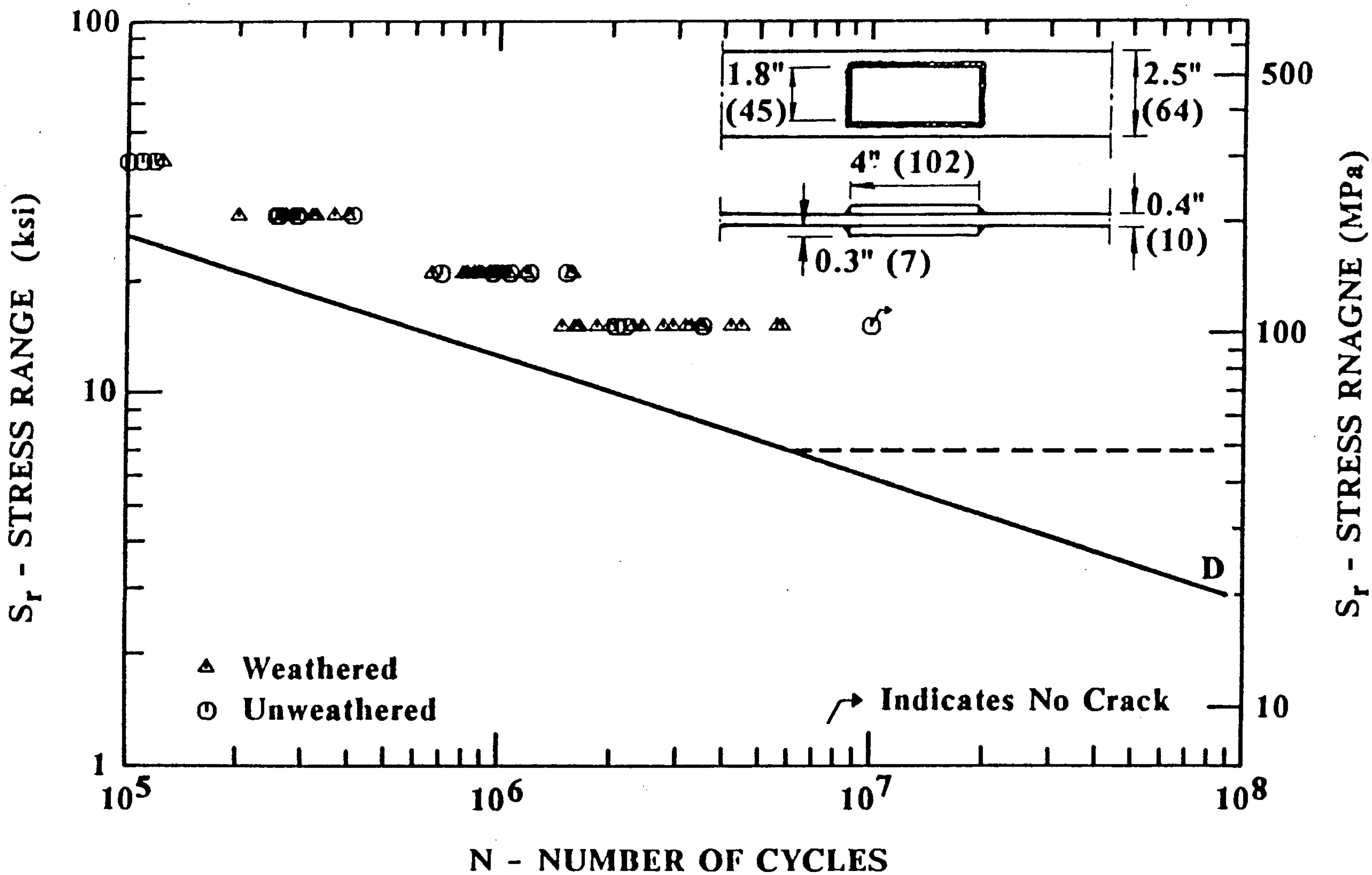

Figure 9. Fatigue resistance of weathered steel simulated attachment specimens.

The Yamada study used two different types of plate specimens: nonload carrying cruciform joints and a gusset plate type specimen. Both weathering steel and standard structural steel were used for the specimens. The cruciform stiffener specimens consisted of two transverse attachments welded to a 1/2 x 3 in (13 x 75 mm) base plate. The gusset specimens were fabricated with two longitudinal attachment plates, each 4 in (100 mm) long, welded on edge to the base plate. The specimens were fatigue tested as fabricated, after they were weathered for 2 years and after 4 years of exposure. In addition, stiffener type details were cut out of the web of an actually weathered steel bridge that had been in service for approximately 5.5 years.

Fig. 10 shows the results of the fatigue failures for the cruciform joints. All failures plot above the AASHTO category C curve.

The gusset plate specimen results are compared with the AASHTO category D resistance curve in fig. 11. Again, the test data fall significantly above the resistance curve.

A full-scale A588 steel W36x230 beam with 1 1/4 x 12 in (32 x 305 mm) cover plates had been subjected to 56 million cycles of stress at a 4 ksi (28 MPa) stress range with no detectable cracking observed at that time (Fisher et al, 1971). The specimen was stored outside of the laboratory for a 2-year period. To simulate exposure to deicing salts, road salt was added to a bucket of water and the suspended solution poured over the cover plate ends at monthly intervals. Undissolved particles were left in place, and the cover plate ends were exposed to the natural weathering process. Fig. 12 shows one of the weathered cover plate ends before the beam was brought into the laboratory and tested. The girder was subjected to a cyclic stress range of 6 ksi (41 MPa). Cracking in the weld throat was discovered after 7.1 million cycles at one end and 7.6 million cycles at the second end, as illustrated in fig. 13. Testing was stopped, and the crack was exposed to reveal its shape. No crack tip had propagated into the girder flange. The residual life for the crack to move

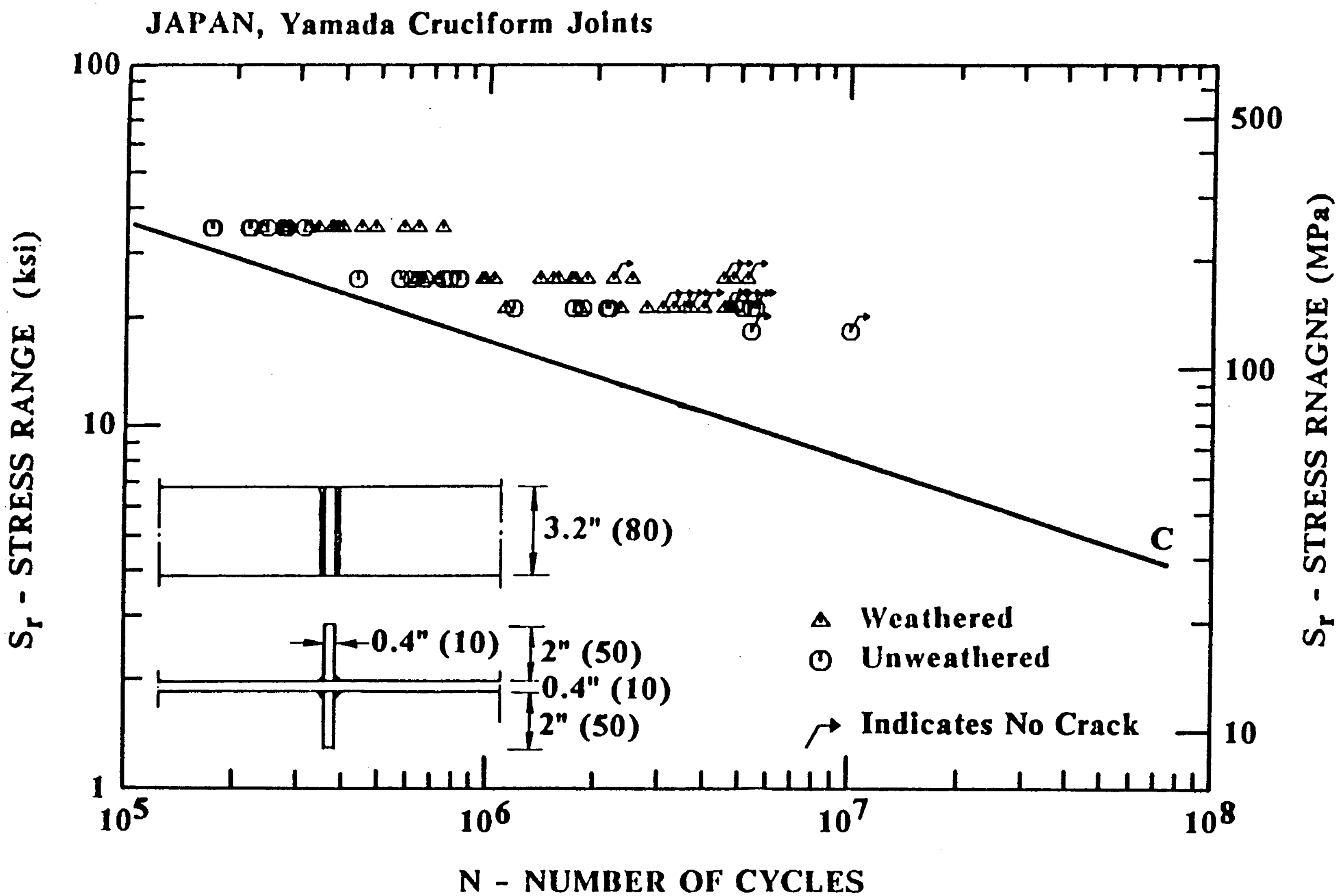

Figure 10. Fatigue resistance of weathered steel nonload carrying cruciform joints tested by Yamada.

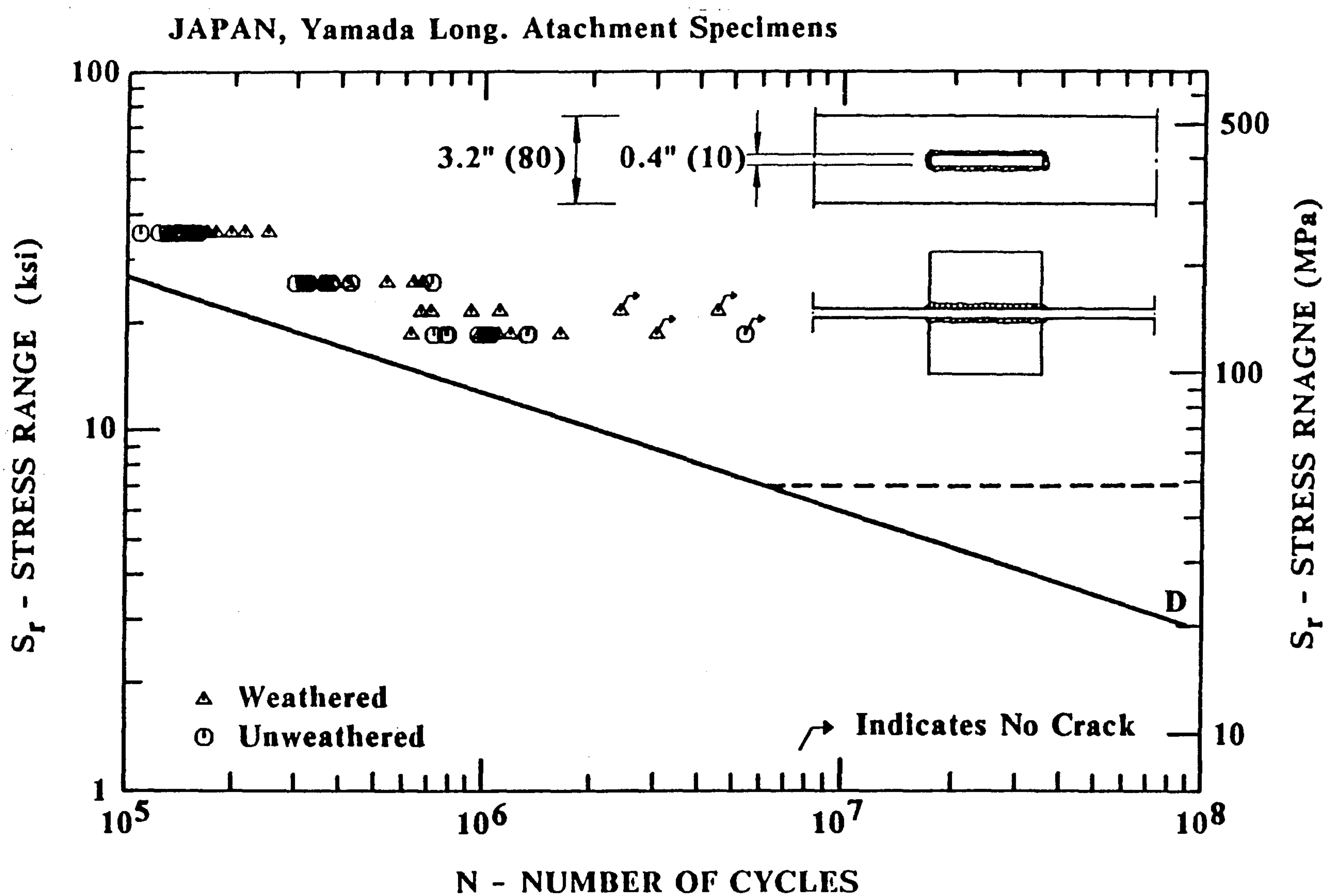

Figure 11. Fatigue resistance of weathered steel, simulated attachment specimens tested by Yamada.

Figure 12. End of cover plate after two years exposure.

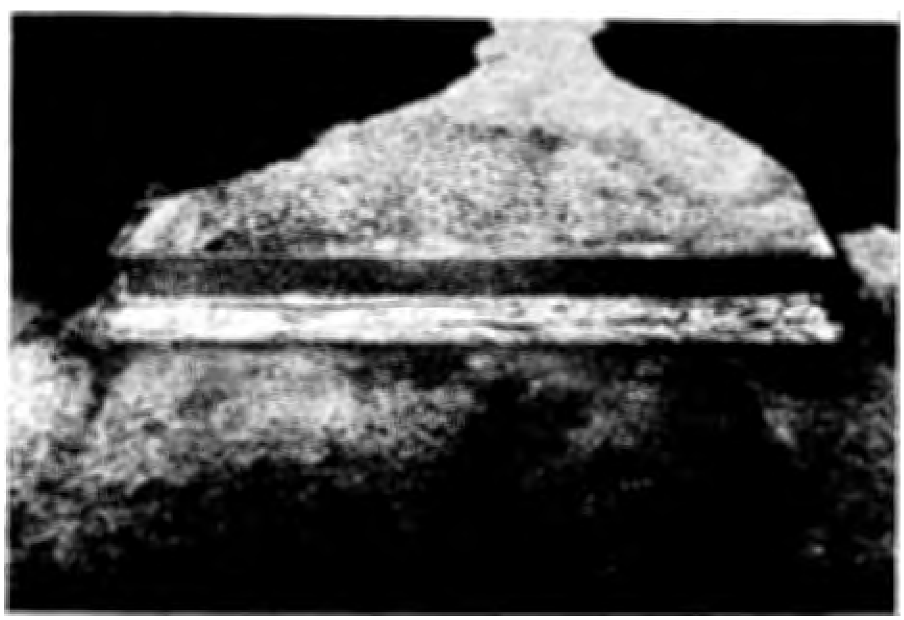

Figure 13. End of cover plate with crack in weld throat.

through the longitudinal welds and through the flange, as was observed with other cover plated beams where similar throat cracks were observed (Fisher et al, 1979), was estimated to be an additional 4.4 million cycles.

The test result is plotted in fig. 14 and compared with the results of other full-scale cover plated beams (Fisher et al, 1979). The weathered details exhibited higher fatigue lives than any of the non-weathered specimens tested at the 6 ksi (41 MPa) stress range that experienced cracking. Examination of the weld toe showed a smoother weld profile with a decreased stress concentration. The same phenomenon was observed by Yamada (1984). Fig. 15 shows a polished cross section through the weld and verifies the smooth weld toe and root crack through the weld throat.

The test data on weathering steel indicate no significant difference existed between the lower bound fatigue resistance of weathered and unweathered steel details. None of the test data for the weathered steel specimens plots below the AASHTO fatigue resistance curve applicable to the welded detail.

<u>Corrosion Notching</u>

Severe notches can develop in sections, particularly where dirt and debris accumulate and create an active corrosion cell. This type of notching is not restricted to A588 steel but will occur in all steel materials. The fatigue resistance of corrosion-notched riveted sections has been examined in full scale members removed from bridges (Fisher et al, 1987; Out et al, 1984). Figs. 16 and 17 show corrosion-notched flange angles of riveted members. In fig. 16, dirt and debris accumulated under a diaphragm, and this provided an active corrosion cell site.

The corrosion notches provide a substantial reduction in fatigue resistance. As several studies demonstrate (Fisher et al, 1987; Out et al, 1987), cracks developed in the gross section at the notches with little or no influence of rivet holes. The fatigue resistance of the severely corrosion notched section was observed to be as low as category E.

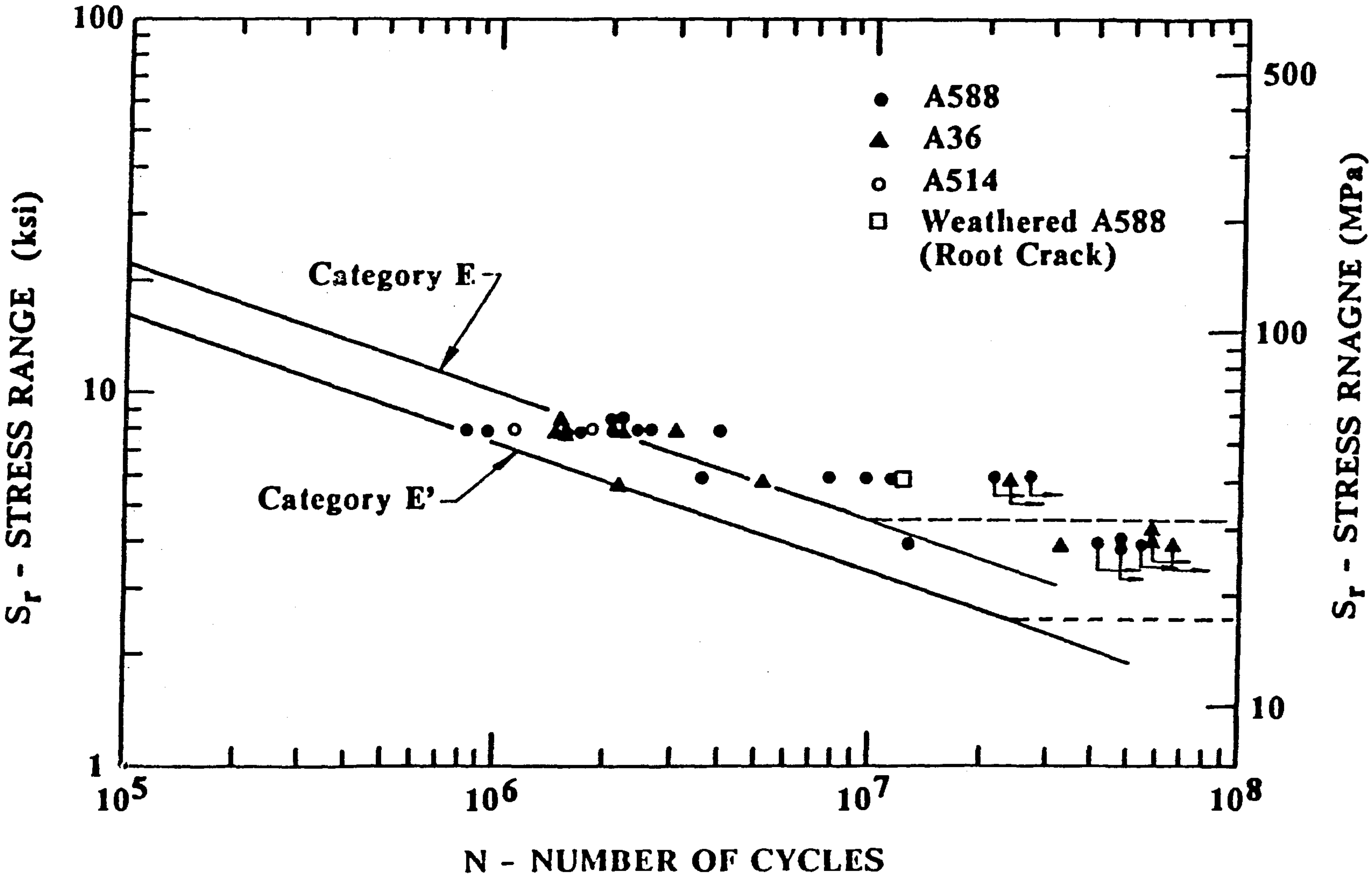

Figure 14. Comparison of weathered A588 steel cover plated beam with unweathered large scale beams.

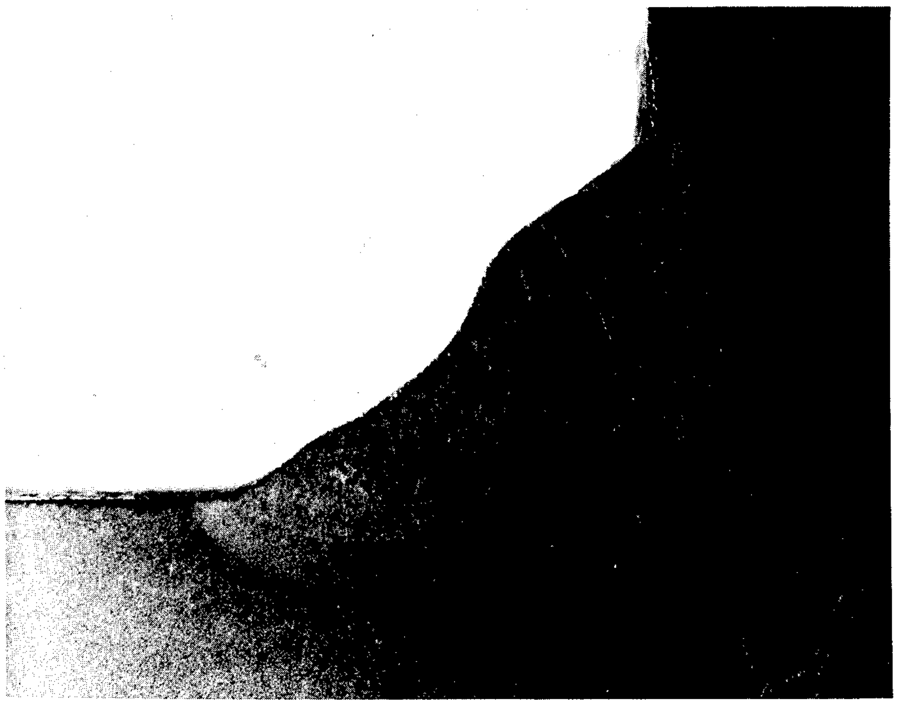

Figure 15. Section through transverse weld at end of cover plate showing root crack.

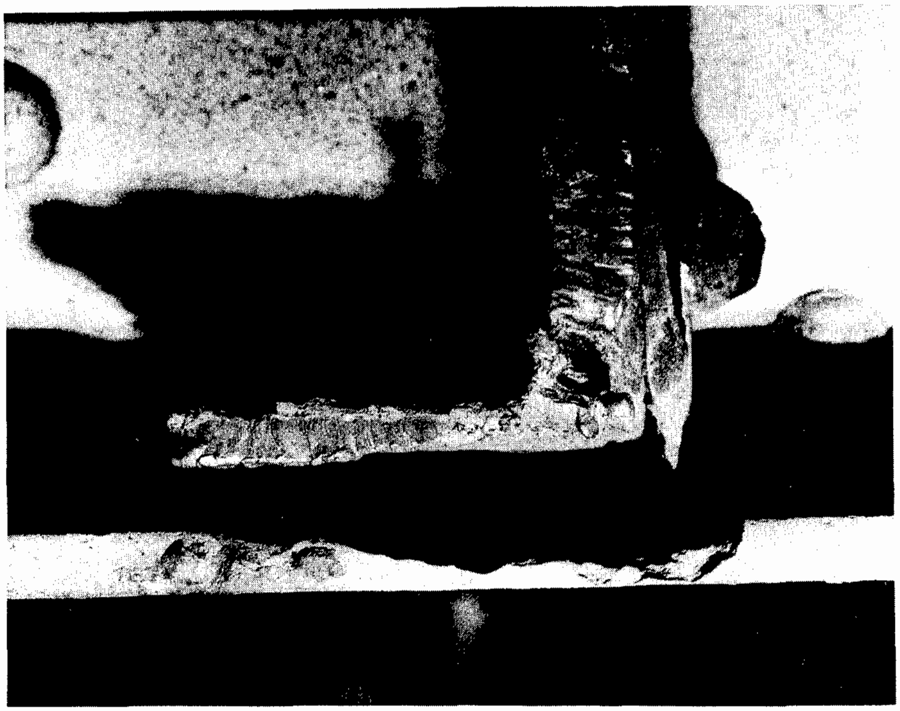

Figure 16. Corrosion-notched flange angle below diaphragm where dirt and debris accumulate.

Figure 17. Severe corrosion notching.

Fig. 18 shows the cracks that developed in the heavily corroded flange angle and web plate of the built-up section. It can be seen that the flange-angle crack was not influenced by the rivet holes.

It is important to note that in actual structures, cracks are seldom observed at the corrosion notched section until significant loss in the total section occurs. This appears to result from the fact that the rate of corrosion in actual structures exceeds the rate for fatigue damage, thus preventing the development of cracks. Any local damage from cyclic loading appears to be removed by the ongoing corrosion process. Cracks only form as the corrosion loss becomes extensive.

Welded Details Subjected to Aqueous Environment During Tests

Tests were carried out by Roberts et al (1986) on welded built-up girders with category E welded flange attachments. Specimens were fabricated from three types of structural steel (A36, A588 and A514). The flanges were all 2 in (51 mm) thick and 7 in (178 mm) wide with 1 in (25 mm) thick, 8 in (204 mm) long plates welded to the top surface. The experiments were carried out in three environmental conditions: air, distilled water and a 3.5 percent sodium chloride solution.

The specimens were subjected to the distilled water and sodium chloride solution by placing cotton cloth strips around each detail and dripping the liquid onto the strips so that they remained saturated throughout the experiments. Fig. 19 shows the plastic tubes used to drip the solution, the cloth strips, and the test beam.

The test results are summarized in fig. 20. This shows that the cycles to failure all exceeded the lower confidence limit for AASHTO category E. All of the beams subjected to the 3.5 percent sodium chloride solution provided cyclic lives equal to or greater than 10 million cycles at a stress range of 8 ksi (55 MPa).

Fig. 21 shows a crack at the flange tip after 13 million cycles. The crack path can be seen to be irregular. Fig. 22 shows the crack tip region and

Figure 18. Cracked components in corrosion-notched member.

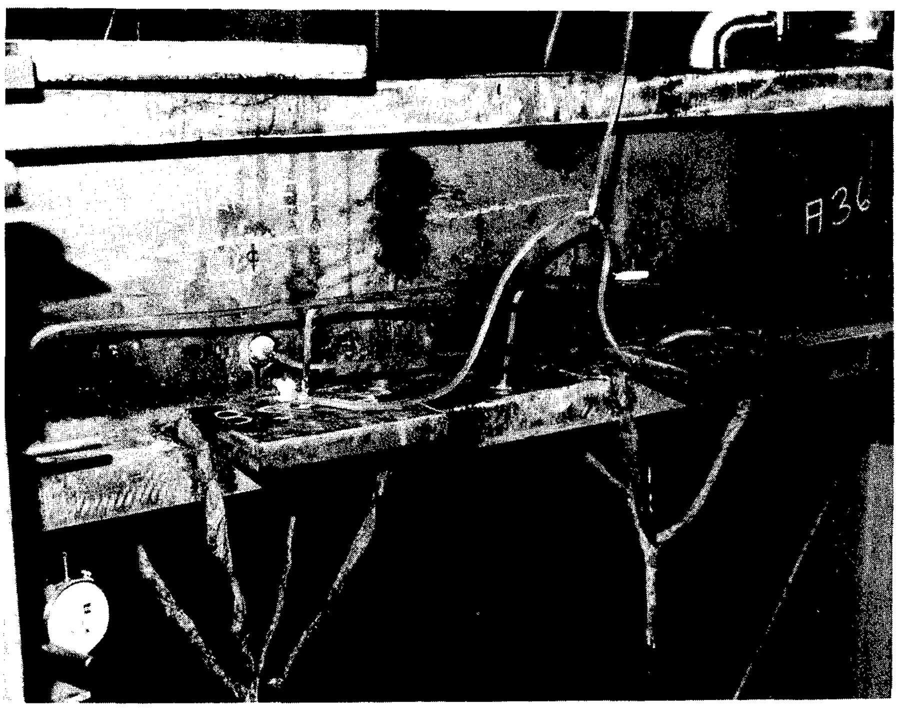

Figure 19. Test set-up showing welded details, tubing to distribute liquid solution, and cotton strips to hold and distribute liquid.

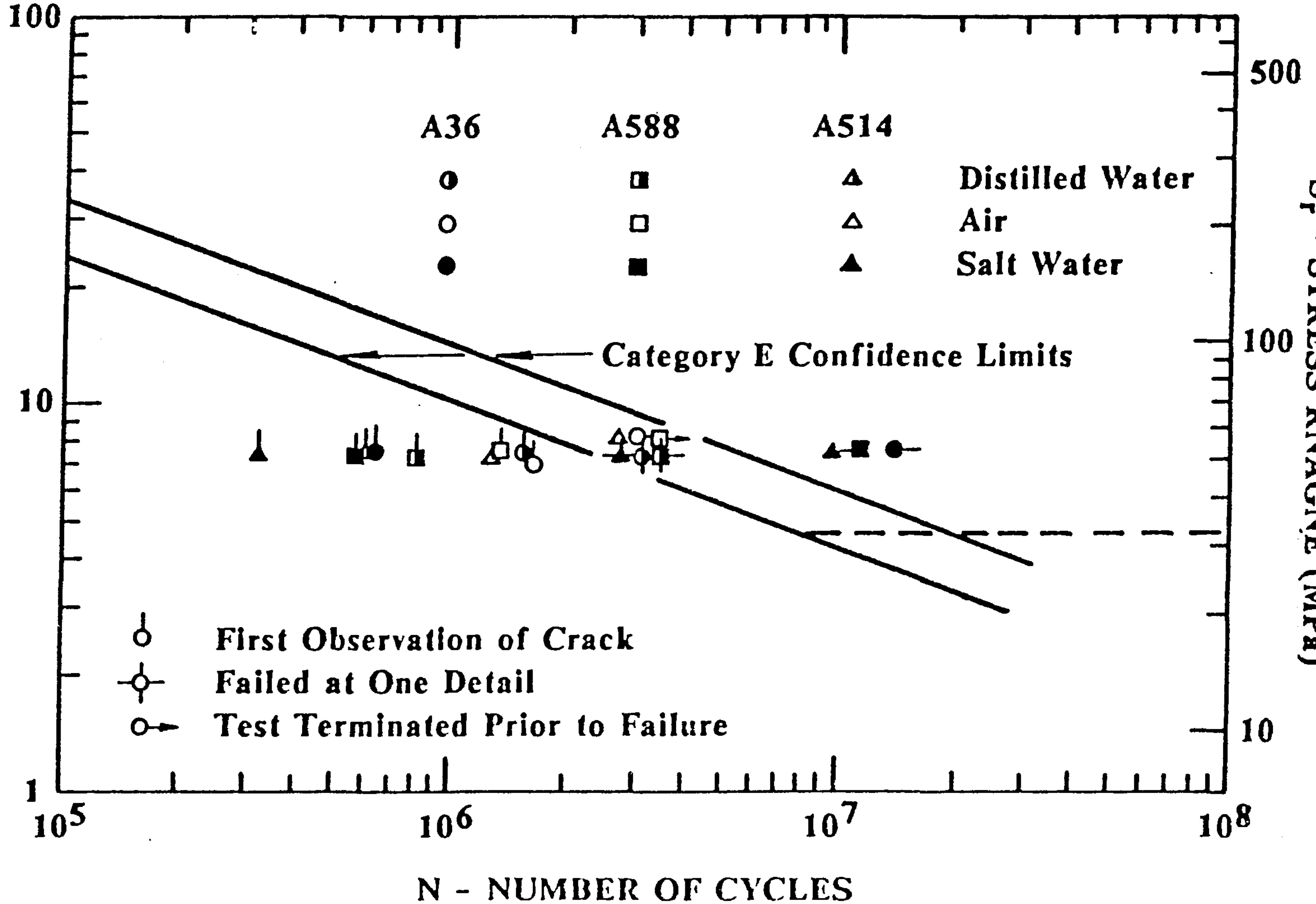

Figure 20. Tests on full scale girders with welded surface attachments in air, water, and salt water.

Figure 21. Flange tip crack at weld toe of A588 beam 3.5 percent NaCl.

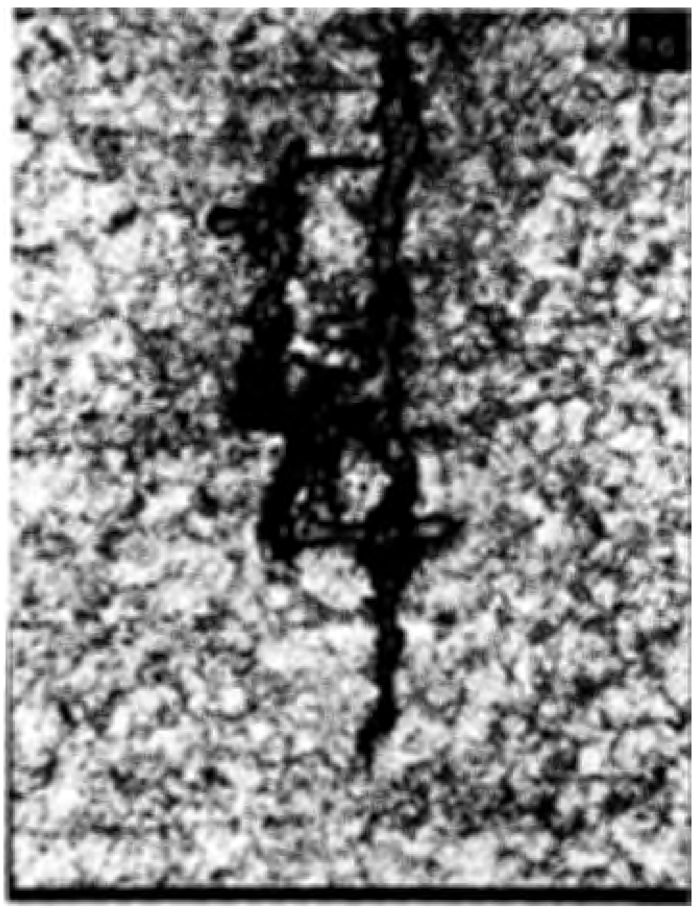

Figure 22. Fatigue crack tip of A588 steel detail at 40X.

illustrates the branching and irregular features that resulted in significantly reducing the crack growth rate by sometimes arresting crack growth.

The experiments on small cruciform specimens by Albrecht and Sidam (1987) were carried out fully immersed, and their small scale resulted in much higher stresses on the crack plane. Hence, no evidence of crack arrest was observed.

The tests on welded girders (Roberts et al, 1986) are compatible with the fatigue crack propagation characteristics reported by Barsom and Novak (1977) and Roberts (1986). Those studies showed an elevation of the fatigue crack propagation threshold when samples were subject to water or a 3.5 percent NaCl water solution. The equation proposed by Barsom for crack growth was found to provide a reasonable bound to the fatigue crack propagation behavior for air, water and 3.5 percent NaCl water solution.

Although cracking was detected first in the welded beam details subjected to 3.5 percent NaCl water environment, these details still provided the longest fatigue lives when influenced by that environment.

The environmental results also confirm that the safety provided by current design rules and specifications is not reduced by corrosion fatigue influences.

Field Experience

Weathering steel has been used in nearly all States with the earliest uses between 1964 and 1969 (Albrecht et al, 1984; AISI, 1982). Although corrosion difficulties were identified in a number of instances where the weathering steel surface remained wet for extended period of time or were subjected to salt contamination, no incident of premature fatigue cracking has been reported. Hence, even where high-volume truck traffic exists, such as on I-95 (New Jersey Turnpike), the field performance of weathering steel bridges with severe fatigue details (i.e. cover-plated beams) has not been observed to be adversely affected by weathering and severe environmental conditions (Noel, 1988).

To provide a broader perspective on the performance of steel bridge structures to environmental conditions, we need to examine the experience of structures in service since the beginning of this century. The issues of corrosion fatigue are not confined to A588 weathering steel. The studies summarized by Roberts (1986) and Barsom (1977) provided similar evidence for all of the structural steels.

Many highway and railroad bridges have been subjected to large numbers of stress cycles in this century. Furthermore, many have not been protected by coatings since their original shop coating after fabrication. Many of these structures have experienced severe corrosion pitting as well as corrosion notching, but very little evidence of fatigue cracking. An examination of the behavior of several of these structures will provide insight into the effects of environmental exposure and corrosion.

Among the earlier welded bridges that have been in service are single-span, single-track bridges on the mainline tracks on the Metro-North System. The New York, New Haven, and Hartford Railroad Company, now a part of Metro North, replaced the superstructures of many of their single span structures in 1940 with through-girder ballasted deck bridges consisting of two rolled beam girders with welded cover plates attached to the flanges and welded stiffeners and welded connection plates for the floor beams. These structures have carried both freight and passenger service since 1940. This has resulted in 10 to 20 million variable stress cycles from the locomotives and cars crossing the structures.

Fig. 23 shows a view of the underside of adjacent girders from two adjacent simple spans and the floor beams that frame into the girders. The cover plates are attached to the W36x300 A7 steel girders with longitudinal welds along the edges of the plate and intermittent plug welds along the center of the cover plate and across each end. This provides a category E′ fatigue resistant detail.

Figure 23. View of cover plated longitudinal girders and floor beams of adjacent single track bridges.

Fig. 24 shows a close-up view of the longitudinal weld termination and also shows the extensive pitting of the rolled beam and cover plate. Measurements on this structure indicated that the stress range varied between 1.5 and 4.9 ksi (10 and 34 MPa) with the effective stress range between 2.1 and 2.7 ksi (14 and 19 MPa). No detectable fatigue crack formation or growth is apparent which is consistent with the fatigue resistance provided by category E'. Should the environmental penalty suggested by Albrecht (Albrecht and Naeemi, 1984; Albrecht and Sidani, 1987) apply, extensive evidence of cracking would be apparent. This is not the case, nor is it generally expected from the other experimental data.

Other details such as the stiffeners welded to the girder web and flanges show the influence of corrosion, as can be seen in figs. 25 and 26. They demonstrate the corrosion activity that develops when dirt and debris accumulate at welded attachments. An examination of the weld toe region of these details indicated that a smoother weld toe transition was created by erosion of the corroded region. Hence, the influence of improved weld toe profile, reported by Yamada (1984) and also observed in fig. 15, is consistent with field behavior.

The corroded areas shown in figs. 23 to 26 do not show severe corrosion cell activity, although the pitting is substantial. More severe corrosion damage was detected on the Williamsburg Bridge in 1988 during field inspections (Williamsburg Technical Advisory Committee, 1988). The loss of section observed in girders, floor beams and stringers of roadways and transit structures was extensive, with the web often completely penetrated. This was a result of active corrosion cells, caused by surface runoff being channeled onto roadway joints and where splash plates stopped channeled water onto the adjacent structure.

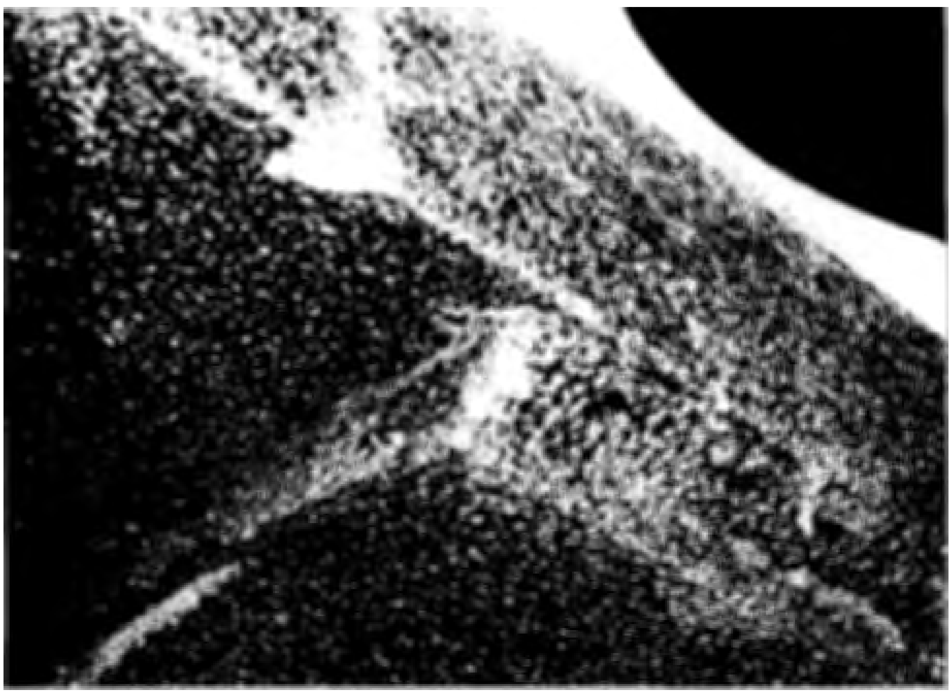

Figure 24. Close-up view of cover plate termination showing weld detail and corrosion pitting.

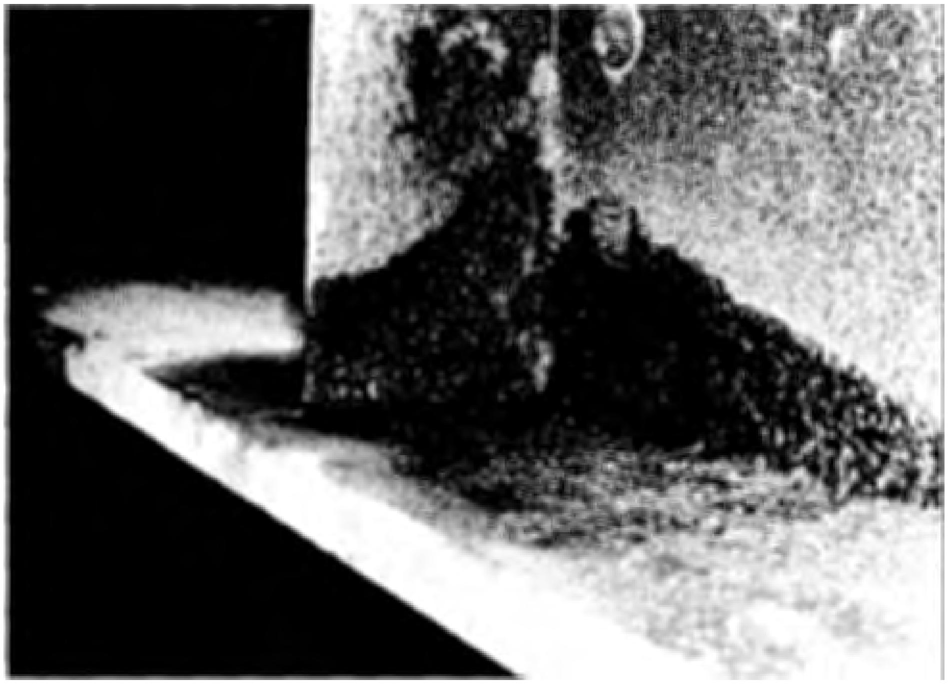

Figure 25. Corroded flange, web, and stiffener welded connection.

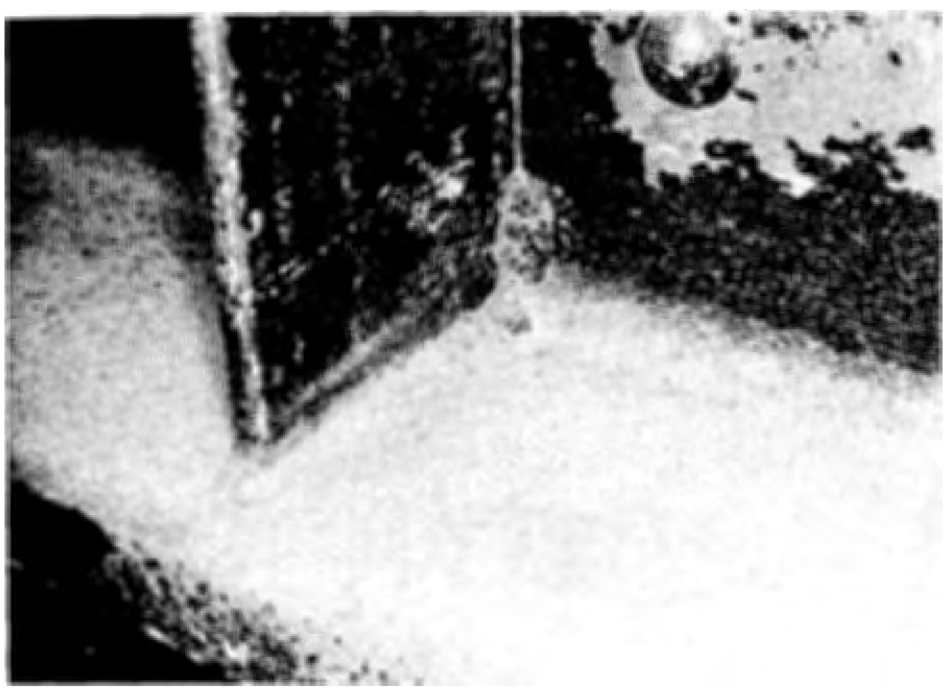

Figure 26. Corrosion at weld toe creates smoother weld transition.

Fig. 27 shows a typical roadway joint and the termination of the splash plate along the edge of the roadway slab. This permitted water, dirt, and roadway salts to be channeled onto the adjacent floor system. Since the support structure is partly sheltered, it does not dry readily. Moisture is retained by the dirt and debris that is piled on the flanges as well as adhered to the web and connection angles, as illustrated in fig. 28. The debris in industrial areas such as New York generally contains sulfates, chlorides, and sulfuric acid. Roadway water and salts add to this, on a continuing basis. This provides ideal conditions for crevice and pitting corrosion cells. These cells can be continuously active as moisture is present even under "dry conditions." When the dirt, coating, and corrosion product shown in fig. 28 were knocked from the web, moisture was observed in the pits and surface shown in fig. 29.

The crevices at the angle web lap joints provide a natural corrosion crevice. Breaks in the coating and mill scale provide ideal pitting conditions. These conditions are easily masked by the dirt and moderate general corrosion which can make visual detection of the corrosion battery difficult unless the area is cleaned.

The runoff from the roadway also enhances the oxygen concentration at the corroding areas. It is well known that the splash zone is the location of maximum corrosion rate in steel piling (Fisher et al, 1984). Oxygen is rapidly transported through the thin layer of water splashed on vertical surfaces. The Williamsburg Bridge provided ideal conditions for accelerated corrosion. The paint coating applied in 1973 did not have the durability to protect the structure from the debris and water it was subjected to. Once breaks developed in the paint film, corrosion cell activity was enhanced and accelerated.

Figure 27. View showing roadway joints and discontinuous splash plates along edge of Williamsburg Bridge Roadway.

Figure 28. Dirt adhering to stringer and floor beam webs, stringer flanges and end connection angles.

Figure 29. Pitting corrosion observed when dirt, paint, and corrosion products were removed from floor beam web.

At the floor beam end connections to the longitudinal girders and at the stringer connections to the floor beams, vertical corrosion slots were found, as illustrated in fig. 30. Fig. 30 shows a location that has not been sandblasted. The web penetration is primarily along the end connection angles. Substantial dirt accumulation can be seen on the bottom stringer flange.

Similar corrosion penetration was observed at the roadway floor beams, as illustrated in fig. 31. Again, the corroded web penetrations extend along the end connection angles and are particularly severe above the lateral gusset plate.

In general, where continuing corrosion occurs, as illustrated with experience on the Williamsburg Bridge as well as on numerous highway and railroad structures where corrosion notching is typical, such as illustrated in figs. 11 and 12, fatigue cracks of significant depth do not develop. The corrosion process erodes the local areas damaged by corrosion-fatigue as rapidly as they develop. This aspect of structural behavior is observed with all steels and is not unique to weathering steel.

Where fatigue cracks have formed, there is no evidence to indicate an acceleration in the crack growth rate attributable to environmental conditions. Bridges are not subjected to the environmental conditions of offshore structures which are submerged in sea water. In submerged structures where the joints are unprotected, reductions in fatigue life are expected if cathodic protection is not provided. This condition does not exist in bridges.

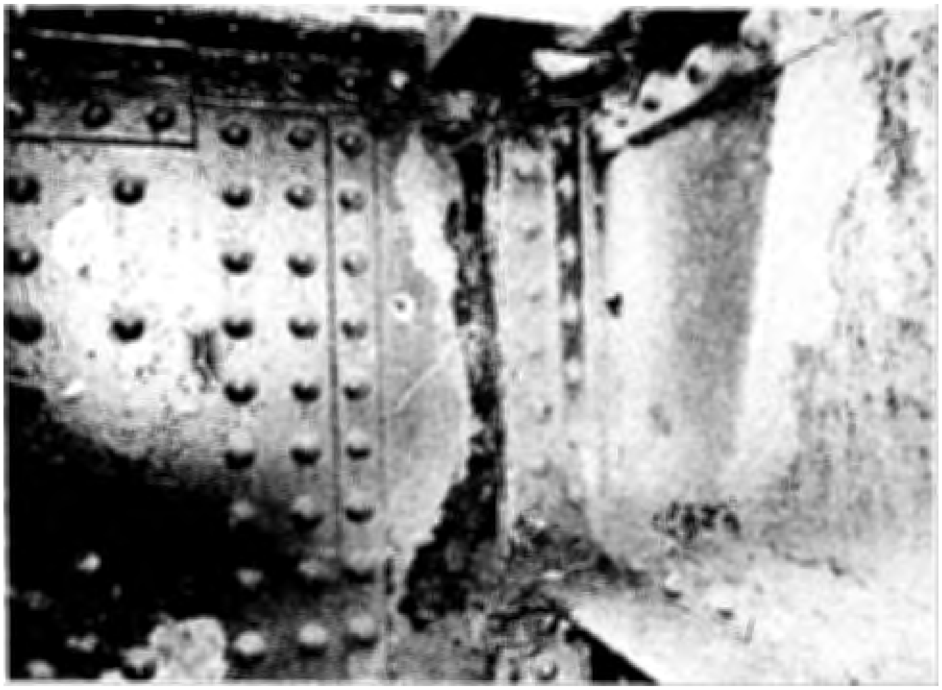

Figure 30. Corroded floor beam web along the vertical angle connection of transit support girders.

Figure 31. Corroded web penetrations between floor beam end connection and stringer seat angles at inner roadway.

Summary and Conclusions

(1) The experimental data indicate that no significant difference exists between the lower bound fatigue resistance of weathered and unweathered welded steel details.

(2) All experimental data except for category A show the fatigue resistance of weathering steel to be above the AASHTO resistance curve applicable to the tested detail.

(3) Available test data indicate that the crack growth threshold is enhanced when the crack tip is subjected to environmental exposure. This observed behavior was consistent with tests on large-scale beams with welded details which also exhibited enhanced fatigue resistance once crack growth developed at the weld toe.

(4) The notch effect due to rust pitting is equally common to all steels. Many steel bridges in service exhibit corrosion pitting. This has not resulted in fatigue cracking. The normal pitting that occurs in weathered steel bridge members decreases the fatigue resistance of as-rolled material from category A to category B. Categories A and B do not contribute to observed cracking in steel bridges.

(5) Severe corrosion notching will substantially reduce the fatigue resistance of all steel members. Generally, the corrosion notch is not related to a detail. It provides its own notch and resistance condition. Experiments have shown that the notch effect can be as severe as category E when large section losses occur causing stress elevation.

(6) Corrosion notching in actual structures seldom leads to significant fatigue cracking. This appears to result from the ongoing corrosion process which continually removes any fatigue damaged material.

(7) Overall, existing specification provisions for the fatigue design on steel structures adequately provide for the design of weathering steel bridge members.

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PANEL DISCUSSION OF FATIGUE RESISTANCE

Moderated by J.M. Barsom

[Dr. J. M. Barsom is a graduate of the University of Pittsburgh, where he received a B.S. degree in physics, an M.S. in mathematics, and a Ph.D. in mechanical engineering. Dr. Barsom is Senior Consultant - Metallurgical and Structural Performance at U.S. Steel Corporation. He is also an Adjunct Professor in the Civil Engineering Department at the University of Pittsburgh.]

Transcript:

NICKERSON: Thank you, John. When we set this program up we randomly selected the order of speakers. We did not intentionally put Dr. Albrecht first and Dr. Fisher second. Dr. Albrecht has asked for a couple of minutes to clarify some of his points. I want to give all of you a chance to be thinking about some of the things that we have heard here. We're going to distribute a test after Dr. Albrecht is done and find out how many of you really understood what was said. Dr. Fisher presented one side, Dr. Albrecht another, and because of the complexity of this issue, Dr. Barsom is going to do the moderation--not me.

Dr. Albrecht is going to give a couple of minutes here.

ALBRECHT: I would like to respond to several comments John has made in his presentation. First, John referred to the data by Yamada, which are the three lowest points in fig. 1 of my presentation. There is no question that these data show an increase in life, and we understand the mechanism that produced this increase. These tests, however, represent only 47 tests out of 1,565. I get the impression from John's comment that he suggests we ignore the loss in fatigue strength in all other tests because Yamada found an increase in his tests. I personally would not ignore all other data just because of Yamada's findings.

The second comment regards the comparison of the strength of a detail to the various AASHTO categories. For example, we tested transverse stiffeners that High Steel Structures had fabricated by the automatic submerged arc welding process. These specimens tested in air were close to category B, as you can tell. We have plotted the data where they belong, close to category B. We have not plotted the data at the category C* grid line (fig. 1). So we are plotting the data in its right place. We also tested specimens with 4 in long, flat plate attachments, which had about category C* fatigue strength. In this case we plotted the data points near the category C* grid line. We did not plot the data at the category D line which is applicable to 4-in long attachments welded to beam flanges. So we are plotting the data in its right place, corresponding to the fatigue strength that the <u>specimen</u> shows it has, not what someone might interpret it should have.

On the third point on cover plates, I would agree. The initial tests we are doing are also showing a rounding-off of the welds, and that is improving the fatigue strength. We have done three tests so far.

The fourth point is that the corrosive environment affects all steels alike. My suggestion is that we apply my recommendations to all steel structures not protected against corrosion. There is no need to distinguish between A588, A7, and other steels.

Fifth, John showed examples of railway bridges and highway bridges and said, well, we don't see any cracks, hence there are no problems. You have to keep in mind that the railway bridges in many cases have not gotten the number of cycles needed to initiate cracks because not that many trains cross bridges in the United States. It's not like in Japan where I saw at one railway station in the suburbs of Tokyo a train go by every 35 seconds. And for highway bridges, just because there are no cracks yet in A588 bridges, the oldest being only 22 years old, doesn't mean that there is no problem and we don't have to do anything about corrosion. I hope there will be no problems, but certainly that is no excuse to do nothing.

Sixth, regarding John's comment about off-shore rigs, I understand that we don't immerse bridges in the Pacific Ocean or in the Atlantic Ocean. But, let's face it, what is good for the goose is good for the gander. I should remind you that Barsom and Novak from U.S. Steel did a study for the NCHRP in the mid 1970's in which they immersed their specimens in water to measure crack growth rate, and John Fisher sat on the advisory panel of that project. So that is the standard in the industry. I want to also remind you that in the study sponsored by FHWA at Lehigh University and performed by Roberts, Fisher, and Irwin, crack growth rates were also measured with specimens immersed in saltwater. That is the standard in the industry. And, as I recall it, John Barsom sat on the advisory panel of the FHWA project. You can ask anyone from other industries--immersion is a standard testing procedure. It doesn't matter whether the steel is in the Pacific Ocean or the Atlantic Ocean or whether it is on a bridge subjected to salt and considerable moisture. The crack tip forms its own environment.

Seventh, I've heard yesterday about caked salt acting as a protective coating that chokes off the oxygen and prevents further corrosion. Today, I hear about salt arresting cracks. Well, my philosophy of designing structures is different.

Eighth, consider also what others are doing. Look, for example, at the specifications of the American Petroleum Institute for off-shore structures contaminated by salt in an aqueous environment. They have different design lines for air and for aqueous environments. Look at the European fatigue specifications for the design of bridges and buildings. You will find a statement saying that these specifications do not apply to steel structures which are not protected against the environment, where protection means painting or cathodic protection. Finally, 2 years ago, at the invitation of the Japan Society for the Promotion of Science, I visited many Japanese laboratories and steel companies. I spent 1 day each with the two largest steel manufacturers in Japan. We obviously discussed the subject of weathering steel and the differences of opinion that exist in this country regarding its fatigue strength. I told them that I was recommending basically a reduction in life by a factor of 2. There was silence in the room. Since I got no reaction I asked them, well, what do you think. They said it was not enough. So I asked how much is enough. They said they would reduce the fatigue life by a factor of 5 to 6. As you know, the Japanese have a good track record of doing things right.

Now, back in the early 1980's I published a paper and John in his discussion agreed that for category A details a reduction to category B was justified. It seems that today John does not agree to anything.

FISHER: No, that's not what I said, Pedro. I'm saying that I think you're only looking at one side of the equation. You've got to look at the fact that there is no bridge subjected to levels of stress range that are above 10 ksi. The laboratory test results show a reduction in the finite life region for category A. I agree with what you had chosen. But I'm just questioning the relevance of it.

ALBRECHT: I know what the stress ranges are in bridges. A student of mine collected 190 stress range histograms from 40 bridges in 9 States. We have documented the work. I think you have to be careful not to combine two different issues here. We know, for example, that two cover plated bridges in Alabama have maximum measured stress ranges of 7 ksi, which is considerably higher than the allowable stress range. On all bridges for which we had data on category E' details, the maximum measured stress range was higher than the calculated stress range. For category A base metal, the measured stress ranges were 10 ksi. I don't think that there are two different worlds as you are suggesting.

NICKERSON: We all heard Dr. Fisher mention the lack of water control on the Williamsburg Bridge, and before we have that same problem here I think we better break, have our coffee. Fill your questions out, pass them to the center, and Dr. Barsom will resume at about 10:20.

We are going to revise the schedule that was set up for the forum. We're obviously running behind schedule at this point, and we're going to consider this afternoon's summary session somewhat sacrificial in the interest of trying to get a better understanding of the issue at hand. So, Mr. Schmidt's presentation and Mr. Wasserman's presentation will follow the conclusion of the panel discussion, which may mean after lunch. If necessary, we will continue the panel discussion up to lunch time. Hopefully, the issues can be brought out and succinctly presented so that people like myself can understand them; I don't know that 3 days is enough time for that, but hopefully, we can try and get a better handle on exactly what is fatigue resistance of A588 steel.

As I had indicated earlier, Dr. John Barsom is going to moderate the discussion. If you do have any questions that have been written out and you pass them to the center aisle, I'll collect them and John will use them to initiate the discussions. If you feel you would rather stand up and ask the question yourself, why please use the microphones--there are three of them spread across the room so they are accessible to virtually everybody. So without further ado, Dr. John Barsom of U.S. Steel.

BARSOM: Good morning, ladies and gentlemen. For a moment I wanted to ask "Why me?" and then I came up with an answer: I'm going to have one on my right, and one on my left, and being short enough, I can duck all the punches. What I'd like to do is, for those of you who have written questions down, to give me your questions. And for those of you who would like to get to the microphone to do so. If you are going to ask questions from the audience, please use the microphone so that everybody can hear you.

I, as you heard, my name has been mentioned several times, spent some of my better years in the wrong field doing corrosion fatigue, and there were a heck of a lot things I realize I could have done that would have been more fun. The problem of corrosion fatigue is extremely complex. It is significantly more complex than fatigue in a benign environment.

I just want to make one point that was mentioned earlier, that quite a bit of the data that has been generated by myself and others has been either in the air or in full immersion. Unfortunately, the reason for it was not because it was the right thing to do. It was because it was the easiest thing to do. We know how to run data in air. We know how to run it in full immersion, and I spent close to 12 years running tests, or trying to run tests, for simulating actual weathering conditions and I gave up. That is why I feel that maybe I would have done a *** of a lot better if I went in to another field.

With that in mind, I am asking all of you that when you ask your questions or make your comments, please make them short, precise, and concise so that we can have everybody that has a question to do so. I am also asking the two presenters--who did a very good job--to try as hard as possible not to argue with each other and to try to address the questions as they arise. So with that I would like to invite Pedro and John, one on each side, and we'll start with the questions.

Now I was told that we can go as long as we need to answer questions and we have till lunch time. Now I hope you can last that long. I know they can last much longer. I have only two questions in here right now and I will start with--actually the both of them are to John--the first question is:

"You indicated that Pedro did not get crack arresting because of the small specimen he was using. Did you use small specimens in developing the original curves, and if so did you get crack arrested?"

FISHER: No, we did not use small specimens. In fact, if you read the trail of NCHRP reports, you will note that we have purposely excluded small specimens because we found--and that's discussed in considerable detail in NCHRP Reports 102 and 147--we found in general that small specimen tests give you higher fatigue resistance than the built-up sections and that, I think, is due to distribution of defects and the residual stress conditions and the geometric variations. The smaller the specimen, particularly for machined specimens, the less likelihood you will get the worst case condition. So the original specifications rely predominately upon the full simulated scale specimens which, as Pedro has indicated, are essentially the same size beams that he was talking about in his exposure tests. Larger specimens have been examined using the heavier wide flange shapes. This led to the development of the E' prime category.

ALBRECHT: The data I have shown to you on rolled beams and welded beams are for wide flange 14 X 30 sections. The same size beams were used in Reports 102 and 147. So those specimens are of the same size. Then we did tensile tests of specimens with transverse stiffeners and 4-in attachments. Those with traverse stiffeners had two qualities of welds. Both were done by High Steel Structures.

The first series had automatic submerged arc welds, giving the stiffener a strength close to category B because the weld equality was excellent. In the second series of tests on the same type of detail, we specifically asked High Steel Structures not to do such a good job. In fact, we asked them to manually weld the stiffeners so that we would get a fatigue strength closer to that of the beams that I've tested at Lehigh University when I was there as a Ph.D.

student working with John. The fatigue strength of the manually welded stiffeners was within 5 percent of the fatigue strength of the wide flange 14 x 30 steel beams with transverse stiffeners and the 38-in-deep girders with transverse stiffeners. Weld quality, not specimen size, affected the fatigue strength in this case.

BARSOM: Next question, I'll have Pedro start on it.

"What ultrasonic equipment is operational to measure steel plate thickness?" Continuation of the same, **"what A588 surface preparation is required, and finally, what is the degree of accuracy?"**

ALBRECHT: That is a good question and I think we know more, considerably more, today than we knew 3 years ago, mainly because of the work that McCrum has done in Michigan. He has extensively measured the thickness of members in a very large number of bridges. The man to answer that question is McCrum. Basically, one can measure with reasonable accuracy--certainly for the purpose of checking cross sectional properties--the thickness of corroded members. This involves grinding, as McCrum has recommended, one side of the plate until about 30 percent of the surface shows blank metal so that the probe can firmly seat on the base metal. So one grinds one side of the plate, applies the coupling agent, and measures the thickness. In general, the ultrasound waves are reflected on the back side of the plate by the metal-oxide interface. They don't penetrate the rust on the back side too much and that seems to give good results. We have tried this ourselves and we agree with McCrum's findings.

BARSOM: What degree of accuracy?

ALBRECHT: I'm not sure what the participant means by the degree of accuracy. The instrument can measure the thickness of a machined block to within 1 mil. In the field, I'm sure the error is somewhat larger than that. How much larger, I cannot tell you for sure. McCrum would be the person to answer that. I'm sure Chuck Arnold would provide you with McCrum's telephone number if you would like to have that information.

FISHER: I think I would agree with Pedro that you can get probably within 1 or 2 mils of the thickness. One of the difficulties you'll have, of course, is with pitting and crevice corrosion. Pitting corrosion may prevent access with the ultrasonic probe. So you may have problems in that regard to measure the residual thickness.

BARSOM: The long, long question to John Fisher.

"Does weathered include severely weathered or just ideally weathered?"

FISHER: Well, I would put for the application of the weathered steel, if there is continuous corrosion cell activity I think that is fairly severe. I don't really think we should be using weathering steel unless we can stabilize the oxide because effectively if there is going to be a continuing corrosion process, I think we're going to have a corrosion problem and not a fatigue problem because we are just going to continually wipe out the section. Who was it, I guess Arnold was saying yesterday--or maybe it was Al Dunn--saying they were losing 5 or 6 mils a year. I can assure you, you lose 5 or 6 mils a year, there are no fatigue cracks that are going to catch up with that rate of corrosion or fatigue crack initiation. So, you're just not going to ever get a fatigue crack; you're going to get perforation. The issue here is developing a stable oxide film.

ALBRECHT: If the question is related to the data that I've shown, let me clarify that all data--which I have called the weathering fatigue S-N data--came from specimens ideally and boldly weathered from 2 to 8 years. Boldly exposed, no salt contamination, just ideal weathering conditions.

BARSOM: Next question:

"Sounds like we are hearing that cleaning (washing) with salt problems with surface weathering on salt-contaminated A588. Are there data to support this conclusion?"

FISHER: I don't know if there have been any data. If you have salt splashing on bridges like Michigan has on both A588 and quasi-painted structures that are filthy, the dirt accumulating on beam flanges is promoting corrosion cell activity. That is not going to be solved by washing because unless they can really keep that dirt off, you probably will still get some of this penetration if you're going to have a continuing process. But I think for modest salt applications we should be able to control the corrosion with the details that minimize water on the structure as was pointed out yesterday in the presentation on the New Jersey Turnpike. Certainly that is as heavily traveled an artery as there is, and there is no evidence of an acceleration in fatigue damage. The traffic is similar on I95 in Connecticut. In a 25-year period, they have seen maybe 40 million cycles. The situation is similar to the point that I was making on the railroad bridge: I know how many cycles these bridges have experienced. They have experienced 20 to 30 million cycles because it is on a heavily traveled line. It's on the East Coast, carrying, since 1940, passenger and freight traffic and a known number of trains. So we have the types of structures that have welded details. In each of these cases, they would have experienced cracking if corrosion fatigue was evident as are category E details. I think salt and debris accumulation, if cleaned off yearly, would make a substantial improvement. I just don't know of any test data to support this view. But I think it is something that we should seriously consider for the future.

ALBRECHT: In my opinion, there are two aspects to the question. Hosing the bridges will certainly wash off loose debris that traps moisture and chlorides, and that is helpful. There is no doubt about that. You want to get rid of the poultice. The second aspect has to do with the salt trapped in the rust coating. There is no evidence, to my knowledge, that hosing bridges will wash out the salt trapped in the rust coating. In fact, there is no way it can do that because the salt migrates through the porous and cracked rust coating to the interface between the base metal and the rust coating. In the beams which we have exposed, below slabs of rust that had fallen off the bottom flange, we found a white layer of salt that had migrated inward. The salt was laying close to the steel. This trapped salt cannot be washed out by hosing.

BARSOM: Now, both of you gentlemen, if you don't understand the question like this one I'm going to read--I don't understand it--you don't have to answer it. And maybe the one that wrote it can explain it. Maybe you do understand it. To you, John:

"You said 'do not have conditions of emergence.' Which data show corrosion rates between 'severe marine' and 'immersion in seawater'? No place for uncoated A588."

FISHER: I would agree we shouldn't be using A588 if we have continuous marine exposure, if that is what that statement means. Insofar as the other, I would interpret that in the way tests have been carried out and there have not been any tests. We've tested only these two conditions of weathering exposure to environmental conditions. In one case, specimens are brought into the laboratory and then tested in a laboratory environment without further exposure. In the second case, there is sodium chloride or distilled water continuously applied because we're trying to compress the test time and that is the point that I was trying to make. We have only tested those two extremes. There are no tests in between. But what I've also indicated is that the evidence indicates that the crack growth threshold increases as a result of continual environmental saturation. In fact, there are tests available where people have stopped testing and where they've had the aggressive environment in place. After a day or two, it takes a devil of a long time to re-initiate the crack. In fact, I think Barsom experienced that on some tests.

ALBRECHT: John referred to a test done by Barsom and Novak on their NCHRP study. After reading the report, I called up Novak and asked him about that. Novak told me that the testing machine had stopped overnight. Next day when they restarted the test it took longer for the crack to reinitiate. Novak told me that he thought the equipment malfunction had applied an overload and that is what caused the crack growth delay. We know that periodic overloads retard the growth. So that is Novak's explanation. That is something Novak didn't come to realize until later, so it is no criticism of Novak.

May I ask a question? You were referring to other data showing a higher threshold in an aqueous environment than in air. What data is that?

FISHER: Both Barsom's crack propagation studies where he diminished the frequency, as well as our own at Lehigh which Roberts carried out.

BARSOM: I want to continue with the questions, but now that my name is mentioned, from both sides, the test that was mentioned was not done by Novak, it was done by myself. After that, I had an MIT student do his Master's degree with me, and we went through much more testing and depending on the region in the crack propagation, crack initiation behavior, you can have the deceleration of crack growth and essentially no crack growth whatsoever. And in other regions, you can have these cracks start very fast. While I'm at that point, I'm going to make a few statements now.

I think, part of the problem, I think, is a matter of trying to sit down and look at the data and analyze it and try to explain it and how it would apply to the real world. I think what John presented about the real world is there and what Pedro presented on data is there. The only complication comes in that fatigue and what we dealt with as fatigue in air and all the AASHTO fatigue curves are easy to generate, and they were generated under a benign environment where frequency of testing is not important; how you go from the minimum load to the maximum load is not important. Most important parameters in corrosion fatigue become very innocuous when you go to fatigue. So we can have an equivalency between constant aptitude loading and variable aptitude loading and develop the actual fatigue curves. When we go to corrosion fatigue--and I'm going to stick to full immersion because wetting and drying is almost impossible to try in duplicate within a lifetime or to get some sense out of it within a lifetime--when you go to corrosion fatigue, it is a completely new different game. And we have to start looking at things a little bit more critically.

For example, bridges see to 2 ksi stress range for, let's say, 100 million cycle. That is say a 60-year period. We cannot run tests for 60 years. At the same time, that is happening, there is a general corrosion going on where some of the materials are being chewed away.

Now what we really have to do is what has been started on variable aptitude loading and those of you that are familiar with the work that has been done by Fred Moses and Chuck Shillings know that to try and apply the variable aptitude loading to retrofit bridges we have to go to the 2 ksi stress range for very long life. We have to start thinking in the same kind of way for corrosion fatigue. Frequency is very important. The way you go from the minimum load to the maximum load. If you apply it very fast as a square cycle, there is no environmental effect. When you apply it very slowly as a sinusoidal curve, there is a big effect. Unless we sit down and try and look at things in terms of how fast the general corrosion is occurring, how fast the cracking is occurring, which one takes over in what region of the behavior, we are going to be arguing about this for a long time and we are going to have a *** of a lot more research that is going to cause us more problems. Now with that statement made--and I'm sure both sides are going to argue with me, and I hope they don't argue with me, but answer the questions from the audience--I'm going to read the last part of this card, which is not a question but I'm sure both sides will have some comments on it.

"Ironically, it sounds like the Williamsburg Bridge might have failed sooner by cracking if corrosion had been controlled."

FISHER: No, I don't think so, because the magnitude of the cyclic stress, had there been no corrosion cell activity, would not have produced cracking and there are enough members that are not corroded to illustrate that. Actually, we are talking about thousands of members and floor beams. We're not talking few. There were only about 50 that were in the condition that was described. I don't think cracking would have developed because it is essentially a riveted system, and the fatigue limit is close to category D and there aren't very many cycles that would exceed that.

BARSOM: Are there any other questions that have been written on cards? These are the people that want to be anonymous when they ask the question. I guess there aren't any, so it is open to the audience for any questions or comments.

PARTICIPANT: I agree with Dr. Fisher, simulating real life situations in a laboratory, I think we all realize it is very difficult and that is why we have accelerated tests. I've been involved with Dr. Albrecht's research right from the day he initiated it. I forget how many years ago, I was a division bridge engineer in the State of Maryland at that time and one of my primary responsibilities was reviewing the regional scope of work for proposed research and it has continued up until today and it is still continuing, because each time we finished one phase of the report we found more questions than we had answered so we continued on. And one of the issues that was addressed at each and every step of this research was how do we simulate real life. We had numerous bridge engineers sitting in the room advising Dr. Albrecht how to do that. I don't recall who actually came up with the method of trying to do this, such as sponges or whatever, but we all agreed it was the closest that we could come. So there were many practical bridge engineers' input into the methods he used in his testing.

But if I understand what has happened here, Dr. Albrecht's research shows proposed reductions in design life that he should be using in new designs and Dr. Fisher says that it doesn't relate to real life, we shouldn't be using those reductions. Yet, the entire AASHTO specification is predicated on exactly the same basis: research data generated in a laboratory. For example, the cover-plated rolled beams. We virtually eliminated those on new structures on any of our major highways because of this laboratory based research. Is the issue really to reevaluate the entire AASHTO specifications using Dr. Albrecht's data and real life data to determine if there is a better way to simulate this for design of our structures? That is a long question, I realize.

FISHER: I think we have to consider that these are the boundaries and the real world condition is somewhere in between. And I think it's much closer to the air, without this violent exposure condition for the reasons I've cited. I think the experience with structures that we have examined demonstrates that the reductions that were alluded to by the petroleum industry for offshore structures are not applicable. These structures offset a corrosion fatigue problem by cathodic protection of the structure and that is the only way they can eliminate it. So if they don't put cathodic protection, they do have to have a very severe penalty, and where they have offshore structures and they do not want to apply that penalty, they put cathodic protection in place so they don't experience the corrosion fatigue problem. The European and API specifications and the

Japanese all provide that option. We've got these two extremes, and my view is that there is nothing wrong with generating the data. The question is how are we going to apply it to the real bridge application. I personally cannot agree that it is reasonable to apply offshore structure design to bridge structures. I do not believe we have the type of environmental condition that these details reside in. There is absolutely no evidence of it.

PARTICIPANT: This has been a fascinating day and a half, and I have a couple of questions and a couple of statements if I may indulge the panel. The first one, you have three very, very highly respected people in fatigue up there, two of them are really the ones that are doing the questions and the research and such. I don't think there is really any question in the way the research is being done or the results as such. As both of you have said, it is the interpretation. And I go back to my own area, which is not so much fatigue, although I've worked in fatigue a little bit, but it is mainly in fracture.

I can do the same thing in fracture. I can make anything fracture. I can subject it to the worst conditions, the highest loading rate, the sharpest notch, the shortest time, etc., and I can make it fail at anytime and I think you can do the same sort of thing in fatigue. But where I really have my problem and, the fascination with it, is to try to decide what is the significance and what is the real world situation and try to model it. I go back to the times that I worked very closely with Bill Palini, back at the Naval Research Laboratory, and Bill took one extreme. It was the explosion bulge test with notches and defects in which nothing would ever fail if you always adhered to Bill's philosophy and yet that was a far cry from what you really saw going on in the real world.

And I think the real issue is: are we having a problem in corrosion fatigue? And, if so, what can be done about it? And, specifically, is the best thing that can be done is to, as Pedro suggests, change the design curves or is the best thing that can be done, as I heard some comments yesterday, not necessarily washing, perhaps better design details? We kind of did this--well, you kind of did this--in the AASHTO fracture control plan for fracture. You did not go to extremes of material toughness but you put limitations on the welding procedures. You put limitations on design details. You put a number of limitations when coupled with the particular material characterizations for the fracture situation. Many people thought, and I hope that is the case, solve the fracture problem.

I think a similar analogy, perhaps, exists in the fatigue situation. Clearly, you can go to no cyclic stress range or very, very small cycle stress range and you won't have a fatigue problem. But, as I tell my students, if you do that on an airplane, you're not going to have a plane that flies, either. I think there must be some rational and fairly straightforward and simple things, maybe beyond just the washing. We talked about the scuppers, the expansion joints, the washing. There are a number of things, which in conjunction with the realization of what the significance of these results are, maybe solve the problem and then my last question is: what is the problem? Have we seen an awful lot of fatigue failures of weathering steel? So I guess my question would be what problem are we trying to solve today?

ALBRECHT: The problem we are trying to solve is to design weathering steel structures in such a way that they have a desired fatigue strength. That is what we are after, right?

I think your analogy with fracture is all right. We are setting minimum standards on fracture toughness, or Charpy V-notch toughness for steels and, in the same fashion, I feel that we should set minimum standards on fatigue strength for bare, exposed steel structures. Certainly good detailing is not only helpful; it is absolutely necessary for the protective oxide to form, and I think everyone would agree. I think that this alone will not address the fatigue issue because the worst data I have shown is not being considered in my recommendation. I'm assuming that severely corroding bridges would be painted. So, again, the weathering fatigue tests were done with ideally, boldly exposed specimens and the corrosion fatigue tests with as-fabricated specimens. There was no severe corrosion condition prior to stress cycling of these specimens.

I would like to caution you on one point. Ideally we would perform a 30- to 40-year test in which we would simulate as closely as possible the alternate cycling and corrosion of specimens, but that is never going to happen. The NCHRP did a major study on corrosion fatigue, and the FHWA then sponsored an even longer study. To my knowledge, there are no other studies down the pipeline and no other weathering fatigue S-N are being done in the United States other than our work. I think I'm the only one who has a few specimens left to test. Maybe U.S. Steel has a few specimens, but that is it. There are plenty of data available, generated by people worldwide who have tried their best under the given circumstances to simulate what they think would apply reasonably well to structures. There is a phenomenal amount of data there, enough to make a decision if one wants to. In the end you have to decide what you want to do. After all, it's your bridges. A last question: What does a prudent engineer do under such conditions?

FISHER: Well, I won't try to answer, because I think that there are data there. That is the point that I was trying to make. We've got bridges that have had exposure that have been in service for 50 years, and there is to my knowledge no evidence of corrosion fatigue. We have fatigue cracks forming that are compatible with the air data in every instance that I know of. So I see no evidence that there is a problem we can attribute to weathering steel. I think we have the corrosion notching problem, and that is the point made yesterday by the New Jersey Turnpike presentation. We shouldn't be using these bridges where they are not providing protection from water and dirt. Whether it is painted or uncoated, there are similar problems from water and debris. The hangers on the Miannis River Bridge were presumably painted. The problems of corrosion are equally applicable to coated steel and weather steel.

ALBRECHT: ... the weathering steel does work and it can be used under the conditions that were stated over and over again. Namely, good detailing and an environment that is not aggressive towards the steel. Under these conditions we can tolerate corrosion.

FISHER: Yes, I think that is true. Weathering steel under proper application will not provide a continuous corrosion cell. I don't think it should be used under corrosion cell conditions. That is not the proper use of it. If we cannot adhere to those points that were cited yesterday, then I think that is an improper application. But we need to pay the same attention to the painted systems.

ALBRECHT: When as-fabricated beams are tested in air, the rolled beams have category A fatigue strength, and the welded beams have category B fatigue strength. So, the rolled beams have higher strength than the welded beams when tested in air.

The salt-contaminated rolled and welded beams had the same fatigue strength. Both dropped down to category D, meaning that in these beams the rust pit was the dominant flaw and the rust pit was more severe than the longitudinal fillet weld in the welded beams. So both had the same strength as that of category D.

FISHER: I don't think I need to add anything more. That is a point that I was making when I addressed that issue to Pedro that if we are going to deal with categories A and B some penalty would apply--although in bridges, I think we have to consider that we'll never likely expose any bridge structure to a stress range that would exceed the crack growth threshold, even in a notched condition. So the real world that the bridge sees versus what we use for the design equivalent are two different things. If we're going to treat the corrosion-notch effect, I think it should be the same for rolled and built-up section.

PARTICIPANT: [inaudible]

FISHER: The corrosion cell problem is predominately associated with dirt accumulation and debris which the oxidation provides. I think by washing members you would knock loose scale off. I think the presentations yesterday on corrosion indicated you have to create a corrosion cell. For the most part you only see very severe corrosion-notching conditions when you allow debris which is a combination of dirt, the oxides and saltwater to be trapped and held. Washing will do marvelous things because it will remove the residual poultice that is going to exist and that is your dominant source of severe corrosion.

BARSOM: I think what, if I understood, the purpose of the washing is not to remove the chlorides, because that is a much more difficult task. As Pedro said, it is very hard to get rid of them; it's to get rid of the debris that sits there and keep the moisture in there. Chuck.

PARTICIPANT: Thank you, John. I would like to make a couple of comments along with the discussion that we have had. Two things, John. You mentioned railroad bridges. One thing we were concerned about, our structures on the highways, as opposed to railroad bridges, is generally that railroad bridges don't get the salt that the other ones do. The second issue in that case being that better pavement policies and use of salt if you looked at those curves, as I know you have, has gone off the boat exponentially since the middle fifties. So the 100-year-old bridges sat there for many years and really didn't corrode all that much. That is certainly not a linear traverse.

FISHER: Chuck, that is not the case. You forget that from the beginning of the century up until 1960's reefer cars were used. Produce was shipped with ice and salt. Bridges were doused continuously from the beginning of the century up until the time that the last reefer car was removed from the railroad system. So they have had the same conditions that you have had for a long period of time, and they in fact have salt-contaminated bridges. So that is not the case.

PARTICIPANT: Do you think it's to the same extent, John, as far as coverage of the structure is concerned?

FISHER: When you see the corrosion problems that railroads had, you will find them similar to highway bridge conditions. The Canadian National Railroad had many main line structures across Canada with extensive corrosion in the webs and flanges. In fact, in the 1950's welded plates were added to offset the corrosion loss and that gave them more problems because cracking developed from the welds. They would have been better off to live with the corrosion.

PARTICIPANT: Okay, thank you. I wasn't aware or thinking of that. Certainly in our highway structures, the exponential use of salt began in the 50's and the 100-year-old bridges that we have had, many of them withstood them very well for the simple reason that we didn't go out and douse them with salt as we have done so much since then.

Another comment I would like to make is I think the most surprising thing that I have found about the data that we have been developing is where the corrosion attack data lie with respect to the other published data which Pedro will summarize in his most recent report and that the data in the 50 bridges, which are not all metropolitan bridges, show that in the worst case we get 6 mils per surface per year outside of a lapped area and in the average case 2 to 4. Now these include bridges that are not of the tunnel type or not the down-in-the-middle-of-Detroit type bridges as well as others. And when you look at the crevice parts, we're up to like 16 or more and when you compare that with the weather data that exists on specimens, it puts you somewhere between the heavy marine environment and saltwater immersion. So that condition that exists, in these areas, which I agree with you, this type of steel should not be used and the other kind is still going to be hard to protect. I think it is still surprising that that curve is swung over there as far as it has.

The only other comment I would like to make is that I agree with you wholeheartedly as well, that it would be much nicer if our structures had been maintained much better than they have, whether they are A588 or whether they are A36. Unfortunately, I don't think our State is very much atypical with respect to what has actually been done as compared to what should have been done. And the details that are out there that are causing many of the problems, had they been changed in design, then the maintenance would not have been necessary. So I see a need perhaps to design a little bit with the fact that in general the political situations being what they are and the money situations what they are, we do not do maintenance at anywhere near, as somebody mentioned yesterday, put them up and forgot them, has been much more the situation than maintaining them and the thruway type or toll road facilities I think have been much better maintained in that respect than the public facilities have. Thank you.

FISHER: Chuck, I think that if you have a high rate of corrosion, you are never going to have a fatigue crack. You're just going to corrode the thing away. And I think that the people who are experts in corrosion will confirm that. I think that it is time as bridge engineers that we stood up and said "Look, this is ridiculous." It takes too much money just to make emergency repairs because we allow this corrosion to get out of hand. I don't think we are doing our job. I think it is time that we made the case that we need to keep structures clean. Now I don't care if they are painted or weathered; I think it is equally a problem, and it is a disservice to the country to do otherwise.

PARTICIPANT: I'd like to presume to ask the speakers to focus on what I see is the real problem. We have almost a million bridges in this country. Very few of them are weathering steel. The purpose of this symposium is to consider weathering steel and its use for the future. It seems to me that--my understanding is that--with certain restrictions weathering steel can be expected to serve as well as the steels that we have had since the turn of the century, and if that is true, what is significant is to focus on how to identify and when to apply the restrictions and know that if the economics of a particular situation, either keep a bridge in the air or to design a new one, you can use weathering steel. If speakers could address that, I think that is what everyone here is to find out.

ALBRECHT: Yes, the NCHRP has funded a study to develop guidelines for fabrication, maintenance, design, and rehabilitation of weathering steel bridges which will address the point you have made. These guidelines are in the final stage of review and, believe me, they went through several reviews. I am the principal author. The others are Seymour Coburn, formerly with U.S. Steel; Pat Gallagher, formerly with U.S. Steel; Gary Tinklenberg, formerly with Michigan DOT; and Fateh Wattar, a colleague of mine. These guidelines will shortly be in the hands of the NCHRP. Bob Reilly, sitting right in front of you there, may want to add any comments.

REILLY: Well I guess, Neal, that that's right. But obviously those guidelines are going to espouse Pedro's view on this issue of fatigue, which I think is just not applicable. I think it is imposing something that is not needed; it's not justified on the basis of the data we have available.

PARTICIPANT: Let me go back again to fracture. When we began to have the problems with the bridges and fracture, AASHTO developed a fracture control plan which considered materials, design, fracture critical members, redundant and nonredundant, well control, and inspection from an initial aspect with not an awful lot of continued aspect once you took care of it and satisfied the aspects of a fracture control plan. If you think conceptually of what is the perhaps fatigue control plan, I think we kind of now take care of it initially in design by the fatigue curves and initial inspection and initial well control and then don't pay an awful lot of attention to it throughout the life of the bridge. We say we do, but we really don't. Whereas a fatigue control plan or the concept of a fatigue control plan really is an ongoing lifetime kind of thing and maybe if you somehow thought of formalizing that--bridge maintenance and inspection which ought to be done gets lost in the budget shuffle. Maybe if there were a fatigue control plan which had to be netted might politically bring it more to the forefront and that it is the time-dependent aspect which ought to be looked at for the life of the bridge and not just an initial design thing and just forgotten. So I might suggest that for your consideration.

ALBRECHT: Well, there are such efforts under way, and I agree with you that they are valuable. For example, the State of Maryland is computerizing the analysis of all bridges so that, for example, if someone with an overloaded truck needs to travel along a certain route, the bridges on that route can be analyzed. Another intent is to determine how much life is left on these bridges and which bridges must be taken care of first.

PARTICIPANT: Thank you, John. For those of you who don't know me, I am Roger Wilt, Manager of Construction Marketing, Bethlehem Steel Corporation. So you can say I have an axe to grind. I do have a concern and I will end up with a question, but I think the question is probably going to be directed more towards the audience and not the panel and the panel may feel free to comment.

I have a concern, I have a multifold concern, and I'll start off I hope without offending anybody upfront, because I have been a college professor and I have done research, that usually the hidden purpose of research is to continue research--it is not necessarily to come to the ultimate solution of the problem at hand. I am very concerned that in doing more and more about finer and finer parts of the problem that we are looking at three significant figure solutions to what might be as John Fisher said a gross problem. And I'll put it more bluntly than he does: wash off the guano in the pigeon nest, and you'll help the life of the bridge. I have a feeling that we are looking at corrosion mechanisms, we are trying to build extra strength into our bridges, we are trying to protect ourselves from the unimagined future problem by increasing the cost of our bridge structures. When, in reality, if we took the same amount of money, and I almost sat down after Neal gave his comments, if we took the same amount of money and diverted it towards maintaining what we have, we would probably be ahead of the game and doing the traveling public a service.

I am very, very concerned about the future outcome of these 2 days. Because if I heard it right at the beginning, it is to develop guidelines, and I'm concerned that these guidelines are going to be oriented towards doing more and more about less and less than the finer and finer details of how to get on with the job, put up a *** good bridge, keep it clean, do what the New Jersey Turnpike has already told us works, maybe avoid some geographic areas where these bridges are performing well, and do the job with some common sense rather than inordinate attention to details.

Now the questions that I have to the audience. If we go ahead to these criteria, which way do you think we should go? The common sense, the maintenance, let's do the job with what information we have, or protect ourself with more and more details? To that the panel may wish to answer.

BARSOM: I'll start with the audience only to get them to participate and then I'll come to the panel. Audience?

PARTICIPANT: Let me address a question that Pedro asked. And I think that this is the key here. With all the research that is going on, with findings that have been presented by John Fisher, by Pedro, you raised the question earlier. What is a designer to do? What is an engineer to do? Well, that is a good question and I would tell you what I would do as a designer. I heard the evidence, I heard the research, the results and then I would say that, well, it's agreed. Everybody here agreed, all the researchers, everybody said we shouldn't use weathering steel in an aggressive chloride environment. I wouldn't use it in that environment. Everybody agrees. But where it would and can be used properly in a nonaggressive environment, I would use it and then I would design my details to avoid water being splashed on my bridge that has salts in it. And then how do you do that? We have been talking for a number of months now and maybe for the last year on the Tennessee experience, which is to eliminate joints. That is a very positive way. And where you can't eliminate joints, you design your detail to provide for either the water to be taken off--like it was discussed yesterday on details that they used in the New Jersey Turnpike and some of the other States--or you protect the steel underneath those joints and under those conditions. And then if you do this and you design a clean bridge, as Roger was just saying, then you indeed have the same conditions for that weathering steel bridge as you had, fatigue-wise, for a painted steel bridge, and there should be no consideration given to reducing the fatigue detail categories. Because your fatigue strength would be the same thing. You know everybody concedes that where you're talking about existing bridges and you have corrosion damage steel, that is a totally different category. But what you are advocating is on new structures, for new designs, that an engineer should approach his design, if he is going to use weathering steel, on a basis for fatigue to lower the fatigue categories. And I think it is a negative way to approach design. I think positively you design the material the way it should be used and prevent the conditions which would present severe corrosion, and we can certainly do those, we can design details to prevent this, then there should be no difference in the fatigue category of the weathering steel bridge and the painted steel bridge, as John has said. And that is what I would do as a designer, and I think most consultants would approach it that way, or should approach it that way.

ALBRECHT: Well, I think there are two different issues there. Everyone would agree with what you have said about controlling corrosion. That is what we need to do in new bridge designs. The mistake we have made in the past was to take standard drawings for a painted bridge and use them for a weathering steel bridge without paying attention to details. I think that is the major mistake. We need proper detailing for good corrosion performance. That would help the severely corroded rolled and welded beams for which I have shown data, but not the weathering fatigue specimens which were ideally and boldly exposed and still had losses in fatigue strength. So just eliminating the severe corrosion doesn't mean that there is no fatigue problem any more.

PARTICIPANT: [inaudible]

PARTICIPANT: You're telling me that your data indicates that weathering steel exposed to the air in the normal conditions has a lower fatigue strength than painted steel. Is that what you are saying?

ALBRECHT: That is correct. I showed the summary of 1,565 tests on which I have based my recommendations.

PARTICIPANT: John is going to answer that question. I don't think he agrees with that. From what I have seen, the data presented, I look at John's curves and I look at your curves, and I don't see where we are necessarily talking about the same thing. Maybe I don't fully understand it. But I don't know if we are putting things on the same basis and superimposing the data, and from there I'll let John pick it up from there.

FISHER: Well, as I showed in the first three slides, there is no degradation in fatigue resistance. The notch factors that Pedro uses for category C and D details are based on a mean. I think a mean has no relevance. As I showed you with his own data, there are no data that demonstrate that the lower bound for weather steel tests are any different than unweathered steel details. The means are different and he used the means to compute the fatigue-notch reduction factor. We do not use the mean. We haven't been using the mean since the AASHTO code was developed in 1974. Means were used before that time. Worldwide, we were no longer using mean data. So I think that is one issue but certainly the real bridge structures are not being exposed, if we use them in the proper places, to continuous sodium chloride solutions. So, I see no evidence that there are any data to justify, with properly sited bridges, an acceleration due to corrosion fatigue.

ALBRECHT: I must say I am a bit surprised about the statement that we are not using means. John is a member of the Load Resistance Factor Design (LRFD) panel that wrote the latest AISC Specifications. The LRFD equation has in the numerator the mean of the resistance and the mean of the load, and in the denominator it has the standard deviations. Of course, we use means. The mean minus 2 standard deviations is a simplified approach that has served us well in the past. Of course, we use means. We use means worldwide.

Coming back to your point, it is exactly right. We test specimens as fabricated, with no weathering at all, and get a certain mean S-N line. We take specimens that have been weathered ideally for a number of years, bring them into the lab, test them in air, and get a lower mean S-N line. There is a loss in life due to weathering between the two sets. I will be glad to send you the U.S. Steel data by Blake and Barsom that shows the same thing. The U.S. Steel data shows it just like the Japanese data, like anyone else's data.

BARSOM: Well, I'm not going to comment on that, but I disagree with that statement. But, I would like to make a statement to show you where confusion comes in. I think what John was talking about is that we do not use means on fatigue analysis, and I would have to agree that the design community has stopped using mean stress range as a criteria. The AASHTO design curves are based on a weakest-link kind of an approach and tolerate a certain amount of imperfection for a given category. So, for fatigue we do not use mean, for other applications we use mean. And if we stick to the fatigue and corrosion fatigue, I think maybe we can clarify things a little better. There was another question, Neal, back there again.

PARTICIPANT: I would like to point out that engineering is applied science and not pure science, and one of the things that we wrestle with as designers is that the legal profession sees itself as the protector of the public as well and that the design guidelines that will come forth will control design because every engineer dealing with design in the real world has to realize that the lawyers read what you publish. And if it is published as a matter of information, that is worthy of consideration, that is one thing. And if it is published as a matter of recommended criteria, you are imposing something on the design of bridges which can have a major effect on the cost to the country on the effect, and I, listening to you and Professor Fisher speaking, I thought it might be worth mentioning that we do have to think about what the lawyers have to say about design as well.

ALBRECHT: I worked 7 years for engineering firms designing structures before I went back to school to get a Ph.D. degree, so I know what the design profession is. In my last position, I was Assistant to the Chief Civil Engineer of the Bechtel Corporation office in Montreal. On the second item, regarding the cost, I agree with you that reducing allowable stress ranges will increase the cost of the bridges to some degree. But I can't, in good judgment, disregard the data for the sake of keeping the bridge cost down. I can also tell you, for example, that we have developed, at the University of Maryland, details that increase the life of the cover plate detail from category E to category B. That is a sixteen-fold increase in fatigue life. Ohio is building five bridges with those details. I don't quite understand why, on one hand, there is such a reluctance to address the weathering steel issue and, on the other hand, when something really good comes up, few seem interested in it. I think we must be careful to separate the technical issues from the economic issues. I'm not here to decide economic issues.

I fully agree that enough research has been done on fatigue of weathering steel. I'd do any research you want until I retire, but there is no need for that. The data are there, if the profession wants to make a decision. And, even if the data were published in the literature only as a body of information, you can be sure a lawyer will use it.

BARSOM: Any other questions? I'll play the devil's advocate and pursue some of the questions. I will throw it to the panel first and then to the audience.

Suppose we go with the so-called seven commandments that were presented by New Jersey Turnpike, to apply each one of them and make cost in terms of maintenance and the like for their experience has been excellent; their bridges are 24, 25 years old, some of them, with severe details, with very heavy traffic they have not experienced any cracking. Therefore, it appears to say, at least up to now, that they do not have a problem with fatigue or corrosion fatigue using A588 in the bare application just like they would use an A588 painted or any other steel painted. Would we, if we applied the rules that they have come up with or were given--the seven commandments--combined with proper site selection, would we solve any questions on the fatigue or the corrosion fatigue problem?

I will start with Pedro to answer that and then John and then to the audience. Before the answer is made, I want to emphasize to you that one of my responsibilities at U.S. Steel, at one time I had been part of the corrosion research group for U.S. Steel, and part of the effort was to send people out, at the request of bridge engineers, to the site where they want to build a bridge and find out if their application weathering steel can be used or cannot be used. As a matter of fact, we are right now working with Delaware on something like that, and I personally informed the State of Florida, when they were considering the Sunshine Skyway Bridge, we put test panels, test wicks, on the bridge that was hit by the ship and I personally told them not to use

A588 for that application. So I think the industry is not anxious to jump in and use steel where it shouldn't be used, and I hope and I know that you continue to do that. With that comment I would like to know if Pedro, John, and the audience have any comments about using the seven commandments and proper location would solve the problem of fatigue corrosion fatigue?

ALBRECHT: I'm glad the New Jersey Turnpike is using what they call the seven commandments. Many more "commandments" can be found in the brochures of the steel producers and in the guideline report that we have prepared for the NCHRP. Eleven of the 12 chapters in the NCHRP report deal with the design, maintenance, construction, and rehabilitation of weathering steel bridges. Only one chapter deals with fatigue.

Remember my last plot comparing calculated and recommended stress ranges in bridges. There was a green line, an orange line, and at the bottom a red line. Observing the guidelines eliminates the need for the red line, and that is why I have not included the red line in my recommendations. The way to take care of the red line is with proper detailing, not with reduced allowable stress ranges. But, you cannot take care of the green and orange lines with proper detailing alone.

FISHER: As I have indicated, I disagree with that position. I think we both look at the same data and I've reviewed his data with you and I do not believe that the data justify the position he takes. I think the data demonstrate that the weathered details that we're concerned about--the category C, D, E, and E' details--show no evidence that there is a degrading of their fatigue resistance. I think if one follows what the New Jersey Turnpike has done, in keeping the structures clean and with proper siting, that we certainly will not have any more fatigue cracking possibilities with weathering steel structures than we have with any other steel structure. I think it is just a matter of the interpretation of the data, and I just disagree with the application that is being suggested with the data. It is a matter of how that should be applied. I just do not see the justification for it.

PARTICIPANT: Unfortunately, I missed the complete details of the New Jersey seven commandments. So my question is regarding a particular member of the total presence here--how can I get a hold of the complete details of the seven commandments? Maybe there is somebody here who can help us, give us some contact number?

BARSOM: You are welcome and there is somebody volunteering in the far corner: Roger Wildt in the corner?

WILDT: Bethlehem will be pleased to make them available to anyone.

BARSOM: Any other questions? Any other comments?

PARTICIPANT: In case you don't know me, my name is Chao Hu; I'm the State Police Engineer for Delaware Department of Transportation. We have no objection of using A588 steel, but we don't want to do it wrong and we hold a lot of selection of site and seven things we should avoid. We try to do our best to do that; however, for you who distribute those seven recommendations, could you make it more specific? For example, we know the joint is not supposed to leak, but it is going to leak. Do we really need a leak-free joint to use A588 steel? If that is the case, we may not be able to use it because we cannot guarantee a joint that is not leaking. As to the tunnel effect, could you tell us how high the beam has to be above the underpass--17 ft or 14 1/2 ft? Or what kind of widths are we talking about--does it require retaining wall to have a tunnel effect, or is a regular 2-to-1 slope going to avoid a tunnel effect? As long as we know more about those details, then we will feel more comfortable in using A588 steel. Until then, we will still be worried.

BARSOM: I think there are; these seven commandments can be made more specific and more useful to the design engineer and should be made more specific, more so than what was presented by Mr. Noel. I think that can be done. I think there is enough information to address these points. Unfortunately, the presentation didn't go through that kind of detail. There is another question back there?

PARTICIPANT: I just had a question for the panel, and I wrote it down so I wouldn't mistake it when I read it. You both seem to agree that corrosion notches reduce fatigue strength, assuming that corrosion is stopped by cleaning and painting. Should the structure be analyzed based on the corrosion notches using a reduced fatigue category? If so, what and what size notches?

FISHER: Yes, I think that if you have corrosion notching, you are going to have to reduce the resistance because if you paint it to stop the corrosion, then you keep the notch, and it will reduce the strength. Now the problem is in how to get an accurate model of that. You could possibly use Peterson to get a notch factor. It may be very difficult. The other option would be to do some judicious grinding and smooth the notched region. You could afford, I think, the loss of the material. In fact, I think that is an area fruitful for research. I think peening might have some great advantages to corrosion notched areas.

ALBRECHT: In the report on the trolley bridge beams we have just written for the State of Maryland, we have recommended a procedure for determining the fatigue strength of a corroded and pitted beam as a function of net section loss and pit depth. You can follow the method we are suggesting. That should answer your question.

Coming back to your comment on the tunnel effect and vertical clearance, in the NCHRP guidelines we cited the British recommendation for a minimum vertical clearance of 24 feet. But we are not recommending that value. We are saying that proper clearance is important for good air circulation, so that the wind can clear away most of that spray.

You mentioned also the leaking joints. I was surprised, coming from Europe, to see how many simple-span bridges there are in this country. Several years ago, I asked Warren Alexander about that. He told me that in the 1950's engineers were uncomfortable designing indeterminate structures so they would rather have determinate structures. That was the explanation he gave. I think we are paying, whatever the reason may be, we are paying for that today, both in terms of the weathering steel and painted steel bridges. Certainly the fewer joints, the better. I was kind of disappointed that Clellon Loveall did not make a stronger point about jointless bridges in his talk because his State just about pioneered their application. That is what we should do, irrespective of whether it is a weathering steel bridge, a concrete bridge, or a regular steel bridge. We are recommending jointless bridges in the guidelines. Do away with the joints as much as possible.

BARSOM: The last paper I read that was from England, I think they recommend something on the order of between 17 and 21 ft.

Pedro may have another paper where someone was 24, but that is the number that I recall. Seventeen, depending on the conditions and the like, up to 21 ft.

Any burning questions or comments? Clellon, do you want to defend yourself? Well, I want to thank Pedro and John for a very interesting morning. I think that you have to agree you got your money's worth for this symposium or forum and maybe you actually owe Bob Nickerson a few bucks. So if you feel like that, you can give it to him. At this point, I will turn it over to Bob to finish the morning session.

NICKERSON: I think, as John said, you got your money's worth on this presentation. We spent considerably more time than we had intended, but I think it was worthwhile to air the issue. I think the real issue--and we are going to get into this a little bit more this afternoon--how do we get the message to the designers that aren't here, the messages that we have heard? Not only related to the fatigue issue, but related to the proper detailing and so forth. And I'm glad to see in the audience Rita Robinson from Civil Engineering magazine who is taking quite extensive notes--I'm not sure how many issues she is going to have to take to devote all of the notes to.

We have got to find a mechanism to get these proper details to the designers that have a tendency not to come to forums such as this. Too many of them don't even seem to read professional literature, and we look at designs continuously day in and day out; they continue to have details that we know will not operate or work successfully. Yet when we point that out, we are continually in a loggerhead position, because it seems that many of them do not have erasers. So I think that we have to come up with a mechanism to do that. Maybe that is really the issue to address with this forum.

SELECTION OF WEATHERING STEEL FOR A GIVEN LOCATION

Presentation by W. Mathay

Highlights:

Key points covered by Mr. Mathay included:

- o Weathering steels are not appropriate for use in marine environments and in areas where they would be subject to heavy pollution or prolonged exposure to moisture.

- o Sites should be carefully studied in terms of their weather and environmental factors before selecting weathering steel for a given location.

- o When locating a structure in a corrosive environment, include design features that will mitigate or offset these weather/environmental effects.

Summary:

Successful performance of weathering steel ultimately depends on corrosion occurring at a controlled rate over the life of the structure. This corrosion performance can differ significantly based on environmental differences such as degree and type of atmospheric pollution, extent of rainfall, level of humidity, and--in coastal environments--the amount of airborne salinity. These weather and environmental factors should be carefully considered via a thorough site inspection before deciding to use weathering steel in a given location.

Weather Factors

Weather factors affecting weathering steel performance include:

- o Moisture--rain or dew forms a moisture film on steel surfaces which leads to corrosion; the length of time steel stays wet determines corrosion extent. Dew stays corrosive for hours or days; rain can wash away contaminants.

- o Wind--this factor is significant because the patina forms through alternate drying/wetting. Also, it is important to know the direction of the wind, since it can blow contamination materials.

- o Sunlight--by drying the steel, sunlight stops corrosion and helps in forming the patina.

Collect as much data as possible regarding the proposed site's humidity, rainfall, wind direction, and exposure to sunlight.

Environmental Factors

Service and maintenance lives are appreciably affected by the environment. There are certain general guidelines regarding the use of weathering steel in various environments. For instance, it takes longer for a film to form in rural rather than industrial atmospheres; also beach sites are the most corrosive. The critical factors here are time of wetness and chloride contamination.

Coastal Environment. Tests have shown the extremely corrosive effects of beach sites on iron, stainless steel, and aluminum. Although weathering steel is more resistant, it too is affected by high chloride concentrations. Note that:

- o Salt can be blown for great distances. Tall structures can deflect this salt, keeping it off the proposed structure.

- o Avoid flat coastal locations where the wind can carry salt.

- o Weathering steel is not suitable where recurrent wetting by saltspray occurs.

Overall, however, there are no specific rules for weathering steel in coastal use, since there are too many variables to be considered. Judge each site individually, and conduct tests where possible.

Chemically Polluted Environment. Although not nearly as aggressive as rain in terms of its corrosivity, acid rain can, with continued exposure, have harmful corrosive effects on weathering steel. Determine the prevailing wind direction in the location. Also be aware of fertilizers used in the area, as certain ones can adversely affect the performance of weathering steel.

Artificial Environments. This category comprises such situations as continued exposure to deicing effects and the tunnel effect. Both of these cause serious corrosion to weathering steel. The key here is to carefully review the proposed design to avoid circumstances where the design works with the environment to corrode the structure. An example of this is the potentially severe corrosion resulting from joints and deicing salts.

Site Inspection

Conduct a detailed site inspection. Examine the topography of the area, the location and heights of buildings, the age and condition of metal objects. This information provides valuable clues as to the expected performance of the proposed structure, and can be obtained from area highway departments, owners, and observation. Compare any specific data received with ASTM atmospheric corrosion data.

Note also the prevailing wind direction to determine if localized chemical pollution will be coming toward the structure. If the location is in a coastal environment, note the locations and elevations of hills, building concentrations, and large trees--these can all serve to block salt air and keep it away from the structure.

Finally, if there is time, perform atmospheric tests using sample panels and chemical analyses for chlorides.

JOINTLESS STEEL BRIDGES

Presentation by E.P. Wasserman

[Edward P. Wasserman is the Civil Engineering Director, Structures Division, with the Tennessee Highway Department. He has been with the Department for more than 20 years, and has a B.E. in civil engineering from Vanderbilt University.]

<u>Highlights</u>:

Key points covered by Mr. Wasserman included:

- o Joints contribute to numerous problems in terms of bridge design and maintenance.

- o Continuous jointless bridges successfully resolve the problems raised by joints. The key to their use is careful, innovative design aimed at providing the movement otherwise supplied by joints.

<u>Summary</u>:

Tennessee is a proponent of continuous, jointless bridges, and utilizes their design wherever possible. Its longest jointless bridges in concrete and steel are 927 ft (283 m) and 416 ft (127 m), respectively. The only exceptions to Tennessee's jointless "policy" are cases where the abutment is keyed to rock or is too massive to reliably move the required amount.

The problems associated with joints include the following:

- o Expensive;
- o Difficult to install;
- o Periodically need to be raised for paving;
- o Generally malfunction over time;
- o Leak, leading to substructure damage;
- o Contribute to misalignment of severely skewed bridges; and
- o Allow expansion bearings to malfunction.

To avoid these problems, Tennessee engineers develop alternative designs that provide the needed movement by some method other than an expansion joint.

Tennessee has eight weathering steel bridges; only one of these has joints. When this bridge was originally built in 1980, there were significant problems with joint leakage (severe uplift) at abutment 2. The solution was to make the abutment integral and use the weight of the abutment as a counterforce. The bridge no longer has a leakage problem, and shows no signs of deterioration.

Mr. Wasserman showed a series of slides to illustrate the various design techniques and methods employed by Tennessee in building its jointless bridges. One set of slides showed an example of a rehabilitated bridge which had been successfully converted as a fully continuous unit with no joints.

In its designs, Tennessee also consciously avoids the use of scuppers to the greatest degree possible.

MAINTENANCE SESSION--INTRODUCTION

Moderated by J.W. Peart

[John W. Peart is a research chemist in the Materials Division, Office of Engineering and Highway Operations Research and Development, Federal Highway Administration. He is involved with research dealing with cost-effective environmentally acceptable coatings and corrosion control methods for steel highway structures. Before joining FHWA, Mr. Peart spent 12 years as R&D manager of the National Shipbuilding Research Program in the area of surface preparation and coatings.]

Summary:

Mr. Peart targeted areas for discussion in developing cost-effective maintenance procedures to extend the life of weathering steel bridges.

The first of these areas was damage assessment. Issues associated with this topic include how to determine a bridge's corrosion "status" through inspection; the comparative ease or difficulty of crack detection; quantitative assessment of pits (how deep are they?). The key is to develop standards for assessing A588 bridge conditions and corrosion products.

A separate set of issues is attached to the area of blasting and painting, including alternative coatings, problems encountered, discoloration and/or moisture after blasting, the feasibility of washing bridges down, and strategies for mitigating the corrosion rate.

MAINTENANCE OF WEATHERING STEEL--VIRGINIA

Presentation by T. Neal

[Thomas Neal is Chief Chemist at the Virginia Department of Transportation. He has worked for the department for the past 20 years.]

Highlights:

Key points covered by Mr. Neal included:

- Virginia requires that all new A588 bridges be coated as a preventive maintenance procedure.

- Little maintenance work has been done with A588 bridges in the State.

Summary:

The Virginia Department of Transportation requires that A588 steel be painted using an inorganic zinc-rich paint primer and intermediate and or topcoat(s). Areas to be painted are 5 ft (2 m) from either side of the expansion joints, the fascias of the exterior beams, and all bearings. This specification helps to forestall A588 maintenance needs, given the concurrent demands on the State's maintenance budget and resources.

All new A588 bridges are painted as per this specification; additionally, six structures with no obvious corrosion problems were painted after erection as a precaution. To date, only a single weathering steel bridge has received any remedial painting. This is due more to budgetary constraints than to the steel's performance; there are many examples of poorly located and/or designed A588 steel bridges in the State which require some maintenance.

One problem in maintaining the A588 bridges derives from their appearance. Because weathering steel bridges always look rusted, there is less sense of urgency in maintaining them. They promote what Mr. Neal referred to as a "false sense of security," leading to situations where the maintenance problem gets out of hand without being resolved.

Points brought out during the presentation's question and answer session included:

- o Virginia uses only VOC-complying coatings.
- o No bridges have been washed to Mr. Neal's knowledge.
- o Bridge inspectors are provided with criteria (regarding section loss, etc.) to determine problems with weathering steel.
- o The State does not keep separate cost figures on A588 maintenance expenditures.

MAINTENANCE OF WEATHERING STEEL--USER'S EXPERIENCE

Presentation by G.L. Tinklenberg

[Gary L. Tinklenberg is with Corrosion Control Consultants and Labs. He worked for the Michigan Department of Transportation for several years.]

Highlights:

Mr. Tinklenberg's presentation focused on two major topics:

- o A detailed discussion of Michigan's history with weathering steel bridges; and
- o An in-depth description of a Michigan study on weathering steel maintenance considerations.

Summary:

Michigan's interest in weathering steel for bridge construction passed through several distinct stages, beginning with moderate curiosity in the A588 concept, increasing to uncritical enthusiasm for--and exclusive use of--the product, and dropping down as problems first became apparent and were gradually recognized as serious issues.

Over time, it became apparent that Michigan's designers didn't completely understand where and how to use weathering steel; nor were these limitations quantifiably recognized and defined anywhere else. In the absence of guidelines for use, Michigan had to discover workable maintenance options.

In 1982, Michigan painted its first weathering steel bridge. The bridge had been performing well; it was painted to stop thermal expansion caused by differential heating on the outside fascia beam. Between 1981 and 1983, the State began a program of painting problem joints.

A curious phenomenon--the "green mold phenomenon"--became apparent as bridges were blasted prepatory to joint painting. Immediately after the blast, the

surface looked fine; a few minutes later, however, a greenish substance appeared on the surface. This odd appearance had little or no effect on the coatings.

At first, Michigan used inorganic zincs in its painting program (only organics are used today). Painting was performed 5 feet out from the joint. However, Mr. Tinkleberg pointed out that this painting practice does not resolve the leaking situation: the water simply increases momentum as it runs down the paint--essentially, the problem has moved down from the web to the bottom flange. To eliminate--rather than just move--the problem, the flange must also be painted.

Another discovery was that blasting weathering steel does not completely clean out the pitting. Therefore, underneath the inorganic paint, there was a small amount of readily visible rust. These rust cells interacted with the water that was allowed through by the extremely porous inorganic paints. The rust consequently multiplied; this was one of the main reasons for stopping usage of inorganic paints.

Mr. Tinklenberg next discussed in depth a long-term study conducted in Michigan to identify the appropriate paints and painting system for dealing with weathering steel corrosion.

One of the early conclusions of this study was that there was no way to simulate weathering steel in the laboratory in order to test the paints. Consequently, testing was done on actual bridge samples.

Based on input from paint companies and laboratory testing, it was determined that organic zinc-rich paint gave the best results; alkyds performed the worst.

The next step was to determine the appropriate surface preparation. Tests of various gradient blast processes were conducted on weathering steel hanger plate panels. The recommendation was to blast to near-white. Washing yielded very little improvement in performance.

Next, testing began to determine the best coating system. A procedure using organic zinc-rich, epoxy intermediate, and urethane top coats was adopted. Mr. Tinklenberg advocated using a system based on theoretical values to obtain appropriate coverage proportions for use with weathering steel. These proportions are: use one-quarter of the theoretical value to determine the primer coat; one-third for the intermediate; and one-half for the top coat. With A36 steel, use one-half of the theoretical value for all three coats.

PANEL DISCUSSION OF MAINTENANCE

Moderated by J.W. Peart

Highlights:

Panel participants were George Meyer, Bernard Appleman, Gary Tinklenberg, and Al Dunn. Key points covered by the panel are included in the Maintenance Session Summary at the end of Section III of this report.

Summary:

The session opened with a series of questions directed to George Meyer, a consulting engineer for the New Jersey Turnpike Authority.

Question: Have you painted or partially painted any of the Turnpike's A588 bridges?

Painting has only been performed on one bridge in the past; this worked well. Presently, a second bridge is being painted following the same procedure: near-white blasting in the area of the joint and application of inorganic zinc paint; no solvent wash was performed. There are many other bridges that this procedure should be applied to, as there are hundreds of joints, dating from the 1969 widening project, which use compression sealers that were placed between A588 joint armor; this armor is now badly scaled.

Question: Does the Turnpike have rust problems equal to that in Michigan?

Yes, the Turnpike has very severe laminar corrosion to the point of web holes. The problem is due to deicing salts: "the Turnpike puts down salt in the winter like they're paving with it." There has been metal lost to webs. There are many areas on major bridges that have a metal loss of 1/4 inch at the top of the lower flanges; these are all restricted to joint areas.

Question: Will any of the new construction be painted?

There are no plans to do so. The plan is to go with strip and compressed seals with galvanized metal work. The Authority does not seem concerned with staining, although drip bars have been tried to stop staining.

Question: How is the difference between Michigan and the Turnpike--the Nation's two biggest users of weathering steel--explained in terms of their very different approaches to the use of A588?
The key is that one is a State and one is a private tollroad. The Turnpike can afford to do any necessary repair, and can spend whatever it costs to resolve any problems encountered.

Question: Are there any estimates as to how much it's going to cost to maintain weathering steel? Isn't the Turnpike going to be replacing a lot of these anyway?
Some of the Turnpike's A588 bridges are nearly 20 years old. If the widening project gets off the ground, they're going to be completely replaced, thereby eliminating the need for a maintenance budget. Under the project, 140 bridges are going to be completely removed; most of these are A588.

Mr. Meyer went on to describe a maintenance project he is currently working on for the Turnpike involving a 16-year-old two-girder and floor beam arrangement similar to the Myannis Bridge in Connecticut. Mr. Meyer also noted that he has not conducted any stress tests on Turnpike bridge hangers.

At this point, attention shifted to Michigan, and Mr. Tinklenberg was asked about the ratio of bad to good weathering steel bridges in the State. He replied that there were about 19 or 20 weathering steel structures in the Upper Peninsula. Of these, 10--out of the State's total 600 A588 bridges--are considered to be in good condition (i.e., clearly not requiring maintenance for many years) and to be good and proper use of weathering steel. However, it is not clear why 10 are good, and the neighboring ones are not.

Question: Do you have any jointless A588 bridges?
Mr. Meyer said that the Turnpike has many continuous span bridges, and under the widening project, will be trying to build nothing but continuous span bridges. He pointed out, however, that with continuous span bridges, there is a sequence of pours in the deck--this is another source of leakage to be considered.

Mr. Tinklenberg said that Michigan had none, but is trying to do them under traffic. The State has got a lot of fixed pins and hangers: the theory was that all joints leak, even construction joints, so these were moved off the piers and non-movable pin and hangers were used to reduce the length of the girder. This costs $4,000 per pin and hanger, and there are 30 pins and hangers per bridge.

Mr. Meyer was next asked about replacing pins and hangers with stainless steel; he detailed how this process was performed. A participant pointed out that since it hadn't been tested, the stainless steel might result in the same corrosion problem as had the A588.

A question was aimed at Al Dunn regarding the use of wash solutions. Mr. Dunn replied that it now seems as if the process may have to be done only every 5 years. If it has to be done more frequently, it will not be cost effective to do it. Louisiana is waiting for test results to be completed.

Next, Bernard Appleman was asked about paint coating systems.

Question: According to your lab tests, what coating systems work well on A588?
Seven coatings have been identified as satisfactory to varying degrees, and are being tested in the field over different surface treatments.

The tests were conducted using actual plates which were pre-corroded to simulate severe pitting. The panels were tested for high and low chloride contamination. Tests were then made regarding different cleaning methods, including wet abrasive blasting, dry abrasive blasting, dusting, hand/power tool cleaning and chemical treatment (this last didn't work well and was not included in later testing).

The seven coatings used are: a control paint (lead containing oil alkyd); inorganic zinc vinyl; organic zinc with epoxy/urethane; a two-coat epoxy system; two epoxy mastic systems; a second organic zinc/epoxy system; and a thermal spray zinc. These were found to give reasonable performance in the lab and did sufficiently well over the surfaces to be selected for field testing. These have been out about a year. Some panels are already showing some signs of failure already; some showed signs of degradation within 6 weeks. The panels are being exposed at three bridge sites in Pennsylvania, Louisiana, and Michigan; as well as at Curie Beach, North Carolina; and an industrial site in Pittsburgh. All told, the study represents a lot of data on several different coating systems using different surface preparation methods, and exposed in five different environments. The bulk of the analysis is not done yet, but preliminary analysis regarding the cleaning shows that wet blasting does a better job in removing chlorides (even for pitted steel); it is not clear if this will translate to a longer paint life, however.

Since the test started, a couple of new cleaning methods have been devised such as SSPC SP11 (power tool cleaning to bare metal); SSPC is performing a new study for FHWA using these methods on the threshold limits of chloride contamination--how much can remain on the surface before painting.

Mr. Tinklenberg remarked that he had just completed a large study on the concentration of rust inhibitors. The study had shown that there may be major problem with every system tested. He also said that the blasting, wetting, and blasting cycle is not effective because of its expense. By only blasting once, however, there would be a tremendous cost savings, particularly in A36 bridges where wet blasting is used to control the release of lead into the atmosphere.

A participant noted to Mr. Tinklenberg that sulfate and chloride levels in tap water will affect the performance of rust inhibitors.

Mr. Appleman was asked if his tests indicate what happens to metal spray on A588. He replied that in the lab, metal spray works quite well. Mr.

Tinklenberg remarked that part of the problem with metal spray is having to maintain the seal coat. Mr. Appleman said the 5-year study would show if there's a problem with the seal.

Mr. Appleman next raised the issue of washing.

Question: Has anyone done washing on a long-term basis as an alternative method for protecting A588?

The ensuing discussion brought out the fact that it's been done in western Pennsylvania and Franklin County, Ohio. Mr. Appleman asked if the technique was something that should be recommended for wider use, stating that if the chlorides--which tend to retain moisture--could be removed, many of Michigan's (for example) worst situations could be avoided.

Mr. Tinklenberg pointed out that simple washing or swabbing does not give enough flow over salt surface to remove contaminant buildup. He said high pressure water is great for removing debris, but it takes long periods of flow for the salt to migrate, and consequently for any kind of washing to be effective.

Mr. Appleman added that if there's any kind of scale built up with chloride in it, it cannot be removed unless the water is ultra high pressure, which is not cost effective. In this case, other surface preparation methods, such as blast cleaning, need to be considered to remove the scale. Steam cleaning alone could be effective, but it would depend on how often it was used. Steam cleaning is better used in combination with blast cleaning to remove build-up which might otherwise become embedded during the blast.

This led to the issue of blasting. Mr. Tinklenberg was asked if there was any benefit in blasting mill scale 5 or 10 years after construction; he responded that he hadn't tested for this, and that it might not be cost effective.

Question: Are there any quantifiable blast rates for A588 compared with A36 steels?
Mr. Appleman said that development of these might be something to recommend to the forum; figures exist for 1984 but not beyond that.

Mr. Tinklenberg said that Michigan had done 12 or 15 such blasting projects when he left. The cost of doing near-white blast with full traffic control on A36 steel was $215 - $230/ton; and between $325 to $330 on weathering steel.

The participants then discussed production rates and costs, noting that A36 has higher production rates. In discussing psi's, Mr. Tinklenberg noted that outstanding effects could be achieved at 250 psi with a strong abrasive; he considered 125 psi to be low pressure, while Mr. Dunn thought 90 psi was adequate. The issue was brought up that contractors are using different aggregates as well as different psi's.

The topic of partial versus complete painting of bridges was raised next. Mr. Tinklenberg pointed out that the cost of painting only the necessary components--joint areas, bottom flanges, top of bottom flanges, and 8 inches up the width--would cost almost as much as complete painting. Further, he said, most cracks are detected by the general public; without paint on A588, the public could not discern and report these cracks. So as to retain the input of the public, Michigan paints its weathering steel.

Question: Is there value to painting just around joints?
Participants pointed out that Virginia and the New Jersey Turnpike follow this procedure, and that it's cheaper to do so, particularly because not as much is expended in meeting environmental considerations. Despite this, however, the participants could not conclude that it was better to do partial painting rather than complete coverage--too many unknowns are involved. Mr. Appleman highlighted this issue of decision making by asking the panel if it is better to paint the joints of five bridges or to paint one bridge completely; the former procedure might enable a department to buy time until more is known regarding corrosion.

The next issue considered was how to conduct a corrosion inspection on A588 bridges. One problem is that many descriptions exist for the same sets of phenomena (e.g., dimpled, pitted, laminar rust, flaky rust, granular, grit, sandy). The participants agreed that standard terms and definitions need to be determined and matched with visual descriptions. It was suggested that such descriptions might already exist through NACE, AISI, Michigan, New Jersey, etc. Perhaps a national agency should sponsor development of a picture/text guide with photographs either professionally commissioned or supplied by highway departments.

Related to the issue of standardizing terminology is the development of standardized procedure: what to do on the first day of inspection, for followup, equipment needs, etc.

The participants also said that they wanted quantifiable standards for determining the severity of problems. This guidance should be given to bridge inspectors to assist them in their reporting responsibility. Mr. Tinklenberg contributed an illuminating story: in Michigan, weathering steel bridges were rated on a scale of 1 to 10; consistently, all A588 bridges were according 9's. When questioned, the bridge inspector replied that the bridges were made of weathering steel, which is supposed to rust; since the bridges were in fact rusting, they merited a nine rating.

The point was made that frequently no action is taken on inspectors' reports. The delineation of standards could make reports carry more weight with administrators and decision makers.

Mr. Appleman suggested that quantitative testing techniques for determining chloride deposits and time of wetness be implemented during bridge inspections. This idea was rejected as impractical and not cost or time effective.

In summarizing and reviewing their consensus decisions, the participants agreed that standards were of the utmost priority.

For a variety of reasons, it was determined that no specific guidance could be given as to when to paint partially versus painting the whole bridge.

Partial painting of new construction (redundant painting) was suggested as a good method of preventive maintenance, especially if it is assumed that all joints are eventually going to leak. The recommendation was made to paint a length that was twice the depth of the girder. Mr. Tinklenberg noted that a nice consequence of painting near joints on new construction is that the mill scale will be removed in the process.

While it was felt that better data are needed on relative production rates for blast cleaning on weathering steel versus carbon steel (and it was agreed that blasting is nearly always necessary to determine the pitting and corrosion), no one felt it was something to fund extensively.

Mr. Appleman suggested the establishment of a clearinghouse on available A588 painting and maintenance information. This clearinghouse should be centrally maintained to serve as an industry resource for "comparing notes" on the issues. Mr. Appleman volunteered using SSPC as the repository of this information as part of its ongoing contract with FHWA. Mr. Dunn suggested that a standard form be designed to capture the desired information so as to keep variables to a minimum and ensure uniformity of response. Suggested data elements included agency, date painted, type of paint, type of surface preparation, age of the bridge, name of the chief engineer--no list of success rate, no comparisons. Mr. Neal said that such data are probably not readily available.

Mr. Appleman also suggested that the group meet to pool its knowledge every 2 or 3 years; and that an attempt be made to obtain copies of published reports.

A final suggestion made by the participants was to evaluate the effectiveness of drip bars.

SUMMARY--DESIGN

Presentation by R.L. Nickerson

The results of the design session comprise three major points.

1. Location-- Weathering steel is not adequate in all environments, and design engineers need to be careful in selecting locations for its use. Chlorides should be measured, if necessary, to determine if the site is adequate. Vegetation growth needs to be considered to ensure that it doesn't trap humidity, etc. There are places where weathering steel is simply not appropriate.

2. Detailing--Engineers must use proper details. The most obvious of these are related to the joint problem. While it might be true that there isn't a joint that won't leak, joints can be properly detailed so as to control that water and keep it from running down the flanges of the girders.

   Other design details to be aware of include those that won't trap moisture, that won't allow debris to build up and that traps moisture. These concerns are equally important whether the bridge is painted or not; also, they are just as applicable to concrete bridges.

3. Value of Good Maintenance-- One such example of good maintenance is periodically washing down bridges. To illustrate the significance of good maintenance, Mr. Nickerson discussed a welded highway truss bridge in New Jersey which is believed to be the first all-welded highway bridge in the U.S. The bridge would not meet any current welding standards; its weld profiles and categories of details are such that it would have to be rejected. Yet, almost 50 years after its construction, carrying very heavy traffic, the bridge looks as if it were brand new.

SUMMARY--MAINTENANCE

Presentation by J.W. Peart

Weathering steel bridges are not all situated in perfect sites. Some of them are associated with marine areas; some with high humidity areas; others are in urban areas with much salting. The maintenance engineer is going to have to provide methods and techniques to preserve these bridges--regardless of their location--and to increase their useful life.

1. Uniform Inspection Standard-- The most significant recommendation made in the maintenance session was to develop a standardized method for inspecting weathering steel bridges. This standard needs to address blasting off corrosion, and defining the degree of corrosion and how to assess the degree of corrosion. The goal of this standard should be to ensure that all engineers can recognize a situation as exemplifying a given condition, and be able to make a decision on the appropriate repair or maintenance options for that condition. Given the array of factors involved in maintenance decision making, however, the engineer will have to make blasting, painting, etc., decisions on a case-by-case basis.

2. Redundancy of Painting-- In new construction of A588 steel bridges, it would be helpful to paint the joint areas to at least twice the depth of the girder. This redundancy should answer to problems with joint sealing.

3. Drip Bars-- This interesting suggestion involves the adding drip bars, not to keep the rust stain from running, but rather--if there is a leak in the joint--to restrict the flow of the saltwater from the rest of the bridge girder.

4. Painting-- A number of questions exist as to the best coating systems to use for weathering steel bridges in terms of paint cost effectiveness and durability. At this time, several studies are being performed on this subject: an FHWA report on the topic is due

by the end of the year; preliminary evaluation work has been done by SSPC; and panels are out on bridges now. As soon as these data become available, they will be published and distributed.

5. <u>Information Clearinghouse</u>--SSPC has offered to serve as an information clearinghouse, collecting and dispensing information regarding what bridges have been painted--partially or completely--what systems have been used, and how these are performing. To this end, SSPC may be requesting such data from highway departments to facilitate the exchange of weathering steel bridge maintenance information.

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