

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
REPORT

333

GUIDELINES FOR EVALUATING CORROSION EFFECTS IN EXISTING STEEL BRIDGES

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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FOREWORD

By Staff
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This report contains the findings of a study that was performed to develop practical guidelines that can be used to assess the effects of corrosion on structural details in highway bridges. Existing research results and state inspection, analysis, and evaluation practices were examined and synthesized. This report presents specific guidelines for field inspection and assessment of corrosion damage on structural steel members and components. In addition, guidelines and simplified methods for the office evaluation and analysis of the structural capacity of corrosion damaged members and components are presented. The report also recommends provisions for incorporation into the *AASHTO Manual for Maintenance Inspection of Bridges* that consider corrosion damage and the associated reduction in load capacity due to corrosion. This report will be of immediate interest and use to bridge engineers, bridge inspection personnel, materials engineers, specification writing bodies, and others concerned with the inspection or evaluation of corrosion damage to structural steel members and components.

Engineers normally assess the detrimental effects of corrosion on steel bridge components in terms of the increased static and fatigue stresses caused by the reduction in cross-sectional area of the components. Limited studies have previously demonstrated that stress concentrations caused by corrosion in steel bridge members can result in fatigue behavior equivalent to an AASHTO Category E fatigue detail. However, corrosion can produce other severe effects such as: freezing of pinned joints which may cause unintended bending moments in members; freezing of bearings which may lead to large forces in piers, abutments, and other members; and the build up of corrosion products causing local forces and distortions transverse to the normal load carrying direction. This last effect has been blamed for several major bridge failures in recent years. Other detrimental effects can be produced by nonuniform patterns of corrosion on bridge members. At present, guidelines do not exist for bridge inspectors and bridge engineers to adequately identify and evaluate the detrimental effects of corrosion on critical details in steel bridges.

NCHRP Project 12-28(7), *Guidelines for Evaluating Corrosion Effects in Existing Steel Bridges*, was initiated with the objective of developing practical guidelines for assessment of the effects of corrosion on structural details in steel highway bridges. Another project objective was the development of specification provisions that could be recommended to AASHTO for inclusion in the *AASHTO Manual for Maintenance Inspection of Bridges*. The research entailed collecting existing literature and data and synthesizing existing practices.

This report documents the work performed under Project 12-28(7). The report is organized into four major parts: Field Inspection Guidelines, Office Evaluation Guidelines, Recommended Specifications for Evaluation of Corrosion Effects in Steel Bridges, and the Research Report.

Part I of the report, Field Inspection Guidelines, provides an overview of the mechanics of corrosion and the importance of adequate corrosion inspection and identification. The Guidelines identify critical details that must be inspected and provide descriptions and photographs that will assist the inspector in the determination of corrosion severity. Recommended corrosion-inspection forms are also provided in the Guidelines.

Part II of the report, Office Evaluation of Corrosion Effects, provides an overview of potential failure modes due to corrosion effects. It presents recommendations and simplified analysis techniques for determining the remaining load capacity of a corroded bridge member or component. Part II treats, in detail, the effects of material loss, the effects of corrosion on fatigue resistance, and the effects of unintended fixities, movements, and pressures caused by corrosion and corrosion products.

Part III of the report summarizes the office evaluation analysis methods and presents them in a specification format. The recommended provisions are intended for inclusion in the *AASHTO Manual for Maintenance Inspection of Bridges*. Part IV is the research report summary, describing the research approach, and providing a discussion of the research findings, applications, and conclusions.

The field and office guidelines are suitable for immediate use in most highway and railroad bridge inspection, analysis, and evaluation programs. It is expected that the AASHTO Subcommittee on Bridges and Structures will consider the recommended specifications for adoption in either 1991 or 1992.

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GUIDELINES FOR EVALUATING CORROSION EFFECTS IN EXISTING STEEL BRIDGES

SUMMARY

Corrosion is the major cause of deterioration of steel bridges. The results of this deterioration can range from progressive weakening of a bridge structure over a long period of time to sudden bridge collapse. The effects of corrosion damage vary with the type of structure and the location and extent of deterioration. Corrosion damage must be carefully appraised and evaluated. In some cases, immediate repair or closure is necessary, while in other cases, the conditions created by corrosion can be tolerated. In all cases, however, the likely progression of corrosion must be considered.

Present inspection procedures do not focus specific attention on the types of corrosion attack on bridges. Also, in many cases, the amount of deterioration is not adequately determined. Bridge inspectors should be given additional instructions regarding inspection with respect to corrosion.

The primary objective of this report is to present practical field and office guidelines that can be used to assess the effects of corrosion on steel bridges along with a description of the field and analytical investigations performed to develop the guidelines. For user convenience, the report's format differs from that of a typical NCHRP series report. It is organized into four parts: Field Inspection Guidelines, Office Evaluation of Corrosion Effects, Recommended Specifications for Evaluation of Corrosion Effects in Steel Bridges, and Research Report. Using the guidelines and the corrosion inspection forms presented in Part I can help inspectors in the appraisal of corrosion damage on bridges. The field inspection guidelines suggested in Part I are based on a two-level appraisal system. The Level I inspection is adequate for routine biennial inspections and is performed by current inspection staffs who have sufficient training to quantitatively assess corrosion performance parameters, such as section loss, surface roughness, unintended fixity, and type of corrosion, and then to clearly report corrosion information. The Level II inspection is performed by a multidisciplinary team including bridge inspectors and specialists in corrosion. The corrosion inspection of a bridge is intended to supplement information gathered under the National Bridge Inspection Standards (NBIS) program.

The AASHTO bridge maintenance inspection manual and the AASHTO standard specifications for design are primarily concerned with good quality materials and bridge structural behavior consistent with its design. A corrosion damaged structure may behave differently from the "design" structure and different failure modes may govern its capacity. Therefore, additional criteria must often be considered when evaluating the capacity of a deteriorated structure. The office evaluation methods presented in Part II include recommendations and simplified analysis techniques aimed at helping the engineer evaluate remaining structural resistance and making him aware of the variety of potential failure modes. Treated in detail are the effects of material loss, effects of corrosion on fatigue resistance, and the effects of unintended fixities, movements, and pressures caused by corrosion.

Part III then summarizes the office evaluation methods of Part II in a guide specification format suitable for consideration by the AASHTO Subcommittee on Bridges and Structures. The recommendations and analysis techniques presented in Part III, for

the evaluation of common conditions created by corrosion that are not covered in the *AASHTO Manual for Maintenance Inspection of Bridges*, are intended to complement that manual and should be used in conjunction with the current AASHTO Standard Specifications for Highway Bridges.

Part IV, the main text of the NCHRP Project 12-28(7) final report, documents the research conducted as background for preparation of the guidelines. The first chapter introduces the research approach in accomplishing the objectives. Subsequent chapters discuss the findings, applications, conclusions, and potentially fruitful areas of research that have become evident as a result of the study. The appendixes (A through G) include a glossary of the terminology used throughout the report, a bridge inspection questionnaire that addresses corrosion effects and current practices, the recommended corrosion and evaluation guidelines, and details of the laboratory tests, field investigations and analytical studies conducted to complement the existing information.* Cited references and an extensive bibliography are provided in Appendix H.

*See Appendix I, herein, for additional information.

Part I

Field Inspection Guidelines

CHAPTER I

INTRODUCTION

1.1 BACKGROUND

In the past decade, corrosion has received increased attention as a cause of structure damage and failure. Several recent bridge collapses have been attributed to corrosion. The estimated cost of repairing corrosion-damaged bridges in the United States is staggering. At present, however, there are no established means of evaluating the extent and effects of bridge corrosion. In some cases, the devastating effects of corrosion go unheeded, with costly or even catastrophic results. In other cases, bridges are unnecessarily closed or replaced because their load-carrying capacity cannot be determined.

The field inspection guidelines documented in Part I of this report are part of an effort to clarify and standardize bridge corrosion inspections and evaluations, with the goal of increasing the safety and serviceability of the nation's bridges.

The guidelines for office evaluation of corrosion effects, presented in Part II, include simplified analysis techniques for routine use and background information for assessing local and overall effects of corrosion. Effects of metal loss, unintended fixities, and distortions caused by corrosion are treated in detail.

1.2 CORROSION INSPECTION

The National Bridge Inspection Standards (NBIS), as regulated by the Federal Highway Administration (FHWA), require existing bridges to be inspected at defined frequencies; inspection reporting is summarized on a Structural Inventory and Appraisal (SI&A) form. The data recorded on the SI&A form do not specifically require information concerning the types or extent of corrosion on a bridge.

The corrosion inspection of a bridge is intended to supplement information acquired under the NBIS program. The corrosion inspection program is concerned with the types of corrosion found, locations of corrosion on bridge elements, metalwork losses as a result of corrosion, rates of deterioration, unintended fixity or restraint of moving parts, ways to mitigate corrosion damage, and environmental effects on the structure. The corrosion inspection is not concerned with the width of the bridge roadway or its skid resistance, but is interested in pavement joints and cracks that allow water and debris to collect on steel support elements and cause corrosion. The corrosion inspection is not interested in the adequacy of the roadway handrail for impact restraint, but is concerned with whether it acts as a splash guard to protect steel elements or if there is galvanic corrosion from dissimilar metals in the handrail-support connections.

The evaluation of corrosion effects in existing steel bridges is

based on a two-level appraisal system. The Level I evaluation is to be performed by current inspection staffs who have sufficient training in corrosion to quantitatively assess corrosion performance parameters (such as section loss, surface roughness, unintended fixity, or types of corrosion), and to be able to accurately report corrosion information. One of the main objectives of a Level I evaluation is to determine whether a Level II evaluation is required. A Level II evaluation is performed by a multidisciplinary team including the bridge inspectors and specialists in corrosion. The objective of this evaluation is to determine the effects of corrosion on the load-carrying capacity of the bridge, to evaluate the corrosion process itself, and to determine a method to interrupt or mitigate the corrosion. This evaluation addresses loss of metalwork section, corrosion-induced stress raisers, consequences of unintended fixity of moving parts, and potential instability or overstress of members due to corrosion-induced distortions or section loss.

1.3 PURPOSE AND SCOPE OF FIELD INSPECTION GUIDELINES

The purpose of the field inspection guidelines provided in Part I of this report is to familiarize the bridge inspector with the many types of corrosion that occur, with emphasis on those commonly found on bridges. Representative of a broad range of subjects covered in this part are a general background on corrosion mechanics, progression, and effects; descriptions of the various forms of corrosion with definitions; typical occurrences on bridges and photographic examples; sketches depicting bridge corrosion conditions and showing where various types of corrosion are found; types of corrosion inspections; samples of corrosion inspection reports based on an example bridge; and a discussion of corrosion prevention and repair.

The aim of the field guidelines is to present methods of gathering meaningful information and conveying it from the bridge inspector to the office evaluator. This guide outlines the requirements of Level I and Level II corrosion inspections. It does not address specific protective coating evaluation techniques, and should not be considered a "text book" on corrosion that will make bridge inspectors experts on the subject.

The office guidelines give the engineer a means of evaluating the information gathered in the field. The office requirements may range from a review of field conditions to determine the need for repainting or the selection of a paint system, to a full evaluation of effects of advanced corrosion upon the bridge's carrying capacity, with prioritizing of repairs.

1.4 CORROSION CASE HISTORIES

Catastrophic failure, with loss of life, is probably the most publicized aspect of bridge corrosion. Collapses of the Point Pleasant (Silver) Bridge over the Ohio River in 1967 and the Mianus River Bridge on Interstate 95 in Connecticut are two widely known bridge failures. The Point Pleasant Bridge, an eyebar chain suspended structure, failed because of corrosion cracks at the pin hole in an eyebar. The Mianus River Bridge failure is attributed to corrosion of components of a pin-and-hanger assembly. It is hypothesized that, over a relatively long period of time, accumulation of corrosion products from an underlying washer shifted the hanger transversely on the pin causing a misalignment of the hanger. This misalignment, with the hanger now bearing nearer the end of the pin, increased the stress range in the pin resulting in a fatigue crack leading to failure of the pin. The critical nature of this detail and the effect of corrosion on the performance of the detail apparently were not noticed during previous inspections. It is not even certain whether each of the critical hanger assemblies were inspected during the previous inspections.

The Mianus River Bridge disaster sparked immediate inspections of similarly constructed bridges nationwide. While misalignments analogous to that hypothesized to explain the Mianus River Bridge failure were not uncovered, at least two other bridges were found with corrosion-related problems at pin-and-hanger assemblies. The Harvard Bridge in Cambridge, Massachusetts, was approximately 100 years old. Its hangers were wrought iron eyebars that had frozen at the pins because of corrosion, which caused a redistribution of forces sufficient to initiate fatigue cracks. A major rehabilitation program was required before returning the bridge to full service. A broken hanger was found on the Yankee Doodle Bridge over the Norwalk River, on Interstate 95 in Connecticut. After blast cleaning the overlying corrosion product, 16 other hangers were found with cracks. Although the cracks were not attributed to significant metal loss due to corrosion, corrosion products between the pin and hanger may have caused the joint to bind. In any case, corrosion had obscured serious structural damage that might well have gone undetected during a normal bridge inspection.

Other dramatic examples of undetected corrosion can be cited. In Philadelphia, severe deterioration caused failure of a main load-carrying member of a Southeastern Pennsylvania Transportation Authority (SEPTA) bridge which had been hidden from view by wall and ceiling panels of a station building. In Detroit, there was a near collapse of a bridge because stirrups supporting a girder had rusted through and broken. The girder (and presumably the stirrups) had been encased in concrete for protection against exhaust fumes from locomotives passing below. Cracks were discovered in beams of a Virginia bridge by a work crew. These cracks were reported to be a result of bearings frozen by rust. This damage had apparently occurred within a 2-year inspection interval.

An historic bridge in St. Paul, Minnesota, provides an example of the speed with which corrosion damage can occur. Wrought iron eyebars on this structure experienced a 10 percent loss of section in only 4 years. Sliding and roller bearings were also frozen, apparently within a 2-year period. The altered behavior of this bridge was sufficient to shift members intended for tension into compression and vice versa.

Somewhat less dramatic examples of bridge corrosion are found in which the operating authority is aware of existing corro-

sion damage and is able to plan for major maintenance programs. Such programs have been described for the Ben Franklin Bridge in Philadelphia, which has experienced extensive corrosion damage to stringer beams and electrolysis damage from a direct current electric transit system operating on the bridge. Corrosion of structural steel and lack of regular maintenance for electrolysis damage are also cited as reasons for a rehabilitation program on the New York City elevated transit system.

Corrosion associated with paint breakdown and scale on cast iron was cited as the impetus for major repairs on the Tower Bridge in London. Here, it was noted that the majority of corrosion occurred where water collected near structural members, either by seepage through the deck or as a result of pockets in its design features.

Cables are of particular interest in the integrity of suspension bridges. A complete recabbling of the Royal Gorge Bridge (after 45 years of service) was performed because of corrosion of the cables in the concrete encased portion of the anchorages. Suspension cable corrosion was studied in great detail for bridges over the Ohio River after the Portsmouth Bridge was recabled for the second time in its 50-year life. The portions of cables most prone to attack are those that remain moist for extended periods. Cables in damp anchorages and poorly drained sections in the main span are especially vulnerable. Many other instances of suspension bridges experiencing cable corrosion damage have been reported.

Recently, attention has been focused on corrosion of cables of cable-stayed bridges. A survey of many of these bridges showed significant cable corrosion. The Lake Maracaibo Bridge in Venezuela has required recabbling and may need it again in the near future. A bridge at Hamburg, Germany, required cable replacement because of corrosion after only 3 years of service. Problems have been encountered because of leaking of cable sheathing. This allows water to contact the cables, leading to corrosion attack.

It should be noted that a lack of redundancy (no alternate paths for applied loads) increases the risk of structural failure due to corrosion damage.

CHAPTER 2

CORROSION OF STEEL

2.1 MECHANICS

Corrosion of steel is the deterioration and eventual destruction of the metal because of its reaction with the environment. Chemically, it is the transformation of a metal to its oxide through a reaction involving oxygen, water, or other agents. Figure 1 depicts the steel life cycle.

Corrosion is an electrochemical process. It requires metal, an electrolyte, and current flow. Corrosion occurs between metal areas having a higher tendency to corrode (anode) and metal areas having a lower tendency to corrode (cathode). An electrolyte which allows current flow must be in contact with the anode and cathode for corrosion to occur. On bridges, this electrolyte is usually water. Electrons flow through the base metal from the

anode to the cathode. As negatively charged electrons leave the anode, positively charged ions of the anode metal are released into the electrolyte. These ions can react with other materials to form "corrosion products." Metal loss occurs at the anode, while the cathode is undamaged.

Different areas on the same steel member can serve as anodes or cathodes because of differences in chemical or metallurgical structure, stresses, or the presence of corrosive deposits. It is possible for an area that is initially cathodic to become anodic, and even reverse back to cathodic again.

A simplified diagram of the corrosion process is shown in Figure 2.

A condition such as that shown in Figure 2 is called a "corrosion cell." An oxygen cell is a type of corrosion cell in which oxygen concentrations in the electrolyte determine the anode and cathode locations. Locations where the electrolyte oxygen concentration is low (such as stagnant standing water) are anodic and prone to corrosion. Metals at point of low ion concentration corrode. Bacteria can affect the rate of corrosion because their metabolic processes can alter the oxygen and metal ion concentrations in the electrolyte.

The corrosion process on an actual steel member is shown in Figure 3.

The corrosion product for iron and steel is iron oxide. It is similar in appearance and practically identical in composition to common forms of iron ore. Iron ions react with oxygen in water to form rust. Other common corrosion products are "white rust" on aluminum and greenish patina on copper.

All metals react with their environment to some extent. Metals can be ranked in the "galvanic series" from most anodic to most cathodic. In general, if two of these metals are in contact, the one nearer the top of the list will be the anode (and corrode), while the less reactive will be the protected cathode.

The following is a condensed list of the galvanic series, showing common metals:

<i>Anodic</i>	1. Magnesium	8. Nickel	<i>Cathodic</i>
	2. Zinc	9. Brass	
	3. Aluminum	10. Copper	
	4. Steel	11. Bronze	
	5. Cast Iron	12. Silver	
	6. Lead	13. Gold	
	7. Tin	14. Platinum	

Corrosion occurs in many specific forms. Those forms which commonly attack bridges are discussed in Chapter 3.

2.2 PROGRESSION

The rate and progress of corrosion on steel are affected by several factors, which include environmental effects, type of steel, surface protection, and such other factors as the presence of pollutants, bacteria, crevices, deposits, and stress. If any one of these factors is changed, the rate and extent of the corrosion will also change.

2.2.1 Environmental Effects

Environmental effects include temperature, humidity, and the exposure of the material. High temperatures increase the rate of corrosion. This is usually not significant in bridges. The amount

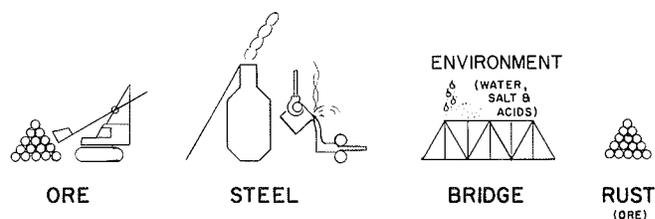


Figure 1. Steel life cycle.

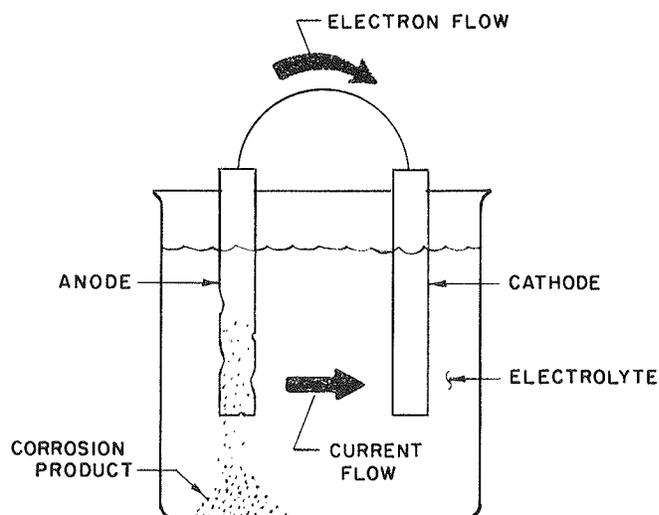


Figure 2. Simplified corrosion process.

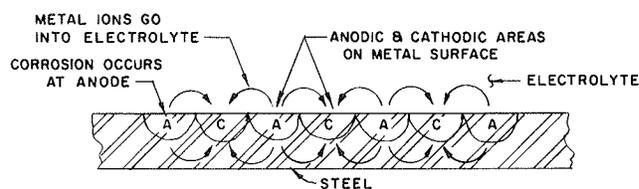


Figure 3. Corrosion process on steel.

of moisture available is very important to the rate of corrosion because water serves as an electrolyte. In arid regions, corrosion may be slow compared to regions with above-average precipitation. Exposure is important in assessing corrosion on a single structure. Areas exposed to wind or sun that can dry quickly are less prone to corrosion than sheltered areas where water can remain in contact with the metalwork (see Figures 4(a) and 4(b)). Inspectors should pay close attention to locations where water can collect, no matter how small these areas may be.

Impurities (such as salt) can make water a more efficient electrolyte and speed corrosion. Because of this, structures in coastal areas—or those exposed to deicing salts—will corrode faster than bridges not exposed to salt. Studies have shown corrosion rates 2.75 times higher when salt is present than when it is not.

2.2.2 Type of Steel.

The type and grade of steel used can have a major effect on the rate of corrosion. In general, structural steels are "carbon steels." They contain carbon as an alloying element to give strength and hardness. Copper as an additive to steel helps improve strength and gives a significant increase in corrosion resistance in atmospheric exposure. Silicon has a similar effect. Chromium increases the hardness and strength of steels, as well as its corrosion resistance. Nickel shows similar behavior as an alloying element. Manganese is used to prevent brittleness and increase toughness of steel.

The grades and characteristics of structural steel typically used for bridges are as follows:

ASTM-A36 (AASHTO M183): This grade is the most common for bridges built since the 1950s. Its main alloying agents are carbon and manganese. Silicon is sometimes added to sections thicker than 1½ in. A36 steel requires coating for corrosion protection. Copper may be added to increase corrosion resistance.

ASTM-A572 (AASHTO M223): This is a higher strength grade than A36. Its main alloying agents are carbon, manganese, silicon and columbium or vanadium (which give a fine-grained structure). Typically, A572 steel is twice as resistant to corrosion as A36, but also requires coating for protection.

ASTM-A588 (AASHTO M222): This grade is one of the high-strength, low-alloy or "weathering" steels. Besides the alloying agents used in A572, it includes nickel, copper, and chromium. Generally, A588 has about twice the atmospheric corrosion resistance of carbon steels containing copper, and four times the resistance of carbon steel without copper. This steel is used both with and without painting. When unpainted, its corrosion product forms a protective film on the steel surface that reduces further corrosion. A588 is available in strengths comparable to A572.

ASTM-A440 & A441 (AASHTO M187 & M188): These grades are not in common use today, having been replaced by A588. Their strength is comparable to A588 or A572, and their corrosion resistance is about twice that of carbon steels.

ASTM-A242 (AASHTO M161): This is the predecessor of A588 steel. Its properties are similar to A588. A242 is not commonly used today.

Older steel types include:

A7: This grade was the most common type of structural steel prior to the 1950s. Carbon was the main alloying element in A7 steel. Its strength and corrosion resistance is comparable to A36 steel.

A94: This grade is also known as "silicon steel." It had a higher carbon and silicon content than A7 steel, giving it a higher strength.

A42: This material—wrought iron—is formed of virtually pure iron or iron silicate rolled with slag inclusions that form fibers through the material. Wrought iron is tough and ductile. Its yield strength is usually less than steel, but its corrosion resistance is higher than A36 steel. Wrought iron can be alloyed, sometimes with nickel.

2.2.3 Surface Protection

Surface protection is the most common type of corrosion defense. Steel is usually painted or galvanized. Epoxy coating is now being used on reinforcing bars for concrete in corrosive environments. Typical forms of surface protection provide a barrier between the metal and its environment, particularly water, the common electrolyte for corrosion. Keeping the electrolyte away from the metal will stop the corrosion reaction. In other words, if the bridge could be protected totally from the environment, no corrosion would occur.

Sacrificial coatings of aluminum, zinc or aluminum-zinc alloy are also used to protect steel. Zinc is applied to steel by electroplating, hot dip galvanizing, liquid applied coatings (organic and inorganic zinc), and flame spray methods. Aluminum and aluminum-zinc are applied by hot dip galvanizing and flame spray methods. Sacrificial coatings protect the steel by being both a barrier coating and a cathodic protection. The steel is protected by cathodic protection at scratches and nicks in the coating.

Other forms of surface protection apply a coating that uses galvanic corrosion to protect the base metal. Zinc, the metal coating applied to steel during galvanizing (and also used in some paints), is closer to the anodic end of the galvanic series than steel. As a result, corrosion attacks the sacrificial zinc coating, protecting the steel beneath. As the zinc is sacrificed, it forms an oxide which further protects the steel. This oxide can cover nicks and scratches in the zinc coating, in effect "healing" injuries to the protective layer.

Copper-bearing steel and high-strength, low-alloy steel ("weathering steels") develop an initial layer of rust when exposed to the environment which adheres to the metal surface to protect it from further exposure and corrosion. It should be noted that these steels are not immune to corrosion damage. Under adverse conditions in which moisture collects on the surface and in certain harsh environments, they can deteriorate like other structural steels (see Chapter 7, section 7.6).

2.2.4 Other Factors

Other factors affecting corrosion include the presence of pollutants in the atmosphere, the action of bacteria or other microorganisms, the effects of animal deposits, the types of bridge details, the presence of stray electrical currents or dissimilar metals, and the presence of deposits, crevices, or stress.

Atmospheric pollutants can have similar effects to salt as discussed earlier. Acids formed from gasses in the atmosphere can also directly attack structural steel (in "acid rain," for example) and increase the rate of corrosion. Inspectors in industrial areas should consider pollution effects just as inspectors in marine environments or in locations where deicing salts are used should consider salt effects.

Bacteria and other microorganisms contribute to corrosion by destroying the protective film on metals, forming deposits on the surface and, sometimes, even attacking the steel itself. Corrosive by-products (generally acids) of their metabolism can directly corrode the metal. The effects of bacterial attack can usually be observed in the field, but a determination of the specific cause cannot be made without special tests.

Animal life, particularly birds, can have similar effects as bacteria. Birds nest and roost in the bridge structure, and their

nests and droppings harbor moisture and leave deposits that can form corrosion cells or chemically attack the metal.

The type of bridge details used on a structure has an important effect on the corrosion rate. Details that collect water or provide ideal nesting spots for birds should be carefully checked for corrosion. Crevices form areas where pack rust can accumulate and are conducive to corrosion cell formation.

Stray electric currents from DC sources, such as transit systems or welding activity, promote corrosion by speeding the rate of the electrochemical process. In effect, the currents force the reaction to occur much faster than it would naturally. This phenomenon is difficult to diagnose without special tests. It is more likely on structures with electrical equipment attached. Cathodic protection, which is being used to prevent corrosion of bridge deck reinforcing bars, is an example of electrical currents being applied intentionally to a structure. If the direction of the current flow can be controlled so the structure always acts as a cathode, it can be protected from corrosion.

Combining dissimilar metals on a structure can promote corrosion due to differences in corrosion tendency. Generally, metals ranked lower in the galvanic series will corrode when in electrical contact with metals ranked higher in the series. This can occur where bronze or copper (a cathode), for example, is attached to the steel (anode) structure.

2.3 CORROSION RATES

At this time, research is continuing into prediction of corrosion rates. The general conclusions are that rates of future corrosion can be projected based on knowledge of prior corrosion rates on the same structure. However, because of the variety of factors affecting the corrosion rate, each structure would have a different rate formula. To further complicate matters, different locations on the same structure could have different rates. It is probably premature to predict corrosion rates on a specific structure. In general, however, uniform corrosion occurs at lower rates than corrosion cell effects which can proceed quite rapidly.

2.4 CORROSION EFFECTS

There are four main categories of corrosion effects on the structural integrity of a bridge:

1. *Loss of section.* This is the most important concern. The reduction in member section dimensions leads to lower bending, axial and shear capacity. This effect should be evaluated based on its location on the member (at middle, end, point of load, etc.). It can also affect the fatigue life of the member because of the increased stress range.
2. *Creation of stress raisers.* The formation of holes and notches due to corrosion creates stress concentrations and can initiate cracks.
3. *Introduction of unintentional fixity.* When corrosion freezes moving parts of the bridge, such as expansion devices or hangers, the structure behaves differently from the way it was designed. Members can be subjected to unexpected high stresses.
4. *Introduction of unintended movement.* According to one

study, built-up corrosion product in constricted areas ("pack rust") can generate pressures up to 10,000 psi. This pressure can bend or move bridge components with damaging effects.

CHAPTER 3

CORROSION FORMS ON BRIDGES

3.1 INTRODUCTION

Corrosion is known to appear in many forms. These forms are classified according to how the corrosion attacks the metal. The corrosion spectrum ranges from uniform corrosion, which can be identified visually, to stress corrosion, which cannot be identified with the naked eye.

Eight forms of corrosion have been identified in Fontana's *Corrosion Engineering*. This list covers the most common types of metal corrosion. The primary forms of corrosion, and the subgroups associated with some of them, are: (1) uniform corrosion; (2) galvanic corrosion; (3) crevice corrosion (deposit attack, underfilm corrosion); (4) pitting; (5) intergranular corrosion (weld decay); (6) selective leaching; (7) erosion corrosion (fretting); and (8) stress corrosion (corrosion fatigue).

All bridge inspectors should be familiar with the forms of corrosion, particularly those that are easily identified by the naked eye. The inspector should also be aware of the forms of corrosion that require the use of a microscope or laboratory tests in order to identify them. In many situations, several forms of corrosion may be present simultaneously.

This chapter discusses the forms of corrosion and their occurrence in bridges. Each of these forms and subgroups are discussed in full, including the formation of the corrosion, the frequency of observation, the locations where the form is generally found on bridges, and the presentation of photographic examples. Following the discussion of corrosion forms, specific bridge details are considered to show the types of corrosion they commonly suffer.

3.2 UNIFORM CORROSION (See Figure 5)

3.2.1 Definition

Uniform corrosion or rusting (also known as general corrosion) is a general thinning of metalwork in a universal or overall manner. It is a natural process exhibited by all bare metals exposed to the atmosphere. On steel bridges, it is observed as a uniform rust over the entire surface. Uniform corrosion can be identified by the naked eye.

3.2.2 Occurrence

One of the simplest examples of uniform corrosion is the formation of the oxide product that protects weathering steel. New weathering steel generally is coated with mill scale that eventually flakes off as a result of weathering and corrosion,

exposing the base metal. A progression of corrosion occurs until the surface is covered by its own corrosion product. The corrosion product reduces the corrosion rate by forming a barrier between the metal and the environment.

Often bridges in arid areas exhibit uniform corrosion because of the lack of moisture which would have caused other forms of corrosion to occur.

Uniform corrosion of steel typically consists of many small pits joined together. With the thinning of a paint system, the peaks of metal are exposed and a uniform coating of rust or corrosion occurs.

Uniform corrosion is most commonly found on steel bridges on plates and shapes with large surface areas that can be uniformly attacked or oxidized. Usually these members can dry quickly, preventing other forms of corrosive attack. Such members include girder webs, vertical gusset plates, and truss verticals and diagonals.

3.3 GALVANIC CORROSION (See Figure 6)

3.3.1 Definition

Galvanic corrosion or dissimilar metal corrosion is caused when metals of different composition are placed together in the presence of an electrolyte. The difference in their corrosive potential produces electron flow, with one of the metals as the anode and one as the cathode. The intensity of corrosion depends not only on the difference in corrosion potential between the metals (see the galvanic series in Chapter 2), but also on the ratio of the exposed area of the metals and their specific corrosion behavior. Galvanic corrosion can usually be identified visually.

3.3.2 Occurrence

Galvanic corrosion most commonly occurs on steel bridges where aluminum light poles, handrails, or electrical conduits are in contact with steel or where galvanized steel is in contact with bare steel (such as weathering steel). Insulating materials are often placed between the metals to prevent the formation of galvanic corrosion. Galvanic corrosion may also occur on steel where mill-scale is exposed.

Galvanic corrosion has a beneficial effect in the application of zinc paints on steel. The intent is that the less resistant metal, zinc, will be sacrificed in the corrosion process and the steel surface will remain free of corrosion.

3.4 CREVICE CORROSION (See Figures 7, 8, and 9)

3.4.1 Definition

Crevice corrosion is a form of localized corrosion occurring at confined locations where easy access to the outside environment is prevented. It is caused by differences in the environment inside and outside of the crevice, such as concentrations of oxygen cells or metal ion cells. The presence of chloride ions also promotes crevice corrosion. Crevice corrosion can usually be visually observed.

3.4.2 Occurrence

Crevice corrosion is one of the most common forms of corrosion found on steel bridges. It occurs within gaps between mating surfaces as small as several thousandths of an inch wide, such as along edge openings of built-up members with multiple plies of plates, between back-to-back angles used for bracing members, between lacing bars and adjoining components, and between closely spaced eyebars. Crevice corrosion can also occur between steel and other materials, such as timber decks or concrete slabs. These gaps are commonly formed by variations in thickness or alignment from mill rolling of plates and shapes, shearing of plate edges in the fabrication process, and excessive spacing of fasteners that fail to seal the components with their clamping action.

Steels that rely on an oxide film for protection, such as weathering steel, are particularly susceptible to crevice corrosion. These films are destroyed by the high concentrations of chloride or hydrogen ions that can occur in crevices.

3.5 DEPOSIT ATTACK (See Figure 10)

3.5.1 Definition

Deposit attack is a localized corrosion of the crevice corrosion form caused by a deposit of foreign material acting as a shield to create a confined space that behaves like a crevice. These deposits can also hold moisture, which provides an electrolyte. Deposit attack can be observed visually.

3.5.2 Occurrence

Deposit attack frequently occurs on bridges at locations of debris deposits harboring moisture. The debris often consists of road dirt or trash deposited on horizontal surfaces either by wind or by water draining off the roadway. The debris deposits can have a local source, such as coal dust in mining areas, grain or other by-products in farm regions, or salts from deicing agents in northern or high altitude regions. Pack rust itself can act as a deposit and promote further corrosion. One of the most annoying types of deposits comes from bird nests and bird excrement. Many of the materials deposited contain very active agents that accelerate corrosion. Coal dust deposits, for example, contain carbon, which can cause galvanic corrosion, and sulfur compounds, which attack steel. Bird droppings contain acids that damage steel members and protective coatings.

3.6 UNDERFILM CORROSION (See Figures 11 and 12)

3.6.1 Definition

Underfilm corrosion is a type of crevice corrosion that occurs beneath paint. It usually begins where the paint has been physically damaged or at defects in the paint film. This form of corrosion attacks the surface between the coating and the metal causing the paint to debond. A special type of underfilm corrosion known as filiform corrosion occurs in the form of threadlike filaments. Filiform corrosion occurs in high humidity conditions. Underfilm corrosion can be classified visually.

3.6.2 Occurrence

Underfilm corrosion starts at locations where there are breaks in the paint. It can occur anywhere on a structure and is seen as cracking, blistering, or peeling of the paint film. Probing of the coating at damaged areas to determine if coating debondment has occurred will often reveal that a much larger area of metal has been corroded than detectable by examining the painted surface.

3.7 PITTING (See Figure 13)

3.7.1 Definition

Pitting is localized corrosion attack which causes the formation of deep, sometimes narrow, penetrations into steel surfaces. Its formation occurs where there are chemical or physical changes in the metal such as imperfections in the metallurgy of steel, at paint protection flaws, or, most commonly, under deposits of foreign material. Pitting can act as a stress raiser and cause failure by cracking. Pitting can be identified with the naked eye.

3.7.2 Occurrence

Pitting is commonly found where debris of any type harbors moisture on a surface, such as deposits of dirt, trash, or bird excrement. Pitting commonly is found where the paint protection is scratched, nicked from flying debris from vehicles, or at imperfections in the application of the paint. Deposits of minute salt particles in coastal regions or where deicing salt is used can lead to extensive pitting. Pitting frequently takes place under deposits of corrosion product such as pack rust.

3.8 INTERGRANULAR CORROSION

3.8.1 Definition

Intergranular corrosion is a corrosion attack of the boundaries between the metal grains. After the grain boundaries deteriorate, the grains fall out and the metal disintegrates. While the effects of intergranular corrosion are visible to the naked eye, a precise diagnosis requires supplementary examination.

3.8.2 Occurrence

The most common form of intergranular corrosion on bridges is weld decay. See section 3.9 for a discussion of this.

3.9 WELD DECAY (See Figure 14)

3.9.1 Definition

Weld decay is the localized deterioration either of weld metal or base metal due to a decrease in corrosion resistance caused when the heat of welding alters the granular structure of the steel. This intergranular corrosion appears as a band of corrosion parallel to the weld. Weld decay usually requires supplemental examination to confirm its presence.

3.9.2 Occurrence

Weld decay is not a common form of corrosion on bridges that have been properly welded under shop-controlled conditions during fabrication. Its occurrence is more likely to be found adjacent to field welds applied without proper control of heat. Paint applied over field welds may be of lower quality than shop paint, contributing to weld decay. It occurs more frequently in association with thin steels, stainless steels, and alloy steels, but can sometimes be found in structural carbon steels.

3.10 SELECTIVE LEACHING

3.10.1 Definition

Selective leaching (sometimes referred to as dealloying) is the dissolution of one component of an alloy. This can result in changes in its mechanical properties. The identification of dealloying may require microscopic examination.

3.10.2 Occurrence

Selective leaching is not commonly found on steel bridges. An example of such corrosion may be occasionally found on bronze (copper-zinc-tin alloy) bearings where the zinc may leach from alloy. Stagnant conditions in confined areas will favor its formation.

3.11 EROSION CORROSION (See Figure 15)

3.11.1 Definition

Erosion corrosion is an attack on a metal caused by the flow of fluid over its surface with sufficient velocity to remove adhering surface corrosion product. Erosion corrosion, as it typically relates to bridges, is in the form of particle erosion, where particles in fluid abrade the metal surface, wearing away the surface coating on protective corrosion products. This allows corrosion to continually attack bare metal, and speeds the rate of attack. Erosion particle corrosion is analogous to water blasting with a grit. The identification of erosion corrosion may require microscopic inspection.

3.11.2 Occurrence

Erosion particle corrosion is not a common form of corrosion on steel bridges but can be dangerous when streams carry particulate matter that erodes steel piling. This can go undetected under water.

3.12 FRETTING CORROSION (See Figure 16)

3.12.1 Definition

Fretting corrosion is caused by relative motion of two surfaces in close contact under load. Fretting involves the rubbing contact

of nonlubricated surfaces where surface oxidation forms, is broken, and reforms, causing abrasion of the surfaces by oxide and debris. Fretting corrosion cannot be positively identified with the naked eye.

3.12.2 Occurrence

On steel bridges fretting can be observed at stringer relief joints and at stringer ends having sliding contact surfaces where slight stringer movement occurs. It may also be found at locations where bridge components vibrate.

3.13 STRESS CORROSION (See Figures 17 and 18)

3.13.1 Definition

Stress corrosion cracking is cracking caused by the simultaneous occurrence of tensile stress (either residual or applied) and a corrosive environment. Corrosion causes the initiation of discontinuities in the metal acting as stress raisers that lead to cracks. The cracks may be either intergranular (around grains) or transgranular (across grains), but normally occur perpendicular to the member stress. Depending on the type of steel and the corrosive environment, the crack may be as simple as a single crack, but could have multiple branches. Stress corrosion cracking appears as a brittle fracture in an otherwise ductile metal. Upon microscopic examination, the corrosion product can be found in the cracks. The adjacent metal surface generally does not show evidence of any damage. Stress corrosion cracking requires microscopic inspection for identification.

3.13.2 Occurrence

Stress corrosion cracking can occur in bridges under adverse environmental conditions, such as found in industrial areas or in marine environments.

An example of stress corrosion cracking was observed in a structure in a corrosive environment where high-strength bolts failed while the connected members showed no indications of corrosion. The bolts, being tensioned to the proof load (near yield point), developed cracks perpendicular to the applied load reducing the bolt cross-section area until the bolt failed. Stress corrosion cracking has also been observed on wires and strands in the main cables of suspension bridges.

3.14 CORROSION FATIGUE

3.14.1 Definition

Corrosion fatigue is a fatigue-type cracking of metal caused by repeated or fluctuating applied stresses in a corrosive environment. It causes the reduction of fatigue life when the affected member is exposed to a corrosive environment compared to its life in a noncorrosive environment. The mechanism of corrosion fatigue is analogous to stress corrosion cracking, with corrosion creating stress concentrations which cause crack initiation. The damage appears to occur only during the tensile stress portion

of the fatigue stress cycle. Corrosion fatigue must be verified by microscopic examination.

3.14.2 Occurrence

The occurrence of corrosion fatigue on steel bridges is limited to fatigue-sensitive members in a corrosive environment. The distinction between corrosion fatigue and normal fatigue is determined by the presence or absence of corrosion.

3.15 CORROSION FORMS AND BRIDGE LOCATIONS

Many forms of corrosion occur on steel bridges. Often more than one form occurs at a particular location on a structure. The following Figures 19 through 47 are presented to illustrate typical locations and forms of corrosive attack on bridge details.

Figure 19 shows locations of potential corrosion attack on approach bents supported by steel piles or columns.

View A-A shows pitting at the pile-to-cap joint. Water may collect on the underside of the cap and speed the attack on the piles. View B-B shows uniform corrosion occurring at the base of a steel column. Water draining down the column may collect at this point. View C-C shows pitting in the splash zone of a pile. The splash zone, the region of the pile exposed to alternate wetting and drying, is especially vulnerable to corrosion. This is particularly true in coastal environments.

Figure 20 shows potential corrosion locations on bridge approach towers. Corrosion is most likely to occur at the joints. Deposit attack due to debris accumulation or bird roosting can occur on horizontal or inclined connection plates. Crevice corrosion may be found between back-to-back angles (as on the diagonal members) and between members and gusset plates. Water may collect and accelerate corrosion where plates (especially inclined plates) touch vertical members.

Figure 21 shows locations of possible corrosion attack on stringer spans. Crevice corrosion can occur between the concrete deck slab and the steel stringer. It most likely occurs at locations where the deck is cracked, which allows water to flow through the deck to the stringer. Deposit attack due to debris accumulation or bird roosting may occur at flange to web joints or stiffener to flange connections. Uniform corrosion of the stringer may be found where it is exposed to water spray. Underfilm corrosion can also be found on painted stringer bridges. Inspectors should give close attention to locations where water drainage from the deck may splash on the stringers. This examination is especially important in areas where deicing salt is applied to roadways.

Section A-A on this figure shows how localized corrosion can completely eat through a steel member. Section B-B shows the location of crevice corrosion between the deck and stringer. Section C-C shows loss of web thickness in areas affected by uniform corrosion.

Figure 22 shows locations of potential corrosion attack on through-girder spans.

Crevice corrosion may be found between mating metal surfaces. Section A-A shows crevice corrosion between stiffener angles and the girder web, and between the concrete barrier and steel girder stiffener angles. Detail B shows crevice corrosion between the girder bottom flange and a connection angle.

Pitting may be found where the steel is splashed with water

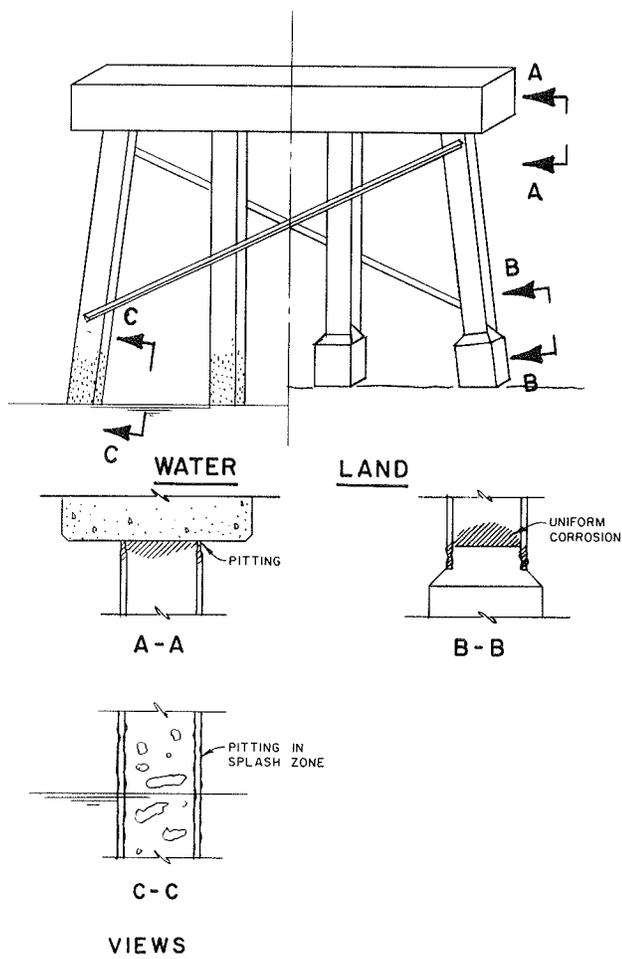


Figure 19. Approach bents.

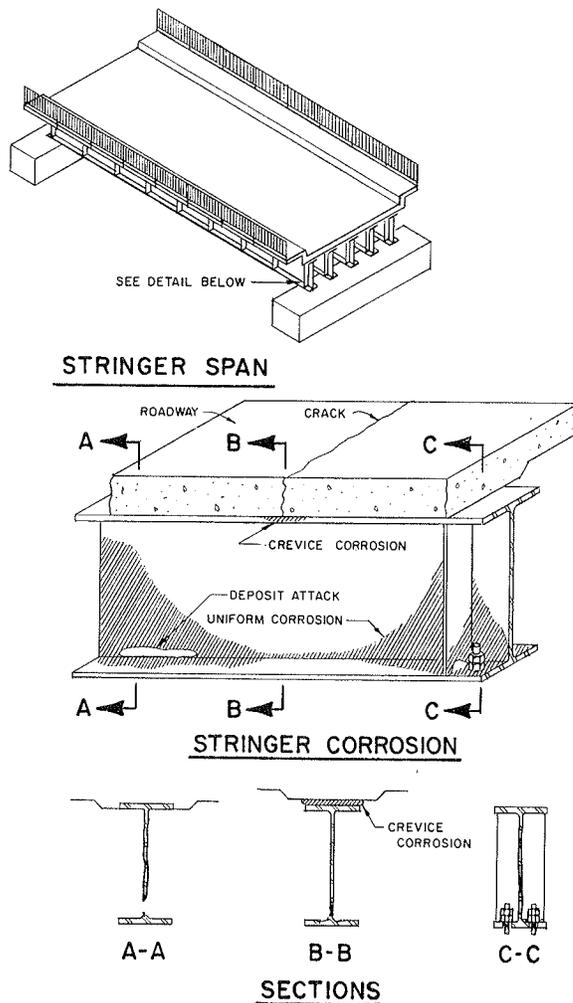


Figure 21. Stringer span.

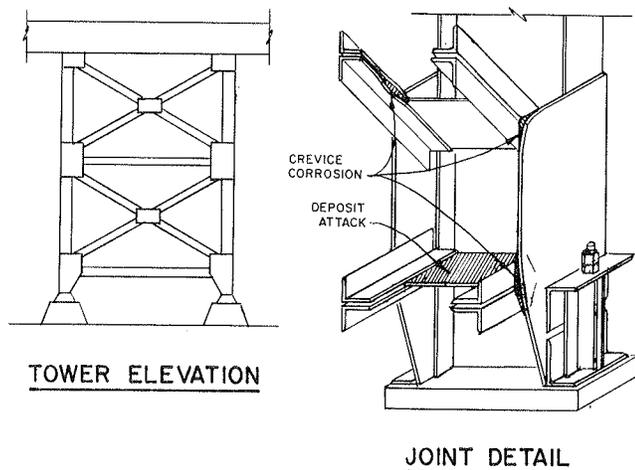


Figure 20. Approach tower details.

from the roadway during storms (see Section A-A). It is especially likely at locations where this water would collect.

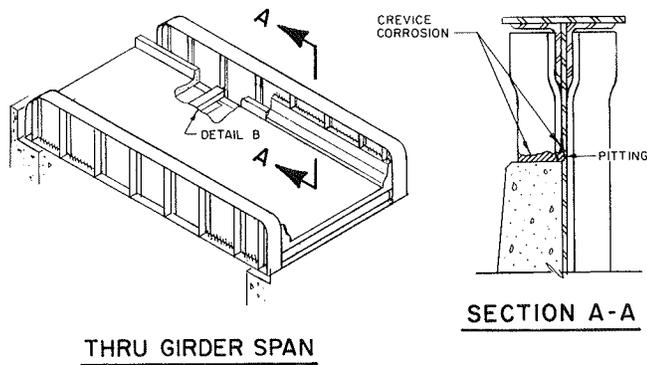
Deposit attack, shown on Detail B, is likely at joints where debris may collect or birds might roost.

Figure 23 shows possible locations of corrosion attack on deck girder spans. See the referenced figures for more information on the specified details.

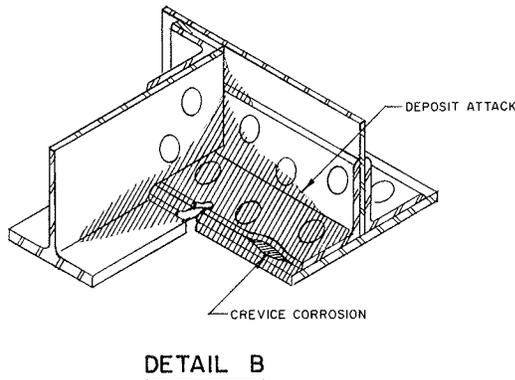
Pitting is likely to occur at low spots on the structure where water accumulates. These locations are typically where stiffeners meet the girder bottom flange. The addition of drip bars to the girder flanges can help prevent corrosion at the stiffener-to-flange joints. Extending drain pipes so that water is not deposited on metalwork can also help prevent corrosive attack.

Uniform corrosion may occur on the girder webs. These are exposed to moisture but do not allow water accumulation.

Proper functioning of bearings is essential to the safety of the bridge. Bearings should be carefully inspected and corrosion damage or bearing "freezing" should be reported immediately. Expansion bearings (typically sliding plate, rocker, or roller design) should be checked to verify that their travel is not restrained by corrosion. Bearings with complicated details that can collect water or debris require closer inspection than those with simpler details.



THRU GIRDER SPAN



DETAIL B

Figure 22. Through-girder span.

Figure 24 shows locations of potential corrosion attack on through trusses. Pitting may occur on exposed members where the surface protection is abraded by debris such as sand or salt stirred up by traffic. Deposit attack as shown on Section A-A and Detail B may be found at locations where debris accumulates. Crevice corrosion is possible between mating surfaces of built-up members.

Figure 25 shows potential locations of corrosion attack on truss connections.

Crevice corrosion can be seen on Sections A-A, B-B, and D-D. It occurs between members and the gusset plate. It could also occur at mating surfaces of the components of built-up members.

Section C-C shows damage from deposit attack. The gusset plate at Section C-C has been eaten through by corrosion. This is not uncommon in cases of severe deposit attack. Truss connections are prone to deposit attack because of their complexity, which promotes both debris accumulation and bird roosting.

Figure 26 shows possible corrosion locations on truss chords. Crevice corrosion can occur between members in contact with one another, such as eyebars or eyebars and pin collars in pin joints. The swelling of pin collars is an indicator that corrosion exists between the pin and collar. Crevice corrosion could also be found where lacing or cover plates are connected to box members. Fretting is possible where eyebars vibrate against truss verticals or other members. Pitting can attack cover plates if water is likely to collect on them.

Any occurrence of corrosion on pins or pin joints should be carefully noted because these areas are critical to structural safety.

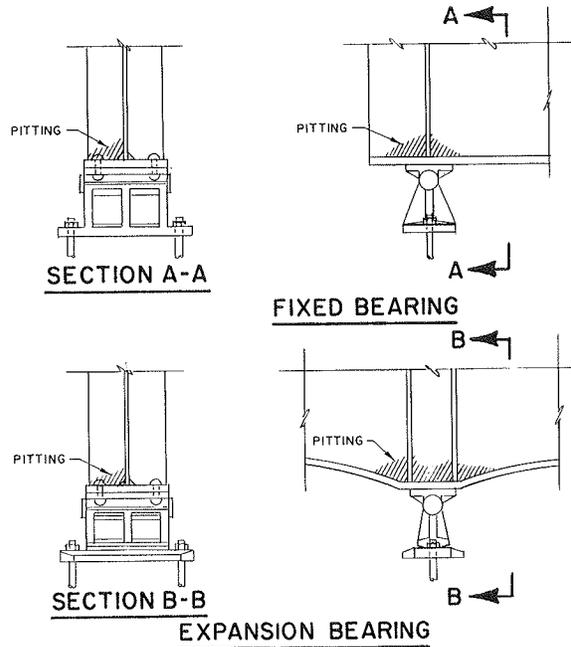
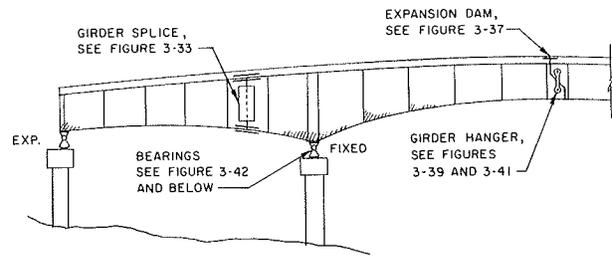


Figure 23. Deck girder span.

Figure 27 shows locations of potential corrosion attack on truss floorbeams. Pitting may occur on surfaces that are exposed to water accumulation, such as the floorbeam bottom flange and the floorbeam top flange adjacent to the roadway. Uniform corrosion might be found on the floorbeam webs. This surface is exposed to moisture but would not collect water. Deposit attack may be found at connections prone to debris accumulation or bird roosting. Crevice corrosion may occur between mating surfaces.

Figure 28 shows potential types of corrosion attack on stringer-floorbeam connections. Crevice corrosion, pitting, and deposit attack can all occur at these locations.

Crevice corrosion may occur at mating surfaces of connecting members.

Pitting is shown on each of these details. It is most likely to occur at locations where waste can collect against the steel.

Deposit attack locations are indicated on fixed connection C and expansion connection B. This form of corrosion occurs at locations prone to debris accumulation or bird roosting.

In general, the more complex the detail, the more likely it is to promote corrosion. Complex details gather more debris and have more locations where water can collect. Complex details

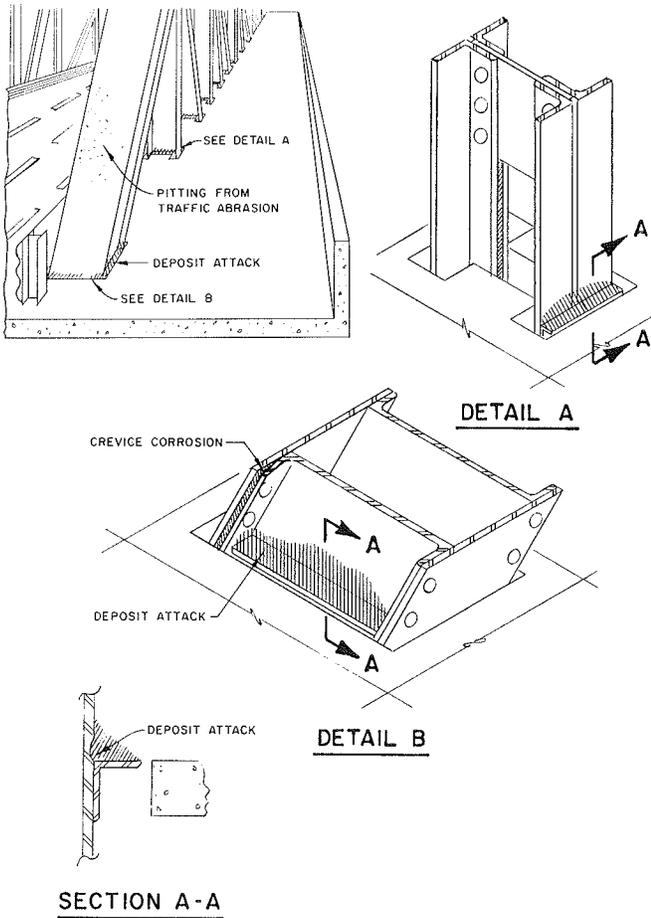


Figure 24. Through-truss details.

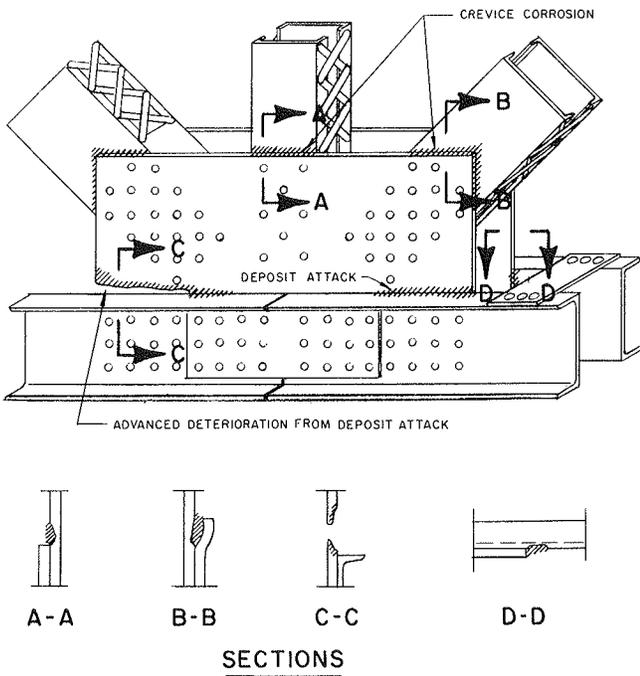


Figure 25. Truss connection details.

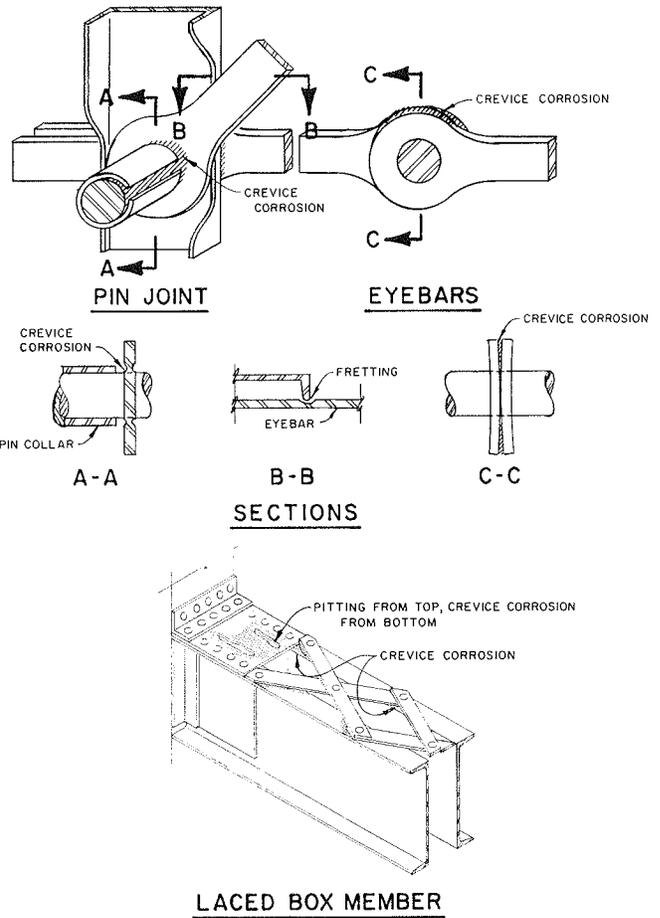


Figure 26. Truss chords.

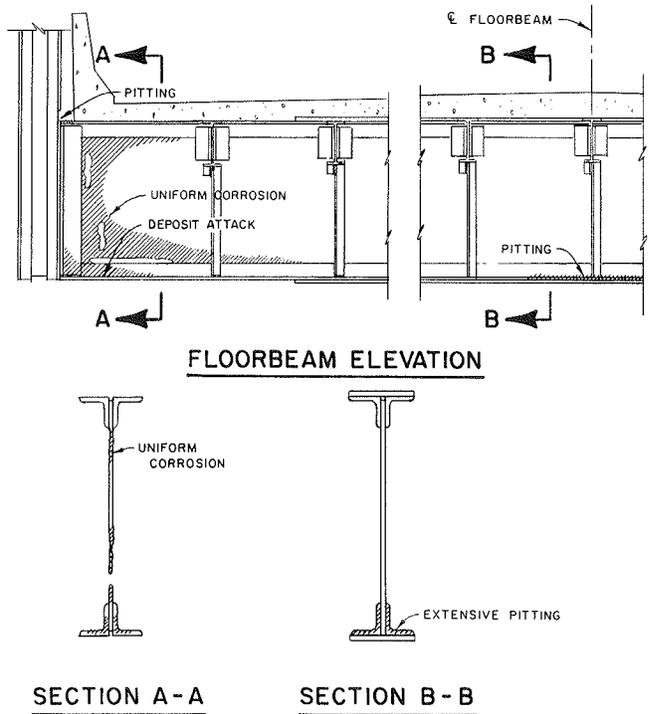


Figure 27. Truss floorbeams.

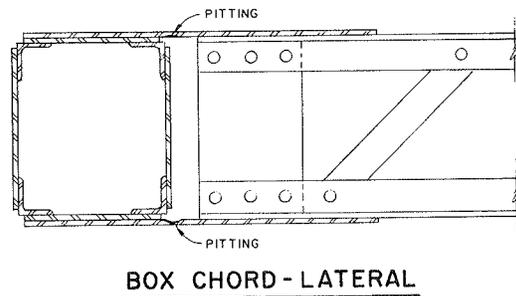
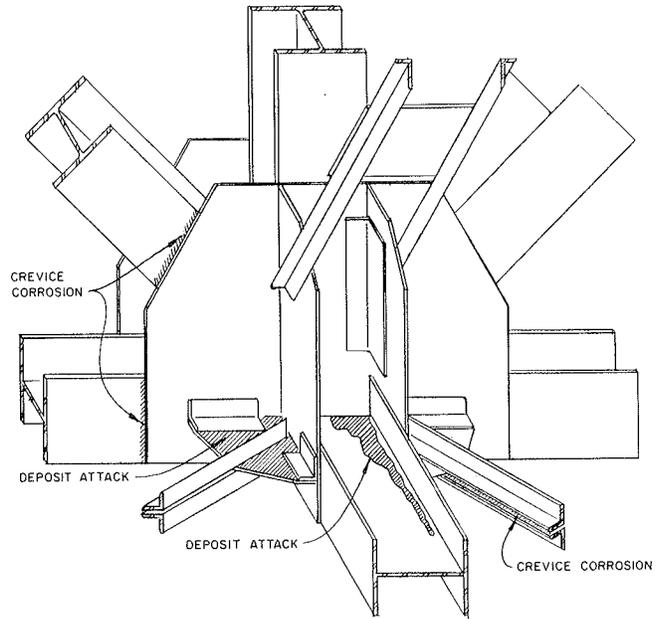
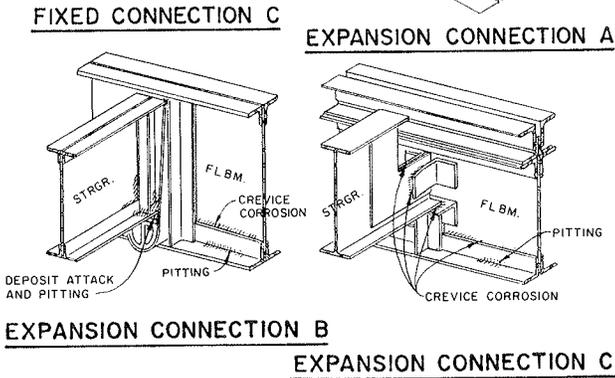
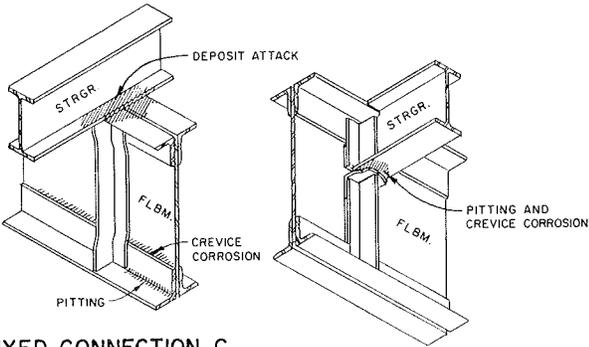
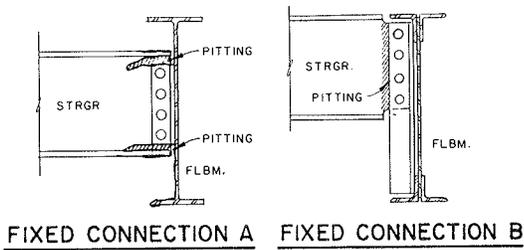


Figure 28. Stringer floorbeam connections.

Figure 29. Truss connection bottom laterals.

are also more difficult to inspect. Stringer-floorbeam connections as shown in expansion connections B and C are more complicated and should be given extra attention during corrosion inspections.

Figure 29 shows locations of potential corrosion attack on truss bottom chord connections. Crevice corrosion can be found between members framing into the joint and gusset plates. It also may occur between back-to-back angles. Deposit attack from debris build-up or bird roosting is likely on horizontal surfaces such as gusset plates or member flanges. Pitting may be found at locations exposed to splashing from roadway runoff or at locations where water can collect.

Figure 30 shows potential corrosion locations on truss top laterals and sway frames. Crevice corrosion can occur between back-to-back angles and at mating surfaces of connections. Pitting can be found at locations where water collects on the steel.

The formation of pack rust between surfaces may cause sufficient swelling to spread components and may even cause fastener failure.

Figure 31 shows potential corrosion locations at stringer-to-bottom lateral connections.

Section A-A shows that pitting can occur wherever water collects on a member—even on its underside. Weld decay due

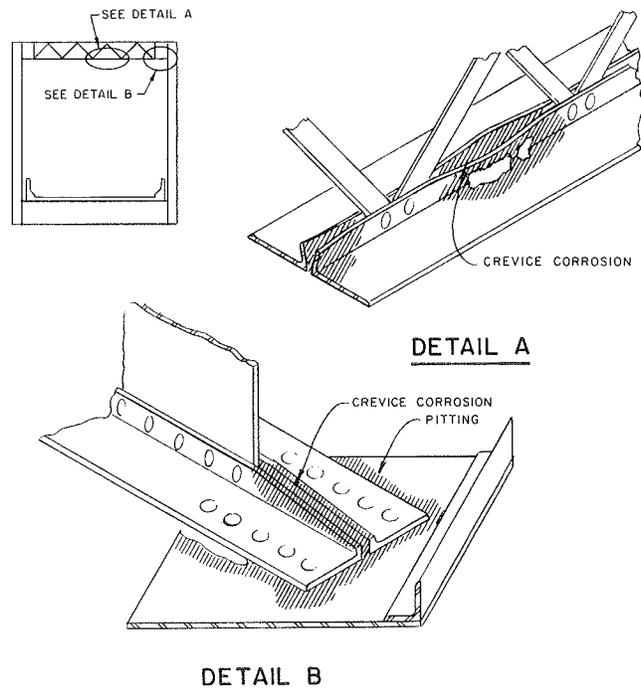


Figure 30. Truss top lateral sway frame.

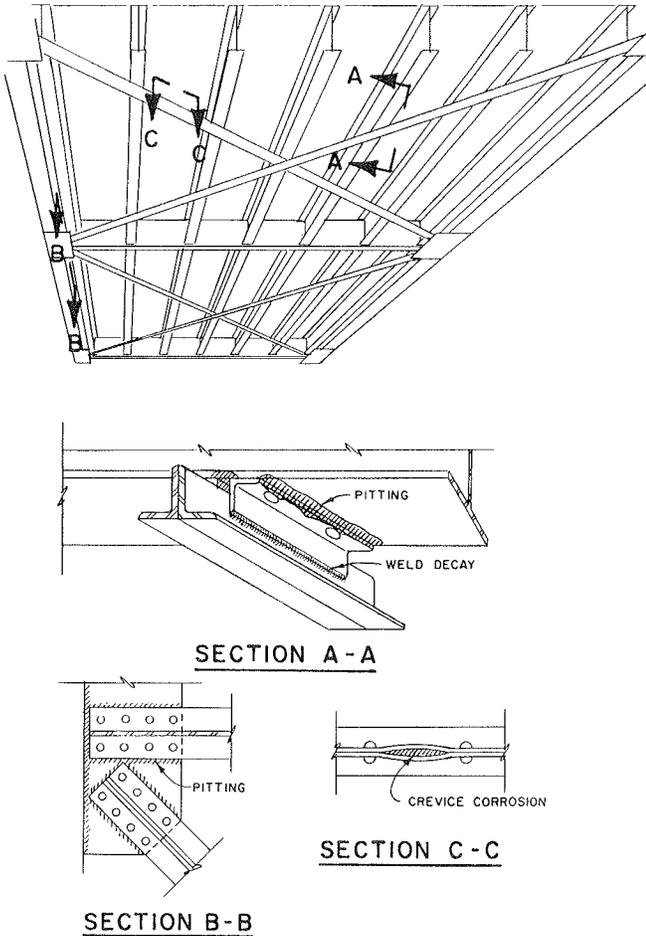


Figure 31. Stringer bottom lateral connections.

to improper welding techniques may be found at welds. It is more common at field welds than shop welds.

Section B-B shows pitting occurring at a bottom connection plate where water can collect.

Section C-C shows crevice corrosion between back-to-back angles of laterals. Crevice corrosion may also be found between laterals and gusset plates.

Figure 32 shows some of the potential locations for corrosion attack on suspension bridges. Suspension bridge cables are susceptible to several forms of corrosion and require thorough periodic inspections.

Crevice corrosion is a frequent form of attack on cables. It can occur between suspender cables and the cable band. It may attack the cable inside its exterior sheathing. Crevice corrosion might attack where the cable enters the anchorage.

Pitting could occur at the suspender connection where the member is exposed to spray from roadway runoff and erosion from particles stirred up by traffic. Pitting may be found at the anchorage at locations where water can accumulate.

Other details on suspension bridges should be inspected in an appropriate manner.

Figure 33 shows location of potential corrosion attack on cable-stayed bridges. For these structures, particular attention should be given to the cables. Cracks in the cable sheathing can allow moisture to attack the cables. Crevice corrosion can form

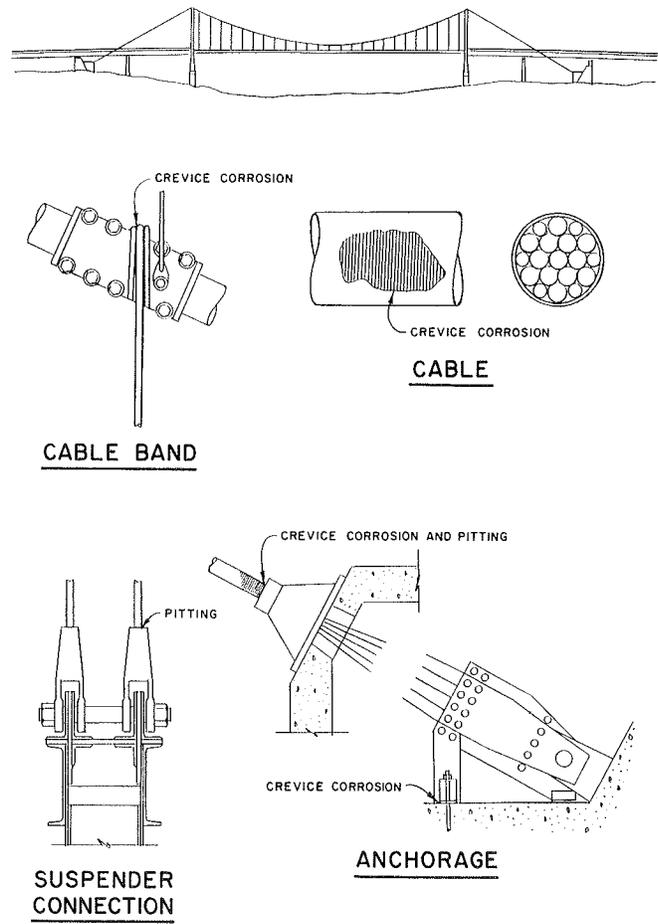


Figure 32. Suspension bridge.

between the strands, and stress corrosion may occur within the individual wires. Crevice corrosion and pitting can occur at the anchorage because of water runoff and debris accumulation, as well.

Other details of cable-stayed bridges should be inspected for corrosion in an appropriate manner.

Figure 34 shows potential corrosion locations at girder-lateral connections. Crevice corrosion may occur between the girder and gusset plate or the lateral and gusset plate. Pitting may occur on the gusset plate where water can accumulate. Extensive pitting can eat away the fastener heads and cause the failure of the connection.

Figure 35 shows potential corrosion locations at a steel member splice. Crevice corrosion may occur between the member ends or between splice plates and the members to be spliced. Pitting can occur on the member bottom flange where water can collect.

Figure 36 shows potential locations of corrosion attack on steel roadway grating. Uniform corrosion may occur on the grating where it is exposed to roadway runoff. Pitting may be found where the grating joins the supporting stringer, because debris and water can collect at this location. Deposit attack is likely on top of the supporting stringer where debris accumulates.

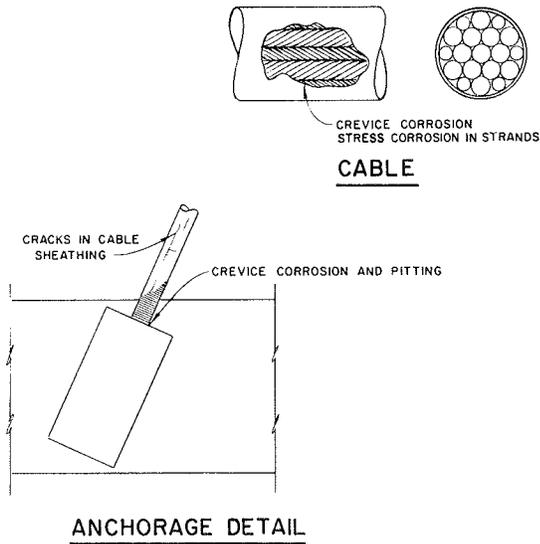
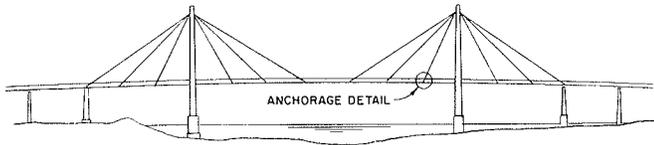


Figure 33. Cable-stayed bridge.

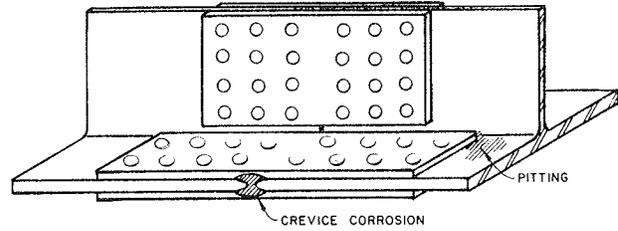


Figure 35. Splices.

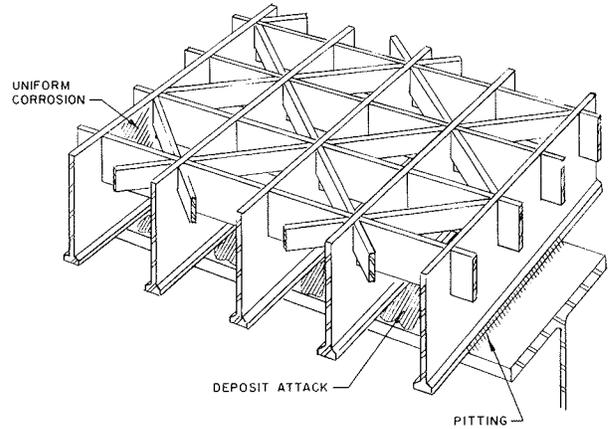


Figure 36. Roadway steel grating.

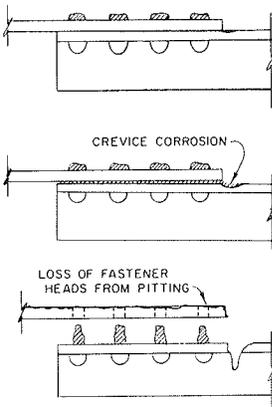
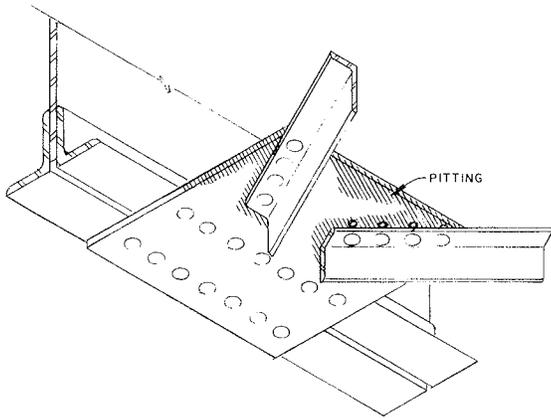


Figure 34. Girder lateral fasteners.

Figure 37 shows corrosion attack on a timber-deck bridge. Crevice corrosion can occur where the timber rests on steel. Crevice corrosion may also be found where a metal restraining clip is attached to the stringers. Pitting occurs where water collects while draining through the structure. Pitting is likely to be found on the steel stringer top flange between timbers and on the stringer or floorbeam bottom flange.

Figure 38 shows potential corrosion locations between a deck slab and an orthotropic bridge deck. Water seeps through the cracks shown in the deck slab, causing crevice corrosion to occur between the concrete slab and the steel deck plate.

Figure 39 shows potential corrosion locations on roadway expansion dams. Expansion dams are prone to corrosion because of their exposure to water, abrasion, and debris accumulation. Crevice corrosion can occur between finger plates and base angles and at sliding plate joints. Pitting may be found at locations exposed to runoff flow or accumulation. Expansion dams can be a major source of structural problems and should be carefully inspected and maintained.

On finger-type expansion dams with excessively long fingers as shown in this figure, crevice corrosion can cause the fingers to lift, which removes the support of the finger tips. This will result in unexpected bending of the fingers and premature failure.

On sliding-plate-type expansion dams, excessive corrosion between plates can cause unintended fixity of the dam plates, leading to failure of the dam.

Figure 40 shows possible locations for corrosion attack on roadway railings. Pitting may occur on the rail because of splashing from roadway runoff or loss of surface coating by abrasion from particles stirred up by traffic. Crevice corrosion is possible

where the concrete deck is in contact with steel members. Crevice corrosion may also be found at connection angles or other locations where steel members are joined. (Galvanic corrosion is also possible if the rail is a different material from the rest of the bridge. See Figure 46).

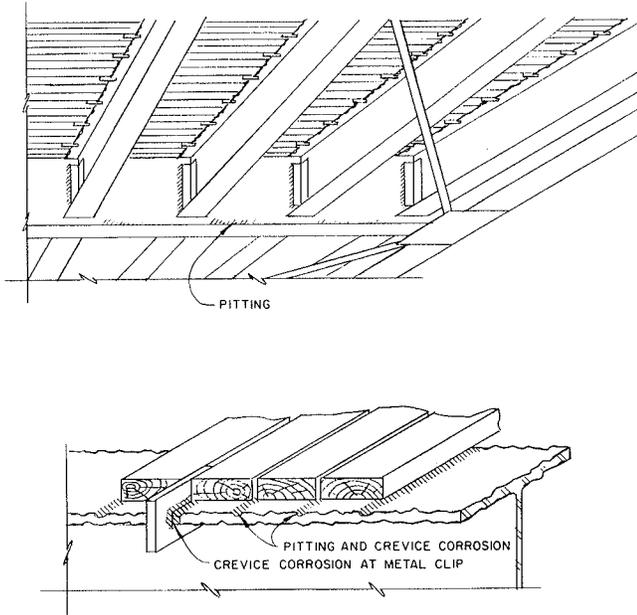
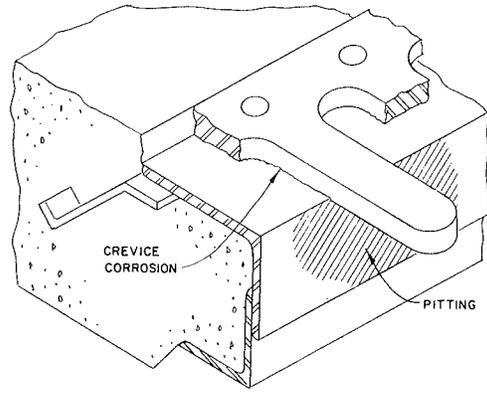


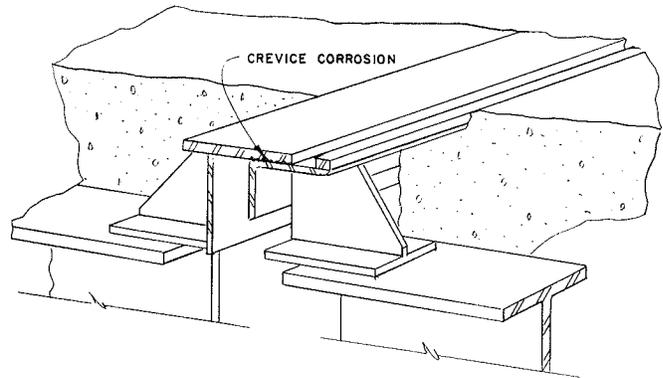
Figure 37. Roadway timber deck.



DETAIL A

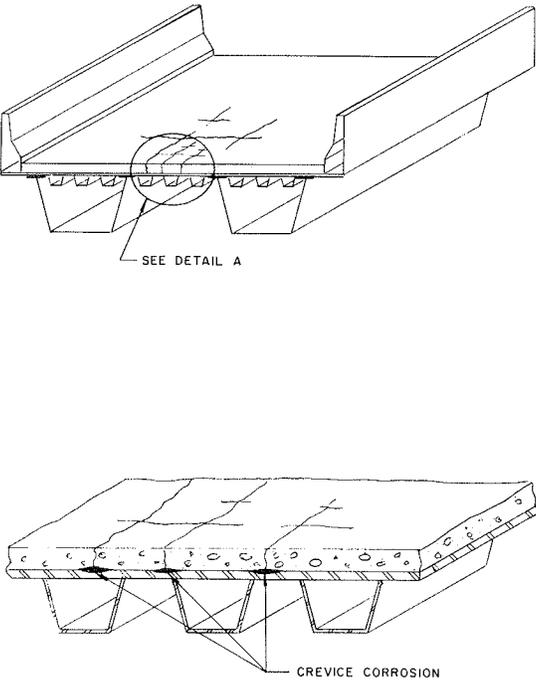
Figure 38. Roadway orthotropic deck.

FINGER TYPE DAM



SLIDING PLATE DAM

Figure 39. Roadway expansion dams.



SECTION A-A

Figure 40. Roadway railing.

Figure 41 shows forms of corrosion attack on pin joints. Crevice corrosion is possible between eyebars and pin caps or nuts, and between eyebars and the pins themselves. Crevice corrosion between closely packed eyebars can produce a build-up of corrosion products ("pack rust"). This pack rust expands as shown in the detail at the bottom of the figure. The expansion forces the pin washer against the cotter pin, which can shear off, leading to failure of the joint as the pin works out of the hole.

Because pin joints are critical to the safety of the structure and are prone to this sort of deterioration, it is important that they be inspected regularly. Any deterioration or "freezing" of joints should be reported immediately.

Figure 42 shows the progression of deterioration of rivets and bolts. The top detail shows the initial condition of these fasteners. After a period of corrosive attack, the fasteners appear as shown in the middle detail. Note the deterioration of the top end of the fasteners. This is due to pitting from water accumulating on or deposits collecting atop the connection. The rivet head deterioration resembles the blossoming of a flower, with the shank under the blossoming pieces in the shape of a cone. The final stage—advanced deterioration—is shown at the bottom of the figure. The bolt has failed because of pitting or even stress corrosion, and has fallen out of the connection. The rivet has lost its head and clamping action.

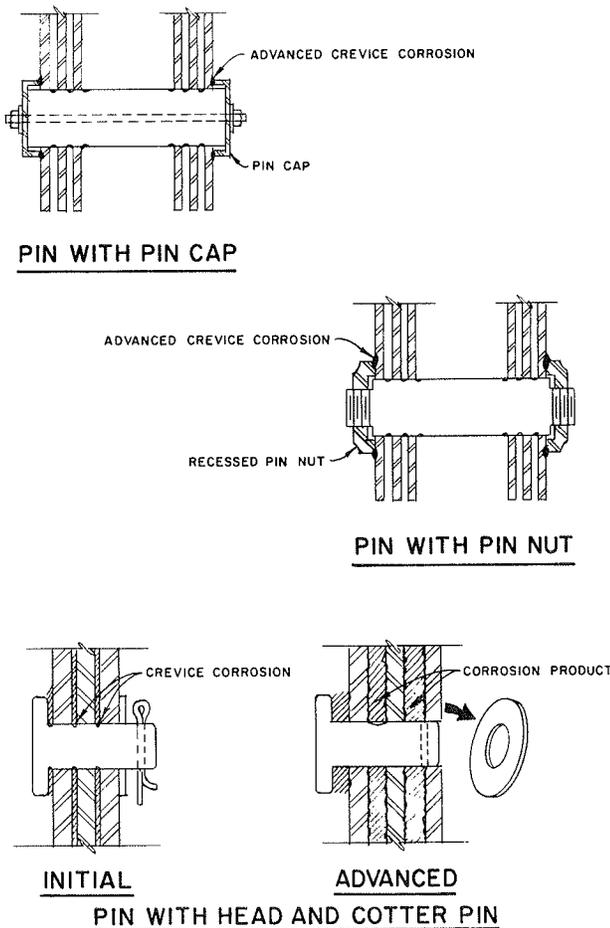


Figure 41. Fasteners pins.

Figure 43 shows a pin-hanger assembly in a cantilever girder bridge. Fixity of the pinned ends of a hanger due to corrosion may result in fatigue cracks in the hanger or the pin. Built-up corrosion product between the elements of the pin-hanger assembly may cause the hanger to shift and overstress the pin. (This problem is discussed in more detail in Chapter 6.)

Figure 44 shows cracking of substructure as a result of a frozen expansion bearing. Other possible damage includes bent or broken anchor bolts; misalignment, distortion, buckling or cracking of connecting members; and damage to the bearing. (This problem is discussed in more detail in Chapter 6.)

Figure 45 shows a potential location of fretting corrosion at a stringer joint. Slight movements of the stringer will lead to fretting between the stringer and the supporting member.

Figure 46 shows potential locations of galvanic corrosion on a bridge. Galvanic corrosion is possible at any location where dissimilar metals are connected. In this case, galvanic corrosion may occur where the aluminum handrail is fastened to the steel post. Galvanic corrosion can also occur where galvanized steel is fastened to bare steel, especially where galvanized steel is fastened to weathering steel.

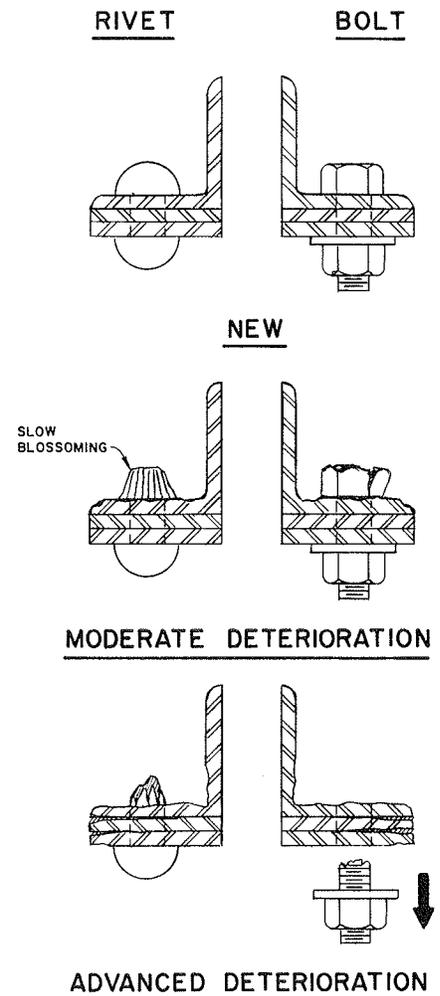


Figure 42. Fasteners rivets and bolts.

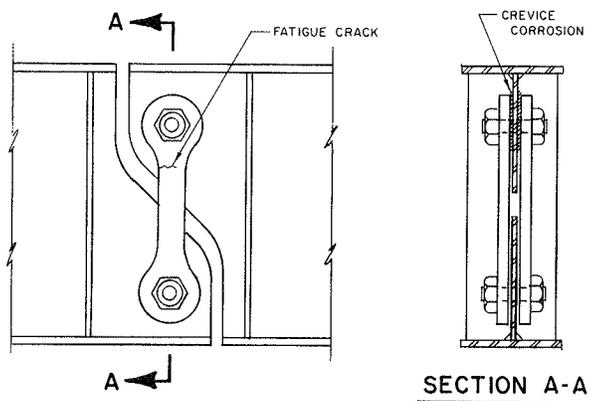


Figure 43. Unintended fixity hangers.

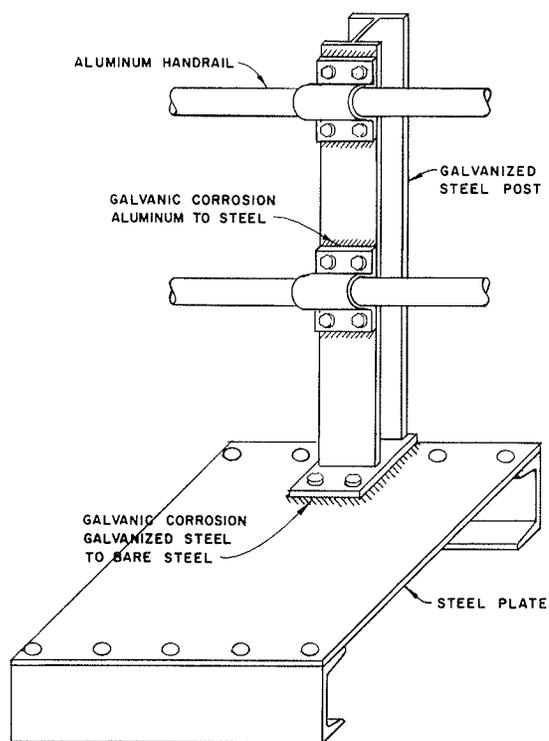


Figure 46. Galvanic corrosion.

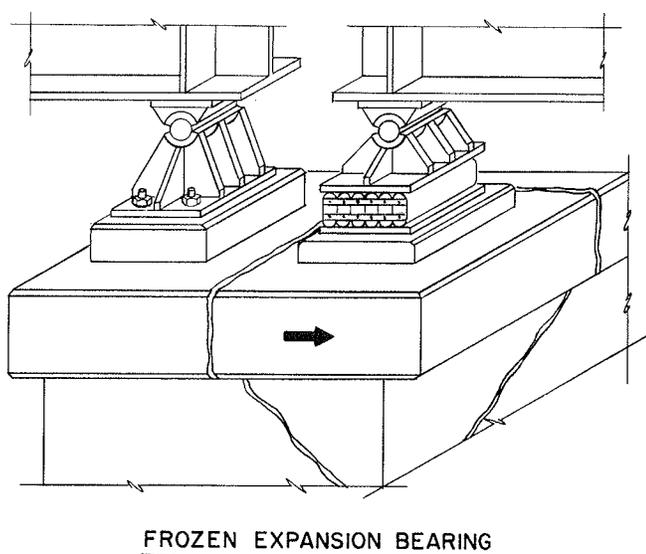


Figure 44. Unintended fixity bearings.

Figure 47 shows the effects of stress corrosion cracking on a connection. High-strength bolts are highly stressed by tightening during installation. If the connection is in a corrosive environment, stress corrosion cracking may attack the bolts. Eventually the bolts fracture as shown and may fall out, leaving the open holes shown on the connection detail.

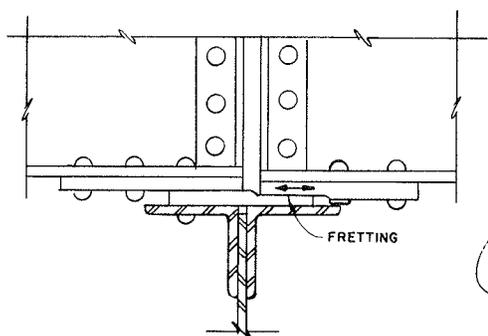


Figure 45. Fretting.

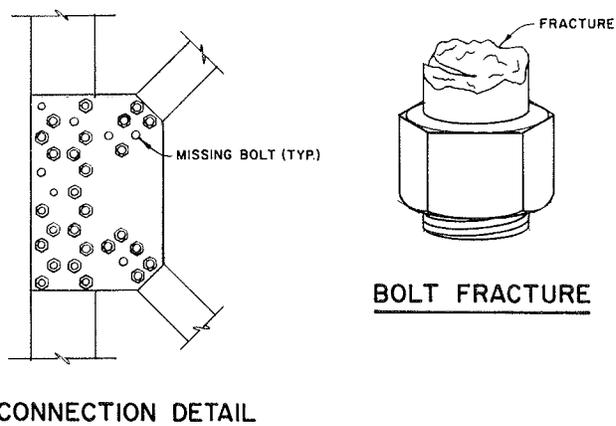


Figure 47. Stress corrosion cracking.

TYPES AND TECHNIQUES OF CORROSION INSPECTION

4.1 INTRODUCTION

The corrosion inspection of steel bridges is similar to the well-established maintenance or safety inspection of bridges generally performed biannually under FHWA guidelines and reported on SI&A forms. The only difference is that the entire emphasis of the inspection is on corrosion. The corrosion inspection is *not* intended to alter the existing National Bridge Inspection Standards but rather to *supplement* them. The corrosion inspection identifies types of corrosion, records their effects, and appraises corrosion conditions. It is a specialized inspection involving bridge inspectors with added training and knowledge of corrosion.

The level of corrosion inspection is generally determined by the government department needing the information or by a private owner with possible guidance from a consulting engineer. The inspection level is based on the intent of the inspection, the amount of information needed to evaluate a specific requirement, and the amount of time and money available.

In Part II of this report, "Office Evaluation of Corrosion Effects," corrosion evaluations are divided into two levels. Level I uses relatively simple analysis methods to determine the structure's load-carrying capacity. Level II is a more exhaustive approach using sophisticated analysis techniques to evaluate the bridge capacity. The information needed for a Level I evaluation is obtained from a cursory or general field inspection. (This information may be gathered during the normal biennial inspection each structure should receive.) From these findings, either the basic overall requirement can be satisfied or a determination made that a more comprehensive inspection is needed. A Level II evaluation involves a multidisciplinary team of inspectors, corrosion specialists, and office evaluators. A detailed field inspection is needed to support this level of evaluation.

Corrosion inspection forms for use in recording inspection results are presented in Chapter 5 of the field inspection guidelines.

4.2 LEVEL I—CURSORY INSPECTION

The cursory inspection provides an overview of corrosion conditions without detailed examination of deficient areas or the use of sophisticated tools and equipment, by using visual observation and experience to evaluate the conditions. The cursory inspection answers such basic questions as: (1) Is extensive corrosion present? (Without actually measuring metalwork losses.) (2) Is corrosion global (found throughout the entire bridge) or localized? (3) Has corrosion caused misalignment of parts, shifted bridge components, or frozen members intended to move? (Without actually measuring the amount of displacement or fixity.)

The cursory inspection offers overall observation, but lacks the close scrutiny that would quantify conditions or find remote effects. Visual estimation of physical losses can be made without measurement by using percentages of section loss or equivalent section thickness loss.

The primary use of the cursory inspection is to determine in a quick and inexpensive manner the overall bridge condition and whether a more extensive Level II evaluation is needed. A cursory inspection may not provide all the information needed for a Level I office evaluation. The cursory inspection report is a brief statement estimating the general and worst conditions, supplemented by the Corrosion Inspection Form and selected photographs. A cursory corrosion inspection can be done at the same time as the routine maintenance inspection of a bridge.

4.3 LEVEL I—GENERAL INSPECTION

The general corrosion inspection is used for a Level I evaluation. The inspection is a "hands-on" approach in which bridge members that are accessible without the need for specialized equipment are climbed and inspected, and where random or spot measurements are taken to quantify the extent of metalwork losses. Both general and worst case conditions are checked. A combination of estimating and measuring is used for determining the extent of corrosion damage.

A general inspection is consistent with the need for an overall condition determination with a sampling of conditions, but lacks the detail of a complete survey. Also, a general inspection provides enough information to determine the need for a Level II evaluation.

The general inspection report includes descriptions of bridge conditions (by component), spot measurements of worst case conditions, and a narrative description of the corrosion found. It is supplemented by the Corrosion Inspection Form and photographs.

4.4 LEVEL II—DETAILED INSPECTION

The detailed inspection is an in-depth inspection covering all corrosion aspects of all bridge elements. If necessary, special access-gaining equipment is used to put the inspector in a "hands-on" position to closely observe each member and make detailed measurements of all metalwork losses. Metalwork surface cleaning is performed, as required, to make accurate surface measurements and precise determination of metalwork losses.

The detailed inspection provides the full range of information required for a complete evaluation of the bridge. The detailed inspection report is a total account of the corrosion conditions on the bridge. Metalwork losses are described in detail. Bridge component conditions are rated, using the Corrosion Inspection Form. A narrative description of the inspection findings and photographs are used to illustrate the inspection results.

4.5 INSPECTION PERSONNEL

The inspection personnel performing corrosion inspections should have the same minimum qualifications as required for the bridge maintenance inspection program. These qualifications include being in good physical condition; a minimum of a high school education; training in bridge maintenance inspection plus added training in corrosion; the physical ability to climb structural steel without difficulty; and the skills needed to inspect, sketch, report, photograph, and measure details. The qualifications of an inspector should be matched to the level of corrosion

evaluation required. For Level II evaluations, corrosion experts may be included in the inspection team.

The inspector should wear the appropriate personal protection equipment consistent with the task being performed. Such items as a hard hat, goggles, face shield, reflective vest, life preserver, life belt, and nonslip shoes are all equipment recommended for use. Many of these items are dictated by either the inspecting agency or by governmental regulations.

Inspection team leaders for Level I inspections should meet the minimum requirements for team leaders of the National Bridge Inspection Standards. Team leaders for Level II inspections should also have specialized training in corrosion inspection.

Precautions should be taken when working in confined areas to ensure that sufficient ventilation and oxygen are present. If questionable, the air should be tested before entering these areas.

4.6 ACCESS EQUIPMENT

For both Level I and Level II corrosion inspections, the inspection team leader must determine the types of equipment needed to gain access to all areas of the bridge. Access equipment normally used for a detailed maintenance inspection should be satisfactory for a corrosion inspection.

Precautions must be taken when using motorized platforms to ensure that the inspector understands and can safely operate the equipment within its recommended range. Associated with the use of motorized equipment is the proper warning signing and signals for traffic control to assure safety for the inspectors as well as traffic. The *Manual on Uniform Traffic Control Devices (MUTCD)* should be consulted for specific traffic control procedures.

Underwater inspection of bridge members may be required to evaluate conditions of steel piles or other substructure components. Guidelines for this can be found in *Underwater Inspection of Bridges*, FHWA-DP-80-01.

4.7 STEEL CLEANING METHODS

Steel cleaning may be necessary to allow the level of corrosion inspection required. Debris and corrosion product can mask defects and prevent accurate evaluation of an element's condition. For general inspections the inspector should be prepared to use a whisk broom, putty knife, and chipping hammer to clean metalwork as needed to make selective measurements.

For detailed inspections, the same cleaning equipment may be sufficient for bridges in relatively clean condition and only having minimal corrosion. For bridges with heavy debris accumulation and large amounts of corrosion product, cleaning may require compressed air to blow off debris, sand blasting or water blasting, or the use of needle guns. Water blasting is especially effective in cleaning and does not disturb the steel surface for examination of the roughness profile (see Figures 48 and 49).

At times the extent of debris accumulation causes members to be buried from view and it becomes necessary to have contract services physically dig out debris to expose the bridge elements. This is most common in the area of bridge abutments.

4.8 METALWORK LOSS MEASUREMENTS

The measurement of metalwork losses should be consistent with the level of evaluation of the bridge. For a cursory inspection,

estimation of a loss from a trained eye is close enough. This can be in the form of a visual estimate, reporting the loss either in terms of the thickness and metalwork width lost or as a percentage of the original section. Experienced inspectors may be able to make an initial estimate of how critical the section loss is to the member capacity. However, it is always wise to report all losses and then determine their consequences in the office evaluation.

For general inspections, loss measurements of an area of metalwork may be estimated (after cleaning) based on caliper measurements, straight edge less pitted areas, equivalent areas, or a series of D-meter (ultrasonic) measurements. The level of accuracy in measurement should be consistent with the intended use of the information.

For detailed inspections, loss measurements may be accurately obtained from caliper readings, D-meter readings, or the use of more sophisticated instruments.

4.9 ADJECTIVE CONDITION RATING

For uniformity in the description of corrosion conditions during the field inspection and for proper interpretation by the office evaluator, an adjective condition rating system is used. This system is based on a scale of 0 to 9, with 9 being the best condition and 0 being an unsatisfactory rating. This condition rating scale is shown in Table 1.

To illustrate the condition rating system a series of photographs (Figures 50(a), 50(b), 50(c), 50(d), and 50(e) showing elements in various stages of deterioration are presented.

Table 1. Condition rating scale.

Rating	Description
N	Not applicable
9	Excellent condition
8	Very Good condition - no corrosion
7	Good condition - minor corrosion with no significant metalwork loss
6	Satisfactory condition - minor corrosion with minor metalwork losses but element functioning as intended
5	Fair condition - moderate corrosion with element functioning at a reduced level
4	Poor condition - major corrosion with element functioning at a marginal level
3	Serious condition - serious corrosion with element functioning at an inadequate level
2	Critical condition - severe corrosion with element not functioning as intended
1	"Imminent" Failure condition - extent of corrosion severe, requires determination if repairable
0	Failed condition - extent of corrosion renders element beyond repair

CHAPTER 5

CORROSION INSPECTION REPORT

5.1 INTRODUCTION

The corrosion inspection report should be consistent with the level of the inspection effort, i.e., a detailed inspection requires

a detailed report while a cursory inspection requires a brief report. This chapter presents examples of suggested report forms to be used for the various report levels. An example of a cursory, general, and detailed report is presented for the same bridge. There are many other report formats that can be used—the main objective is to provide useful information consistent with the level of inspection effort that will meet the needs of the party requesting the information.

Report elements suggested for the levels of inspection effort should include the following as a minimum:

Level I—Cursory Inspection:

- Structure identification
- Environmental conditions
- Steel protective system
- Narrative covering general and worst case conditions
- Photographs

Level I—General Inspection:

- Structure identification
- Environmental conditions
- Steel protective system
- Survey of corrosion conditions by members with selective estimates of metalwork losses
- Narrative covering general and specific case conditions
- Photographs

Level II—Detailed Inspection:

- Structure identification
- Environmental conditions
- Steel protective system
- Survey of corrosion conditions by members
- Sketches and detailed measurements of metalwork losses
- Narrative covering general and specific case conditions
- Photographs

5.2 CORROSION INSPECTION REPORT FORMS

A simple and uniform method for reporting corrosion conditions is to use a set of forms that can be adapted for the various levels of inspection. The following Corrosion Inspection Report forms (Figure 51) have been developed for the indicated purposes:

Summary—For identification of structure, environment, steel protective system, overall conditions by major structural elements, and recommendations.

Survey—For rating each major structural component per span.

Metalwork Losses—For listing member losses by location and sketch, per span.

Narrative—For the discussion of conditions both general and specific, per span.

Photographs—For support of conditions noted and use by the party requesting the inspection to review the conditions reported.

5.2.1 Corrosion Inspection Report—Summary

This form is the cover sheet for each level of inspection report. The bridge identification should be that used by the requesting

party. For example, a state highway department might use the identification normally found on SI&A forms.

Environmental information should include descriptions of site conditions that would affect the steel, such as: arid, dry, wet, coastal, salt mist, rural, urban, industrial, deicing agents, and so on.

Paint system identification and date when last painted are useful in evaluating the compatibility of that system with the site environment.

A short narrative summary is presented for each of the bridge main segments.

Recommendations should be given for repairs, maintenance, and further needed investigation. A cursory report may recommend that a detailed inspection be performed.

5.2.2 Corrosion Inspection Report—Survey

This form is intended to be used as a part of a general or detailed inspection report. It can be used as a summary for an entire bridge or multiple sheets can be used and only the applicable portions are used per span. The survey is intended to rate each bridge element and list the types of corrosion affecting that element.

5.2.3 Corrosion Inspection Report—Metalwork Loss

This form is intended to be used in a detailed inspection for reporting all metalwork losses by listing the losses by member and providing sketches as needed to ensure that the reviewer has sufficient information for evaluation.

5.2.4 Corrosion Inspection Report—Narrative

This form is intended to be used at each level of inspection to report the general and specific corrosion condition in a narrative. The narration is useful in reporting observations concerning the cause of deterioration and the immediate and long-term effects if corrective measures are not taken. The extent of narration should be consistent with the level of the inspection.

5.2.5 Corrosion Inspection Report—Photographs

This form is to be used with each report to support conditions observed and provide the reviewer a means for concurrence with the reported information.

5.3 EXAMPLE BRIDGE INSPECTION

For illustration of the various corrosion inspection reports a hypothetical bridge inspection is used. The bridge is located over a small river in a rural environment with no industrial activities nearby. The climate is generally humid with frequent periods of rain. Ice and snow conditions exist for which deicing salts are used. The bridge consists of a simple span 360 ft long through-truss over the river supported by concrete piers. The approaches are 75 ft long steel beam spans supported by concrete caps with steel pile bents.

Because of the condition of the bridge deck and known corrosion activity the owner has requested that a corrosion inspection be performed to assist in evaluation of the load capacity and determination of the extent of repair efforts needed.

Summary sheets (Figures 52, 53, 54) for each level of inspec-

tion for this example bridge follow. Complete corrosion inspection reports for the example bridge are contained in Chapter 8. It should be noted that actual corrosion inspections could be much more detailed than the examples given.

CORROSION INSPECTION REPORT - SUMMARY								
INSPECTION: <u>CURSORY</u> GENERAL DETAILED				DATE: 4/15/89		-		
BRIDGE NAME OR FEATURE U.S.A. River Bridge				STRUCTURE NUMBER 1234-56-7890				
OWNER State DOT		CUSTODIAN DOT		BUILT 1952				
SPANS-TYPE-LENGTH 75' Rolled beam Approaches: 360' 12 panel through truss								
ENVIRONMENT Humid, Rural								
PAINT SYSTEM Green vinyl over zinc				LAST PAINTED 1978				
OVERALL CONDITION R=6 CONTROL ITEM Frozen Bearings, Leakage								
APPROACHES R= 6 CORROSION: LOCATION-TYPES-EXTENT								
Beam spans in fair condition due to extensive corrosion on beam ends.								
E4 (LS) has end web hole. Debris blocks further viewing. Deicing salts are affecting all beam ends. Bearing conditions are unknown. Need to clean off concrete caps for full inspection.								
MAIN BRIDGE R= 7 CORROSION: LOCATION-TYPES-EXTENT								
Main bridge is in generally good condition. Crevice corrosion is common at gusset plates and seams of built-up members, but no section loss is visible on any primary member. Truss expansion bearings show heavy corrosion.								
FLOOR SYSTEM R= 6 CORROSION: LOCATION-TYPES-EXTENT								
Main bridge floorbeams are in generally good condition. Main bridge stringers are in fair condition, with section loss near member ends, especially near roadway joints.								
SUBSTRUCTURE* R= 6 *STEEL CORROSION: LOCATION-TYPES-EXTENT								
Approach pile bents have crevice corrosion at pile-cap interface with some section loss.								
MISCELLANEOUS R= 6 CORROSION: LOCATION-TYPES-EXTENT								
Bridge needs cleaning. Debris has accumulated on main bridge bottom chord and around bearings on approaches. Deck leaks through cracks and joints.								
RECOMMENDATIONS								
Bridge needs at least a general corrosion inspection, with particular attention paid to deck leaks and bearings.								
9-EXC. NO REPAIR	8-V.GD. MIN MT	7-GOOD MAJ MT	6-SAT. MIN REHAB	5-FAIR MAJ REHAB	4-POOR IMMED R	3-SERIOUS URG. R	2-CRIT. URG. R	1-IMM.FAIL. CLOSE

Figure 52. Corrosion inspection summary sheet—cursory level.

CORROSION INSPECTION REPORT - SUMMARY								
INSPECTION: CURSORY <u>GENERAL</u> DETAILED			DATE: 4/15/89					
BRIDGE NAME OR FEATURE U.S.A. River Bridge			STRUCTURE NUMBER 1234-56-7890					
OWNER	State DOT		CUSTODIAN	DOT	BUILT	1952		
SPANS-TYPE-LENGTH 75' Rolled Beam Approaches; 360' 12-panel through truss								
ENVIRONMENT Humid, Rural								
PAINT SYSTEM Green vinyl over zinc			LAST PAINTED 1978					
OVERALL CONDITION R=6 CONTROL ITEM Frozen Bearings, Leakage								
APPROACHES R=5 CORROSION: LOCATION-TYPES-EXTENT								
Approach steel is in fair condition. Beam ends have extensive corrosion due to water leaking through deck and joints. E4 LS OS stringer has web hole, section loss and 100% stiffener loss. Concrete cap at W4 has cracks - extent unknown.								
MAIN BRIDGE R=6 CORROSION: LOCATION-TYPES-EXTENT								
Main bridge steel is in generally good condition. No significant section loss seen on any primary members. Crevice corrosion is attacking gusset plates and built-up members. Deposit attack has occurred on gusset plates. Inclined top struts show pitting where they collect water. Truss expansion bearings show severe corrosion.								
FLOOR SYSTEM R=6 CORROSION: LOCATION-TYPES-EXTENT								
Main bridge floorbeams are in generally good condition, with some losses due to deck and joint leakage at beam ends. Stringers are in fair condition, with corrosion at ends. Spot checks showed losses up to 3/16".								
SUBSTRUCTURE* R=6 *STEEL CORROSION: LOCATION-TYPES-EXTENT								
Main bridge - concrete piers are OK								
Approach bent piles are in fair to good condition with crevice corrosion at concrete caps. Ext. pile at E4 has web hole.								
MISCELLANEOUS R=6 CORROSION: LOCATION-TYPES-EXTENT								
Bridge needs cleaning to prevent deposit attack.								
RECOMMENDATIONS								
Bridge should receive detailed corrosion inspection. Cleaning should be done to expose bearings and gusset plates as required.								
9-EXC. NO REPAIR	8-V.GD. MIN MT	7-GOOD MAJ MT	6-SAT. MIN REHAB	5-FAIR MAJ REHAB	4-POOR IMMED R	3-SERIOUS	2-CRIT. URG. R	1-IMM.FAIL. CLOSE

Figure 53. Corrosion inspection summary sheet—general level.

CORROSION INSPECTION REPORT - SUMMARY								
INSPECTION: CURSORY GENERAL <u>DETAILED</u>			DATE: 4/15/89					
BRIDGE NAME OR FEATURE U.S.A. River Bridge			STRUCTURE NUMBER 1234-56-7890					
OWNER	State DOT		CUSTODIAN	DOT	BUILT	1952		
SPANS-TYPE-LENGTH 75' Rolled Beam Approaches; 360' 12-panel through truss								
ENVIRONMENT Humid, Rural								
PAINT SYSTEM Green vinyl over zinc			LAST PAINTED 1978					
OVERALL CONDITION R=6 CONTROL ITEM Frozen Bearings, Leakage								
APPROACHES R=4 CORROSION: LOCATION-TYPES-EXTENT								
Approach span beams are in fair condition. Beam ends have extensive corrosion due to water leaking onto beams. Section loss of up to 3/16" typical. E4 LS OS beam has 3/8" section loss and 6" web hole, with 100% bearing stiffener loss. Concrete slab lifted 1/8" at this location by pack rust. W4, W2, and E1 are similar. Concrete cap at W4 has cracked due to frozen bearing.								
MAIN BRIDGE R=6 CORROSION: LOCATION-TYPES-EXTENT								
Main bridge metalwork is in generally good condition. No significant section loss seen on any primary members. Pitting has occurred from flying debris and crevice corrosion is attacking built-up members. Deposit attack has occurred on gusset plates, especially at bottom chord connections. Inclined top struts collect water and show extensive pitting. Truss expansion bearings show serious corrosion attack.								
FLOOR SYSTEM R=6 CORROSION: LOCATION-TYPES-EXTENT								
Main bridge floorbeams are in generally good condition. Some losses of up to 1/4" have occurred due to water draining through the deck and open joints. Similar drainage has attacked the stringers, which are in generally fair condition with up to 3/16" loss at ends. Galvanic corrosion is occurring at guard rail connections.								
SUBSTRUCTURE* R=6 *STEEL CORROSION: LOCATION-TYPES-EXTENT								
Main bridge - piers are concrete and in good condition. Approach bent H-piles are in generally good to fair condition. Drain leakage has attacked pile at E4 causing a web hole. Crevice corrosion has caused up to 1/8" section loss at pile-cap interface.								
MISCELLANEOUS R=6 CORROSION: LOCATION-TYPES-EXTENT								
Bridge needs cleaning (some cleaning was done for this inspection.)								
RECOMMENDATIONS								
Bridge deck and joints should be repaired or replaced to stop leaking. All bearings should be repaired as required. Bridge should be cleaned regularly to prevent deposit attack. Drain holes should be drilled as required.								
9-EXC. NO REPAIR	8-V.GD. MIN MT	7-GOOD MAJ MT	6-SAT. MIN REHAB	5-FAIR MAJ REHAB	4-POOR IMMED R	3-SERIOUS	2-CRIT. URG. R	1-IMM.FAIL. CLOSE

Figure 54. Corrosion inspection summary sheet—detailed level.

STRUCTURAL EVALUATION OF CORROSION DAMAGE

6.1 INTRODUCTION

This chapter briefly covers the information required by the office engineer for the Part II office evaluation of corrosion effects.

Part II contains the methods for the office evaluations. That part is intended to provide direction in evaluating the effects of corrosion damage on bridge capacity, in the office. Topics covered include material loss, unintended fixity of hangers and bearings, and distortion of members. Part II also discusses the differences between Level I and Level II corrosion evaluations, as mentioned in Chapter 4.

6.2 MATERIAL LOSS

The effects of material loss on bridge capacity are discussed in detail in Part II. These effects include member loss of strength, loss of stability, reduction in stiffness, and increased susceptibility to fatigue. Tension, compression, and bending members can have different responses to material loss.

6.3 UNINTENDED FIXITY

Unintended fixity of pins, hangers, and expansion devices occurs when these components "freeze" because of corrosion between contacting surfaces. This fixity causes the redistribution of forces in a structure in ways not anticipated in the original design. As a result, the structure's behavior is changed and some members may be overstressed or suffer other distress, potentially leading to structural failure.

6.4 MEMBER DISTORTION

Build-up of corrosion in confined areas (typically due to crevice corrosion) can lead to distortion of member components. This distortion, such as bending of cover plates or back-to-back angles, can cause changes in a member's section properties. The final result can be a loss of member capacity due to an increase in load eccentricity or a change in the moment of inertia or radius of gyration.

6.5 COMPONENT DESTRUCTION

Severe corrosion attack can cause the destruction of a component of a member. Destruction of a member component can lead to redistribution of loads or the failure of the member. A component does not have to be missing completely to be "destroyed." If it cannot perform its primary function, it can be considered destroyed.

6.6 MEMBER CAPACITY VS. STRUCTURE CAPACITY

Deterioration and capacity reduction of a single member may not have significant effects on the capacity of the entire bridge. Structural redundancy, allowing the redistribution of forces, can keep a single member failure from causing a total bridge failure. Part II discusses this fact and offers procedures for determining the structural capacity with deteriorated members.

CHAPTER 7

CORROSION PREVENTION AND REPAIRS

7.1 INTRODUCTION

The rate and extent of corrosion attack on a bridge will vary according to the environmental conditions, structural details, cleanliness, surface coating, and maintenance history of the structure. It should be recognized that no two bridges have identical corrosion susceptibilities. Different locations on the same structure may even show different rates of corrosion. Therefore, each bridge and detail should be considered individually. In unusual conditions, a special analysis of corrosion factors may be required.

Various techniques are available to prevent corrosion on bridges. The sections of this chapter will discuss these techniques and their applications.

7.2 BRIDGE DETAILS

At the time a bridge is inspected, it is too late to change many of the structural details. Inspectors should be aware, however, of details requiring special attention and maintenance.

The open details of older bridges promote rapid drying after wetting. They are also less prone to debris accumulation. Because of these factors, bridges with open details may be less prone to corrosion.

Inclined members may tend to hold water, producing ideal situations for corrosion. Holes can be drilled in these members in noncritical locations to provide drainage.

Protected areas that allow debris collection or bird nesting should be inspected carefully. Cross-frame angles should be installed with the legs pointing down to discourage nesting.

Details that allow structure movement or rotation are particularly important. Rockers, rollers, and pin-connected joints or hangers must be well-maintained to prevent "freezing" of joints and bearings. As discussed in Chapter 6, the consequences of unintended fixity can be catastrophic; therefore, maintenance of these details is very important.

While it is difficult and costly to change many details after a bridge is constructed, some retrofits can be made to improve corrosion resistance. Clean-outs can be added to drains to make

cleaning easier. Drip bars could be added to both the top and bottom flanges of sloping girders to prevent water accumulation, and drain pipes should be extended to clear all metalwork. Bearings with complicated details that are prone to corrosion damage can be replaced by simpler systems (such as elastomeric bearing pads). Bridge joints can be replaced by systems that prevent deck run-off attack on structural members. Enclosed areas of bridges can be protected by screens that prevent bird roosting.

7.3 BRIDGE CLEANING

Bridge cleaning can reduce corrosion of members at a relatively low cost. In arid or semi-arid regions, regular bridge cleaning can be nearly as effective as painting in preventing corrosion. Cleaning washes away corrosive deposits, such as salt, atmospheric pollutants, and bird droppings.

Some areas of bridges should be given special attention when cleaning. This would include areas near finger dams and expansion joints, the bridge bearings, members subject to splashing by roadway runoff, and areas prone to bird roosting and nesting.

The schedule for bridge cleaning should depend on the exposure of the structure to corrosive agents. A bridge in an area of industrial pollution should be cleaned frequently. Bridges exposed to deicing salts should be cleaned in the spring of each year. Structures in isolated areas may not require as frequent cleaning.

7.4 BRIDGE PAINTING

Bridge painting is obviously an important part of corrosion prevention. Inspectors should note the condition of the paint on each bridge inspected.

The proper selection of paint is also important. Factors to be considered in paint selection include its corrosion resistance characteristics, cost, ease of application, resistance to wear or cleaning, and availability.

It may not be necessary to repaint an entire bridge if limited areas of the existing paint are in poor condition. Spot repainting is an option. Also, extra coats of paint could be applied to bridge members affected by deicing salts or debris accumulation.

Special protection should be considered for specific needs. Splash zones of piles can be sheathed in corrosion resisting material. This has been shown to provide up to 25 years of protection. Full length sheathing of exposed piles is preferable to partial sheathing. Cathodic protection is becoming more common. It is applied to both new and existing structures to reduce corrosion. New products are frequently introduced to protect structures against corrosion and they may merit consideration in some cases.

Paint and other protection systems are becoming more sophisticated; therefore, specialists in the field should be consulted as necessary.

7.5 BRIDGE REPAIR

In general, repair of corrosion damage is based on restoration of the damaged member to its original cross-sectional area. Repair may not be required in every case where corrosion damage exists. An engineering evaluation of the inspection findings

should be made to determine the required repairs for corrosion damaged structures.

When repairs are made, details should be kept as simple as possible to avoid promoting further corrosion.

7.6 WEATHERING STEEL

High-strength low-alloy steel, or weathering steel as it is commonly known, is not immune to corrosion. A thin layer of initial oxidation protects weathering steel from further corrosion. When this layer does not form, because of either too dry or too moist conditions, the steel will corrode continuously.

Recent examinations of numerous unpainted weathering steel bridges have provided significant corrosion-related observations. Crevice corrosion, uniform corrosion, pitting, and galvanic corrosion were identified. These were promoted by deicing salt contamination, accumulated dirt and debris, capillary action of the corrosion product itself, and mill scale. Affected details included pin and hanger assemblies, beams, cover plates and weldments; cross bracing, diaphragms and stiffeners near joints; and expansion joints.

The effects of angle and extent of exposure of weathering steel have also been examined in great detail. The most severe condition for corrosion is one which promotes prolonged dampness without allowing washing of the surface. Thus, beam flanges and lower portions of their webs located near leaking joints are particularly susceptible. This calls for careful selection, design, and maintenance of expansion joints. Probably the best solution for corrosion of weathering steel is cleaning and painting of the affected areas.

Experience has shown that the use of galvanized components in contact with weathering steel can lead to rapid deterioration of the galvanized coating. If galvanizing is used, the coated component should be electrically isolated from the weathering steel.

CHAPTER 8

COMPLETE EXAMPLE BRIDGE CORROSION INSPECTION REPORTS

This chapter contains the complete corrosion inspection reports for the example bridge discussed in Chapter 5. Three types of reports are shown: (1) cursory inspection reports—Figures 55 (a, b, c, and d), (2) general inspection reports—Figures 56 (a, b, c, d, e, f, and g), and (3) detailed inspection reports—Figures 57 (a, b, c, d, e, f, g, h, i, j, k, and l).

CORROSION INSPECTION REPORT - SURVEY							
BRIDGE NAME OR FEATURE U.S.A. River Bridge				STRUCTURE NO. 1234-56-7890		DATE: 4/15/89	
ELEMENT				R		TYPE CORROSION	
SUBSTRUCTURE* R = 6				APPROACHES R = 5			
PILES	-			BEAMS	6	Ends	P,C,D,U
BENTS	6	C @ Cap, P generally		GIRDERS	-		
TOWERS	-			B. CHORDS	-		
PIERS	-			T. CHORDS	-		
				DIAGONALS	-		
				VERTICALS	-		
*STEEL ONLY				B. LATERALS	6	C, spread angles	
FLOOR SYSTEM R = 6				T. LATERALS	-		
STRINGERS	7	P ends, C @ slab		PORTALS	-		
FLOORBEAMS	6	D bot. flg., U web		SWAY FRAMES	-		
BRACING	7	C between angles		BEARINGS	-	D, C, P,F	
CONNECTIONS	7	C		LINKS-PINS	-		
GRATING	-			FASTENERS	8		
DECK PLATES	-			WELDS	8		
EXP. DAMS	6	C, D, P		CONNECTIONS	7	U	
RELIEF JTS.	7	C, interface					
BARRIERS				MAIN BRIDGE R = 6			
RAILINGS	6	G-- no isolation		BEAMS	-		
INSP. WALK	-			GIRDERS	-		
DRAINAGE	5	D, P		B. CHORDS	6	P, D inside, C seams	
				T. CHORDS	7	U top surface	
MISCELLANEOUS R = 6				DIAGONALS	6	C seams	
MAIN CABLES	-			VERTICALS	6	C seams P @ deck	
SUSP. CABLES	-			B. LATERALS	6	C spread angles	
STAY CABLES	-			T. LATERALS	7	C spread angles	
CTWT ROPES	-			PORTALS	6	P at deck	
MACHINERY	-			SWAY FRAMES	6	Collision	
FENDERS	-			BEARINGS	5	L6 rollers F,P	
DOLPHINS	-			LINKS-PINS	5	L0 pin F	
CLEANLINESS	6	Pigeon nests, sand from trucks		FASTENERS	7	P - deck leaks	
				WELDS	8		
				CONNECTIONS	6	D horizontal surfaces	
F = Frozen							
U=UNIFORM; C=CREVICE; G= GALVANIC; P=PITTING; D=DEPOSIT ATTACK							
9-EXC. NO REPAIR	8-V.GD. MIN MT	7-GOOD MIN MT	6-SAT. MAJ MT	5-FAIR MIN REHAB	4-POOR MAJ REHAB	3-SERIOUS IMMED R	2-CRIT. URG. R
							1-IMM.FALL. CLOSE

Figure 56(b). Corrosion inspection general survey report. (See also photographs—Figures 56e through 56g)

CORROSION INSPECTION REPORT - NARRATIVE			
BRIDGE NAME OR FEATURE U.S.A. River Bridge		STRUCTURE NO. 1234-56-7890	
DATE: 4/15/89		-	
SUBSTRUCTURE			
Main Bridge - No steel elements.			
Approaches - Approach bents are composed of braced H-piles with concrete caps. The bent H-piles are in generally good-to-fair condition with minor-to-modest section loss from poutlice corrosion at the cap interface. Flange and web losses are noted and appear to 1/16 to 1/8 inch. Pile #1, Bent E4 has a web hole at the top. Some minor section loss from crevice corrosion exists where bracing crosses the piles. Paint is thinning primarily on the south side.			
FLOOR SYSTEM			
Main bridge floorbeams are in generally good condition, with some minor losses on top flanges. This deterioration may be due to water leaking through the deck or open joints. No significant losses are seen on other sections of the floorbeams.			
The main bridge stringers are in fair condition. Section losses are seen in flanges and webs near the stringer ends. Corrosion is found most frequently near roadway joints. Heavy corrosion has occurred on the bottom flange of stringers at relief joint seats, which may interfere with the movement of the joints. Holes have developed in the webs of stringers at Panel Point 3', and a notch has developed in the cope of Stringer 4 at Panel Point 0.			
Some galvanic corrosion can be seen at the aluminum rail connection to steel supporting members.			

Figure 56(c). Corrosion inspection general narrative report. (See also photographs—Figures 56e through 56g)

CORROSION INSPECTION REPORT - NARRATIVE			
BRIDGE NAME OR FEATURE U.S.A. River Bridge	STRUCTURE NO. 1234-56-7890	DATE 4/15/89	-
<u>APPROACHES</u>			
<p>The approach beam span metalwork is in fair condition due to corrosion on beam ends and bearings caused by deck water leaking on these elements. At locations other than beam ends, the metalwork is in full section.</p> <p>Affected beam ends have section loss on webs of 1/8" on the lower 6 inches from splash and bottom flange loss of about 3/16 inch for a distance of 24-36 inches. The worst location is at W4 Left Side exterior beam which has a 6" high hole and bearing stiffener with 100% section loss. The bottom flange is buried in debris and cannot be examined without further access and cleaning.</p> <p>The condition of the bearings is not fully known due to the extent of debris covering the sliding plates. Cracks on the side of the concrete cap at W4 were suspicious and debris removed enough to expose the bearing and note that it was frozen. All bearings need to be uncovered and examined.</p>			
<u>MAIN BRIDGE</u>			
<p>The main bridge truss is in generally good condition. There is no apparent significant section loss on any of the primary members. Crevice corrosion is common at gusset plates and seams of built-up members. Uniform corrosion is occurring where paint has peeled. Debris accumulation is widespread along the bottom chords.</p> <p>Crevice corrosion has spread the back-to-back angles of the bottom laterals. Deposit attack has corroded the bottom lateral plates at the floorbeam to bottom chord connections.</p> <p>The main bridge truss bearings are in fair to good condition. Fixed bearings show no deterioration. Expansion bearings show advanced corrosion deterioration. Anchor bolts at these bearings have sheared off or necked down. Crevice corrosion can be found between and beneath the rollers, possibly inhibiting their movement.</p>			
<u>MISCELLANEOUS</u>			
<p>The main bridge and approaches need cleaning. Debris has accumulated on main bridge lower chord members and connection plates. Girder ends at the abutments are completely buried.</p>			

Figure 56(c). Continued

CORROSION INSPECTION REPORT - PHOTOGRAPHS			
BRIDGE NAME OR FEATURE	STRUCTURE NO.	DATE	-

Figure 56(d). Sample form shown for photographs used in completing all corrosion inspection reports.

CORROSION INSPECTION REPORT - SUMMARY								
INSPECTION: CURSORY GENERAL DETAILED				DATE: 4/15/89		-		
BRIDGE NAME OR FEATURE U.S.A. River Bridge				STRUCTURE NUMBER 1234-56-7890				
OWNER State DOT		CUSTODIAN DOT		BUILT 1952				
SPANS-TYPE-LENGTH 75' Rolled Beam Approaches; 360' 12-panel through truss								
ENVIRONMENT Humid, Rural								
PAINT SYSTEM Green vinyl over zinc				LAST PAINTED 1978				
OVERALL CONDITION R=6 CONTROL ITEM Frozen Bearings, Leakage								
APPROACHES R=4 CORROSION: LOCATION-TYPES-EXTENT								
Approach span beams are in fair condition. Beam ends have extensive corrosion due to water leaking onto beams. Section loss of up to 3/16" typical. E4 LS OS beam has 3/8" section loss and 6" web hole, with 100% bearing stiffener loss. Concrete slab lifted 1/8" at this location by pack rust. W4, W2, and E1 are similar. Concrete cap at W4 has cracked due to frozen bearing.								
MAIN BRIDGE R=6 CORROSION: LOCATION-TYPES-EXTENT								
Main bridge metalwork is in generally good condition. No significant section loss seen on any primary members. Pitting has occurred from flying debris and crevice corrosion is attacking built-up members. Deposit attack has occurred on gusset plates, especially at bottom chord connections. Inclined top struts collect water and show extensive pitting. Truss expansion bearings show serious corrosion attack.								
FLOOR SYSTEM R=6 CORROSION: LOCATION-TYPES-EXTENT								
Main bridge floorbeams are in generally good condition. Some losses of up to 1/4" have occurred due to water draining through the deck and open joints. Similar drainage has attacked the stringers, which are in generally fair condition with up to 3/16" loss at ends. Galvanic corrosion is occurring at guard rail connections.								
SUBSTRUCTURE* R=6 *STEEL CORROSION: LOCATION-TYPES-EXTENT								
Main bridge - piers are concrete and in good condition. Approach bent H-piles are in generally good to fair condition. Drain leakage has attacked pile at E4 causing a web hole. Crevice corrosion has caused up to 1/8" section loss at pile-cap interface.								
MISCELLANEOUS R=6 CORROSION: LOCATION-TYPES-EXTENT								
Bridge needs cleaning (some cleaning was done for this inspection.)								
RECOMMENDATIONS								
Bridge deck and joints should be repaired or replaced to stop leaking. All bearings should be repaired as required. Bridge should be cleaned regularly to prevent deposit attack. Drain holes should be drilled as required.								
9-EXC. NO REPAIR	8-V.GD.	7-GOOD MIN MT	6-SAT. MAJ MT	5-FAIR MIN REHAB	4-POOR MAJ REHAB	3-SERIOUS IMMED R	2-CRIT. URG. R	1-IMM.FAIL. CLOSE

Figure 57(a). Corrosion inspection detailed summary report. (See also photographs—Figures 57f through 57l)

CORROSION INSPECTION REPORT - SURVEY								
BRIDGE NAME OR FEATURE U.S.A. River Bridge			STRUCTURE NO. 1234-56-7890		DATE: 4/15/89		-	
ELEMENT R			TYPE CORROSION		ELEMENT R		TYPE CORROSION	
SUBSTRUCTURE*R =				APPROACHES R =				
PILES				BEAMS				
BENTS				GIRDERS				
TOWERS				B. CHORDS				
PIERS				T. CHORDS				
				DIAGONALS				
				VERTICALS				
*STEEL ONLY				B. LATERALS				
FLOOR SYSTEM R = 6				T. LATERALS				
STRINGERS 7 P @ Ends, C @ slab				PORTALS				
FLOORBEAMS 6 D bot. flg., U web				SWAY FRAMES				
BRACING 7 C between angles				BEARINGS				
CONNECTIONS 7 C				LINKS-PINS				
GRATING -				FASTENERS				
DECK PLATES -				WELDS				
EXP. DAMS 6 C, D, P				CONNECTIONS				
RELIEF JTS. 7 C Interface								
BARRIERS -				MAIN BRIDGE R = 6				
RAILINGS 6 G no isolation				BEAMS -				
INSP. WALK -				GIRDERS -				
DRAINAGE 5 D, P				B. CHORDS 6 P, D inside, C seams				
				T. CHORDS 7 U top surface				
MISCELLANEOUS R = 6				DIAGONALS 6 C seams				
MAIN CABLES -				VERTICALS 6 C seams, P at deck				
SUSP. CABLES -				B. LATERALS 6 C spread angles				
STAY CABLES -				T. LATERALS 6 C spread angles				
CTWT ROPES -				PORTALS 6 P at deck				
MACHINERY -				SWAY FRAMES 6 Collision				
FENDERS -				BEARINGS 5 L6 rollers F,P				
DOLPHINS -				LINKS-PINS 5 L0 Pin F				
CLEANLINESS 6 Debris in bottom chords				FASTENERS 7				
				WELDS 8				
				CONNECTIONS 6 D horizontal surfaces				
F = Frozen								
U=UNIFORM; C=CREVICE; G= GALVANIC; P=PITTING; D=DEPOSIT ATTACK								
9-EXC. NO REPAIR	8-V.GD.	7-GOOD MIN MT	6-SAT. MAJ MT	5-FAIR MIN REHAB	4-POOR MAJ REHAB	3-SERIOUS IMMED R	2-CRIT. URG. R	1-IMM.FAIL. CLOSE

Figure 57(b). Corrosion inspection detailed survey report. (See also photographs—Figures 57f through 57l)

CORROSION INSPECTION REPORT - SURVEY								
BRIDGE NAME OR FEATURE		STRUCTURE NO.		DATE: 4/15/89		-		
U.S.A. River Bridge		1234-56-7890		SPAN: Approaches				
ELEMENT	R	TYPE CORROSION		ELEMENT	R	TYPE CORROSION		
SUBSTRUCTURE *R = 6				APPROACHES R = 4				
PILES	-			BEAMS	6	Ends P,C,D,U		
BENTS	6	C @ cap, P general		GIRDERS	-			
TOWERS	-			B. CHORDS	-			
PIERS	-			T. CHORDS	-			
				DIAGONALS	-			
				VERTICALS	-			
*STEEL ONLY				B. LATERALS	-			
FLOOR SYSTEM R =				T. LATERALS	-			
STRINGERS				PORTALS	-			
FLOORBEAMS				SWAY FRAMES	-			
BRACING				BEARINGS	4	F,D,C,P		
CONNECTIONS				LINKS-PINS	-			
GRATING				FASTENERS	7	Rivets full section		
DECK PLATES				WELDS	-			
EXP. DAMS				CONNECTIONS	7	No losses		
RELIEF JTS.								
BARRIERS				MAIN BRIDGE R =				
RAILINGS				BEAMS				
INSP. WALK				GIRDERS				
DRAINAGE				B. CHORDS				
				T. CHORDS				
MISCELLANEOUS R =				DIAGONALS				
MAIN CABLES	-			VERTICALS				
SUSP. CABLES	-			B. LATERALS				
STAY CABLES	-			T. LATERALS				
CTWT ROPES	-			PORTALS				
MACHINERY	-			SWAY FRAMES				
FENDERS	-			BEARINGS				
DOLPHINS	-			LINKS-PINS				
CLEANLINESS	3	Poor on cap tops		FASTENERS				
				WELDS				
				CONNECTIONS				
F = Frozen								
U=UNIFORM; C=CREVICE; G= GALVANIC; P=PITTING; D=DEPOSIT ATTACK								
9-EXC. NO REPAIR	8-V.GD. MIN MT	7-GOOD MAJ MT	6-SAT. MAJ MT	5-FAIR MIN REHAB	4-POOR MAJ REHAB	3-SERIOUS IMMED R	2-CRIT. URG. R	1-IMM.FAIL. CLOSE

Figure 57(b). Continued

CORROSION INSPECTION REPORT - NARRATIVE			
BRIDGE NAME OR FEATURE		STRUCTURE NO.	DATE 4/15/89
U.S.A. River Bridge		1234-56-7890	-
SUBSTRUCTURE			
Main Bridge - Main piers are concrete and in good condition with only minor surface shrinkage cracks.			
Approaches - The bents are composed of braced H-piles with a concrete cap. The bent H-piles are in generally good-to-fair condition with minor-to-moderate section loss from poutlice corrosion at the cap interface. Losses to flanges and webs range to 1/8 inch, but are generally 1/16 inch or less. Bent E4 exterior pile (#1) has a hole through the web at the underside of the cap. The deterioration is caused by leakage of deicing salt water from the overhead drain onto this pile (see detail). A general uniform corrosion has occurred on the southside piles exposed to direct sunlight with only 3 mils paint remaining. Paint deterioration has occurred due to the gunshot blast peppering the piles of Bent E2 and bare areas are now pitted.			
FLOOR SYSTEM			
Main bridge floorbeams are in generally good condition, with typical minor metalwork losses at the edges of the top flange due to deterioration from water penetrating the deck and coming thru open roadway joints. The losses, checked with ultrasonic equipment, are as much as 1/4" at the very edge of the flange and taper to 0" in toward the web. Pitting of the webs to 1/8" deep just below stringer connection angles and around stringer relief seat castings is common. Losses to the bottom flanges of interior floorbeams at or near mid-span were < 3/32" and only in a few locations. The web and latter flange losses should be evaluated by rating the floorbeams.			
Top flanges of end floorbeams beneath expansion dams show up to 1/8" loss in thickness, and there is much minor deterioration of the webs around the jacking stiffeners. Bottom flanges show up to 1/16" loss at the bottom of the back to back jacking stiffener angles. Debris accumulations have caused deterioration and spreading of these angles. Also beneath expansion dams at the exterior stringers, the angles used to tie the end floorbeam flange to the stringer show moderate deterioration.			

Figure 57(c). Corrosion inspection detailed narrative report. (See also photographs—Figures 57f through 57l)

CORROSION INSPECTION REPORT - NARRATIVE

BRIDGE NAME OR FEATURE U.S.A. River Bridge	STRUCTURE NO. 1234-56-7890	DATE 4/15/89	-
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FLOOR SYSTEM (CONT.)

Main bridge stringers are in generally fair condition, with some minor repairs needed. Typical areas of deterioration are in the vicinity of roadway joints in the top flanges where the section losses are $\leq 1/8"$, in the bottom flanges and webs at the bottom flanges where losses in both areas are $\leq 3/16"$, and in the bottom flanges at relief joint seats where losses are also $\leq 3/16"$. These losses are generally within 3' of the stringer ends. There is also heavy corrosion around the bolts in the bottom flange of stringers at relief joint seats. The roughness of the stringer bottom flange will prevent proper movement/relief.

A 1/4" loss in the bottom flange of Stringer 4 (stringers numbered 1 to 4, left to right, when looking ahead station) was found in the end 2' of the stringer adjacent to the relief seat at Panel Point 3'. Deterioration of the web at the bottom flange has produced holes in stringers on the east side of the floorbeam at Panel Point 3'. These two areas should be patched. A notch has developed in the cope of Stringer 4 at Panel Point 0 from deterioration. No crack has developed, but the area should be monitored.

Galvanic corrosion is attaching the structure where aluminum railing is bolted to steel members. There is no insulation between the rail and the steel supports.

Figure 57(c). Continued

CORROSION INSPECTION REPORT - NARRATIVE

BRIDGE NAME OR FEATURE U.S.A. River Bridge	STRUCTURE NO. 1234-56-7890	DATE 4/15/89	-
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APPROACHES

The approach beam span metalwork is in fair condition due to corrosion on beam ends and bearings caused by extensive leakage of winter deicing salts in water being deposited on the metalwork. At locations other than the beam ends, the metalwork is in full section with only minimal uniform surface corrosion on bottom flange edges and bottom and minor pitting on the webs.

At the beam ends, in general the effects of salts has cause extensive loss of paint protection and the webs have loss about 1/8 inch section in the lower 6 inches from splash, the tops of the bottom flanges have lost 3/16 inch thickness for a distance of 24-36 inches and the end bearing stiffener bases are knife edged. The worst case location is at E4 left exterior beam where drainage water and joint leakage both have concentrated water on the metalwork. There the beam web end 4 inches has loss 3/8 inch section full height plus has developed a hole 6 inches high at the base. The bearing stiffener has 100% section loss and does not contact the bottom flange. The bottom flange has 1/2 inch section loss. At the top flange poulitice corrosion has cause pack rust to form between the flange and concrete and has lifted the concrete slab 1/8 inch. Similar conditions occur at W4, W2, and E1 (see Metalwork Loss sheet).

Diaphragms between beams are in full section.

At the fixed ends of the beams the sole plates and anchor bolts have lost section from deposit attack. When uncovered from 4 inches of dirt, the metalwork was found pitted and 50% of the anchor bolt shank cross section is lost. At the expansion ends, debris was removed and sliding plate found frozen to the masonry plate. At W4 the concrete cap has cracked in an attempt to relieve the pressure.

Figure 57(c). Continued

NOTES



(a)



(b)

Figure 4. Photographs illustrating the effects of environmental exposure on corrosion. Opposite sides are shown of the same vertical member at the same level. The angle in (b) is on the windward side of the member, exposed to the prevailing wind carrying salt mist from the ocean. The angle in (a) is on the leeward side of the member, protected from the wind.



Figure 5. Photograph shows uniform corrosion of bridge member.



Figure 6. Photograph shows galvanic corrosion—aluminum bracket with carbon steel bolt.



Figure 7. Photograph shows crevice corrosion between back to back angles and gusset plate.

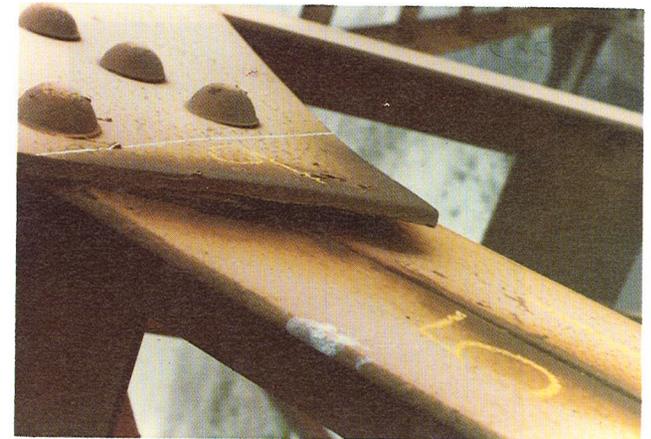


Figure 8. Photograph shows crevice corrosion—pack rust causing bending of gusset plate. Excessive edge distance of rivets contributed to this condition.

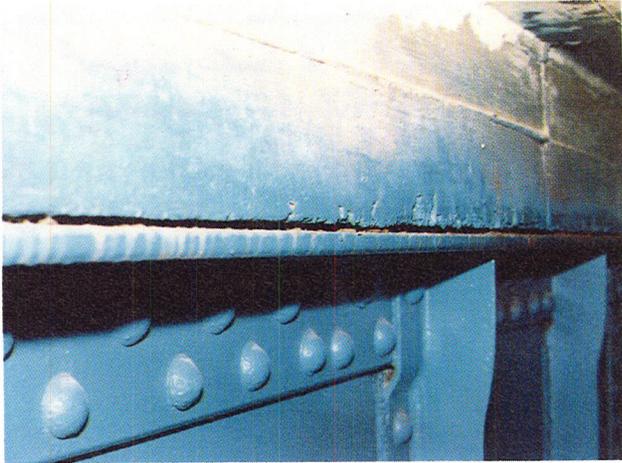


Figure 9. Photograph shows crevice corrosion between concrete haunch and steel beam.

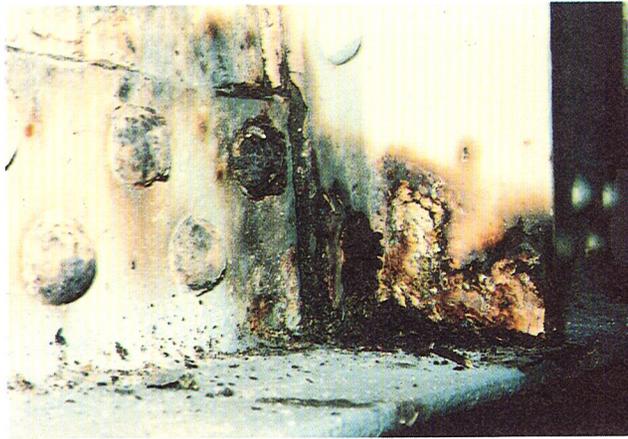


Figure 10. Photograph shows corrosion due to deposit attack at stiffener-to-flange connection. Debris accumulation was removed for photograph.

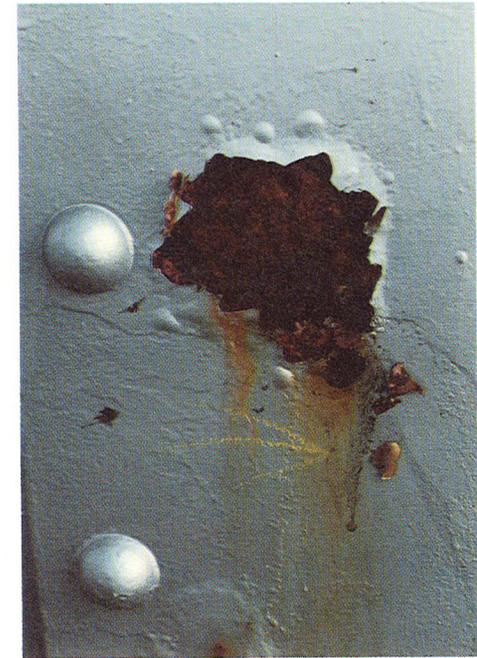


Figure 11. Photograph shows paint blistering due to underfilm corrosion.

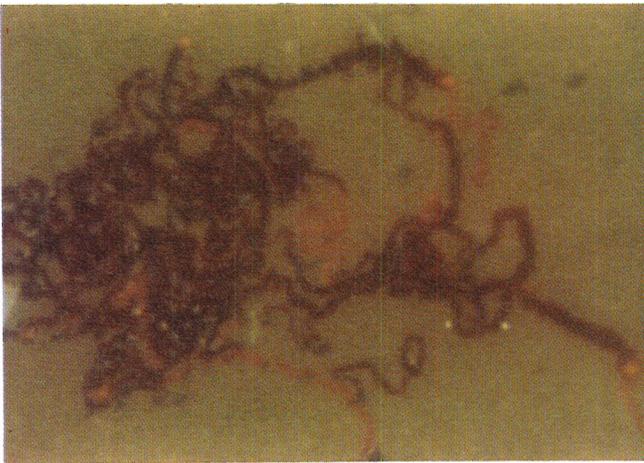


Figure 12. Photograph shows filiform corrosion.

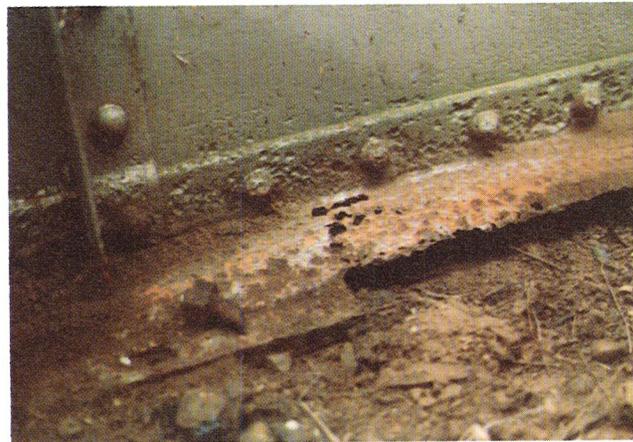


Figure 13. Photograph shows pitting of girder web and bottom flange. Note active pitting on bottom flange as well as old pits on web that has been painted.



Figure 14. Photograph shows weld decay.



Figure 15. Photograph shows potential erosion corrosion location of a steel-encased pier in a river carrying abrasive particles.

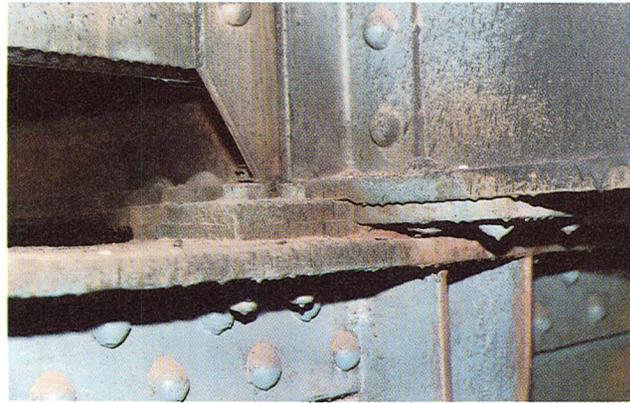


Figure 16. Photograph shows fretting corrosion at stringer joint.



Figure 17. Photograph shows stress corrosion cracking.



Figure 18. Photograph shows corrosion cracking of bolts in industrial environment.



Figure 48. Photograph shows stringer with corrosion and debris accumulation.



Figure 49. Photograph shows same stringer after cleaning, revealing crack and hole.



Figure 50(a). Photograph shows progression of deterioration—rating 6.



Figure 50(b). Photograph shows progression of deterioration—rating 5.



Figure 50(c). Photograph shows progression of deterioration—rating 4.

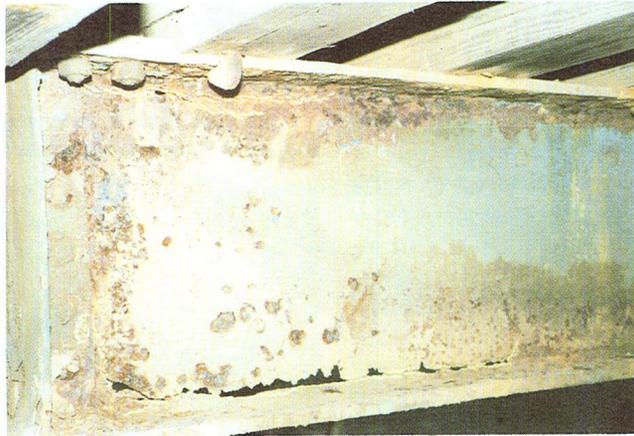


Figure 50(d). Photograph shows deterioration of web at support—rating 1.

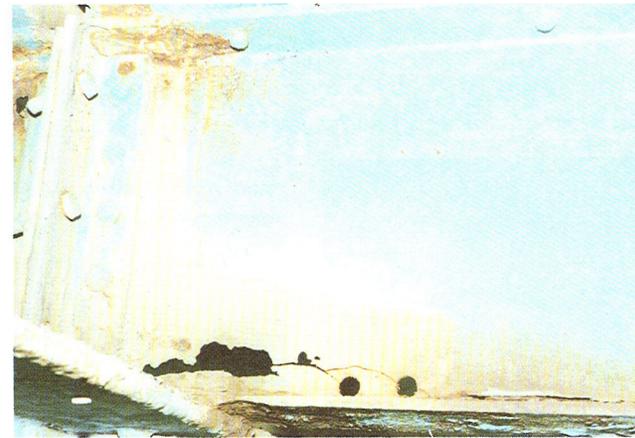


Figure 50(e). Photograph shows poor stringer condition but adequate shear capacity at support—rating 3.

CORROSION INSPECTION REPORT - PHOTOGRAPHS

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Figure 55(c). Photograph 1 shows typical condition of top flange and web of end floor beams beneath expansion dams. (See also cursory reports—Figures 55a, 55b)



Figure 55(d). Photograph 2 shows typical condition of anchor bolts on expansion bearings. Many bolts have sheared off just below the nuts. Note the extent of corrosion in the roller nest. (See also cursory reports—Figures 55a, 55b)



Figure 56(e). Photograph 1 shows typical cracking of main bridge deck. (See also general reports—Figures 56a, 56b, 56c)

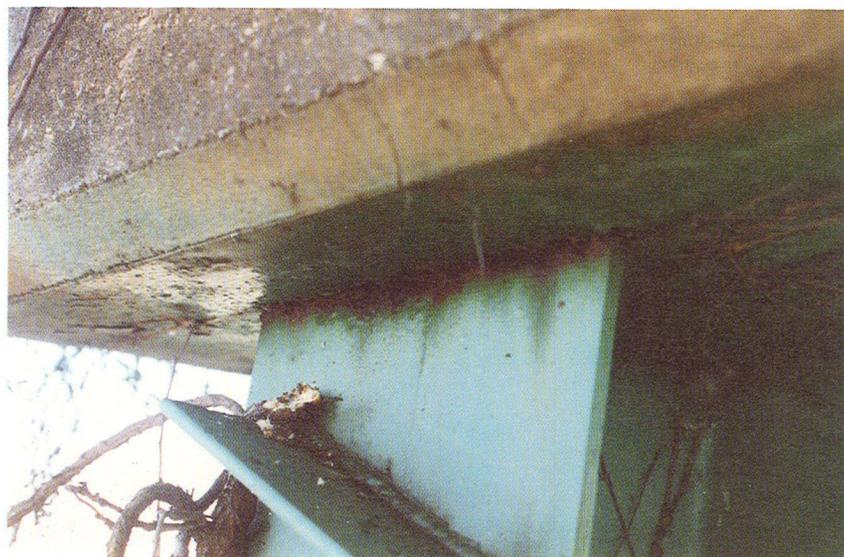


Figure 56(f). Photograph 2 shows typical area of deteriorating metalwork at bottom of bent cap concrete. (See also general reports—Figures 56a, 56b, 56c)



Figure 56(g). Photograph 3 shows typical areas of section loss to stringer flanges and web. (See also general reports—Figures 56a, 56b, 56c)



Figure 57(f). Photograph 1 shows typical patched, cracked condition of concrete deck on approaches, which allows leakage. (See also detailed reports—Figures 57a through 57e)

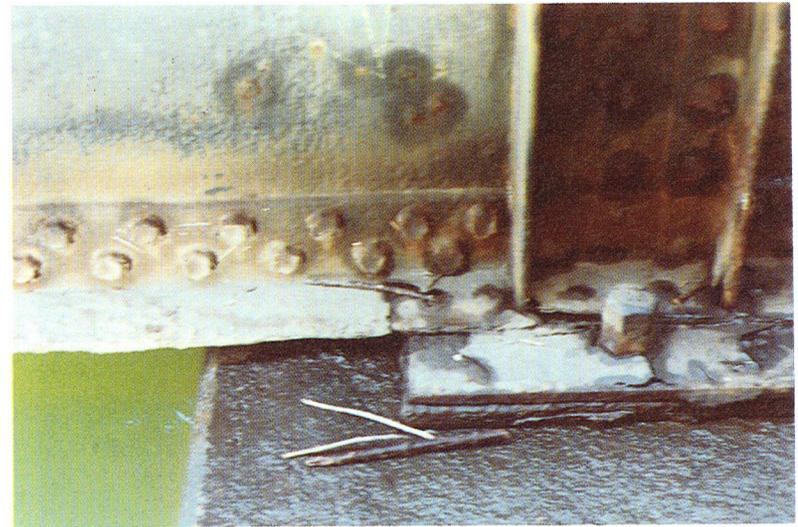


Figure 57(g). Photograph 2 shows deteriorated expansion bearing on approach spans. (See also detailed reports—Figures 57a through 57e)

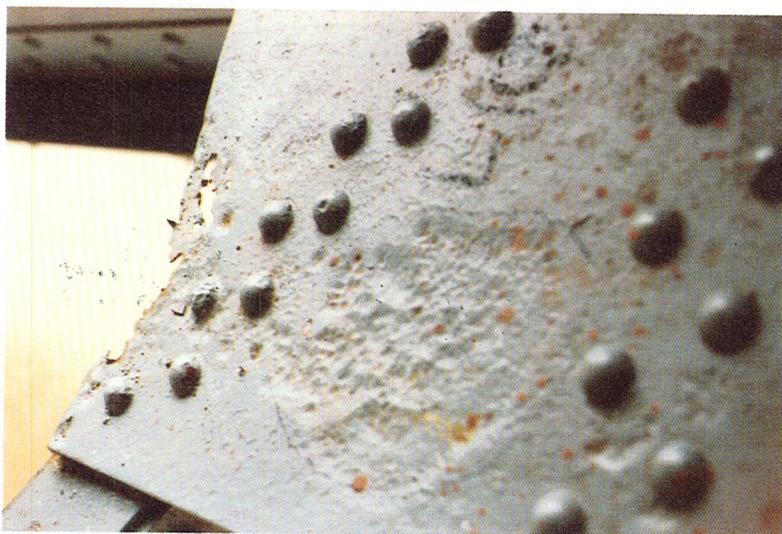


Figure 57(h). Photograph 3 shows pitting of lateral connection plate. (See also detailed reports—Figures 57a through 57e)



Figure 57(j). Photograph 5 shows typical debris accumulation in bottom chord connections. Arrows point to area of deteriorating metalwork. (See also detailed reports—Figures 57a through 57e)



Figure 57(i). Photograph 4 shows typical area of metalwork deterioration at H-pile encasements. Crack in concrete from flange of H-pile is from expansion of deteriorating flange metalwork at interface with the concrete. (See also detailed reports—Figures 57a through 57e)



Figure 57(k). Photograph 6 shows typical condition of expansion bearings. Many bolts have bent or sheared off. Note the extent of corrosion in the roller nest. (See also detailed reports—Figures 57a through 57e)



Figure 57(l). Photograph 7 shows typical area of deterioration of floorbeams, top flange and web at stringer connections. (See also detailed reports—Figures 57a through 57e)

Part II

Office Evaluation of Corrosion Effects

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Corrosion can affect steel bridges in many ways. It can cause loss of section, create unintended fixities and shift, or deform bridge components. Some forms of corrosion, such as stress corrosion or corrosion fatigue, can cause damage that is not easily identifiable and can lead to unexpected failures. The conditions created by corrosion can reduce the static, fatigue, fracture, or buckling strength of bridge members, affect structural behavior, and reduce overall bridge strength and load-carrying capacity.

Guidelines for evaluating the load-carrying capacity of existing highway bridges are provided in the AASHTO *Manual for Maintenance Inspection of Bridges*. The AASHTO maintenance manual provides a basis for computing the maximum loads that may be allowed on a bridge. It contains general provisions regarding the evaluation of member resistance, the type of loading to be used, the method of analysis, and the safety levels required. All matters not definitely covered in the manual are referred to the current AASHTO *Standard Specifications for Highway Bridges*. The evaluation guidelines included in the maintenance manual and in the supporting sections of the AASHTO specifica-

tions for design are primarily concerned with good quality materials and a bridge structural behavior consistent with its design. Bridge deterioration and its effects on member capacity and structural response are only covered in general terms.

1.2 PURPOSE AND SCOPE OF OFFICE GUIDELINES

The objective of these office guidelines is to provide practical and comprehensive guidance for assessing the effects of corrosion on steel bridges. Accurate analysis of many conditions created by corrosion often requires refined and expensive analysis techniques that are not readily available to the bridge engineer. The office guidelines include simplified techniques for the analysis and evaluation of common conditions created by corrosion not covered by the AASHTO *Manual for Maintenance Inspection of Bridges*. The recommendations and the methods of analysis suggested are intended to complement the AASHTO maintenance inspection manual and to provide the necessary guidelines for a more accurate deterministic evaluation of the nominal resistance of a corroded member.

CHAPTER 2

GENERAL APPROACH

This chapter provides an overall description of the office evaluation approach used, the background information needed for understanding the office guidelines and their organization, and the definition of terms used throughout this study. It includes a general description of the possible effects of corrosion, an overview of the elements involved in the evaluation process of the load-carrying capacity of a bridge and of those elements that are directly affected by corrosion. A general description of the effects of the location of corrosion and of the method used for analysis, a review of the evaluation criteria to be considered, and a section on stress concentrations are also included.

2.1 EFFECTS OF CORROSION

The effects of corrosion on steel bridges can vary from non-structural maintenance problems to catastrophic failures. The most common effect of corrosion is loss of metal. The loss of metal can be uniform and evenly distributed or it can be localized in the form of pits, holes, or edge scallops. The loss of section and the creation of stress raisers can reduce the static, fatigue, fracture, and buckling resistance of bridge members. If the loss of metal is significant, the load distribution characteristics of the structure may also be affected, resulting in an increase in load in members adjacent to the deteriorated member.

Corrosion can cause freezing of moving parts of a bridge such as pin-hanger assemblies and bearings. These unintended fixities cause the structure to behave differently from the way it was

designed. High unexpected stresses may be induced in the frozen elements and in the adjacent bridge members.

Build-up of corrosion products in confined areas can result in shifting of elements or deformation of bridge components. This may have a significant effect on the overall buckling or local buckling resistance of bridge members.

Corrosion forms such as stress corrosion and corrosion fatigue are very difficult to detect and can result in unexpected fracture in tension elements.

2.2 BRIDGE EVALUATION

The process of evaluating the load-carrying capacity of an existing bridge involves: (1) selecting an evaluation method or approach, (2) evaluating the strength of bridge members, (3) defining the type of loading to be used, (4) calculating the resulting member loads, and (5) defining the required safety levels.

A deterministic approach such as specified in the AASHTO *Manual for Maintenance Inspection of Bridges* or a higher level, reliability-based approach such as described in Ref. 6.10 could be used.

The purpose of bridge evaluation is to help the engineer assess the capability of the bridge to safely carry its intended traffic. If the bridge is found deficient, a decision must be made regarding posting, regulating permits, closing, rehabilitation, replacement, or further in-depth study.

Corrosion on bridges reduces the strength of bridge components and members, which in turn can reduce the overall strength of the structure and, thus, cause a reduction in load-carrying capacity. It is important to emphasize the difference between the terms "member strength or capacity," "overall strength or capacity of the bridge structure," and "load-carrying capacity of the bridge."

The term "strength or capacity of a member" refers here to a nominal value that is calculated in order to assess the actual strength of the member. It depends on the member properties and dimensions, but it also depends on the method used for its evaluation (like service load or load factor), and the factors of safety used. The reduced nominal capacity of a member damaged by corrosion will henceforth be referred to as the residual capacity of the member. The concept of residual capacity is adopted from Refs. 7.24, 7.25, 7.26, 7.52.

The "overall strength or capacity of the bridge" is also defined here by nominal values, and it depends on the strength of its members and on the structural behavior of the bridge as a whole. Thus, the evaluation of the "bridge capacity" will also depend on the method used for bridge analysis (like linear elastic or nonlinear analysis, or even line methods, two-dimensional methods or three-dimensional methods). The reduced nominal capacity of a bridge damaged by corrosion will henceforth be referred to as the residual capacity of the bridge.

The term "load-carrying capacity of a bridge" also refers to a nominal value, and it depends on the bridge strength and also on the type of loading, evaluation method, and safety levels used. It is only indirectly related to the effects of corrosion damage, through the reduced strength of the bridge.

This study focuses on the evaluation of corrosion effects on the nominal strength of bridge members and of the bridge structure as a whole. Evaluation of member strength is the only element in the evaluation process of the load-carrying capacity of a bridge directly affected by deterioration. Also, the evaluation

of the strength of a member is not influenced by a large number of factors (like the type of loading). Thus, focusing on member strength facilitates easier parametric investigations into the effects of corrosion. The concept of a residual member capacity factor (7.24, 7.25, 7.26, 7.52), defined as the ratio of the deteriorated capacity of a member to its undeteriorated capacity, will be used throughout this report. The residual capacity factor is not sensitive to the method used to calculate the member strength and the factors of safety used. Therefore, its use can further add to the clarity of the parametric investigations.

When evaluating the residual strength of a deteriorated bridge structure one must identify possible *modes of failure* and evaluate the condition based on the applicable *failure criteria* that predict the modes of failure. An analysis of the whole structure is preferable, but in many cases an analysis at the member level only may be sufficient.

2.3 EFFECTS OF LOCATION OF DETERIORATION

Corrosion can affect a large area of a bridge member and result in a relatively uniform deterioration. It can also take the form of localized deterioration limited to a confined area. In some cases, the location of deterioration on a member will have little or no effect on the capacity of that member. In other cases, the effect of the location of damage can be significant. Likewise, a reduction in the capacity of some members may have no effect on the overall capacity of the bridge, while deterioration of other members has a significant effect on the overall capacity of the structure. In order to evaluate these effects, a distinction must be made among: (1) localized effects of deterioration, (2) effects of localized deterioration on the behavior of the member, and (3) effects of deterioration of a member on the behavior of the bridge structure as a whole.

Localized deterioration can cause increased nominal stresses, stress concentrations, and localized yielding or buckling, and thus result in a local reduction in strength. Redistribution of stresses may take place at the local level and the overall behavior of the member may be affected. However, a local reduction in strength does not necessarily mean that the same reduction in the overall strength of the member will result.

The effect of localized deterioration on the overall behavior of a member will depend on the type of member and the location, nature, and extent of deterioration. For example, a reduction in the moment capacity of a girder in a low moment region is not as critical as a reduction in bending strength at a location of maximum moment.

Extensive deterioration of a member can affect the behavior of the bridge structure as a whole. It can affect the load distribution characteristics of the structure and result in increased load effects in adjacent members and a reduction in load in the deteriorated member. The overall reduction in bridge capacity will depend on the type of structure, the type and location of the deteriorated member and its degree of deterioration. Some members are more critical than others. For example, a reduction in the strength of the suspending hangers of a cantilever truss bridge will directly affect the load-carrying capacity of the structure with no possibility for load redistribution. Severe deterioration of a girder in a multigirder span will result in redistribution of loads before failure. In general, the more redundant the structure, the lower the effect of single member deterioration.

The evaluation of the effects of deterioration at the local,

member, and structure level will depend on the method of analysis and the evaluation criteria used.

2.4 EFFECTS OF METHOD OF ANALYSIS

2.4.1. Analytical Modeling Versus Actual Response

All methods of analysis are based on idealized analytical models of the bridge structure. The analytical evaluation of the behavior of a bridge is usually quite conservative. Results of field tests in the elastic range have shown that the actual elastic response of a bridge is less than the analytical predictions. Load testing of bridges to failure and actual failures of bridge components have shown that bridges often have the ability to resist higher loadings even after severe damage or even failure of a primary structural member. The increased load-carrying capacity which the analytical procedures do not usually account for is mostly due to interaction among various bridge components, i.e., unexploited load paths, and to restraint conditions at joints, bearings or girder web boundaries.

Thus, bridges will generally have a greater structural capacity than indicated by the most commonly used methods of analysis.

2.4.2 Elastic Analysis Versus Nonlinear Analysis

The analysis of the behavior of a bridge structure can be limited to its linear elastic range or may be expanded to include the post-linear elastic regime of the structure. A wide range of possible stress and load redistributions usually exists as a structure passes from an initial localized yielding in a member, to failure of that member, and finally to total bridge collapse. Analysis in the inelastic range can account for these load redistributions and quantify the ability of the structure to resist additional loads even after initial yielding of a bridge member. However, methods of analysis involving inelastic behavior at a structure level are quite complex and computer-based, and are not readily available to the office engineer. Also, in most practical cases recommendations for bridge repair are usually made before resorting to inelastic analysis.

Therefore, it is recommended that the structural analysis for evaluating corrosion effects be linear elastic, and only when a more refined analysis is needed that a nonlinear analysis be used. For example, a more refined analysis may be used as an emergency interim solution to keep a vital bridge in service until it can be repaired. In any case, an initial elastic analysis is required in order to assess the effects of corrosion on serviceability and fatigue resistance.

2.4.3 Changes in the Method of Analysis

More accurate methods of analysis are available today to the bridge engineer. They are the result of recent developments in computer hardware and software and extensive analytical and experimental studies on bridge behavior. It can be useful to relate the method of analysis used for the evaluation of corrosion effects on a given bridge to the original design method used for that bridge. Use of an analysis scheme more refined than the original design may indicate the existence of additional reserve capacity in excess of any strength reduction caused by corrosion losses.

2.5 EVALUATION CRITERIA

2.5.1 General

The conditions created by corrosion can result in various modes of failure that are not necessarily those that controlled the original design of the bridge. In order to evaluate the condition of a bridge affected by corrosion, several criteria need to be considered: strength, deformation, stability, fatigue, fracture, redundancy, and criticality of member or detail.

For each condition, the applicable criteria must be identified and addressed. It can be beneficial to relate these criteria to the original design criteria of the bridge, if available. A brief description of the evaluation criteria is given below.

2.5.2 Strength Criteria

The residual strength of a deteriorated member may be determined by using a service load approach or a load factor approach.

Service Load Approach. The service load or the allowable stress approach uses the attainment of first yielding as a basis for defining the limit state of structure or a member. Safety is ensured by limiting stresses to allowable values, which are below the elastic limit of steel. This facilitates the use of a linear elastic method of analysis. In some cases, the service load approach recognizes the possibility that yielding due to stress concentrations or residual stresses may take place at service load levels without resulting in unrestrained plastic flow and section failure. For example, in axially loaded members, uniform stress distribution is assumed in spite of the possible existence of bolt or rivet holes, residual stresses, or other stress concentrations. The service load approach has been used for the design of most of the existing steel bridges and it still is the most common approach to bridge design and rating. It is needed for serviceability, fatigue and fracture evaluations, even if a load factor approach is used.

Load Factor Approach. A strength or a load factor approach uses the ultimate strength of a member as a limit state. Safety is ensured by limiting the load to a level below that which would cause failure of a member or collapse of the structure. A load factor approach can recognize reserves of strength beyond first yield that may result from stress redistributions. Current practice is to use a linear elastic method of analysis to determine member loads and then use a strength approach at the member level. Strength type approaches are becoming more and more accepted in bridge design and rating. They include the current AASHTO load factor method and the more recent, a trend towards probability-based load and resistance factor approach. The load factor method specifies load factors for the different types of loads encountered, while the load and resistance factor approach forms a basis of defining load factors and resistance factors. In many cases, a strength type approach will yield a higher member capacity. For example, the plastic moment of a beam with a rectangular cross section is 50 percent larger than its yield moment. A comparison of bridge ratings based on allowable stress and ultimate strength methods can be found in Ref. 5.31. The example bridges considered in Ref. 5.31 include composite and noncomposite steel beam and girder bridge structures with a wide range of span lengths ranging from 27.0 ft to 112.5 ft. Both simple and continuous spans are included. For the cases studied, increases from 11 percent to 21 percent were achieved for the inventory and operating rating factors, respectively, by using a load factor rating.

It is recommended that a service load approach be used for an initial evaluation of corrosion effects. This approach is consistent with the original design of most existing bridges and is also needed for fatigue, fracture, and serviceability evaluations. If the results are not conclusive, a load factor approach that is able to recognize some reserves in strength may be employed.

2.5.3 Deformation Criterion

The deformation criterion is primarily related to the serviceability of the structure. Loss of material due to corrosion may lower the stiffness of the structure and result in unacceptable deflections and deformations. When the deformations become inelastic, the strength of the structure may also be affected. The approach used to verify the deformation criterion is the service load approach with an elastic method of analysis. Corrosion can reduce the stiffness of members and thus result in increased deformations.

2.5.4 Stability Criterion

The stability criterion includes local instability, member instability, and structural instability. Instability can initiate in the elastic or the plastic range. In many cases the stability criterion will control the design of a member or structure. Stability is ensured by modifying allowable stresses if a service load approach is used or by modifying the ultimate strength criteria. Corrosion can induce eccentricities and reduce section properties such as moment of inertia and radius of gyration and thus lower the resistance to local or overall buckling.

2.5.5 Fatigue Criterion

The fatigue criterion addresses the behavior of the structure under repeated loading. It has to ensure that no fatigue cracks develop during the expected life of the structure. Fatigue cracks are generally initiated in regions of maximum tensile stresses at points of stress concentration such as holes, notches, or other imperfections and discontinuities. The technique used to verify the fatigue criterion is the service load approach, with an elastic method of analysis. Some of the conditions created by corrosion can affect the fatigue resistance of a structure. Uniform corrosion results in surface roughness which corresponds to localized stress raisers on the surface. Further discussion on this subject is given in Chapter 5 of the office guidelines. Localized corrosion can create eccentricities, holes, and other abrupt discontinuities which can result in a reduced fatigue resistance. Corrosion fatigue, which may occur when the structure is exposed to a corrosive environment, can also reduce the fatigue resistance. Fretting corrosion can initiate cracks and thus adversely affect fatigue performance. In general, the effect of corrosion on a member will depend on its original condition. The effect on a rolled member will be more significant than the effect on a member with poor fatigue details such as weldments or rivet holes.

2.5.6 Fracture Criterion

The fracture criterion addresses the possibility of a member

fracture. The fracture can be either brittle or ductile. Brittle fracture occurs without prior yielding, while ductile fracture is generally preceded by some local plastic deformation. Certain service conditions such as low temperature, impact loading of members with severe discontinuities and conditions of high constraint that restrict the capacity for local yielding greatly affect the susceptibility to brittle fracture. Fracture also occurs at discontinuities that grow to a critical size as a result of fatigue or stress corrosion. The fracture criterion is based on service load conditions.

The fracture of a critical bridge member can cause the bridge to collapse without prior warning. Therefore, special attention should be given to fracture critical bridge members affected by corrosion. Guidance for the inspection and evaluation of fracture critical members can be found in Ref. 6.8.

2.5.7 Redundancy

2.5.7.1 General

A redundant structure is a structure where failure of a single member cannot lead to total collapse. Redundancy is related to the ability of a structure to redistribute loads after one or more of its components fail. Evaluation of structural redundancy requires a good understanding of the behavior of the structure and of the importance of the damaged member. Redundancy is becoming increasingly accepted as a criterion in the design of new bridges and in the evaluation of existing bridges. The effects of corrosion on a highly redundant bridge structure will not be as significant as on a bridge structure with very little redundancy. Many existing bridges are unintentionally redundant. There are many actual cases in which failure of a bridge member or connection, even though considered critical in the original design, did not result in total bridge collapse. There are cases in which one channel of a two channel built-up bottom chord in a truss bridge failed and the bridge still carried dead and live load, cases in which the whole bottom chord of a truss failed and the floor system carried the load, and cases in which a girder of a two girder continuous bridge failed and the bridge did not collapse. In other cases, however, failure of one member, such as the end post of a truss, or a connection, such as a pin-hanger or eyebar joint, caused a bridge collapse.

2.5.7.2 Qualitative Evaluation of Redundancy

At present, the general approach to redundancy has more of a qualitative than quantitative nature.

In the FHWA manual "Inspection of Fracture Critical Members," redundancy is divided into three parts: load path redundancy, structural redundancy, and internal redundancy. In a load path redundant structure there are members that are capable of temporarily carrying the load of a damaged or deteriorated member. For example, a girder bridge with three or more girders is defined as load path redundant. A structurally redundant member has continuity within the load path. For example, a continuous girder, or any statically indeterminate structure, may be defined as structurally redundant (AASHTO, however, does not consider a continuous girder as redundant). A member with internal redundancy is made of a number of independent ele-

ments such that failure of one element does not result in the failure of the other elements. For example, built-up riveted girders have internal redundancy while welded plate girders do not. Welded elements are not independent, and a crack that develops in one element can spread to other elements.

2.5.7.3 Quantitative Evaluation of Redundancy

Recently, considerable progress has been made in defining ways to quantify the effects of redundancy. It has been suggested (7.24, 7.25, 7.26, 7.52) that redundancy be defined in terms of a structural redundancy factor, R , as follows:

$$R = \frac{C_s}{C_s - C_{ds}} = \frac{1}{1 - RCFs} \quad (1)$$

where C_s is the capacity of the intact structure, C_{ds} is the capacity of the structure in a damaged condition (for example, after one member has failed), and $RCFs$ is the residual capacity factor of the structure. The residual capacity factor is defined as the ratio of the deteriorated capacity to the intact capacity. Thus, the redundancy factor, R , can be expressed in terms of the residual capacity factor of the structure, $RCFs$. The value of the redundancy factor, R , ranges from 1.0, when the damaged structure has no residual capacity, to infinity, when the damaged member has no effect on the overall capacity of the structure. The redundancy factor, R , will depend on the effect of damage of a member on the behavior of that member, the location of the damaged member, and the effect of the damaged member on the behavior of the structure. Results of analytical investigations into the effects of member damage on the structural redundancy factor, R , of a girder and a truss bridge model may be found in Refs. 7.25 and 7.26. The analytical investigations were based on three-dimensional, nonlinear finite element analysis and included various types of material behavior (i.e., brittle, ductile, semibrittle) and damage scenarios.

In general, by taking into account the redundancy of a structure affected by corrosion, a more accurate evaluation can be obtained. However, in order to fully account for the reserve in strength of a redundant structure, a nonlinear analysis procedure is needed.

2.5.8 Criticality of Member or Detail

The criticality of a bridge member or detail is related to the consequence of failure of that member or detail. In some cases, failure of a member has little effect on the structural integrity of the bridge, while in other cases it can cause sudden collapse. The criticality of a bridge member is determined by the following factors: location and function, redundancy, and mode of failure.

Not all members of a bridge control its load-carrying capacity. For example, the posts in a Warren through-truss provide bracing to the top chord but do not carry any primary dead load or live load. The top chord members and the bottom chord members, however, directly carry the compression and the tension loads from dead load and live load. The importance of web members increases from mid-span to the end of the span.

Not all members of a bridge that control its load-carrying capacity are equally critical. If a member is highly redundant (made of several parallel elements) and is not required for stabil-

ity, it will be able to sustain failure of one of its elements without serious consequences. Damage of a member which is not internally or structurally redundant, however, can result in the collapse of a single load-path structure.

The mode of failure a member is likely to undergo also affects its criticality. Slow deterioration of a bending member is not as critical as sudden failure of a tension member due to fracture or sudden failure of a compression member due to instability.

Thus, the most critical members in a bridge structure are nonredundant members which control the load-carrying capacity and whose failure would be expected to result in a sudden bridge collapse. They include tension members defined by AASHTO as fracture critical members (FCM's), and compression members which can fail through instability. These members should receive a more rigorous evaluation.

2.5.9 Changes in Design Criteria

The developments in methods of analysis, the continuing bridge-related research, and the experience accumulated over the years have led to changes in the criteria for bridge design. It may prove useful to relate the criteria used for evaluating corrosion effects on an existing bridge to the original design criteria of that bridge. In most cases, it will be found that the original design criteria regarding the assessment of the resistance of bridge members are more conservative. The evaluation criteria used may be related to the original design criteria through a code factor, CF , defined as the ratio of the capacity of a member calculated based on the present criteria, to the capacity of that member calculated based on the original design criteria. A member code factor, CF , larger than 1.0 would indicate that the member has some capacity in excess of that assumed in the original design.

When taking into account the changes that occurred in the design criteria related to member resistance, the changes in the loading of bridges that occurred over the years should also be considered if a load-carrying capacity evaluation is made.

2.5.10 Factors of Safety

The factors of safety used affect the evaluation of the residual capacity and the remaining load-carrying capacity of a bridge member or structure. They have little effect on the evaluation of residual capacity factors. When calculating residual capacity factors, the same safety factors will appear in both the numerator and the denominator and in most cases they will cancel out.

In the service load approach, factors of safety are included in the allowable stresses specified by AASHTO. In the load factor approach, factors of safety are included as load and capacity reduction factors and they account for the uncertainties of loadings and structural response. The ultimate cross-sectional capacity is assumed for determining member resistance. In the load and resistance factor approach, factors of safety are included in the form of load factors and resistance factors. They are probability-based and account for uncertainties in both the loadings and the structural resistance.

The factors of safety used for bridge evaluation are usually smaller than those used in design. AASHTO uses an inventory and operating level for bridge rating, and the Ontario Highway Bridge Design Code (6.12), permits a reduction in the live load

factor from 1.40 to 1.25 for the evaluation of existing bridges.

2.6 STRESS CONCENTRATIONS

2.6.1 General

Stress concentration is the localization of high stresses in the vicinity of a hole, notch, pit, groove or other discontinuity. Some of the conditions created by corrosion can result in stress concentrations. Because both the AASHTO *Manual for Maintenance Inspection of Bridges* and the AASHTO specifications for design do not address stress concentrations directly, an overview of the effects of stress concentration is presented here.

Stress concentrations are measured by stress concentration factors. Three types of stress concentration factors can be identified: elastic stress concentration factors, effective stress concentration factors, and fatigue stress concentration factors.

2.6.2 Elastic Stress Concentration Factors

The elastic or theoretical stress concentration factor, K_t , is defined as the ratio of the maximum local stress to the average stress, calculated based on net section. It assumes linear elastic behavior of the material. Values of K_t , determined for different geometries and loading conditions, may be found in a number of references (7.32, 7.38, 7.44, 7.45, 7.46). In order to illustrate the effect of geometry on the theoretical stress concentration factor, K_t , consider a round hole in the center of a plate axially loaded in one direction. When the ratio of the radius of the hole, r , to the width of the plate, d , is reduced, the theoretical stress concentration factor, K_t , increases, approaching 3.0 as r/d approaches zero. For an elliptical hole with its major axis perpendicular to the load the stress concentration factor at its edge is

$$K_t = 1 + \frac{2a}{b} \quad (2)$$

where a is the large radius and b is the smaller radius of the ellipse. For large values of a/b , K_t can be approximated by

$$K_t \simeq 2 \left(\frac{a}{r} \right)^{1/2} \quad (3)$$

where r is the radius at the end of the major axis. This expression shows that K_t will increase with the length of the hole, a , and approach infinity for a very sharp corner, as r approaches zero.

In most practical cases, however, steel will behave in a ductile manner and yield ahead of the stress concentration. The plastic yielding and the resulting redistribution of stress, which may even occur in relatively brittle materials, cause stress concentrations to have less effect on strength than might be expected from considering elastic stresses only. The practical significance of stress concentration will depend on factors such as the type of metal and the type of loading.

2.6.3 Effective Stress Concentration Factors

In the case of ductile materials, the theoretical stress concentration factor, K_t , has very little practical significance when

evaluating static strength. For loadings beyond yielding, K_t becomes meaningless and the use of an effective stress concentration factor, K_e , is more appropriate. The effective stress concentration factor takes into account plastic yielding and redistribution of stress. It approaches unity as the load increases and the whole section is at yield stress. Thus, stress concentrations in ductile materials will not reduce their static strength (other criteria, however, may be affected, i.e., deformation, stability, or fatigue). In most cases the structural steel used on bridges has good ductility.

In the case of brittle materials under static loading, only limited yielding may take place and the effective stress concentration factor, K_e , will approach the K_t factor.

In some cases such as impact loading, low temperature, or conditions of high restraint such as might be found in thick plates, ductile materials may behave in a brittle manner in the presence of severe notches. These cases can be evaluated using fracture mechanics methods. In most practical cases, however, corrosion will not result in the type of very severe notches which would require a fracture mechanics approach.

2.6.4 Fatigue Stress Concentration Factors

The effect of stress concentrations on fatigue resistance can be quantified by using a fatigue stress concentration factor or fatigue notch factor, K_f . The factor K_f is the ratio of the fatigue strength without stress concentration to the fatigue strength with the given stress concentration. It may vary with the type of material and size of member as well as with the number of applied load cycles (7.45). The available experimental data show that the fatigue stress concentration factor, K_f , is almost always less than the factor K_t . The notch-sensitivity ratio, q , defined as

$$q = \frac{K_f - 1}{K_t - 1} \quad (4)$$

is used (7.38) to relate the fatigue stress concentration factor, K_f , to the theoretical stress concentration factor, K_t . The value of q varies from 0, for no stress concentration effect (when $K_f = 1$), to 1.0, for full theoretical effect (when $K_f = K_t$). A chart showing the relation between the notch radius, r , and the notch sensitivity, q , for different steels, based on test data, is given in Ref. 7.38. The test data used were found to be within reasonable scatter bands. It shows that as the radius of the notch decreases to values under, say, 0.05 in., a sharp decrease in the notch sensitivity, q , occurs. Thus, very small holes or knife edge discontinuities appear to result in a smaller fatigue strength reduction than that predicted by theoretical stress concentration factors.

Several methods for calculating K_f from K_t have been proposed (7.45). One of the formulations suggested, developed by Heywood, is

$$K_f = \frac{K_t}{1 + 2 [(K_t - 1)/K_t] [5000/(F_u r)]^{1/2}} \quad (5)$$

where r is the root radius of the notch in inches and F_u is the ultimate tensile strength in pounds per square inch. This formula is relatively simple and it showed good agreement with a large number of test results (7.45). Tables showing the fatigue stress concentration factor as a function of the geometry of the notch and the type of steel can be found in Ref. 7.44.

It is important to emphasize that there are many limitations to the evaluation of fatigue stress concentration factors and that they cannot be applied with confidence to all situations. At present, the most reliable way to estimate the effects of stress concentration on fatigue strength is through full-scale fatigue tests on the actual conditions, if possible.

The British Code of Practice for design of bridges (6.4) uses the fatigue stress concentration factor concept to define the fatigue resistance of details such as unreinforced apertures and reentrant corners.

In the case of discontinuities and holes created by corrosion it is very difficult to define a clear transition or notch radius. The actual surface or boundary of a corroded section is very irregular with small and sharp notches. The effects of the surface roughness created by corrosion on fatigue resistance are discussed in Chapter 5.

CHAPTER 3

EVALUATION PROCEDURE

3.1 INTRODUCTION

3.1.1 General

The procedure suggested to evaluate corrosion effects on bridges has three phases.

Phase 1 includes preparatory steps the engineer in charge of the evaluation should take before the field inspection. The purpose of these steps is to increase the effectiveness of the inspection. They include collecting the necessary bridge data, understanding the structure behavior, and coordinating the purpose of the inspection and type of information needed with the bridge inspector.

Phase 2 is a qualitative evaluation of corrosive effects. It identifies the criticality of the conditions created by corrosion and the urgency of required corrective actions. It provides a fast initial evaluation of the condition of the bridge and identifies the details or the members to be examined in more detail in a quantitative evaluation.

Phase 3 is a quantitative office evaluation, which determines the residual capacity of a deteriorated bridge. The approach suggested for the quantitative evaluation is described in the following section.

3.1.2 Quantitative Evaluation

A two-level evaluation approach is suggested to keep the analysis effort at reasonable levels. If, for example, a simple, approximate, analysis procedure can give a clear indication of the condition of a bridge, and determine whether or not there is a need for repair, there will be no need for a more accurate and expensive analysis.

Because each case of corrosion can have different effects, no definite criteria are given as to what constitutes a Level I or Level II office evaluation.

In general, it is recommended that for an initial (Level I) evaluation, an approach philosophy similar to that used in the original design of the bridge be employed. This could include, for example, using an elastic analysis with a service load method. By using an approach consistent with the original design, the effects of corrosion can be related to the intended capacity of the bridge according to its original design criteria. When determining if the bridge is acceptable by today's standards, changes in design criteria and loading must be taken into account.

For a more accurate (Level II) evaluation, a different approach philosophy may be used. For example, a three-dimensional linear or nonlinear analysis and an ultimate strength method for determining bridge resistance may be employed. A Level II evaluation may show additional strength that was not taken into account in the original design of the bridge. While a Level I evaluation can be done by an entry level engineer, a Level II evaluation will usually require at least a professional engineer (P.E.) background, some post-graduate education or equivalent experience, depending on the situation.

Before a Level II evaluation is performed, its expense should be weighed against the actual cost of the needed repair.

3.2 FIELD DATA NEEDED FOR OFFICE EVALUATION

The office evaluation of an existing bridge is directly related to its field inspection. The extent and the accuracy of the office evaluation is dependent on the type, amount, and accuracy of the field data available. A separate set of guidelines for field inspections has been developed in Part I of this report. The office evaluator should be familiar with the type and accuracy level of field information which is described in the field inspection guidelines. The minimum field information needed for the office evaluation of the effects of corrosion damage should include the following items:

1. *Location of damage*—on which members or details and where on those members.
2. *Nature of damage*—material loss (uniform, localized), frozen bearings (open position, closed position), frozen pin-hanger assemblies, shifted members or elements, buckled members or elements, deformed components, and misalignments.
3. *Amount and geometry of damage*—amount of section loss, location of loss within the cross section, and degree of roughness; degree of fixity for bearings or pin-hanger assemblies; and amount of movement or distortion for shifted, buckled or deformed elements.
4. *Extent of damage*—how many members or elements are affected, and what is the extent of damage along each member.
5. *Environmental conditions*—arid, humid, coastal, industrial, and exposure to deicing salts.

3.3 PHASE 1—STEPS BEFORE BRIDGE INSPECTION

It is important to understand the purpose of the evaluation in advance and to discuss the goals and techniques of the evaluation with the inspector before the field inspection.

The following steps are suggested before the bridge inspection actually takes place:

- Step 1*—Understand the intent of the bridge evaluation. Are there any specific problems or is it for the purpose of load rating? What is the urgency?
- Step 2*—Collect bridge data (plans, specifications, design criteria, previous rating and inspection reports if available, and results of any stress measurements or load tests if performed).
- Step 3*—Understand the bridge behavior and identify critical members.
- Step 4*—Discuss and coordinate with the inspector the purpose, the extent, and the type of information needed.

3.4 PHASE 2—INITIAL QUALITATIVE EVALUATION

Examine the inspection report and address the following items in a qualitative manner:

1. *Location of damage*—Is the damaged member or detail critical to the bridge integrity? (e.g., fracture critical member or main compression members in nonredundant systems). Is the damage on the member located at a critical location? (i.e., flange loss at midspan of a simply supported girder or web loss at a girder support).
2. *Nature of damage*—Can the damage result in a sudden catastrophic failure without warning? (e.g., distortions in critical compression members or stress corrosion). Is the nature of the damage progressive, resulting in a gradual deterioration? (e.g., uniform corrosion loss).
3. *Amount and geometry of damage*—Is the amount of damage significant? (e.g., high percentage of section loss, large distortions, or full fixity of bearings). Is the geometry of damage such that areas of high stress concentrations (like deep pits or notches) or eccentricities are created?
4. *Extent of damage*—Are there many members or details affected by corrosion? Is the damage limited to a small portion of the member or does damage extend along its entire length? Extensive deterioration of a member may affect the load distribution characteristics of the bridge.
5. *Environmental conditions*—What are the environmental conditions? (e.g., arid, humid, coastal, industrial, exposed to deicing salts). They may reflect the cause and the form of corrosion and give an indication of the future progression rate.

3.5 PHASE 3—QUANTITATIVE EVALUATION

Using the guidelines and the simplified analysis techniques given in the chapters that follow, perform an initial, Level I quantitative office evaluation; the extent and the accuracy of the evaluation should be based on the findings of Phase 2:

- Step 1*—Identify possible failure modes and define relevant evaluation criteria (e.g., buckling or fatigue).
- Step 2*—Assess local effects (e.g., localized yielding or buckling).
- Step 3*—Determine effects on overall member strength.
- Step 4*—Determine effects on the bridge structure as a whole.
- Step 5*—Determine if a more refined, Level II office evaluation is needed. If the initial evaluation does not give a clear indication of the condition of the bridge, a more accurate evaluation may be used. However, the expense of a more sophisticated analysis should be weighed against the actual cost of the needed repair.

- Step 6*—Determine if a Level II field inspection is needed. If more accurate field data are needed, or corrosion forms not readily identifiable by visual observation are suspected, a Level II field inspection may be necessary. The inspection should be coordinated with the office engineer in charge of the evaluation (for example, there is no need to provide field data more accurate than the method of analysis used in the evaluation process).

The results of the office evaluation should enable the engineer to determine the need for bridge repair and the urgency of any required repair. The engineer could also consider the need for posting, regulating permits or even bridge closure. Factors such as frequency of further inspections, maintenance procedures, amount of traffic on the bridge and bridge importance should also be taken into account.

3.6 QUANTIFICATION OF DAMAGE REPORTED

In order to quantify the corrosion damage measured and reported by the field inspector, the following parameters are used throughout this study:

1. *Percentage of section loss, % loss*—The percentage of section loss defines the amount of metal loss at a given location on a bridge member. It relates the amount of section loss to the original section of the member:

$$\% \text{ loss} = \left(1 - \frac{Ad}{A} \right) 100 \quad (6)$$

where A is the original cross-sectional area and Ad is the reduced section area. The term “percentage loss” is familiar to bridge inspectors and is frequently used in inspection reports.

2. *Loss coefficient, Q* —The loss coefficient, Q , also describes the amount of metal loss at a given location along a member. It is defined as the ratio of original section area, A , to the reduced section area:

$$Q = \frac{A}{Ad} = \frac{100}{100 - \% \text{ loss}} \quad (7)$$

The advantage of this parameter is that it is linearly proportional to the increase in stress in an axially loaded bar. Thus, it facilitates comparison of the effects of material loss in various members to the simple case of loss of section in an axially loaded bar.

3. *Length of loss, ℓ* —The length of loss, ℓ , defines the extent of loss along a member. It is usually assumed that along the length, ℓ , the percentage of section loss is constant.

4. *Transition from reduced to full section*—The type of transition from reduced to full section has to be defined as well. The transition can be abrupt or gradual, at a given rate or have a given radius.

3.7 PARAMETERS FOR QUANTIFYING THE EFFECTS OF DAMAGE

In the following, the parameters, terms and abbreviations used

to quantify the effects of corrosion damage throughout this study are summarized:

1. *Percentage increase in stress*—The percentage increase in stress is used in the analytical studies (Appendix E) in order to quantify the effects of material loss or unintended fixities on the damaged member or on other adjacent members.

2. *Percentage increase in deflection*—The percentage increase in deflection is used in the analytical studies in order to quantify the effects of material loss on member or structure stiffness.

3. *Residual capacity factors*—The residual capacity factor concept is used in the office guidelines in order to define and illustrate the effects of corrosion on structural resistance. It is used to assess damage at the local, member, or structure level.

The damage effects at the local level are quantified by using a local residual capacity factor, $RCFl$, defined as the ratio of the local strength of the damaged member at the location of the damage to the original strength of the member at the same location. In the case of a pure tension member, the local residual capacity factor equals the ratio of the reduced section area, Ad , to the original section area, A , and can be expressed in terms of the percentage of section loss;

$$RCFl = \frac{100 - \% \text{ loss}}{100} \quad (8)$$

or in terms of the loss coefficient, Q :

$$RCFl = \frac{1}{Q} \quad (9)$$

The damage effects at the member level are quantified by using a member residual capacity factor, $RCFm$, defined as the ratio of the capacity of the damaged member, Cdm , to the capacity of the original member, Cm :

$$RCFm = \frac{Cdm}{Cm} \quad (10)$$

In the case of a pure tension member the member residual capacity factor equals the local residual capacity factor, i.e.:

$$RCFm = RCFl \quad (11)$$

For other members the member residual capacity factor may be smaller or larger than the local residual capacity factor. For example, in a compression member the residual capacity factor at the member level may be smaller than that at the local level if overall buckling governs (see Section 4.3), while in beams the member residual capacity factor is usually larger than the local residual capacity factor when the location of the damage is not at a critical section (see Section 4.6.3).

The damage effects at the structure level are quantified by using a structure residual capacity factor, $RCFs$, defined as the ratio of the capacity of the damaged structure, Cds , to the capacity of the original structure, Cs :

$$RCFs = \frac{Cds}{Cs} \quad (12)$$

The structure residual capacity factor is usually larger or equal to the member residual capacity factor. For example, a local residual capacity factor of 0.6 may result in a member residual capacity factor of 0.8 and a structure residual capacity factor of 0.9. Usually, in order to calculate residual capacity factors at the structure level, a Level II evaluation will be required.

CHAPTER 4

EFFECTS OF MATERIAL LOSS

4.1 MEMBERS LOADED IN AXIAL TENSION

4.1.1 General

Members loaded in axial tension are usually encountered in trusses as main truss members or bracing members, in girder bridges as bracing members, in arch or cable suspension bridges as vertical suspender members, or in cable-stayed bridges as cable stays. They include flexible members such as cables, rods, plates and eyebars, and nonflexible members such as rolled shapes and built-up members.

A tension member may fail to perform its intended function as a result of yielding, rupture, excessive deformation, fatigue, or fracture. Sometimes, under unusual conditions, a member that functions primarily as a tension member will experience small compressive forces that can result in member buckling. For example, the bottom chord of a truss may experience compressive forces when the bearings of the bridge are frozen.

Corrosion forms such as uniform corrosion, localized corrosion, stress corrosion, or corrosion fatigue can have a significant effect on the load-carrying capacity of tension members.

Failure of some tension members may result in the collapse of the bridge. These members are defined by AASHTO as fracture-critical nonredundant bridge members and are subject to more stringent requirements. Failure of other tension members which are highly redundant may not be as critical to the structural integrity of the bridge.

4.1.2 Strength Criteria

4.1.2.1 Uniform Corrosion Losses

a. *General.* Uniform corrosion losses in a tension member are considered here to result in a uniform reduction in the cross-sectional area of the member over most of its length. Using the reduced member area, the residual strength of the member can be determined in accordance with the current AASHTO *Standard Specifications for Highway Bridges*. A service load or a load factor method may be used.

For a tension member, both the service load and the load factor method are based on an ultimate strength approach. The strength of the member is reached when all fibers of its cross section have yielded.

b. *Service Load Method.* For tension members with holes,

such as for rivets or bolts, the current AASHTO service load specifications, Article 10.32, consider both general yielding of the gross area and rupture of the net area as failure limit states (yielding of the net area at fastener holes is not considered to be a limit state of practical significance). In determining the gross area to be used, the AASHTO specifications allow the designer to neglect any open holes smaller than $1\frac{1}{4}$ in. diameter, up to an area equal to 15 percent of the gross member area. When determining the net area to be used for verifying the tensile strength at the reduced section of a tension member, the AISC specifications use an "effective net area" that is less than the actual net area. The effective net area accounts for effects of eccentricities or stress concentrations on the tensile strength of the reduced section. (These effects are not taken into account when verifying the yield strength of the member.)

Prior to 1977, the AASHTO specifications considered yielding of the net area as the only failure limit state for tension members with holes. This provided for a more conservative design.

The difference between current AASHTO requirements and the original design requirements may be taken into account by using a code factor, CF , as defined in section 2.5.9.

Uniform reduction in the cross-sectional area of a tension member causes a proportional reduction in the capacity of the member. Therefore, the residual capacity factor of a deteriorated member may be expressed as a function of the ratio between the remaining area and the original area of the member, as follows:

$$RCFm = \frac{Ad}{A} \quad (13)$$

where Ad is the remaining area of the deteriorated member, and A is the original area. The original and the remaining areas of the deteriorated member may be calculated based on the net or the gross section area defined in AASHTO specifications, Article 10.32, as applicable.

c. *Load Factor Method.* The limit state used by the load factor method for tension members is yielding of all fibers of its cross section. Because the same limit state is also used by the service load method, the evaluation of the residual capacity of a member will be similar. There is a difference, however, in the way factors of safety are applied. While the service load method assigns a factor of safety to the yield strength of the member, the load factor method applies certain load factors to the different types of loads. As a result of this difference, the estimated remaining load-carrying capacity of a deteriorated member calculated based on the load factor method may be different from that calculated based on the service load method. For higher ratios of dead load to live load plus impact (over 0.67), using the load factor method will usually result in a higher load-carrying capacity estimate. However, the residual capacity factor, $RCFm$, of a deteriorated tension member calculated based on the load factor method is the same as that calculated based on the service load method.

4.1.2.2 Localized Corrosion Losses

a. *General.* Localized corrosion losses in a member result in localized reductions in area such as pits, holes, and edge scallops. In addition to the increase in the average stress due to the loss of section, these discontinuities also cause stress concentrations.

A general discussion on stress concentrations is given in section 2.6.

In most practical cases, stress concentrations may be neglected, when evaluating static resistance, because structural steels have good ductility and yielding followed by stress redistribution will most likely occur before sectional failure. In some instances, however, ductility has occurred. Also, it is important to note that other criteria, such as fatigue or fracture, may be affected.

If the resulting stresses are below yielding and the maximum elastic stresses need to be estimated, theoretical stress concentration factors or a special analysis may be used. Diagrams of theoretical stress concentration factors for the most common conditions created by localized corrosion are shown in Figures 58 through 66(b), which are derived from Refs. 7.38 and 7.45. Based on these figures, some observations regarding the effects of the geometry of the deterioration on the elastic stresses in a tension element can be made. For a finite number of multiple notches the stress concentration of the intermediate notches is considerably reduced. The maximum stress concentration occurs at the end notches and is also reduced, relative to that of a single notch, but at a lesser degree. Figure 66(a) shows that, as the extent of the notch, ℓ , increases, the theoretical stress concentration factor, K_t , decreases. The decrease in K_t with ℓ becomes insignificant for values of ℓ larger than 10 to 15 times the transition radius, r . A ratio of $D/r = 18$ was considered in this figure. Figure 66(b) shows that for ratios of D/r larger than 18, the decrease in K_t with D/r becomes insignificant, and Figure 66(a) could then be used. The relation between the extent of the notch and the resulting stress concentration is further investigated in the analytical studies of localized flange losses in girders (see Appendix E).

b. *Service Load Method.* The service load method accepts the possibility of localized yielding in a tension member and defines the allowable stress based on an overall yielding state with unrestricted plastic flow, and not a state of first yield. This is why a uniform stress distribution is usually assumed in axially loaded members, when bolt or rivet holes, residual stresses or stress concentrations are present.

As discussed in section 4.1.2.1, the current AASHTO specifications for the design of tension members even allow overall yielding of the net area at the location of holes smaller than $1\frac{1}{4}$ -in. diameter if the fracture criteria at that location are satisfied. According to AASHTO's approach a tension member with a localized corrosion loss may also be evaluated based on yielding of the gross area and fracture of the reduced area. However, when the length of the reduced area exceeds the member depth or constitutes an appreciable portion of the member length, yielding of the net area may lead to excessive elongation (of the member). Also, when evaluating a member for fracture of the net area, effects of eccentricities and stress concentrations may be significant and should be considered. Localized corrosion losses are irregular in shape and their extent is often difficult to define and report. Therefore, it is suggested that when evaluating effects of localized corrosion on the strength of tension members, yielding of the reduced area be considered as the governing failure limit state. This will also keep the evaluation process simple. Thus, the residual capacity factor, $RCFm$, may be calculated from

$$RCFm = \frac{Ad}{A} \quad (14)$$

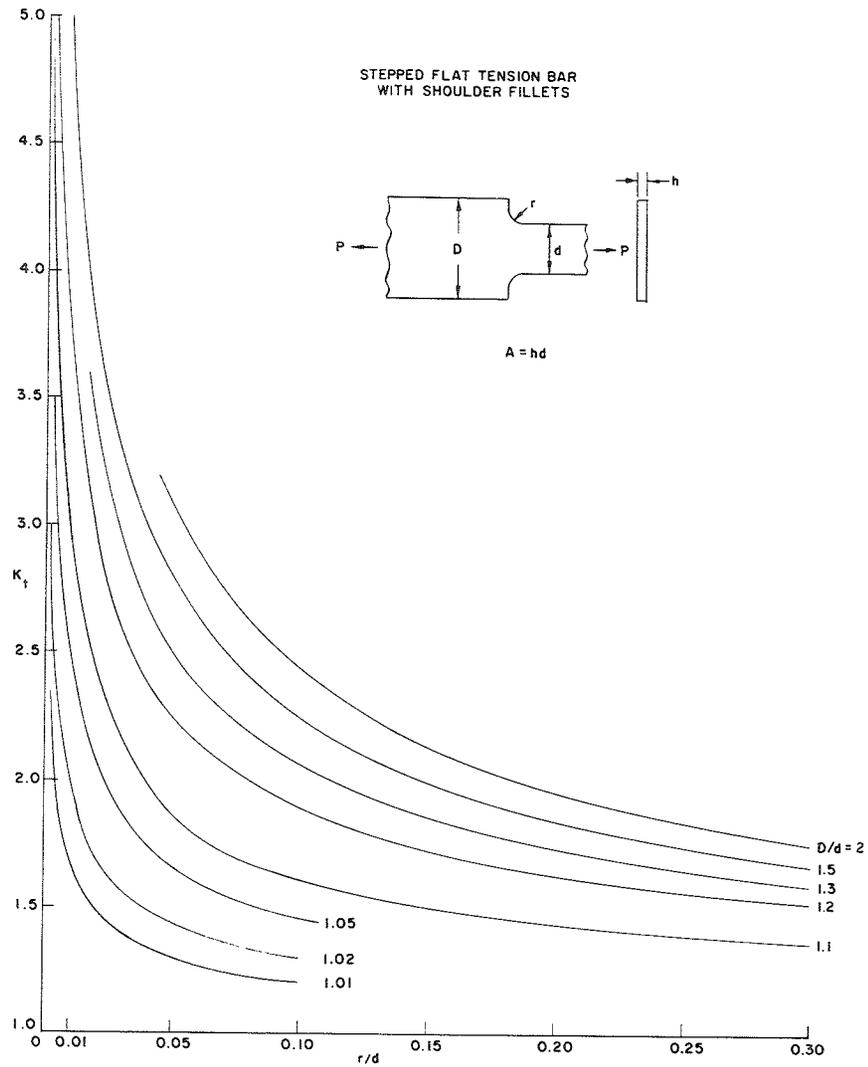


Figure 58. Theoretical stress concentration factors—Case 1.

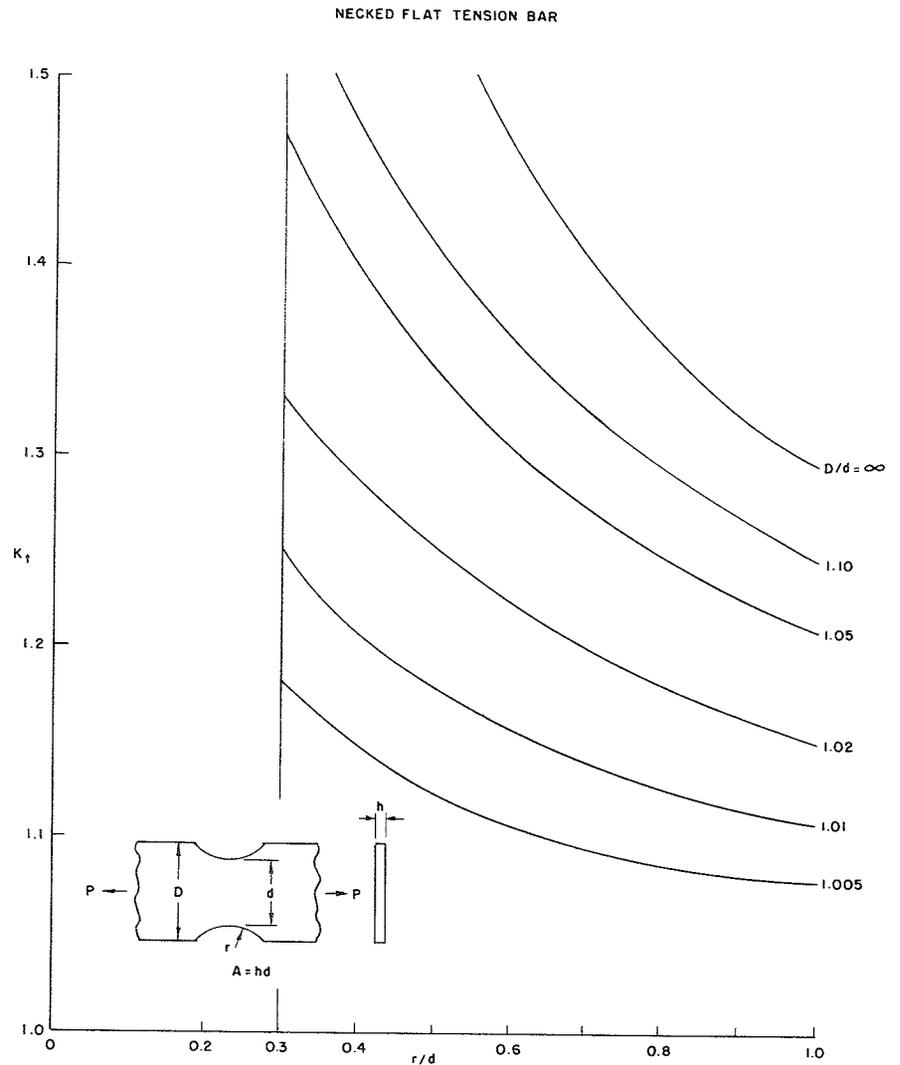


Figure 59. Theoretical stress concentration factors—Case 2.

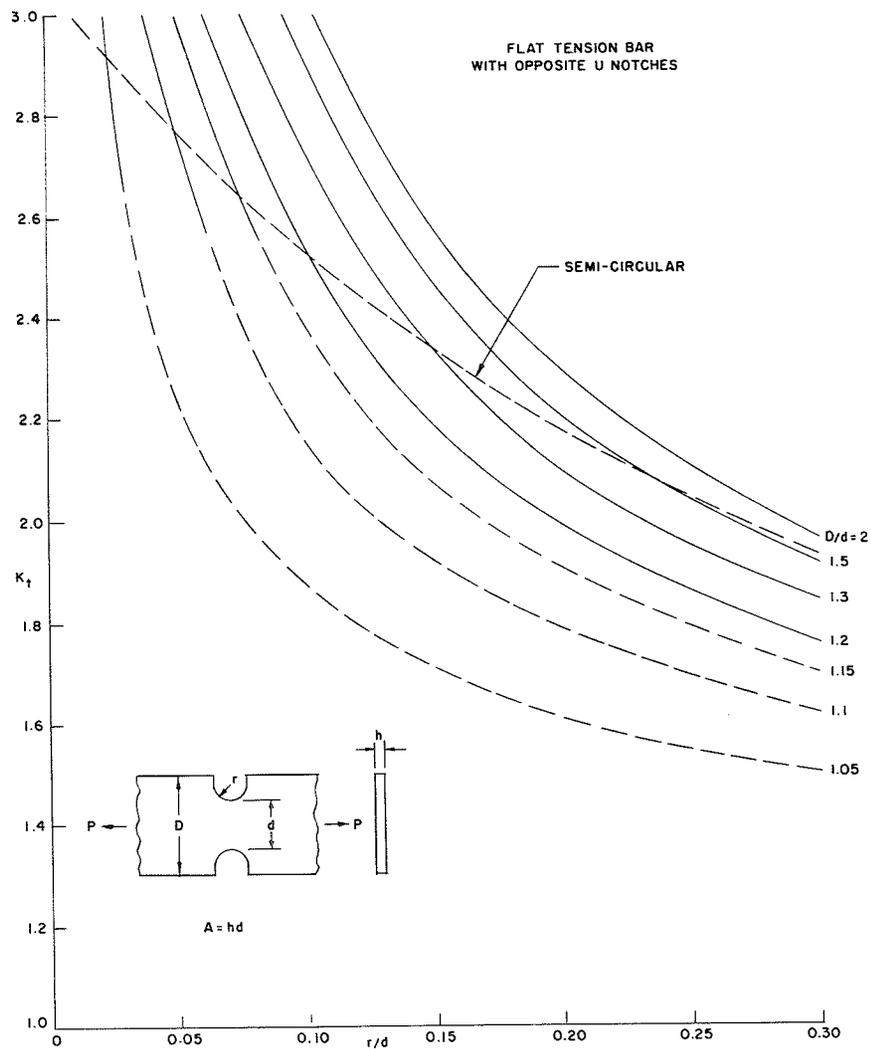


Figure 60. Theoretical stress concentration factors—Case 3.

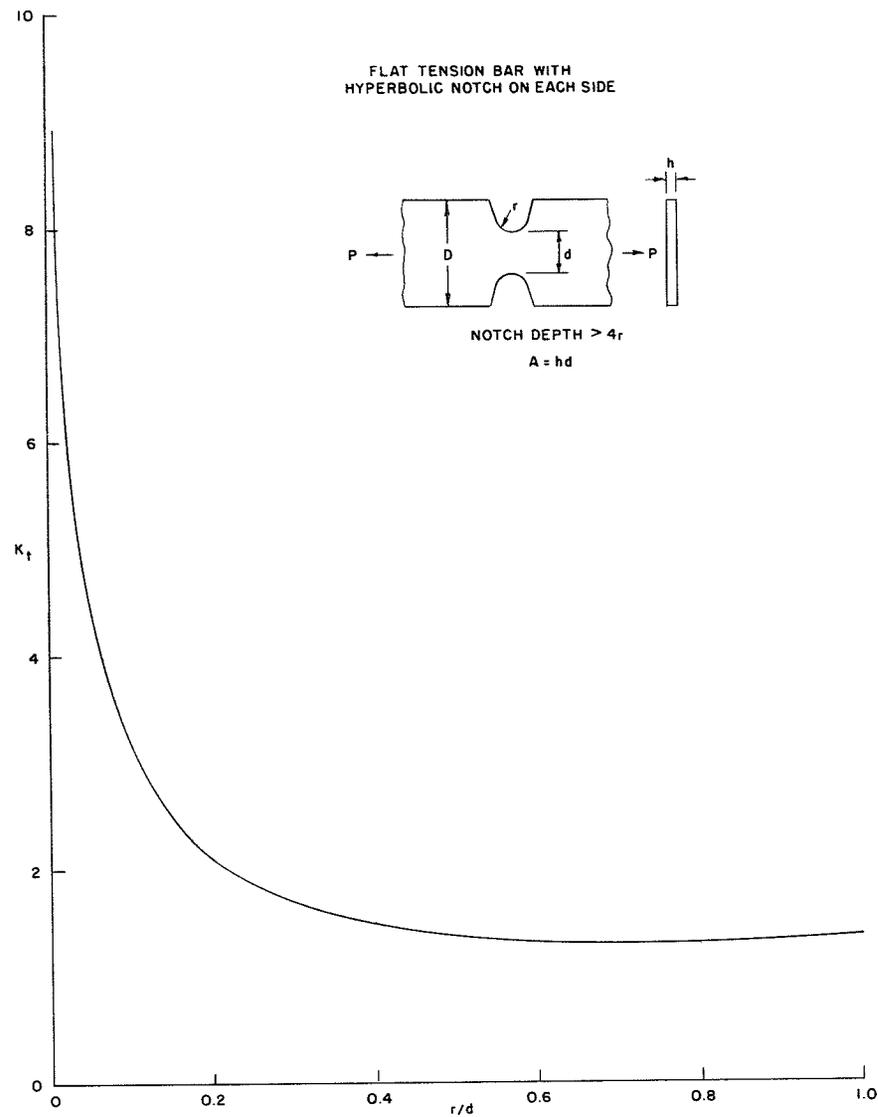


Figure 61. Theoretical stress concentration factors—Case 4.

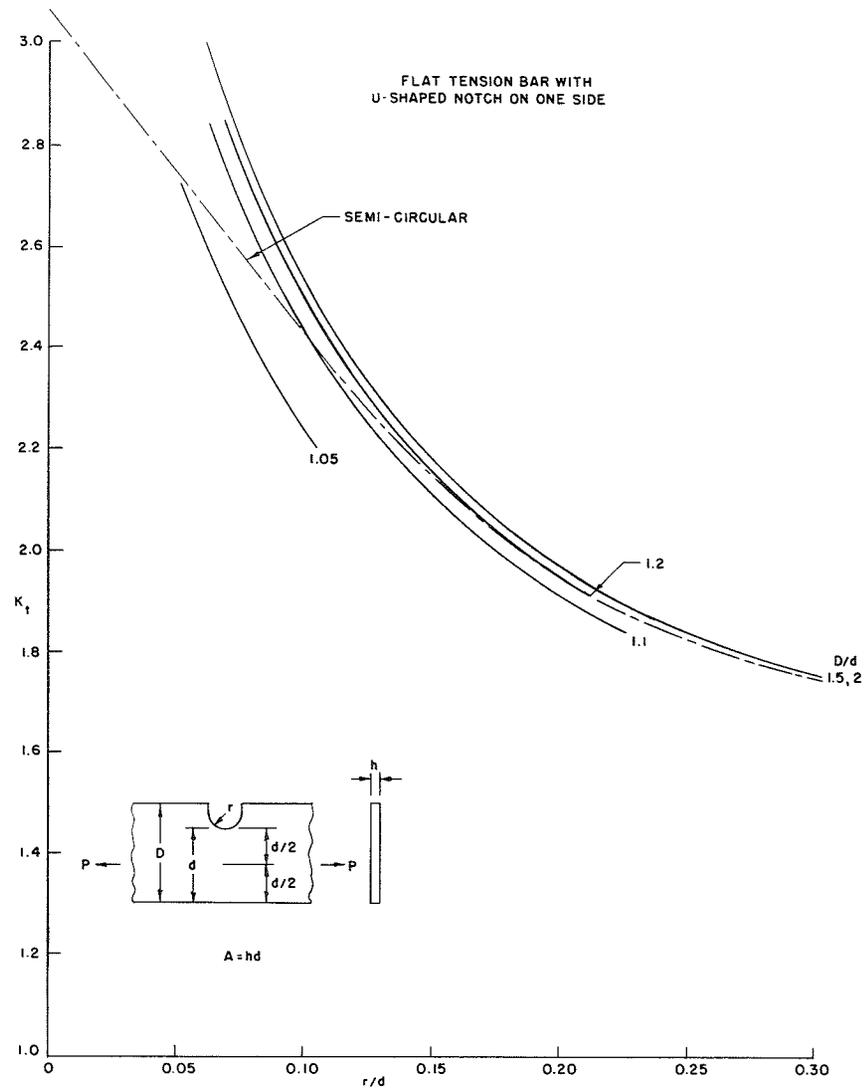


Figure 62. Theoretical stress concentration factors—Case 5.

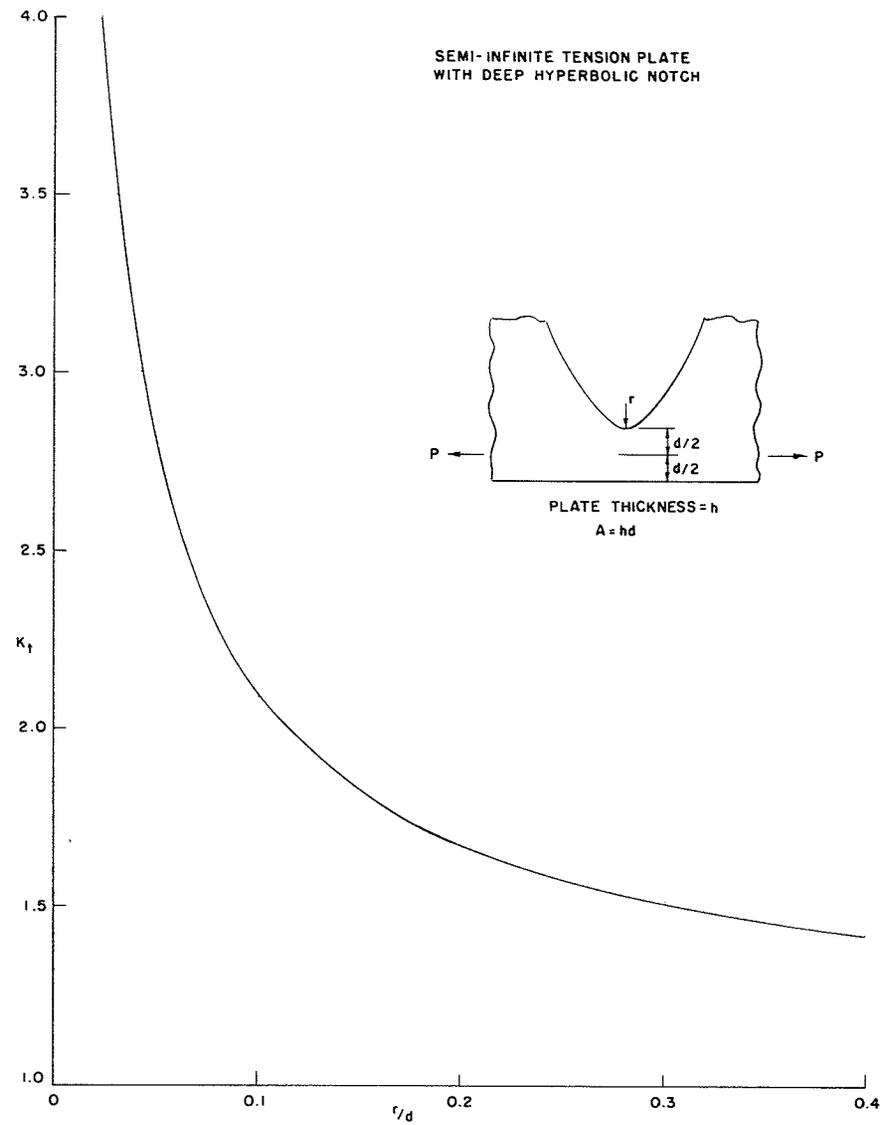


Figure 63. Theoretical stress concentration factors—Case 6.

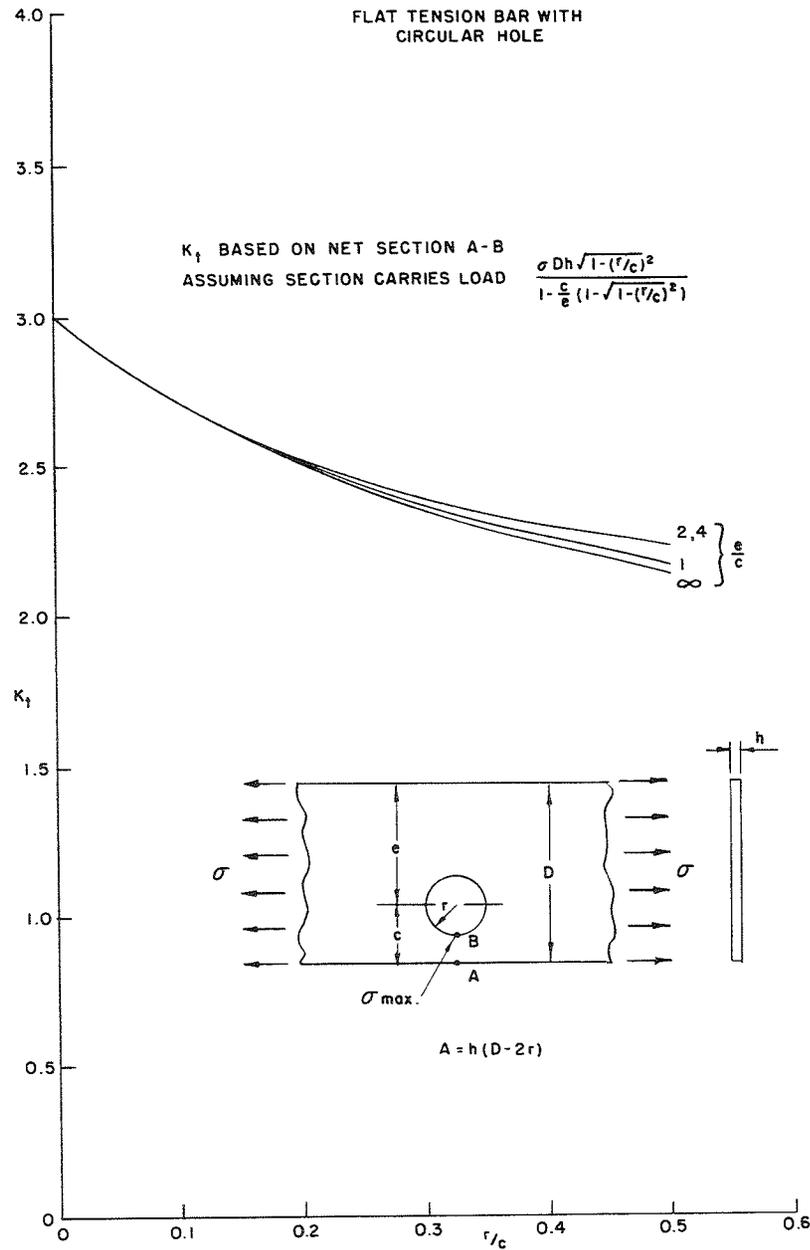


Figure 64. Theoretical stress concentration factors—Case 7.

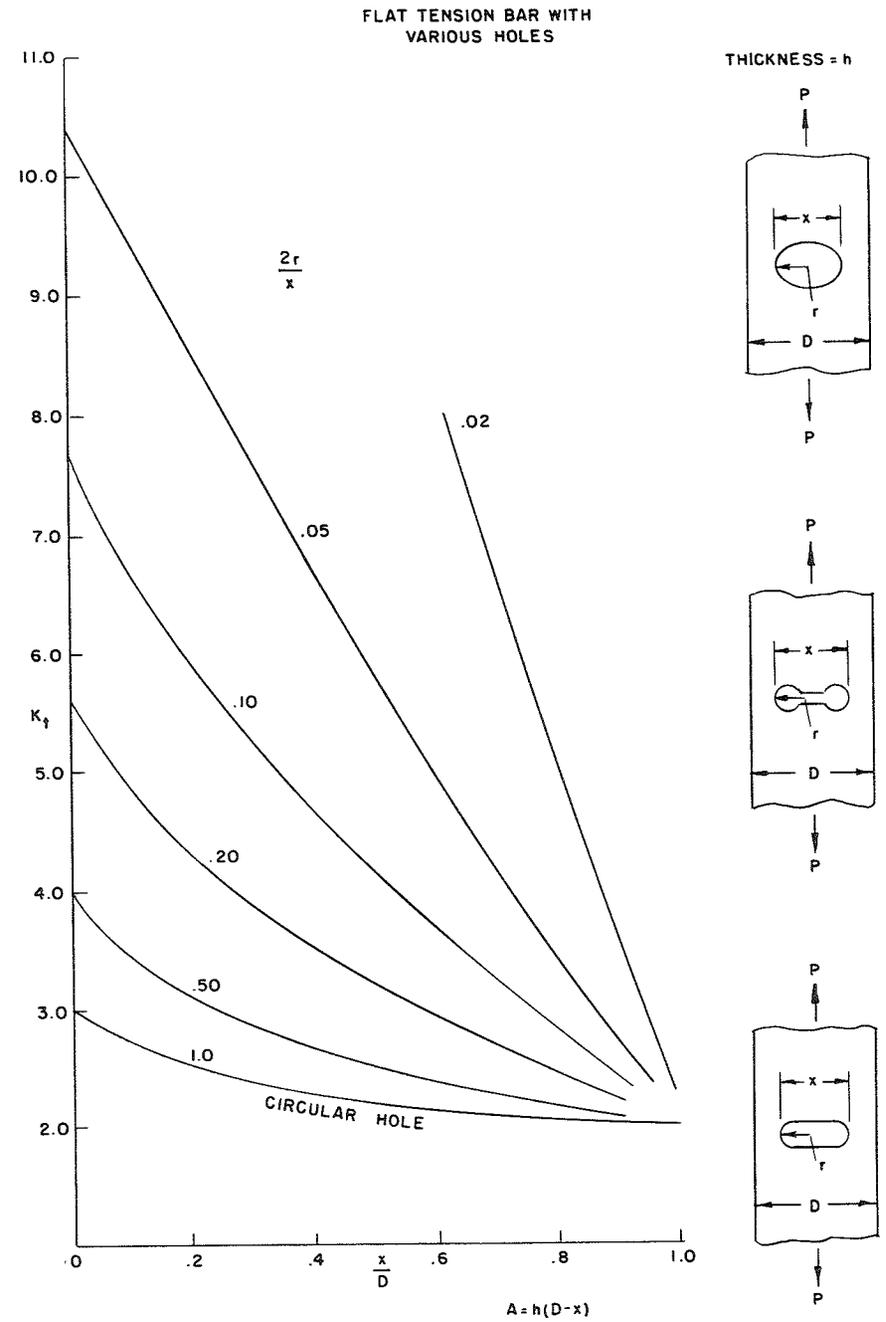


Figure 65. Theoretical stress concentration factors—Case 8.

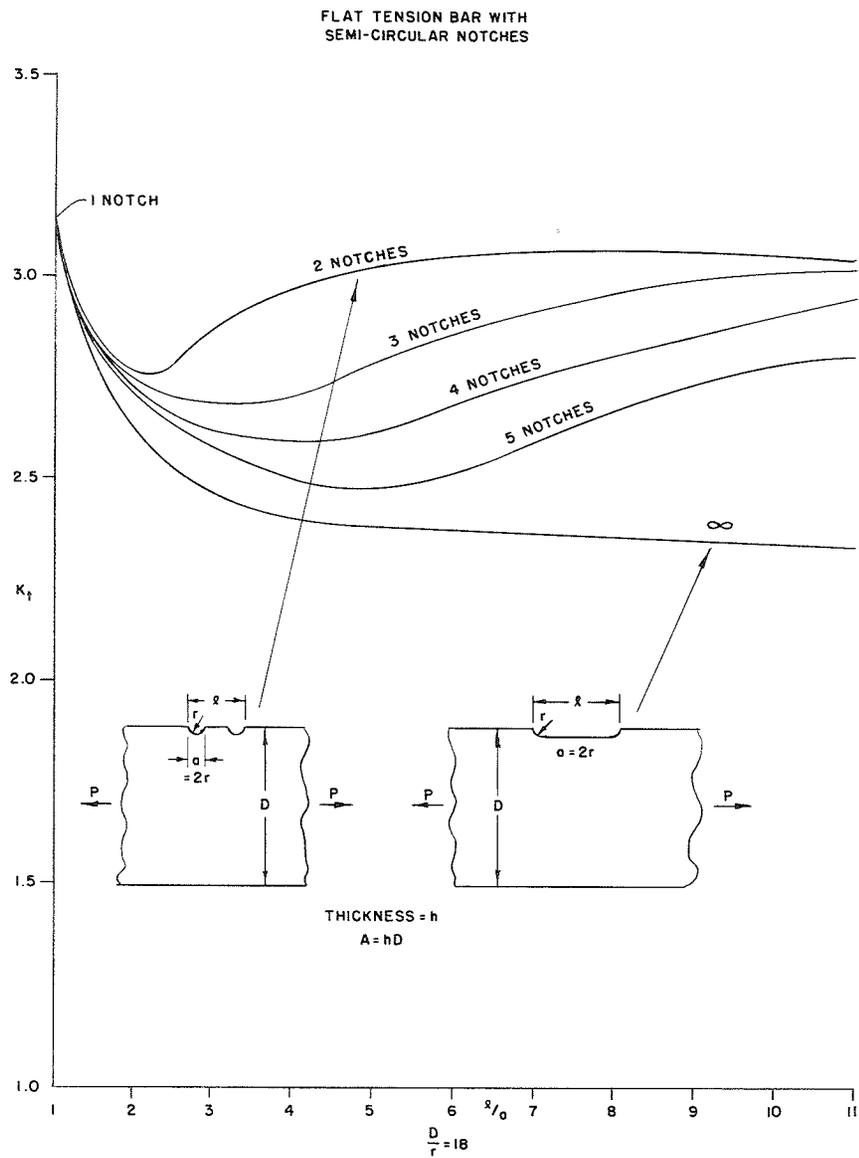


Figure 66(a). Theoretical stress concentration factors—Case 9A.

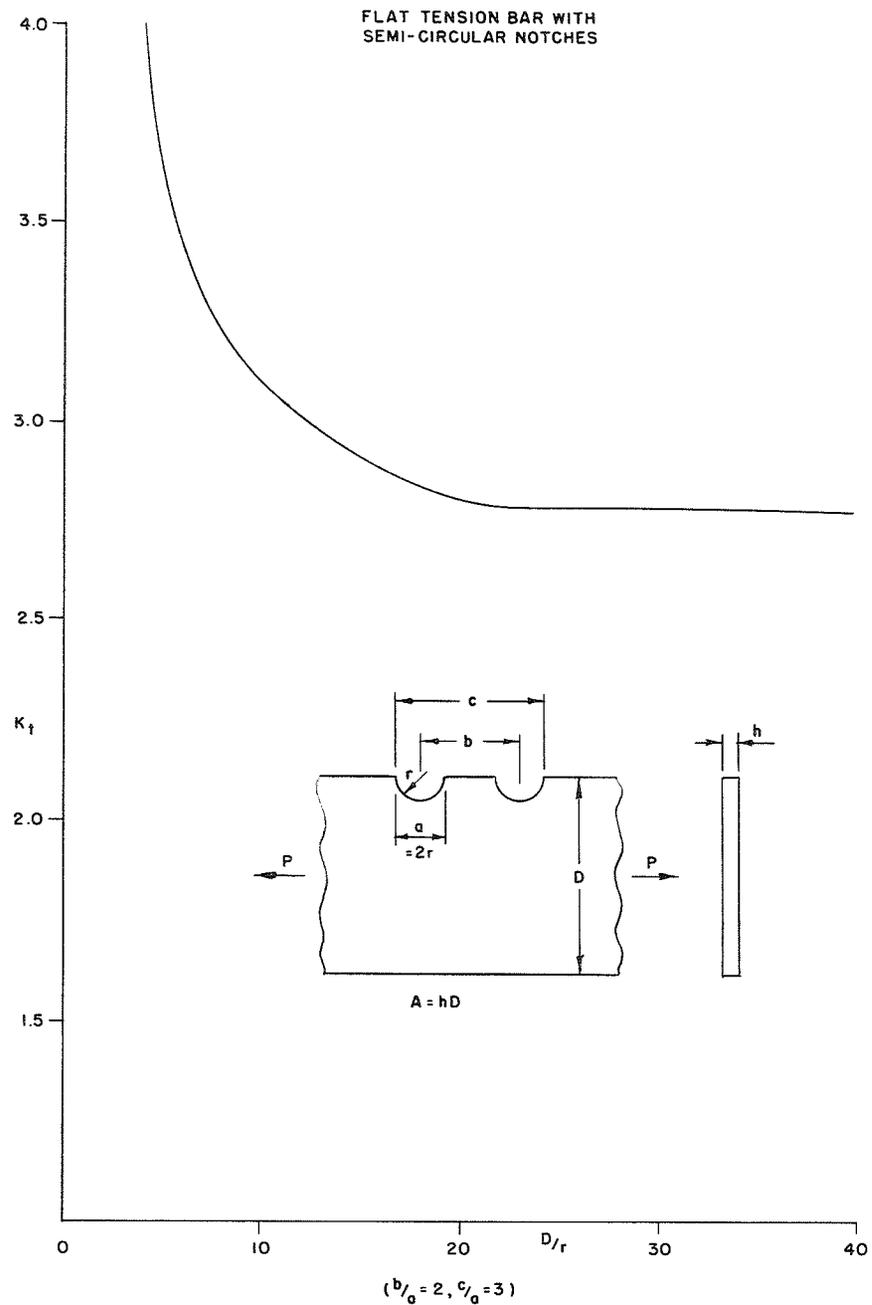


Figure 66(b). Theoretical stress concentration factors—Case 9B.

where A_d , the remaining area, and A , the original area, represent the net area of the member after appropriate deductions for fastener holes and corrosion section losses are made.

c. *Load Factor Method.* As in the case of uniform corrosion, using a load factor method gives the same residual capacity factor as the service load method.

d. *Effects of Location of Deterioration Along the Member.* A localized section loss in a tension member has the same effect on the member strength regardless of its location along the member. Therefore, for a tension member the residual capacity factor at the local level equals the residual capacity factor at the member level, i.e.,

$$RCF_l = RCF_m \quad (15)$$

e. *Effects of Local Eccentricities Due to Asymmetrical Deterioration.* At first glance, it would appear that an eccentricity in a tension member reduces its static capacity. Elastic tension stresses are increased by the bending stresses induced by the eccentricity. However, as the axial stresses increase, localized yielding will take place and result in a redistribution of stresses until the entire section is plastified. Also, because of its self-stabilizing nature, a tension member will tend to deflect in a direction such that the effects of the eccentricity are reduced. Extensive investigations on the behavior of statically loaded tension members with joint eccentricities have shown that as the ultimate load is approached the centroidal axis of the member approaches the loading axis over most of its length, and that its ultimate load approaches the ultimate load of an axially loaded member with no eccentricity. AISC specifications (6.4) allow the designer to neglect joint eccentricities in statically loaded tension members and the Ontario Bridge Design Code (6.12) in its section "Evaluation of Existing Bridges" assumes ductile steel behavior and allows the neglecting of eccentricities induced by asymmetrical deterioration in tension elements.

In general, the effect of eccentricities in asymmetrical deteriorated tension elements may be neglected when estimating static capacity if the degree of deterioration is relatively low. However, eccentricities should be considered when evaluating the fatigue criterion and in cases of significant losses and abrupt changes. If local yielding under working load is undesirable, the diagrams with theoretical stress concentration factors (see Figures 59 to 66b) may be used to estimate maximum stresses, or the deteriorated section may be verified by using a combination of axial load and bending due to eccentricity.

4.1.3 Deformation Criterion

4.1.3.1 Uniform Corrosion Losses

An overall reduction in the cross-section area of a tension member will proportionally reduce its axial stiffness. This may result in increased deflections and possible redistribution of loads. The bridge deflection criteria specified in AASHTO Article 10.6 should be verified and any possible increase in load effects in adjacent bridge members considered. This is further discussed in section 4.8.

4.1.3.2 Localized Corrosion Losses

Localized corrosion losses have little effect on the axial stiff-

ness of a member, and elongations can be calculated on the basis of the gross area. Unless the member is severely deteriorated, very little load redistribution can be expected.

4.1.3.3 Summary

The effect of localized corrosion on the axial stiffness of a tension member and its elongation may be neglected. Uniform corrosion can reduce the axial stiffness of a tension member. The reduction in stiffness is proportional to the reduction in section area. This may result in increased deflections and possible redistribution of loads (see section 4.9). The bridge deflection criteria specified in AASHTO Article 10.6 should be observed and any increase in load in adjacent bridge members verified.

4.1.4 Stability and Slenderness

Tension members are self-stabilizing and therefore stability in itself does not constitute a criterion for evaluation. Nevertheless, the limiting slenderness ratio values specified in AASHTO Article 10.7.5 must be observed in order to ensure adequate rigidity and prevent excessive sag, vibration, or lateral movement. Vibrations may lead to fatigue problems. When calculating the slenderness ratio the reduced section should be used in cases of uniform corrosion losses. The effects of localized corrosion, unless severe, may be neglected.

4.1.5 Minimum Thickness of Metal

Corrosion losses result in a reduction in the thickness of metal. The minimum thickness values specified in AASHTO Article 10.8 may be used as a guide for evaluating the remaining thickness. Smaller thickness values may be allowed because the AASHTO provisions provide some allowance for corrosion loss, but in no case should a thickness below $\frac{3}{16}$ in. be allowed.

4.1.6 Fatigue Criterion

The effects of material loss due to corrosion on the fatigue resistance of a tension member may be divided into: (1) effects of surface or edge roughness created by corrosion and (2) effects of eccentricities, holes, or other abrupt discontinuities that result from significant material loss.

A detailed discussion on the effects of surface roughness on fatigue resistance is given in Chapter 5. A distinction is made in Chapter 5 between a case of active corrosion and a case where the corroded area has been blast-cleaned and repainted to stop the corrosion process.

In order to evaluate the effects of holes or other significant discontinuities, fatigue stress concentration factors (see section 2.6.4) may be used as a guide. A lower bound estimate of fatigue resistance may be obtained by using theoretical stress concentration factors (see section 2.6.4 and Figures 58 to 66b) in order to calculate maximum elastic stresses. The question still remains open, however, as to how significant the effects are of the roughness of the surface or the edges of the discontinuities created by corrosion.

4.2 BUILT-UP TENSION MEMBERS

4.2.1 General

Built-up members are very common in existing truss bridges. They usually consist of channels or other structural shapes connected by lacing bars, batten plates, or perforated cover plates. The connecting elements have to ensure load-sharing action among the main tension elements. Load differences that may occur are transferred from one tension element to another through shear in the lacing or the open web. The action of the lacing or the open web in a tension member is a function of how many components of the member frame into the joint and how the components are connected to the joint. When significant corrosion of a main component occurs, lacing may allow local load redistribution to other components. The connecting elements also provide lateral support to the main tension elements. Provisions for the dimensions and spacing of batten plates and lacing bars are given in the AASHTO specifications, Articles 10.16.9 and 10.16.10. A maximum clear distance between batten plates of 3 ft is allowed for tension members. Deterioration of lacing bars and batten plates is common on bridges affected by corrosion. Both the overall capacity of the member and its resistance to local failure may be affected.

4.2.2 Effects on Overall Capacity

Deterioration of lacing bars or batten plates may adversely affect the load sharing among main tension elements and result in some elements having higher stresses than others. Thus, an overall reduction in the service load capacity and fatigue resistance may result. The effect on the ultimate load capacity is less significant because yielding will result in load redistribution among the main tension elements and failure will occur only when all elements have yielded.

4.2.3 Local Effects

Deterioration of lacing bars or batten plates may also result in an increased slenderness ratio of individual segments of the tension elements. The maximum slenderness ratio to be allowed should conform to the AASHTO specifications, Article 10.7.5, in order to ensure adequate rigidity.

4.3 MEMBERS LOADED IN AXIAL COMPRESSION

4.3.1 General

Members loaded in axial compression are mainly encountered in truss bridges as main truss members or bracing members and in girder bridges as bracing members. Compression members are usually single rolled-shape members or built-up members.

The load-carrying capacity of a compression member is often limited by its resistance to overall or local buckling. Buckling can take place at stresses below the yield stress. As opposed to tension members that are self-stabilizing, compression members are susceptible to failure through instability. Eccentricities or transverse loads cause destabilizing moments that can further reduce the available resistance to compressive loads. In addition

to the cross-sectional area, factors such as the member length, the radius of gyration, end restraint conditions, eccentricity of the axial load, the existence of transverse loads, out-of-straightness and the existence of residual stresses will also affect the strength of a compression member.

Corrosion forms such as uniform corrosion can reduce the section area and other section properties, including the moment of inertia and the radius of gyration, and thus affect the stability of the member. Localized corrosion can introduce eccentricities or affect the end restraint conditions of the member. The formation of pack rust between compression elements may shift the elements and result in increased eccentricities or may deform components of a built-up member and result in localized buckling. Failure due to instability can occur instantaneously and without prior warning. Overall buckling results in a complete loss of member capacity, while localized buckling reduces member capacity and may subsequently lead to overall member failure. If the buckled member is a critical bridge member, a sudden bridge collapse may result. Therefore, it is important that critical bridge compression members be identified and the effects of corrosion on their stability be carefully evaluated.

4.3.2 Strength and Stability Criterion

4.3.2.1 Uniform Corrosion Losses

Uniform corrosion losses are considered here to result in a uniform reduction in the cross-sectional area of the member over most of its length. In addition to the reduction in the cross-sectional area, section properties such as the moment of inertia and radius of gyration, which affect the stability of the member, may also be affected. Uniform corrosion losses can cause both localized buckling and overall failure of the member.

a. *Overall Member Failure.* Overall failure of a compression member may result from overall buckling, compression yielding or a combination of both buckling and compression yielding. Depending on the governing failure mode, compression members are divided into three groups: (1) short members (failure occurs by compression yielding), (2) long members (failure takes place because of elastic instability), and (3) intermediate length members (failure is affected by both the tendency for buckling and compression yielding).

The intermediate length range is usually defined by

$$20 \pm \leq \frac{KL}{r} < C_c \quad (16)$$

where K is the effective length coefficient, L is the member length, r is the radius of gyration, and (KL/r) is the slenderness ratio. The constant C_c is defined:

$$C_c = \left(\frac{2 \pi^2 E}{F_y} \right)^{1/2} \quad (17)$$

and equals 126.1 for 36,000 psi steel and 107.0 for 50,000 psi steel. Most compression members encountered in bridges are within the intermediate length range.

When $(KL/r) > C_c$, the members will most likely fail through elastic buckling. Uniform corrosion losses can affect both the resistance to compression yielding and buckling. The suitability

of a uniformly corroded compression member may be checked against the current AASHTO specifications for design. Based on the AASHTO specifications, the residual capacity factor for a uniformly corroded compression member may be expressed as follows:

$$\text{short members} \quad RCF_m = \frac{Ad}{A} \quad (18)$$

$$\text{long members} \quad RCF_m = \left(\frac{Ad}{A}\right) \left(\frac{rd}{r}\right)^2 = \frac{Id}{I} \quad (19)$$

$$\text{intermediate members} \quad RCF_m = \left(\frac{Ad}{A}\right) \left(\frac{1 - \frac{F_y (KL)^2}{4\pi^2 E} / rd^2}{1 - \frac{F_y (KL)^2}{4\pi^2 E} / r^2} \right) \quad (20)$$

For long and intermediate members, if the loss in cross-sectional area is accompanied by a reduction in the radius of gyration, the member residual capacity factor is further reduced. The higher the slenderness ratio, the larger this reduction will be. In the case of solid rectangular sections, with losses uniformly distributed along all sides, the effect of the section loss on the radius of gyration and on the residual capacity factor can be quite significant. This is illustrated in Figure 67. Figure 67 shows the residual capacity factor as a function of the percentage of section loss for a short member, a long member ($KL/r = 120$), and three intermediate members ($KL/r = 40, 60, \text{ and } 80$). It can also be seen that the effect of the reduced radius of gyration is more pronounced for higher slenderness ratios.

The most common compression members in bridges, however, are made of box shapes or H-type shapes. In the case of a box type cross section, such as that of built-up members using perforated plates, lacing or batten plates, the radius of gyration is not as sensitive to uniform section losses. As a result, very little effect on the residual capacity factor of intermediate or long members, relative to short members, can be expected. This is shown in Figure 68, where a square type box section with losses uniformly distributed along all sides has been considered. Figure 68 shows the residual capacity factor as a function of the percent of section loss for a short member ($KL/r = 10$), an intermediate member ($KL/r = 60$), and a long member ($KL/r = 120$). It can be seen that the slenderness ratio does not have a significant effect on the residual capacity factor of the member and, thus, Eq. 18, which applies to short members, may be applied to intermediate and long members as well.

The effects of losses uniformly distributed along all sides of an H-shape member are shown in Figure 69. In Figure 69, bending about the major axis has been considered. As in the case of a box-shape member, the residual capacity factor is not significantly affected by the slenderness ratio. The effect is even less for bending about the minor axis when the flange width is unchanged. When the losses considered are limited to the flanges, the effect of the losses on the radius of gyration can be more pronounced, and when the losses are limited to the web, the radius of gyration can actually increase.

In general, for box-shape or H-shape members where the radius of gyration is not sensitive to uniform corrosion losses, the residual capacity factor may be approximated by the ratio of the reduced section area to the original section area (regardless of its initial slenderness ratio).

b. *Local Buckling.* Portions and individual components of compression members are designed and made stiff enough to eliminate local buckling as a control on member capacity. However, uniform corrosion can increase the width-thickness ratio, b/t , of portions or elements of a compression member and result in localized buckling at stresses below those which would cause overall failure. The warped portion of a member cannot carry load anymore and thus the compressive capacity of the member is reduced. Eventually, an overall member failure may result.

The susceptibility of a corroded member to localized buckling may be evaluated using the limiting values of b/t specified by AASHTO, Article 10.35.2, as a guide.

If, as a result of corrosion, the measured width-thickness ratio exceeds the values recommended by AASHTO, the American Institute of Steel Construction (AISC) specifications, Appendix C, may be used to evaluate the remaining compressive capacity of the member. Appendix C of the AISC specifications (6.4) provides formulas for calculating a stress reduction factor, Q_s , for unstiffened compression elements whose width-thickness ratio exceeds the applicable limit specified for design. In the case of stiffened compression elements, formulas for calculating a reduced effective width, b_e , are specified for calculating the permissible axial load.

4.3.2.2 Localized Corrosion Losses

The localized corrosion losses considered here include very confined losses such as pits, holes, and edge scallops and also losses uniformly distributed over a limited length along the member. In addition to the reduction in area, localized losses can induce eccentricities and changes in the moment of inertia along the member. They may also change the nature of the end restraint conditions.

When determining the resistance to buckling of a member with localized corrosion, the standard stability formulations for straight members with uniform cross section no longer apply. In the following, guidelines for evaluating the effects of localized corrosion on the capacity of compression members are presented.

a. *Local Versus Overall Effects.* Localized corrosion will affect the area in the immediate vicinity of the damage which, in turn, may affect the overall stability of the member. Most of the main compression members in a bridge fall in the intermediate length range. In this range, failure is affected by both local yielding due to compression stress and the overall tendency towards buckling. The effect of localized damage on the overall stability of the member will depend on its slenderness ratio. For shorter members the local strength of the deteriorated area will govern.

b. *Effects of Location of Deterioration Along the Member.* As opposed to tension members, in compression members the location of localized deterioration can have an effect on its strength. For example, in a compression member with pinned end conditions, a reduction in the moment of inertia of its middle portion has a much greater effect on the buckling capacity than a reduction in the moment of inertia of its end portions. Deterioration at the ends of fixed end compression members may result in a change in the end restraint conditions and reduce its buckling

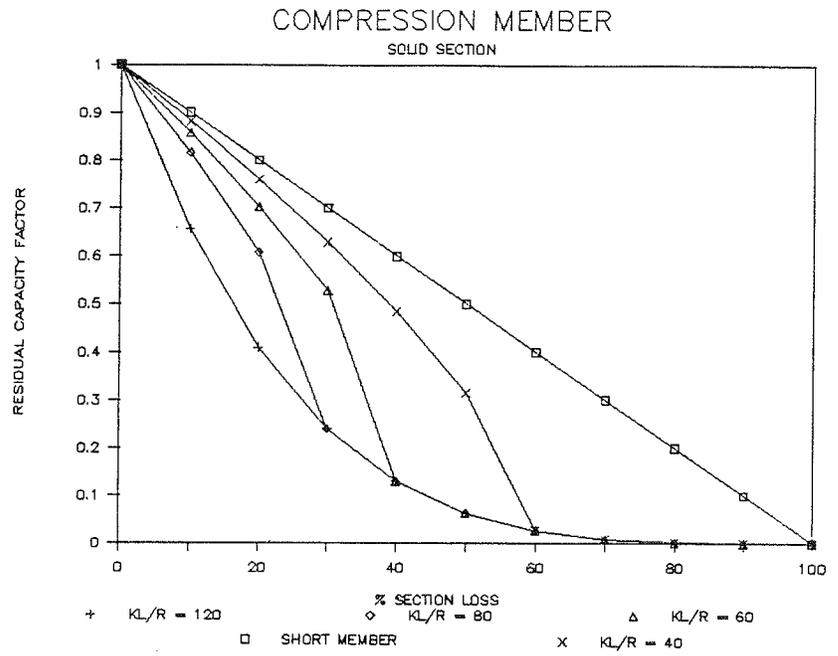


Figure 67. Residual capacity factor for a compression member with a solid cross section.

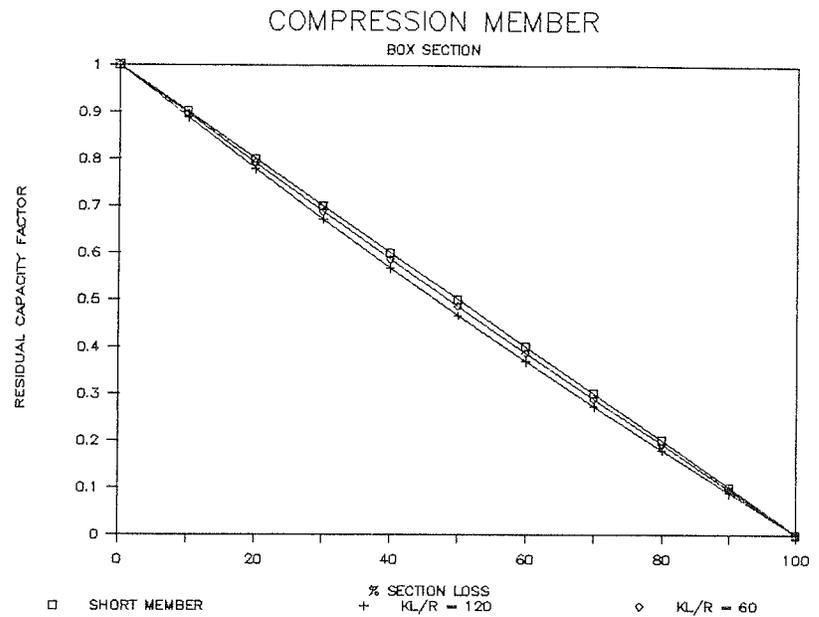


Figure 68. Residual capacity factor for a compression member with a box-type cross section.

COMPRESSION MEMBER

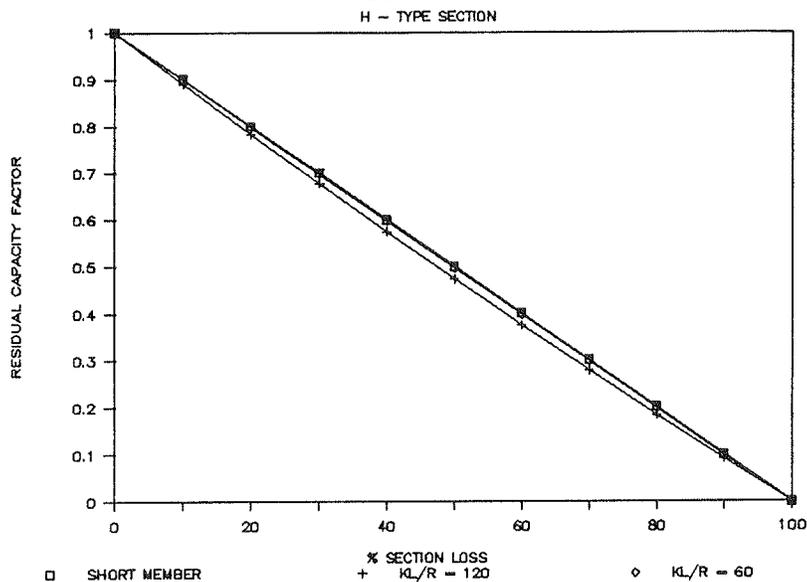


Figure 69. Residual capacity factor for a compression member with an H-type cross section.

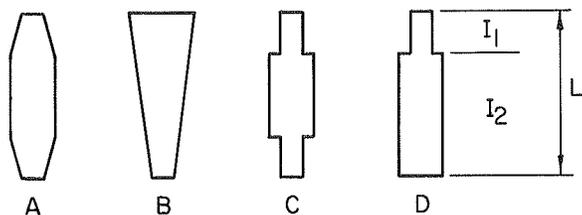


Figure 70. Columns with variable moment of inertia.

$$Q_c = \frac{P_{cr}}{\frac{\pi^2 EI_2}{L^2}} \tag{21}$$

The diagrams in Figures 71 and 72 assume a pinned-pinned condition ($K = 1$). In most practical cases, however, the end connections of the member provide some restraint (AASHTO uses $K = 0.75$ for riveted or bolted end connections and $K = 0.875$ for pinned end connections). For an approximate evaluation of such cases it is recommended that the length L , for use in Figures 71 and 72, be the effective length of the member and its ends be located at the assumed inflection points. The length A will then be measured relative to these inflection points. If there is loss of section between an assumed inflection point and the end of the member, it is suggested that the inflection point be moved, such that the loss is included in the length L , and the K -coefficient be changed accordingly.

The equivalent moment of inertia, I_{eq} , and the equivalent length, L_{eq} , may be calculated from:

$$I_{eq} = Q_c I_2 \tag{22}$$

$$L_{eq} = Q_c^{-1/2} L \tag{23}$$

Thus, the residual capacity factor for a compression member with a reduced cross section over only a portion of its length may be expressed as follows:

$$\text{short members} \quad RCFm = \frac{Ad}{A} \tag{24}$$

$$\text{long members} \quad RCFm = \frac{I_{eq}}{I} = Q_c \tag{25}$$

capacity. The effects of the location of deterioration are less significant for shorter members.

c. *Effects of Change in Stiffness Along the Member.* Corrosion losses can result in a compression member with a variable stiffness along its length. Solutions for several cases of columns with a variable moment of inertia (see Figure 70) have been developed and tabulated.

Some of these cases may be used to approximate the effects of localized corrosion losses. Case "A" represents a member with both end portions tapered similarly. Formulas for evaluating its elastic stability may be found in Ref. 7.45. Case "B" represents a tapered member. A procedure for evaluating the compression capacity of such members may be found in the AISC specifications, Appendix D (6.4). Cases "C" and "D" represent symmetrically and unsymmetrically stepped columns, respectively. Because these two cases are most likely to be used when approximating corrosion losses, solution diagrams developed in Ref. 7.16 are reproduced here, in Figures 71 and 72. These diagrams may be used to calculate an equivalent moment of inertia, I_{eq} , or an equivalent length, L_{eq} , which in turn can be used with the AASHTO column formulas to evaluate the residual member capacity. From Figures 71 or 72 a reduction factor, Q_c , may be estimated.

intermediate members: $RCFm = \left(\frac{Ad}{A}\right)$ (26)

$$\left(\frac{1 - \frac{F_y(KL)^2}{4\pi^2 E Q_c r^2}}{1 - \frac{F_y(KL)^2}{4\pi^2 E r^2}} \right)$$

When the effect of section loss on the moment of inertia of the member is not significant, the residual capacity factor may be approximated by the ratio of the reduced section area to the original section area (regardless of the member slenderness ratio).

d. *Effects of Eccentricity.* Very often localized corrosion results in asymmetrical deterioration which induces local eccentricities. Eccentricities can cause bending stresses and initiate or increase lateral deflection. Bending stresses in a compression member are actually magnified by an amplification factor that depends on the ratio of the applied axial load to the Euler buckling load. If the maximum compressive stress reaches the yield stress, localized yielding will occur. Because the yielded portion of a cross section contributes little to stiffness, yielding will result in a reduction in stiffness, which in turn will further increase the transverse deflection and bending. As a result, the member may fail through instability. Thus, eccentricities in a compression member can have a destabilizing effect and cause a reduction in the available resistance to compressive stress.

The column formulas used in design specifications account for the existence of small irregularities and eccentricities (and asymmetrical residual stress distributions), because they are usually unavoidable. For example, out-of-straightness as high as $\frac{1}{4}$ in. in 20 ft in a newly fabricated column may be encountered. This out-of-straightness can reduce the strength of an intermediate length column with a slenderness ratio between 50 to 120 by as much as 25 percent, and still be considered acceptable, because it is included in the column formulas for design.

An initial estimate of the effects of eccentricity can be made by calculating the eccentric ratio, (ec/r^2) , where e is the eccentricity, c is the distance from the neutral axis to the extreme fiber in compression, and r is the radius of gyration. In column design, an eccentric ratio smaller than 0.25 is considered acceptable.

If the eccentric ratio is large or if an accurate estimate of the residual capacity of the member is needed, the formulas for steel columns given in the *AASHTO Manual for Maintenance Inspection of Bridges*, Article 5.4, may be used. The formulas given in the AASHTO manual express the permissible average unit stress for compression members as a function of a given eccentricity and an initial out-of-straightness. They are based on the secant equation that has been widely used in design in the past. The effects of large eccentricities created or out-of-straightness may also be evaluated by calculating the resulting moments and using the interaction formulas given in the AASHTO specifications for design, Article 10.36, or using a Rayleigh solution or a method based on finite differences.

Eccentricity has greater significance for columns with intermediate slenderness ratios, i.e., $KL/r > 60$.

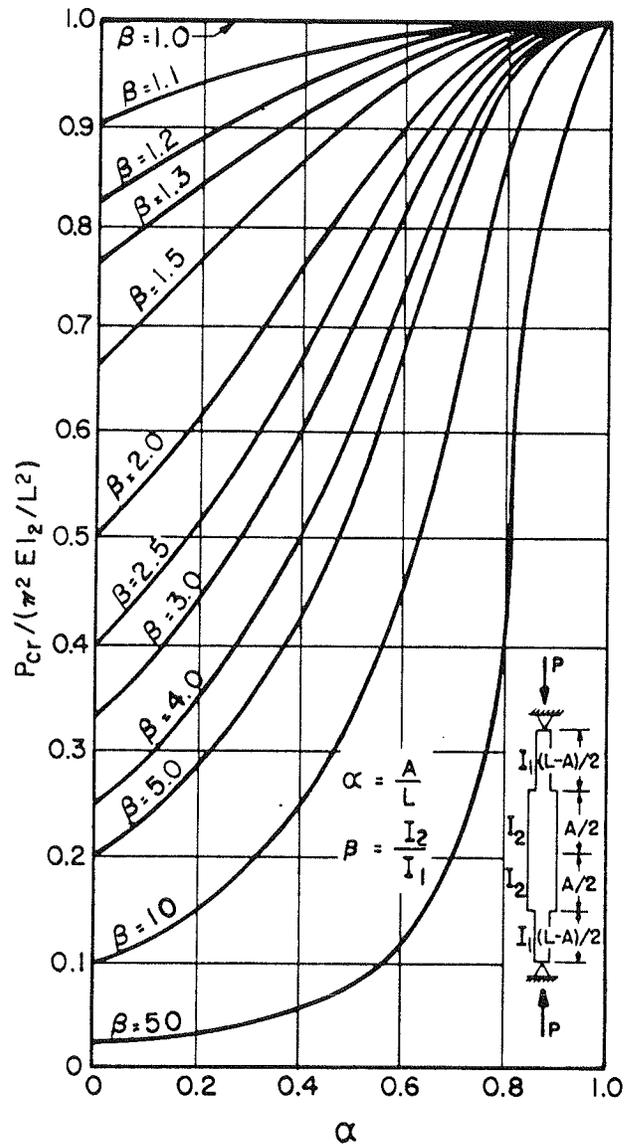


Figure 71. Elastic buckling load for a symmetrically stepped column. (Source: Ref. 7.16)

4.3.3 Deformation Criteria

4.3.3.1 Uniform Corrosion Losses

A uniform reduction in the cross-sectional area of a compression member results in a proportional reduction in its axial stiffness. This will result in increased deflections. If the loss of section is significant, a redistribution of loads may take place.

The bridge deflection criteria specified in the AASHTO specifications, Article 10.6, should be verified and the possible increase in load effects in adjacent components considered.

4.3.3.2 Localized Corrosion Losses

Localized corrosion losses have little effect on the axial stiffness of a member, and axial deformations can be calculated on

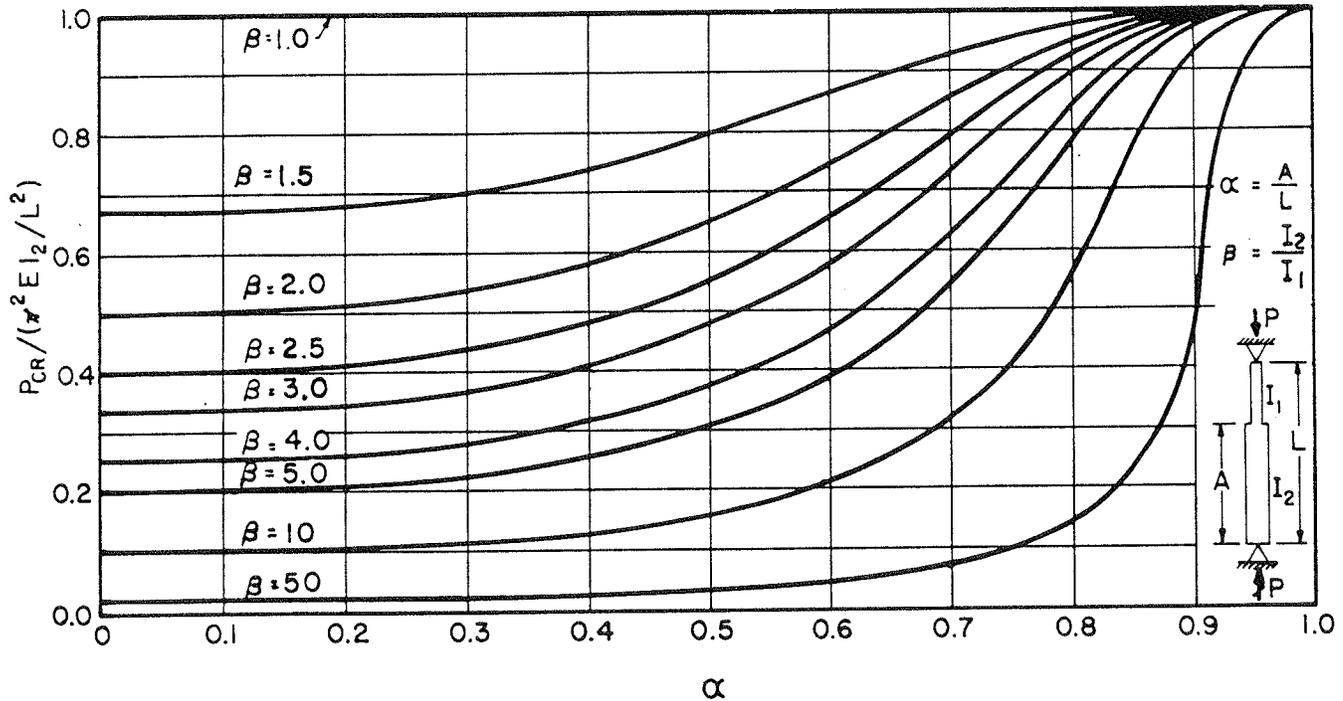


Figure 72. Elastic buckling load for an unsymmetrically stepped column. (Source: Ref. 7.16)

the basis of the gross area. Very little load redistribution can be expected.

4.3.4 Fatigue and Fracture

Fatigue and fracture mostly occur in regions of tensile stresses. Seldom, if ever, does brittle fracture occur in a compression zone and even then only if the compression zone is in the path of a running crack.

4.4 BUILT-UP COMPRESSION MEMBERS

4.4.1 General

Built-up compression members are very common in existing truss bridges. They are usually composed of channels or other structural shapes connected by lacing bars, batten plates, or perforated cover plates. They may also contain solid plates as part of the web system or attached to the main components of the built-up section.

The main function of the lacing or the open web is to resist the shear forces that result from buckling of the member about an axis perpendicular to the lacing or the open web. The resulting shear deformation that occurs reduces its buckling strength. While the effect of the shear forces on the usual single component cross-sectional shapes is negligible, the effect on latticed members can be quite important. Results of theoretical and experimental investigations into the behavior of built-up members may be found in Ref. 7.3. The magnitude of the forces in latticed members was found to vary from about 1 to 3 percent of the axial load.

Another important function of the lacing or the batten plates is to act as lateral bracing for the main components of the built-up member.

Deterioration of the lacing bars and the batten plates is very common in bridges affected by corrosion. While regarded as secondary members when intact, their effect on the structural integrity of compression members when deteriorated may be quite significant.

4.4.2 Deterioration of Lacing Bars

4.4.2.1 General

The lacing system in a compression member resists shear in truss action and, therefore, each individual bar can be loaded in either tension or compression. Provisions for the design of laced members are given in Articles 10.16.10 and 10.35.1 of the AASHTO specifications for design. The provisions specify design loads and minimum requirements regarding dimensions and spacing of lacing bars. Stay plates are required at the member ends and at intermediate points where lacing is interrupted. The lacing system is designed such that the effect of shear deformation on the overall buckling strength of the member is minimal, and the lacing elements do not fail before the overall capacity of the member is reached. However, when deteriorated, the lacing bars may affect the behavior of the member as intended in its design. A reduction in the resistance to shear deformations, which can affect the overall buckling strength of the member, may occur. Also, local buckling may become the controlling factor in the capacity of the member.

4.4.2.2 Effects on Overall Member Resistance

Corrosion of lacing bars reduces the shear resistance of the built-up member and therefore a reduction in its overall buckling strength may result. It is very difficult to accurately quantify this reduction, especially when the lacing bars have various degrees of deterioration. In the following, formulations for an approximate evaluation of the overall effects of deterioration of lacing bars are included. It is assumed that all bars experience the same degree of deterioration.

Analytical solutions for the buckling resistance of latticed members are given in Refs. 7.3 and 7.45. The solutions developed apply to both single and double lacing and also to battened members. On the basis of these solutions, a reduction coefficient, Q_e , can be formulated and used to calculate an effective modulus of elasticity, E_{eff} :

$$E_{eff} = Q_e E \quad (27)$$

or an effective member length, L_{eff} :

$$L_{eff} = Q_e^{-1/2} L \quad (28)$$

where E is the modulus of elasticity of steel and L is the length of the member.

For the case of a single laced member (see Figure 73), the coefficient Q_e can be calculated from Ref. 7.45:

$$Q_e = \left(1 + \frac{4.8 I}{A L^2 \cos^2 \theta \sin \theta} \right)^{-1} \quad (29)$$

where I is the moment of inertia of the built-up section about an axis normal to the lacing bar, A is the cross-sectional area of a lacing bar, and θ is the angle between a lacing bar and an axis perpendicular to the member in the plane of the lacing.

For the case of a double laced member, 2.4 may be used in place of 4.8 in Eq. 29.

Equation 29 shows that a reduction in the cross-sectional area of the lacing bars will reduce Q_e and will thus result in a reduction in the effective modulus of elasticity of the member (see Eq. 27). The degree of reduction in Q_e will depend on the length and the moment of inertia of the member, and on the original area and the geometry of the lacing. For most built-up compression members encountered in bridges the coefficient Q_e varies from 0.85 in more extreme cases, to 1.0. Equation 29 shows that for values of Q_e close to 1.0 the loss of section in lacing bars does not have a significant effect on Q_e . For example, a 20 percent loss in the cross-sectional area of the lacing bars will result in less than a 5 percent reduction in Q_e , and 40 percent loss in the cross-sectional area of the lacing bars can result in about a 10 percent reduction in Q_e .

The effective modulus of elasticity, E_{eff} , and the effective member length, L_{eff} , calculated based on Q_e , can be substituted for E or L in the AASHTO column formulas and used to evaluate the effects of deterioration of lacing on the overall compressive strength of the member. Therefore, when overall strength governs, the residual capacity factor for a built-up compression member with deteriorated lacing may be expressed as follows:

$$\text{short members} \quad RCF_m = 1.0 \quad (30)$$

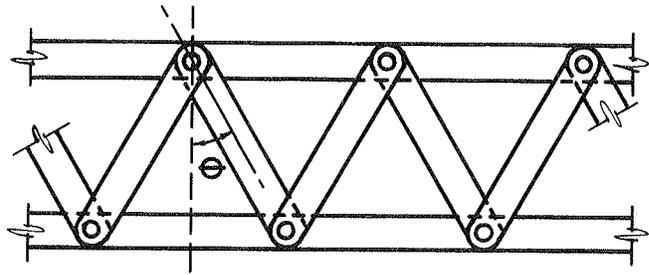


Figure 73. Single laced member.

In the case of short members, failure occurs by compressive yielding, and buckling does not govern. Even if the lacing is severely deteriorated and cannot help in redistributing load between the main member components, yielding of these elements will redistribute the loads, provided these elements are also "short" without bracing. It is most likely, however, that localized buckling will occur prior to yielding of the main member components (see section 4.4.2.3).

$$\text{long members} \quad RCF_m = \frac{E_{eff} d}{E_{eff}} = \frac{Q_{ed}}{Q_e} \quad (31)$$

where $E_{eff} d$ and Q_{ed} apply to a deteriorated condition. For long members the effect of deterioration of lacing on the overall residual capacity of the member can be determined from Q_{ed}/Q_e . For example, 20 percent reduction in the area of the lacing may cause a 5 percent reduction in the residual capacity factor of a long member.

$$\text{intermediate members} \quad RCF_m = \frac{1 - \frac{F_y (KL)^2}{4\pi^2 E Q_{ed}} / r^2}{1 - \frac{F_y (KL)^2}{4\pi^2 E Q_e} / r^2} \quad (32)$$

In the case of intermediate members, the effect of deterioration of lacing on the residual capacity factor of the member is less than for long members. The maximum reduction in residual capacity equals the reduction in Q_e and it occurs when the member slenderness ratio approaches C_e (see discussion on long members).

Based on the above one can conclude that moderate deterioration of up to about a 25 percent loss of the original cross section of lacing bars has very little effect on the overall member capacity, as long as the resistance to local failure is satisfactory.

4.4.2.3 Effects on Local Failure

Deterioration of the lacing bars may also lead to yielding or buckling of individual lacing bars or localized buckling of a main component because of loss of lateral bracing.

The lacing system in a compression member is based on a conservative design which would prevent local failure from occurring before the overall capacity of the member is achieved. For example, the AASHTO specifications, Article 10.16.19,

limit the slenderness ratio of the portion of a main component included between the lacing bar connections to a value smaller than 40 or smaller than two-thirds of the slenderness ratio of the member. Therefore, a certain degree of deterioration of the lacing system may be allowed to occur before localized buckling becomes the controlling failure mode, by allowing larger slenderness ratios. However, the deteriorated lacing system has to be able to resist the transverse shear force, V , given by AASHTO specifications, Article 10.35.1, formula (10-36) and the slenderness ratios allowed should not be larger than those allowed by AASHTO for main compression members. The maximum force in the lacing system, P_ℓ , for members with single lacing (see Figure 73) is given by:

$$P_\ell = \frac{V}{\cos \theta} \quad (33)$$

and for members having double lacing:

$$P_\ell = \frac{V}{2 \cos \theta} \quad (34)$$

where V is the transverse shear force per plane of lacing, and θ is the angle between the lacing bar and an axis perpendicular to the member in the plane of the lacing.

The resistance to buckling of individual lacing bars and the local resistance to buckling of a main component which has lost some of its lateral bracing can be evaluated using the AASHTO column formulas.

The minimum thickness requirements of Article 10.35.2 of the AASHTO specifications should apply.

4.4.3 Deterioration of Batten Plates

4.4.3.1 General

The batten plates in a compression member resist shear through Vierendeel action. Provisions for minimum dimensions of batten plates are given in the AASHTO specifications, Article 10.16.9. Provisions for assessing the overall compressive strength of battened members are given in the AASHTO *Manual for Maintenance Inspection of Bridges*, Article 5.4.2. The maintenance inspection manual specifies factors that allow for the reduced strength of battened compression members. These factors result in increased slenderness ratios to be used with the AASHTO column formulas. They are given in a table form as a function of the spacing center-to-center of batten plates and the actual slenderness ratio of the member. The minimum dimensions of the batten plates and their spacing were specified in the original bridge design such that localized failure would not occur before the overall member capacity is reached. Corrosion of the batten plates can reduce the overall buckling capacity of the member and it can also cause localized buckling in the deteriorated area before the overall member capacity is reached.

4.4.3.2 Effects on Overall Member Resistance

Corrosion of the batten plates can reduce the shear resistance of the built-up member and thus reduce the overall buckling

strength of the member. Severe corrosion of some of the batten plates may result in a built-up member with an increased spacing between the remaining batten plates. An estimate of the effect of a larger spacing between batten plates may be obtained by using the AASHTO manual, Article 5.4.2, where the spacing center-to-center of batten plates is related to the compressive capacity of the member. When the batten plates are only partially corroded, the analytical solution developed in Ref. 7.3 may be used to assess the effects on the overall buckling capacity of the member. By assuming that all batten plates have experienced the same degree of deterioration, a reduction coefficient Q_b , such as the one defined for the case of lacing bars, may also be defined for a batten plated member (see Figure 74).

The coefficient Q_b for a batten plated member can be calculated from Ref. 7.45.

$$Q_b = \left[1 + \frac{\pi^2 I}{L^2} \left(\frac{ab}{12I_2} + \frac{a^2}{24I_1} \right) \right]^{-1} \quad (35)$$

where L is the length of the member, I is the moment of inertia of its cross section, a is the distance center-to-center of batten plates, b is the length of a batten plate between rivets, I_1 is the moment of inertia of one flange about its own centroidal axis normal to the plane of the battens, and I_2 is the moment of inertia of a pair of batten plates:

$$I_2 = \frac{tc^3}{6} \quad (36)$$

where t is the batten plate thickness and c is the batten plate width. The coefficient Q_b can be used to calculate an effective modulus of elasticity, E_{eff} :

$$E_{eff} = Q_b E \quad (37)$$

or an effective member length, L_{eff} :

$$L_{eff} = Q_b^{-1/2} L \quad (38)$$

Equations 35 and 37 show that an increase in spacing between batten plates or a reduction in their moment of inertia reduces the effective modulus of elasticity, E_{eff} . The degree of reduction will depend on the length and moment of inertia of the member,

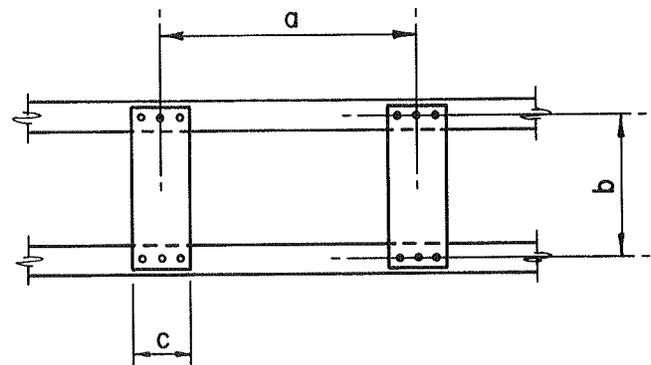


Figure 74. Batten plated member.

the original moment of inertia of the batten plates and their spacing, and the moment of inertia of the flanges of the built-up member. For most common cases encountered in bridges, Q_b varies from 0.80 to 1.0. Equation 35 shows that for this range of values, Q_b is not very sensitive to section losses in the batten plates.

In order to evaluate the effects of deterioration of batten plates on the overall compressive strength of the member, the effective modulus of elasticity, E_{eff} , or the effective length, L_{eff} , can be substituted for E or L in the AASHTO column formulas. Thus, the residual capacity factor for a built-up compression member with deteriorated batten plates, for the case when overall strength governs, may be expressed as follows:

$$\text{short members} \quad RCF_m = 1.0 \quad (39)$$

In the case of short members, batten plates have a minor effect on the overall resistance to yielding, which is the governing failure mode. A short member will fail when all its main components yield, assuming no localized buckling occurs prior to that.

$$\text{long members} \quad RCF_m = \frac{E_{eff}d}{E_{eff}} = \frac{Q_{bd}}{Q_b} \quad (40)$$

where $E_{eff}d$ and Q_{bd} apply to a deteriorated condition. For long members, the effect of deterioration of batten plates on the overall residual capacity can be determined from Q_{bd}/Q_b .

$$\text{intermediate members} \quad RCF_m = \frac{1 - \frac{F_y(KL)^2}{4\pi^2 E Q_{bd}}}{1 - \frac{F_y(KL)^2}{4\pi^2 E Q_b}} \quad (41)$$

For intermediate members, the effect of deterioration of battens on the residual capacity factor is less than for long members. The maximum reduction in residual capacity equals the reduction in Q_b and it occurs when the member slenderness ratio approaches C_c .

In general it appears that moderate deterioration of batten plates (below about 25 percent loss) does not significantly reduce the overall member capacity, as long as the resistance to local failure is satisfactory.

4.4.3.3 Effects on Local Failure

Deterioration of the batten plates may lead to local failure of the battens or to localized buckling of a main component of the built-up section due to loss of adequate lateral support.

The design of the batten plates is usually conservative, so that local failure will not occur before overall member failure. Thus, some deterioration of the batten plates will not necessarily result in a direct reduction in member capacity, until local failure becomes the controlling factor. The deteriorated batten plates must be able to resist the transverse shear force given by AASHTO specifications Article 10.35.1, Eq. 10-36. The maximum value of longitudinal shear V_b causing bending of the battens (see Figure 74) is:

$$V_b = V \frac{a}{b} \quad (42)$$

where V is the transverse shear force per plane of battens, a is the distance center-to-center of batten plates, and b is the length of a batten plate between rivets.

The susceptibility to buckling of one of the components of the built-up section as a result of loss of adequate lateral support can be determined based on the AASHTO column formulas, using the section properties of that component and its unsupported length.

The minimum thickness requirements of AASHTO specifications Article 10.8 should apply.

4.4.4 Deterioration of Attached Plates

When the thickness of plates attached to components of a built-up compression member is reduced as a result of corrosion, local buckling may occur. If the plate is attached by rivets or bolts, it may buckle between the points of attachment. In order to develop the full yield strength of the plate the ratio of the unsupported length, ℓ , to the reduced plate thickness, t_r , has to be smaller than the value given by the formula (7.45):

$$\frac{\ell}{t_r} = 0.52 \left(\frac{E}{F_y} \right)^{1/2} \quad (43)$$

where E is the modulus of elasticity of steel and F_y is the yield stress. For 36,000 psi yield stress steel this ratio equals 15.

4.5 MEMBERS LOADED IN BENDING

4.5.1 General

Members loaded in bending are usually encountered in bridges as members of the floor system such as stringers and floorbeams or as main girders in girder bridges. They include rolled members, prismatic and nonprismatic built-up members, and plate girders. They may be of riveted, bolted, or welded construction.

The evaluation of the performance of bending members affected by corrosion can be governed by quite a few different criteria. They include strength, deformation, stability of web and compression flange, fatigue and fracture. In the design of bending members a distinction is made between compact and noncompact sections. Compact sections are not susceptible to local or overall instability, and the full strength of the beam can be used without earlier instability failures. Such a distinction will not be made here because the deterioration of a beam due to corrosion may cause a compact section to become noncompact and fail as a result of buckling.

However, in the following, laterally supported beams with stocky webs, plate girders, and beams with web holes are treated separately. In the case of laterally supported beams with stocky webs it is assumed that failure through instability is very unlikely. The effects of various instabilities are considered in the section on plate girders, and the effects of corrosion holes in the web are considered in the section on beams with web holes. Because of their complexity, plate girders and beams with web holes are treated independently in separate sections, while this section addresses only stocky, laterally supported beams.

4.5.2 Strength Criterion

4.5.2.1 Uniform Flange Losses

Uniform losses in the flanges of a member loaded in bending result in a uniform reduction in the section modulus of the member. Using the reduced section modulus, the remaining bending capacity of the member can be determined in accordance with the AASHTO specifications for design. A service load or a load factor approach may be used.

a. *Service Load Method.* The service load method limits the maximum stresses in the extreme fibers of the member cross section to allowable values below the yield stress, F_y . The maximum moment capacity, referred to as the yield moment, M_y , is reached when the extreme fibers of the cross section are at yield. The yield moment, M_y , is calculated from

$$M_y = F_y S \quad (44)$$

where S is the section modulus.

The residual bending capacity factor of the member then becomes:

$$RCF_m = \frac{S_{dm}}{S_m} \quad (45)$$

where S_{dm} is the section modulus of the deteriorated beam and S_m is the section modulus of the intact beam.

The effect of uniform losses in the tension or compression flanges is further discussed in sections 4.6.3 and 4.6.4.

b. *Load Factor Method.* The load factor approach takes advantage of the additional bending moment capacity obtained by taking into account the full plastification of the bent section. The plastic moment capacity, M_p , is calculated from

$$M_p = F_y Z \quad (46)$$

where Z is the plastic modulus.

The residual capacity factor based on a load factor approach then becomes:

$$RCF_m = \frac{Z_{dm}}{Z_m} \quad (47)$$

where Z_{dm} is the plastic modulus of the deteriorated beam and Z_m is the plastic modulus of the intact beam.

In most practical cases, the residual capacity factor, RCF_m , calculated based on the load factor method, will be very close to that calculated based on the service load method. However, using a load factor method will show a higher residual capacity. The increase in the residual capacity is proportional to the shape factor, f , given by

$$f = \frac{Z}{S} \quad (48)$$

For rolled I -shaped sections bent about their major axis, the shape factor varies from about 1.10 to about 1.20. Thus, in the case of a deteriorated beam originally designed based on a service load approach, using a load factor approach can show an additional reserve in strength. Another source of reserve in load-

carrying capacity, although only artificial, may result from the way factors of safety are applied in the service load and the load factor methods. For higher ratios of dead load to live load plus impact (over 0.67), using a load factor method will usually result in a higher residual capacity estimate.

c. *Nonlinear Analysis.* In addition to the recognition of the additional strength of a section above first yield, using a plastic approach for analysis can also recognize the additional capacity due to load redistribution such as after the formation of a plastic hinge in a continuous beam. Nonlinear analysis, however, should be limited to Level II office evaluations.

4.5.2.2 Uniform Web Losses

The residual capacity factor in shear, RCF_m , obtained based on either a service load or a load factor approach, is

$$RCF_m = \frac{t_{wd}}{t_w} \quad (49)$$

where t_{wd} is the reduced web thickness. Using a load factor method will not show any additional reserve in strength over that determined based on a service load method, with the exception of that resulting from the way factors of safety are applied in these methods.

The reduction in the web thickness may also result in instability failure. This criterion is treated in detail in section 4.6 on plate girders.

4.5.2.3 Localized Losses in a Beam

The effects of localized losses in the web or the flanges of a beam are treated in sections 4.6 and 4.7.

4.5.3 Deformation Criteria

4.5.3.1 Uniform Corrosion Losses

The stiffness of a beam is proportional to its moment of inertia. Thus, uniform corrosion of the flanges will result in increased deflections, while the effects of uniform web losses will be less significant. The increase in deflection can be determined by using the reduced moment of inertia. The deflection limits recommended in the AASHTO specifications Article 10.6 should be observed.

4.5.3.2 Localized Corrosion Losses

Localized corrosion in the flanges or the web of a beam were found to have very little effect on the stiffness of the beam (see further discussion in sections 4.6.3.3 and 4.7.7).

4.5.4 Fatigue Criterion

The general approach for evaluating the effects of corrosion on the fatigue strength of beams is similar to that used for axial members (see section 4.1.6). Fatigue resistance of beams with web holes is treated in section 4.7.8.

4.6 PLATE GIRDERS

4.6.1 Uniform Web Losses

Uniform corrosion of the web is considered here to result in a uniform reduction in the thickness of the web, tw . This reduction in thickness can affect the resistance to (1) web buckling due to bending, (2) web buckling due to shear, (3) shear yielding, (4) combined bending and shear, (5) web buckling due to vertical compression stresses from the compression flange, (6) web crippling, (7) fatigue loading, and (8) tensile field capacity.

The suitability of a girder with a uniformly reduced web thickness may be checked using a service load or a load factor method. In the following, provisions for evaluation according to the current AASHTO service load specifications for design are given.

a. *Web Buckling Due to Bending.* Web buckling due to bending is addressed by Article 10.34.3 of the AASHTO specifications. The specifications limit the thickness of the web such that elastic web buckling due to pure bending is prevented, and does not control the failure mode. Formula 10-23 of the AASHTO specifications relates the allowable slenderness of the web to the compressive bending stress in the flange. It was adopted by AASHTO in 1965. The minimum ratio of the thickness of the web, tw , to its clear depth, D , is limited to $1/170$ for webs without longitudinal stiffeners. AISC specifications, Article 1.10.6, allow a higher slenderness ratio if a reduced maximum allowable bending stress is used. The reduced allowable bending stress, Fb' , may be obtained from

$$Fb' \leq Fb \left[1.0 - 0.0005 \frac{A_w}{A_f} \left(\frac{D}{tw} - \frac{760}{Fb^{1/2}} \right) \right] \quad (50)$$

where Fb is the applicable bending stress, A_w is the area of web at the section under investigation, and A_f is the area of the flange.

The post-buckling strength of the web is taken into account indirectly by using a relatively low factor of safety of 1.19 against elastic buckling. The web is assumed conservatively to be simply supported at all edges.

Although designed not to control the capacity of the web, under certain corrosion conditions web buckling due to bending may become the governing failure mode. If the web buckling due to bending governs, the residual capacity factor at the local level may be expressed by

$$RCF\ell = \left(\frac{twd}{tw} \right) \left(\frac{twd}{tw} \right)^2 = \left(\frac{twd}{tw} \right)^3 \quad (51)$$

where twd is the reduced web thickness. This expression is based on Eq. 10-23 of Article 10.34.3 of the AASHTO *Standard Specifications for Highway Bridges* and reflects reduction in capacity due to the increase in stress, (twd/tw) , and the reduction in the resistance to web buckling due to bending, $(twd/tw)^2$.

b. *Web Buckling Due to Shear.* Web buckling due to shear is addressed by Article 10.34.4 of the AASHTO specifications. A distinction is made between girders with transverse stiffeners and those with no transverse stiffeners, and between end web panels and intermediate web panels.

(i) Girders with no transverse stiffeners.—According to Article 10.34.4.1 of the AASHTO specifications, transverse intermediate stiffeners may be omitted if the ratio of the thickness of the

web, tw , to its clear depth, D , is larger than $1/150$, and the calculated average shear stress at the section considered is less than the allowable value given by Eq. 10-25 of the AASHTO specifications. Equation 10-25 relates the allowable shear stress in the web to its slenderness, D/tw . It is based on elastic web buckling due to shear. The web is assumed conservatively to be simply supported at all edges.

A relation similar to Eq. 10-25 was first introduced by AASHTO in 1965. Prior to 1965 transverse intermediate stiffeners could only be omitted if the ratio of the thickness of the web to its clear depth was larger than $1/60$ for carbon steel, $1/55$ for silicon steel, and $1/50$ for 50,000 psi low alloy steel.

The latest revisions of the current AASHTO specifications have increased the allowable shear stress given by Eq. 10-25 by about 30 percent (this increase was specified in order to provide a factor of safety against shear buckling consistent with the factor of safety against shear and tensile yielding already employed by the AASHTO specifications). According to Article 10.34.4.5, for calculated shear stresses below the allowable shear stress, transverse intermediate stiffeners may be omitted if the ratio of the thickness of the web to its clear depth is larger than $1/68$ for 36,000 psi yield steel or $1/58$ for 50,000 psi yield steel, for example.

If shear buckling controls the capacity of the web, the residual capacity factor at the local level may be calculated from

$$RCF\ell = \left(\frac{twd}{tw} \right)^3 \quad (52)$$

This expression is based on Eq. 10-25 of the AASHTO specifications and it shows that, when shear buckling governs, a reduced web thickness, twd , affects the residual capacity factor at the third power. Also, for the case when shear buckling controls, a code coefficient, CF , as high as 1.3 may apply.

(ii) Interior girder panels.—According to Article 10.34.4.2 of the AASHTO *Standard Specifications for Highway Bridges* the calculated shear stress in an interior panel should be less than the allowable value given by Eq. 10-26. The maximum spacing between interior stiffeners, do , is limited to the smaller of the value of $3D$ or $D[260/(D/tw)]^2$. This formulation allows stiffener spacings of $3D$ for webs with D/tw smaller than 150. Equation 10-26 of the AASHTO specifications relates the allowable shear stress in an interior panel to its slenderness, D/tw , and the aspect ratio, do/D . It takes into account the post buckling strength of the web of an interior panel. The post buckling strength results from diagonal tension fields formed by shear forces greater than those associated with the theoretical buckling load. The combination of the web tension fields and the transverse stiffeners can provide a Pratt truss action that is able to resist an additional shear force unaccounted for by the linear buckling theory.

Prior to 1973, a maximum stiffener spacing for interior panels equal to the depth of the girder, D , was required. This limit was specified because the equation used for calculating the allowable shear stress was valid only up to this spacing. In 1973 the maximum stiffener spacing for interior panels was increased to $1.5D$ and the equation used for calculating the allowable shear stress was formulated based on post-buckling strength. The 1988 revisions of the AASHTO specifications have further increased the limitation on stiffener spacing for interior panels to $3D$, as a result of numerous tests that indicated that the formulation used gives a good prediction of the shear strength of panels with

this spacing. Therefore, corrosion of some interior stiffeners of existing girders may not necessarily make the girder web inadequate.

If buckling controls the capacity of an interior panel, then, on the basis of Eq. 10-26 of the AASHTO specifications, the residual capacity factor at the local level may be expressed

$$RCF\ell = \left(\frac{twd}{tw} \right) \frac{0.87 + C_d[(1 - (do/D)^2)^{1/2} - 0.87]}{0.87 + C[(1 - (do/D)^2)^{1/2} - 0.87]} \quad (53)$$

Coefficients C_d and C apply to the reduced web and the intact web, respectively, and are defined in Article 10.34.4.2 of the AASHTO specifications. They depend on both the slenderness, D/tw , and the aspect ratio, do/D .

(iii) End girder panels.—According to Article 10.34.4.3 of the AASHTO specifications the calculated shear stress in an end panel should be less than the allowable value given by Eq. 10-28. The maximum spacing of the first intermediate stiffener is limited to $1.5D$. Equation 10-28 is based on the buckling capacity of the web. The reason for using the buckling capacity as a limit for an end panel is to ensure that the panel can anchor the post-buckling tension field that may develop in the adjacent interior panel.

Up until 1965, the AASHTO specifications allowed a maximum end panel stiffener spacing equal to the depth of the girder D . After 1965 the end panel spacing was limited to $0.5D$. The 1988 revisions of the AASHTO *Standard Specifications for Highway Bridges* have increased the maximum spacing to $1.5D$. Therefore, a girder designed after 1965 with a deteriorated first intermediate stiffener may still be adequate if evaluated by today's standards.

In cases when transverse intermediate stiffeners are required and they are deteriorated, their suitability may be checked against Article 10.34.4.6-10 of the AASHTO specifications. The suitability of deteriorated longitudinal stiffeners may be checked against Article 10.34.5.

If buckling controls, the residual capacity factor for an end panel may be expressed

$$RCF\ell = \left(\frac{twd}{tw} \right)^3 \quad (54)$$

c. *Shear Yielding.* The AASHTO specifications for the design of girder webs are primarily based on buckling criteria. However, all formulations also include a limit of $F_y/3$ on the allowable shear stress for the cases when shear stress yielding may control the web capacity. Shear yielding criteria will usually govern for lower web slenderness ratios of:

$$\frac{D}{tw} \leq \frac{6,000 (k)^{1/2}}{(F_y)^{1/2}} \quad (55)$$

where $k = 5 + [5/(do/D)^2]$.

When shear yielding governs:

$$RCF\ell = \frac{twd}{tw} \quad (56)$$

d. *Combined Bending and Shear.* Equation 10-29 of the AASHTO specifications, Article 10.34.4.4, provides an allow-

able bending stress in a girder panel that is subjected to simultaneous action of shear and bending moment. If the shear stress on the section considered is lower than 60 percent of the allowable shear stress, no reduction in the allowable bending stress is required.

e. *Web Buckling Due to Vertical Compression Stresses from the Compression Flange.* When the thickness of the web near the compression flange is reduced, the web may fail to provide the necessary resistance to the vertical compression stresses from the compression flange and result in vertical buckling of the compression flange. A limiting slenderness ratio, D/tw , is specified by the AISC specifications (6.4) in order to prevent this type of failure. The formula provided for the maximum slenderness ratio allowed is a function of the yield stress, F_y , of the flange material. It yields a maximum slenderness ratio of 322 for 36,000 psi y.p. steel and 243 for 50,000 psi y.p. steel.

f. *Web Crippling.* Loss of web thickness can also result in crippling of the web at points of concentrated loads where no stiffeners are provided. Beams with no bearing stiffeners will most likely be rolled sections.

The AISC specifications, Article 1.10.10, provide formulas for calculating the compressive stress in the web due to interior loads and end-reactions. The resulting compressive stresses are limited to $0.75F_y$. The AASHTO specifications limit the maximum bearing stresses to $0.80F_y$. Rolled sections are proportioned such that localized buckling will not govern. However, corrosion of the web in the area of concentrated loads, where no stiffeners are provided, may result in localized buckling at lower stress levels.

In order to ensure that localized buckling will not occur at the end supports of beams with no bearing stiffeners, an approach based on plate theory can be used (5.22). If one assumes that the portion of the web above an end bearing behaves like a rectangular plate with one edge free and three edges fixed, the following formulation applies:

$$t_w = \frac{b (fa)^{1/2}}{162.3} \quad (57)$$

where fa is the compressive stress in the web, above the bearing, expressed in ksi. It can be calculated from the formula provided in the AISC specifications, Article 1.10.10.1, for end reactions. The length b is the distance over which the bearing stress, fa , is assumed to be distributed. Equation 57 uses a plate buckling coefficient of 1.28 and a factor of safety of 1.25.

In order to ensure that buckling of the web will not occur between stiffeners when loads are applied directly to the flange, the AISC specifications, Article 1.10.10.2 may be used.

g. *Conclusions.* In concluding this section on uniform web losses, it is noted that the AASHTO specifications for the design of girder webs have been modified over the years to take into account results of newer experimental and theoretical research work. Most significant were the increase in allowable shear stress in unstiffened girder webs, the recognition of the post-buckling capacity of stiffened girder webs, and the increase in the maximum spacings of intermediate transverse stiffeners. As a result of these changes, a girder web designed in the past based on more conservative criteria may be found satisfactory according to current criteria even after experiencing a certain degree of corrosion loss.

In most plate girders the buckling criteria will govern the capacity of the web. The formulations for the buckling capacity

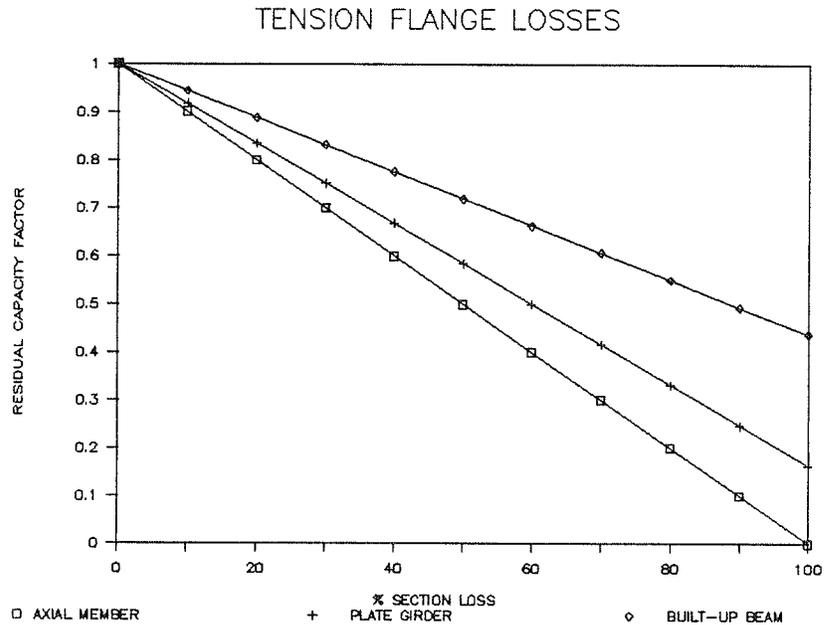


Figure 75. Residual capacity factors for an example beam with uniform tension flange losses.

of stiffened and unstiffened girder webs are based on simply supported boundary conditions. This is a conservative assumption since tests have shown that the rotational restraint offered by the flanges provide fixed edges to the web. The plate buckling coefficient corresponding to a clamped boundary condition at the flanges can be more than 60 percent greater than that corresponding to a simply supported condition. This indicates an additional reserve in buckling capacity that is not accounted for by the AASHTO specifications.

4.6.2 Localized Web Losses

Localized web losses are treated in detail in section 4.7.

4.6.3 Tension Flange Losses

A conservative approach to evaluating the effects of corrosion of the tension flange of a beam is to assume that the tension flange is an independent member loaded in axial tension. Then, section 4.1 will apply.

A more accurate evaluation must take into account the participation of the web in the load redistribution resulting from flange losses. In order to study the effects of flange losses on the elastic stresses in beams, a finite element investigation of a built-up beam model has been carried out, and is reported in Appendix E, Section 2. The conclusions of this investigation are incorporated in the following guidelines.

4.6.3.1 Uniform Losses

The effects of uniform flange losses on the residual capacity of a beam can be evaluated on the basis beam theory, using a

reduced section modulus, S_{dm} . The residual capacity factor then becomes

$$RCF_m = \frac{S_{dm}}{S_m} \quad (58)$$

where S_m is the original section modulus. The effects of tension flange losses on the residual capacity factor in the case of a plate girder and a built-up beam are shown in Figure 75. In the case of the built-up beam, the legs of the flange angles connecting to the web were not included in the tension flange area. Figure 75 shows that the web participates in the stress redistribution resulting from flange losses and that the contribution of the inside legs of the flange angles in a built-up beam can be quite significant.

4.6.3.2 Localized Losses

When the extent of the loss is limited to a small portion along the flange the resulting stress distribution can be quite different from the stress distribution predicted by beam theory. Localized losses can cause higher stresses than those obtained based on beam theory. The increased stresses depend on the amount of flange section loss, the extent of loss along the flange, and the type of change from reduced to full section; i.e., gradual or abrupt. They are localized mainly at the location of the transition from reduced to full section area, but can also affect the overall stress level in the center area of the loss. The increased stresses were found to attenuate rapidly into the web away from the area of the loss.

a. *Service Load Method.* When using the service load definition of failure, which is the condition where the outer fibers of the beam reach their yield level, the stress concentrations caused

by localized flange losses can affect the calculated residual capacity of the beam. This is illustrated in Figure 76 for the case of the built-up beam model used in the analytical investigation reported in Appendix E, Section 2. The length of loss considered is 12 in. and the percentage of flange section loss is related only to the outstanding legs of the bottom flange angles and the existing cover plate. The lower curve in Figure 76 shows the effects of flange section loss when the flange is assumed to behave like an independent axially loaded member, and the upper curve represents results based on beam theory. The curves in between were determined based on finite element analysis and include the stress concentrations which resulted at the location of the transition from reduced to full section and in the center area of the loss. It can be seen that the stress concentrations have lowered the service load residual capacity factor to levels between those predicted by beam theory and those predicted by an axially loaded bar analogy. For a greater length of loss the residual capacity factor curve will be closer to that predicted by beam theory, and for a smaller length of loss it will approach the curve predicted by the axially loaded bar analogy. It appears that beam theory provides an upper bound and the axially loaded bar analogy provides a lower bound to the service load residual capacity factor. In the case of the built-up beam model studied, the residual capacity factor obtained was only about 5 percent higher than that predicted by beam theory for a length of loss of over 3.0 ft when a 30 percent flange section loss was considered. When a length of loss below 6 in. was considered, the results obtained approached those predicted by the axially loaded bar analogy in an asymptotic manner. Therefore, highly localized losses in the tension flange should be evaluated using the axially loaded bar analogy. When the tension flange losses are uniformly

distributed along the flange over a distance larger than 3.0 ft and have gradual transitions from reduced to full section, beam theory may be used to evaluate losses below 30 percent. When the length of loss exceeds 5.0 ft beam theory may be used for flange losses of up to 50 percent.

b. *Load Factor Method.* If a load factor method is used, the stress concentrations at points of localized corrosion of the flange may be neglected when evaluating static resistance.

c. *Effects of Location of Deterioration Along the Flange.* The location of the flange loss can affect the residual capacity of the beam. If the loss is located at a point of maximum moment the effect on the residual capacity of the beam will be most significant.

4.6.3.3 Deformation Criterion

The effect of uniform flange losses on beam deflection can be determined using beam theory with a reduced moment of inertia. The effect of corrosion losses in a flange on the moment of inertia of the beam is not as pronounced. For example, in the case of the built-up beam model studied, a 30 percent uniform reduction in the bottom flange area caused an increase of about 9.5 percent in deflection.

As the length of flange loss along the beam decreases, the effect of the loss on beam deflection also decreases. For example, in the case of the built-up beam model studied, a 30 percent loss over 25 percent of the length of the flange in the central portion of the beam increased the deflection by only about 4 percent. Therefore, localized flange losses will have a negligible effect in terms of shedding load into adjacent structural members.

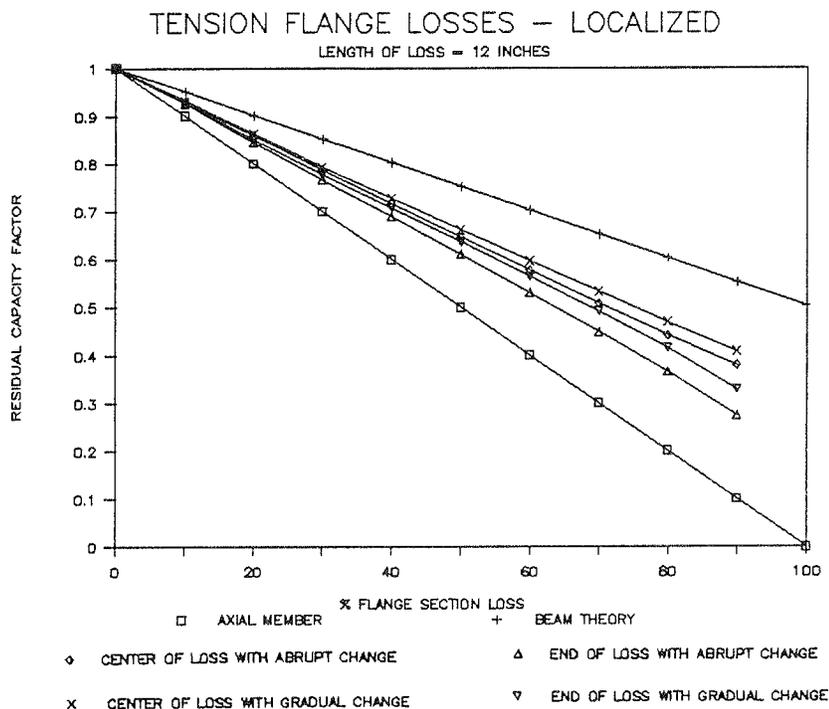


Figure 76. Residual capacity factors for beams with localized flange losses.

4.6.3.4 Fatigue Criterion

In order to evaluate the effects of localized flange losses on the fatigue resistance of the beam the guidelines that apply to axial members (see section 4.1.6) may be used.

4.6.4 Compression Flange Losses

4.6.4.1 Strength Criteria

For the evaluation of the effects of compression flange losses on the strength of beams, the guidelines provided for tension flange losses (see section 4.6.3) apply.

4.6.4.2 Stability Criteria

Uniform or localized corrosion of the compression flange of a girder may also affect the resistance of the flange to buckling and result in instability failure. The buckling modes of the compression flange in a girder include lateral buckling, vertical buckling, and torsional buckling.

a. *Lateral Buckling.* The resistance to lateral buckling is provided by the resistance of the flange to lateral bending and the resistance of the beam to torsion. In order to prevent lateral buckling of the compression flange, Article 10.32 of the AASHTO specifications reduce the allowable compression stress in the extreme fibers of beams by an amount proportional to $(\ell/b)^2$, where ℓ is the unsupported length of the compression flange and b is its width. The formulation used by AASHTO accounts only for the resistance of the flange to lateral bending. If corrosion of the compression flange reduces its width b , the resistance to lateral buckling will decrease and may, therefore, result in a reduction in the residual capacity of the beam. If lateral buckling governs, the residual capacity factor for the case of uniform compression flange losses may then be expressed as:

$$RCF_m = \left(\frac{S_d m}{S_d} \right) \left(\frac{1 - \frac{3F_y I^2}{\pi^2 E} / b_d^2}{1 - \frac{3F_y I^2}{\pi^2 E} / b^2} \right) \quad (59)$$

where b_d is the reduced flange width.

b. *Vertical Buckling.* Vertical buckling of the compression flange may occur when the vertical support of the compression flange provided by the web is reduced. This possibility is treated in section 4.7.5.

c. *Torsional Buckling.* Torsional buckling of the compression flange is essentially a local buckling problem. It can be regarded as the buckling of a uniformly compressed plate free along one edge and hinged at the location of the web. In order to prevent torsional buckling, design specifications limit the value of the ratio of the width of the outstanding leg (or half of the flange width) to its thickness. In order to evaluate corrosion effects on the localized buckling resistance, the guidelines provided for compression members (see section 4.3.2) may be used.

4.6.4.3 Deformation Criterion

The effects of compression flange losses on the deflection of beams are similar to the effects of the tension flange losses, as discussed in section 4.6.3.3.

4.7 BEAMS WITH WEB HOLES

4.7.1 General

Localized corrosion losses can create an area of reduced web thickness or web holes of irregular shape or size. A significant change in the stress field near the deterioration and a reduction in the overall capacity of the beam may result.

Several analytical and experimental investigations on the behavior of beams with web holes have been done in the past because of the need to provide holes in webs of beams used in buildings and industrial applications. On the basis of these investigations, approximate methods of analysis have been formulated—some based on the theory of elasticity and others based on Vierendeel analysis (a review of the relevant literature on the subject is given in the next section). According to the Vierendeel analogy, the beam, near a hole, is assumed to act like a Vierendeel frame. A distinction is usually made between two effects of holes: (1) local stress concentrations occurring at the boundaries of the hole, and (2) change in the configuration of the overall stress field in the area affected.

Experimental results have shown that, except for stress concentrations at the boundaries of the holes, Vierendeel analysis can adequately predict the overall stress distribution on a transverse cross section. It has also been shown that the concentration of stresses at the hole corners attenuate rapidly away from the hole and have little effect on the static behavior of the beam. Most of the experimental and theoretical investigations, however, have been limited to circular or rectangular holes centered on the centroidal axis of the beam. Fewer investigations have considered holes located off the centroidal axis.

The type of holes created by corrosion have not been studied or tested. An analytical investigation of the effects of holes, which better resemble the type of holes created by corrosion, has been carried out as part of this study and is reported in Appendix E, Section 1. The study is based on finite element analysis of a built-up beam with typical corrosion losses in its end panel. The results of this investigation are in agreement with the findings of the previous studies. The stress concentrations at the corners of the holes were found to attenuate rapidly in the surrounding web media, and the resulting normal stress distributions at transverse sections clearly indicated Vierendeel action. Small holes resulted in stress concentrations, but they had little effect on the overall stress field. Thus, the literature on beams with intentional web holes may also be applied to holes caused by corrosion.

The existence of holes in the web of a beam can affect the behavior of the beam in different ways. It can reduce the resistance of the beam to shear, bending or buckling; it can reduce the buckling resistance of the compression flange above the hole; and it can also reduce the fatigue life of the beam. Therefore, failure may occur as a result of increased bending and shear stresses, buckling of the compression flange, buckling of the web, and fatigue.

This section defines the evaluation criteria relevant to the above failure modes. It provides a method of analysis for de-

termining overall bending and shear stresses and guidelines for the evaluation of the effects of web holes on strength, buckling, and fatigue resistance.

4.7.2 Review of Relevant Literature

Several papers are available on the topic of beams with web holes. The majority of these papers address circular or rectangular holes centered on the centroidal axis of the beam. However, some of the papers available also address eccentric holes, holes in composite beams, and effects of holes on ultimate strength, buckling, and fatigue resistance.

An analytical method for calculating the elastic stresses around an elliptic hole centered on the centroidal axis of a simply supported beam is presented in Ref. 7.5. The equations developed for calculating the stresses around the hole are based on the theory of elasticity. The applicability of this method, however, is rather limited and it depends on the size and shape of the hole and the moment-shear ratio at the center of the hole.

Analytical and experimental investigations of beams with concentric holes are reported in Refs. 7.6 and 7.48. The laboratory tests described in Ref. 7.48 were limited to rectangular holes and the analytical investigations performed were based on Vierendeel analysis. A method was presented to determine the amount, as well as the necessity for web reinforcement. In Ref. 7.6 both circular and rectangular holes were considered. The results of the tests performed were compared with analytical results based on the theory of elasticity and on Vierendeel analysis. In both studies, Vierendeel analysis was found to provide reasonably accurate predictions of stresses in the vicinity of the holes, except for local stress concentrations. Analysis based on the theory of elasticity was found to predict stress concentrations, but its use was relatively complex.

An experimental and analytical investigation of the ultimate strength of beams with rectangular holes centered on the centroidal axis is summarized in Ref. 7.7. The beams tested were designed such that yielding, rather than buckling, governed. The results of the tests showed that the first yielding in the beam occurred at the corners of the hole as a result of stress concentration. The load at which first yielding occurred could adequately be predicted by the theory of elasticity. As the load was increased, yielding spread through the web and flanges. The ultimate strength in the region of the hole was then limited by full plastification of the cross sections through the corners of the hole.

Guidelines for the design of beams with web holes are suggested in Refs. 7.6 and 7.50. Both elastic and plastic design criteria are considered. The guidelines apply to static loading and are based on Vierendeel analysis. Although developed for concentric rectangular holes, some recommendations regarding applications to other cases are also included. For example, for an elliptic hole with its major axis in the longitudinal direction of the beam, it is suggested that an equivalent rectangular hole of equal maximum depth as the actual hole be used and that length be determined on the basis that the assumed hole and the actual hole have equal areas. It is also recommended that, for computing stress concentrations, the theory of elasticity be used.

An investigation of the behavior of beams with rectangular off-centered holes is described in Ref. 7.19. A generalized Vierendeel frame analysis which applies to beams with eccentric holes is presented. Stress frozen photoelastic models were used to

verify the method of analysis proposed. The test results showed good agreement with the predicted stress distributions, except in areas of high stress concentration.

Relatively little work on the effects of web holes on beam stability has been reported. Some tests of beams with web holes have shown that the possibility of vertical flange buckling at the location of the hole can be quite severe. Other tests have indicated the possibility of web buckling between adjacent holes and web crippling near support points. The possibility of local web buckling at the hole due to the creation of an unsupported edge of web loaded in compression was investigated in Ref. 7.41. Only limited data on the overall buckling resistance of webs with holes are available. An investigation of the stability of plates with centrally located rectangular holes is reported in Ref. 7.8 for various in-plane loadings. Plates subjected to shear loading showed a significant reduction in elastic stability. The plates subjected to other forms of loading did not show any definite trends. An approximate equation for computing the shear buckling stress for a web with a central rectangular hole is given in Ref. 7.37.

An experimental investigation of the fatigue resistance of beams with rectangular holes is reported in Ref. 7.27. The investigation showed that the fatigue resistance was directly related to the stress concentrations at the corners of the hole, which, in turn, depended on the corner radii. Elastic analysis was found to give a good approximation of the magnitude of the fatigue stress range and the experimental results showed good agreement with the predicted fatigue lives. The fatigue lives for the corner stresses were found to correlate reasonably well with fatigue lives for plane plates subjected to axial loads. In all tests in which fatigue cracks developed the theoretical stress range was greater than 24 ksi. A corner radius of 1 in. was suggested as a minimum for the design cases.

Several recent investigations address the behavior of composite beams with web holes. An experimental study of composite beams with rectangular holes centered on the centroidal axis of the steel section is reported in Ref. 7.13. The results of the tests indicated Vierendeel action; the lower the moment-shear ratio the more pronounced the Vierendeel effect. An analytical model for the strength of composite beams with web holes is presented in Ref. 7.12. The model includes the contribution of the concrete slab to shear as well as flexural strength. Practical design procedures for composite beams with web openings are presented in Ref. 7.17. The methods of analysis proposed are compared with predictions of other analysis techniques and test results.

4.7.3 Analysis Procedure Suggested for Beams with Web Holes

4.7.3.1 General

The analysis procedure suggested here is based on the generalized Vierendeel frame analysis presented in Ref. 7.19. It applies to eccentric, rectangular web holes.

According to the Vierendeel analogy, the sections of the beam above and below the hole are assumed to behave like two independent beams rigidly connected at their ends to the rest of the beam. The shear forces on these sections produce secondary bending stresses that are maximum at the ends of the hole. Thus, the total bending stress distribution in the beam will include, in addition to the primary bending stresses caused by beam bend-

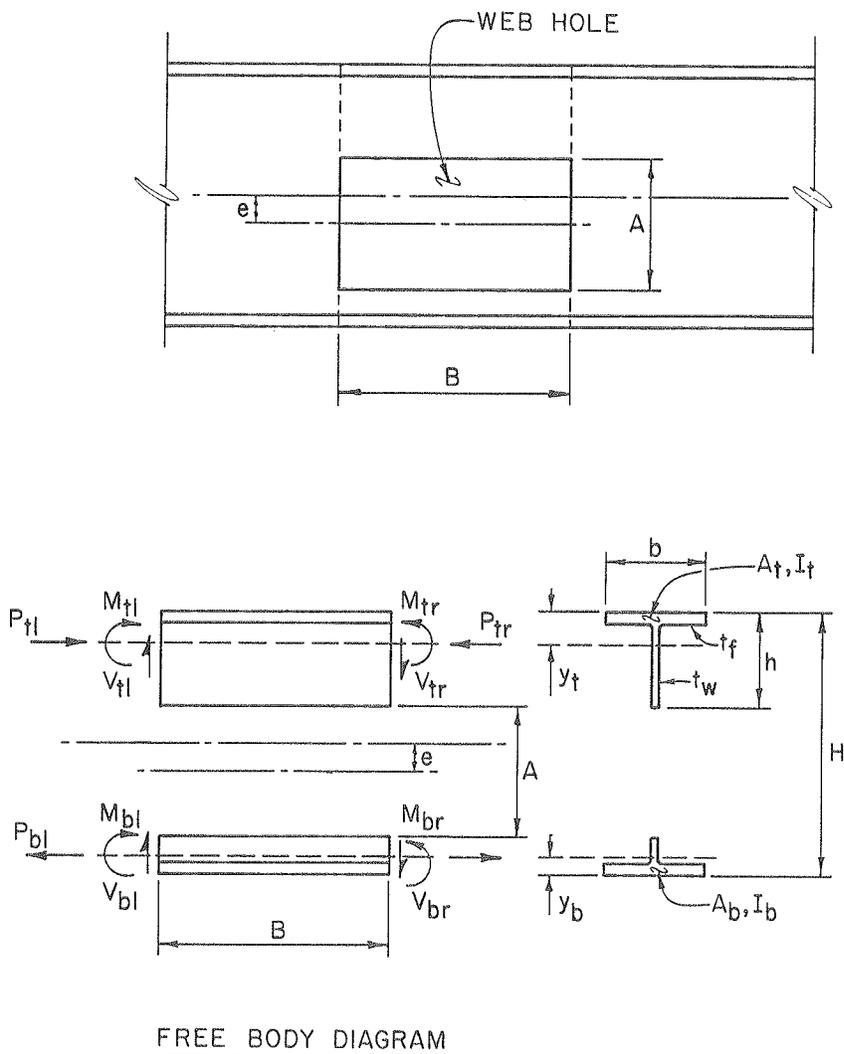


Figure 77. Model of analysis for beams with web holes.

ing, secondary bending stresses caused by Vierendeel bending (stress concentrations are not included). The distribution of the shear force between the beam section above the hole and the beam section below the hole depends on their relative stiffness. If the depth of these sections is large relative to their length, the contribution of the shear deformation relative to that of the bending deformation can be quite significant.

The analysis procedure is derived from equations which define equilibrium and continuity conditions at the ends of the beam sections above and below the hole (see Figure 77). It includes the effects of the shear deformation along the length of the hole. Shear coefficients, as defined in Ref. 7.14, are used. No concentrated loads are assumed to exist directly over the hole, so that a constant shear force could be considered.

The test data available limit the applicability of this procedure to holes not deeper than two-thirds of the beam depth. For the calculation of stress concentrations it is suggested that a procedure based on the theory of elasticity be used. However, because the stress concentrations are highly localized and depen-

dent on the exact geometry of the hole, and because structural steel has good ductility, their calculation is not needed for the evaluation of static strength. Even when fatigue loading is involved, the diverse nature and high irregularity of hole boundaries created by corrosion make the calculation of stress concentrations impractical (see section 4.7.8).

4.7.3.2 Summary of Procedure

The procedure suggested refers to Figure 77. It includes the following steps:

- Step 1*—Determine the shear force V , at the location of the hole, the moment M_p , at a section on the left side of the hole, and the moment M_r , at a section on the right side of the hole.
- Step 2*—Calculate the following section properties: A_p , A_b , I_p , I_b , y_p , and y_b . Calculate constant K from:

$$K = \left(\frac{1}{A_t} + \frac{1}{A_b} \right) \left(\frac{1}{H - y_t - y_b} \right)$$

Calculate shear coefficients R_t and R_b . For a T-section, use:

$$R_t \text{ or } R_b = \frac{(1 + 4m)^2}{1.2 + 9.4m + 27.0m^2 + 19.7m^3 + 3.2n^2m(1 + 1.1m + 0.1m^2)}$$

where $m = (bt_p)/(ht_w)$ and $n = b/h$.

Step 3—Find V_t ($V_t = V_{t\ell} = V_{tr}$) and V_b ($V_b = V_{b\ell} = V_{br}$) from the following equations:

$$\begin{aligned} V_t + V_b &= V \\ \frac{V_t}{V_b} &= \frac{\frac{B^2}{6EI_b} + \frac{2}{GA_bR_b}}{\frac{B^2}{6EI_t} + \frac{2}{GA_tR_t}} \end{aligned}$$

where $E = 29,000$ ksi, and $G = 11,500$ ksi (G being the shear modulus).

Step 4—Find P ($P = P_{t\ell} = P_{tr} = P_{b\ell} = P_{br}$) from:

$$P = \frac{M\ell + \frac{B}{2}V}{K(I_t + I_b) + (H - y_t - y_b)}$$

Step 5—Find $M_{t\ell}$ and $M_{b\ell}$ from:

$$\begin{aligned} M_{t\ell} &= I_t K P - \frac{B}{2} V_t \\ M_{b\ell} &= I_b K P - \frac{B}{2} V_b \end{aligned}$$

Step 6—Find M_{tr} and M_{br} from:

$$M_{tr} = M_{t\ell} + BV_t \text{ and } M_{br} = M_{b\ell} + BV_b$$

Step 7—Calculate normal and shear stresses at the corners of the hole and at the flange level above and below the corners of the hole using basic equations of mechanics. Critical stresses can occur at each of these locations.

4.7.3.3 Applicability to Holes of a Different Shape

The analysis procedure suggested was developed for holes of a rectangular shape. It has been found (7.4, 7.6, 7.50), however, that the results obtained based on Vierendeel analysis for rectangular holes can also be applied to holes of a different shape, by replacing them by equivalent rectangular holes.

Circular holes have been suggested to be replaced by a rectangular hole whose height is 0.9 the diameter of the circular hole and whose total length is 0.45 the diameter of the circular hole (7.50).

Elliptical holes with their major axis in the longitudinal direction of the beam have been suggested to be replaced by rectangular holes of equal maximum depth as the actual hole and with a length determined such that the assumed and the actual holes have the same area (7.4, 7.6).

It is recommended here that the irregular holes created by corrosion be conservatively replaced by an equivalent rectangular hole using the above suggestions as guidance.

4.7.3.4 Derivation of the Analysis Procedure

The analysis procedure suggested (refer to Figure 77) is based on the solution of the equations of equilibrium and continuity at the ends of the sections above and below the hole, as defined in Ref. 7.19:

- Equilibrium conditions for the top T-section yield:

$$P_{t\ell} = P_{tr} = P_t; V_{t\ell} = V_{tr} = V_t; \text{ and } M_{tr} = M_{t\ell} + V_t B$$

- Equilibrium conditions for the bottom T-section yield:

$$P_{b\ell} = P_{br} = P_b; V_{b\ell} = V_{br} = V_b; \text{ and } M_{br} = M_{b\ell} + V_b B$$

- Equilibrium conditions for the left joint yield:

$$P_t = P_b; V = V_t + V_b; M_{\ell} = M_{t\ell} + M_{b\ell} + P(H - y_t - y_b)$$

- Continuity conditions yield:

$$\theta_t = \theta_b; \Delta y_t = \Delta y_b$$

where θ_t , θ_b , Δy_t and Δy_b are rotations and vertical displacements at the ends of the T-sections.

• Using Castigliano's theorem, end rotations and displacements can be expressed as follows:

$$\theta_t = \frac{B}{EI_t} \left(M_{t\ell} + \frac{V_t B}{2} \right)$$

$$\theta_b = \frac{B}{EI_b} \left(M_{b\ell} + \frac{V_b B}{2} \right)$$

$$\Delta y_t = \frac{B^2}{EI_t} \left(\frac{M_{t\ell}}{2} + \frac{V_t B}{3} \right) + \frac{V_t B}{A_t R_t G}$$

$$\Delta y_b = \frac{B^2}{EI_b} \left(\frac{M_{b\ell}}{2} + \frac{V_b B}{3} \right) + \frac{V_b B}{A_b R_b G}$$

where R_t and R_b are shear coefficients defined in Ref. 7.14 for various cross sections.

4.7.4 BENDING AND SHEAR FAILURE

a. *Bending Stresses.* Web holes have little effect on the primary bending stresses in a beam because most of the bending moment capacity is provided by the flanges. However, the bending stresses resulting from secondary bending moments due to Vierendeel action can be quite significant. These secondary bending stresses are additive to the primary bending stresses. The nominal stresses that result can be calculated using the procedure suggested in section 4.7.3. The largest bending stresses can occur at the corners of the hole or at the location of the top or bottom flanges right above or below the corners of the hole (see Figure 78). Therefore, stresses at all these locations must be computed.

b. *Shear Stresses.* Most of the shear capacity of a beam is provided by the web. When a hole exists in the web, the transverse shear is distributed between the section of the beam above the hole and the section of the beam below the hole, according to their stiffness properties. The shear forces on the top and bottom sections can be calculated using the procedure suggested in section 4.7.3. The shear stress distribution for each section can then be determined. Maximum shear stresses occur at the top and bottom flange levels above or below the hole (see Figure 78).

c. *Combined Bending and Shear.* The critical locations in the area of the hole should also be checked for combined bending and shear stresses. Equation 10-29 of the AASHTO specifications, Article 10.34.4.4, may be used.

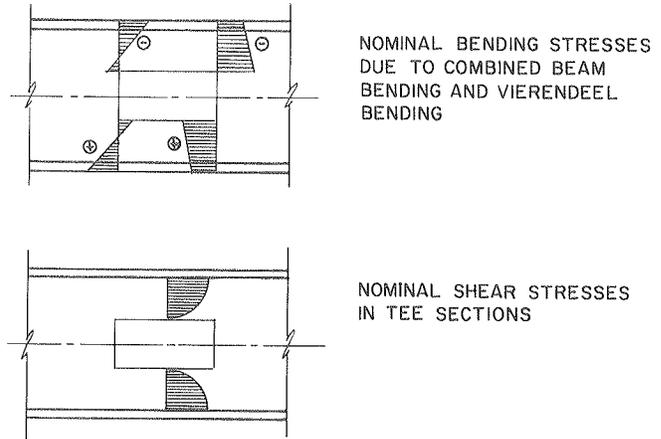


Figure 78. Nominal stress distributions at the location of web holes.

4.7.5 Compression Flange Buckling

Web holes can reduce the resistance of the compression flange to buckling and result in a number of different flange failure modes such as vertical or overall lateral buckling.

a. *Vertical Buckling of the Compression Flange.* If the hole is located close to the compression flange, vertical buckling of the T-section formed by the hole is most likely to occur. This type of buckling has been observed in tests of beams with holes even of moderate length. The T-shaped flange over the hole may be treated as an isolated column with a constant compressive stress along its length. The AASHTO column formulas can be used to evaluate its susceptibility to elastic or inelastic buckling. The effective member length should be based on the length of the hole and on an effective length factor, K . A value of $K = 0.65$ is recommended (7.50). This value reflects the high restraint provided by the stiffer gross section of the beam. It was also confirmed by test results.

b. *Lateral Buckling of the Compression Flange.* Lateral buckling of the compression flange is discussed in section 4.6.4. The resistance to lateral buckling is provided by the torsional resistance of the beam and the resistance offered by lateral flange bending. A web hole can reduce the torsional resistance of the beam, but has little effect on the lateral bending resistance of the compression flange. The AASHTO provisions for ensuring the lateral stability of beams consider only the resistance of the compression flange to lateral bending (with no portion of the web included). Therefore, for beams evaluated based on the current AASHTO specifications, web holes will not indicate any effect on the lateral stability of the compression flange.

4.7.6 Web Buckling

Tests of beams with web holes have shown the possibility of localized web buckling between adjacent holes, web crippling at support points, and localized web buckling at an unsupported edge of the hole. The possibility of overall buckling of a web panel with a hole also exists.

a. *Localized Web Buckling at the Boundary of the Hole.* The possibility of localized web buckling at or near the hole is due to the creation of an unsupported web boundary, which may be subject to high compressive stresses. The maximum width-thickness ratio of unstiffened elements under compression recommended in Article 10.35.2 of the AASHTO specifications can be used as a guide. If the width-thickness ratio of the un-

ported portion of the web exceeds the values recommended by AASHTO, the AISC specifications (6.4), Appendix C, may be used to evaluate the allowable compressive stresses on the web. This approach for verifying localized web buckling assumes that the length of the hole is longer than the theoretical buckling wavelength corresponding to the maximum width-thickness ratios specified by AASHTO. Therefore, in the case of shorter holes the results will be on the conservative side.

b. *Web Crippling.* Web holes in the vicinity of a support can reduce the bearing resistance of the beam. If there is a bearing stiffener, the web hole may take away from the web area assumed to participate with the stiffener in bearing capacity. If no bearing stiffener exists, web crippling may occur. The susceptibility to crippling of webs without holes may be assessed using the guidelines given in section 4.6.1. Because of the lack of appropriate experimental data on crippling of webs with holes, it is suggested in Ref. 7.50 not to tolerate holes too close to the edge of the bearing, if no bearing stiffeners are present. The minimum distance allowed between the edge of the hole and edge of the bearing is equal to half the beam depth. Also, if no stiffeners are present, holes under concentrated loads may have quite severe effects.

c. *Overall Web Buckling.* The existence of web holes can also reduce the overall buckling resistance of the web. It has been shown (7.8) that central holes in plates loaded in shear are capable of significantly reducing the elastic stability. The AISC specifications address web holes in shear panels by not allowing the design of panels with web holes for tension field action. Because of the limited information available and the complexity of the problem, no specific recommendations can be made, at present, regarding the effects of size and location of web holes on overall buckling strength.

4.7.7 Deflection of Beams with Holes

A method for calculating the deflection of beams with web holes is outlined in Ref. 7.50. It consists of calculating the deflection of the beam without the hole, as a first step, and of calculating the secondary deflections due to a modified Vierendeel be-

havior at the location of the hole, as a second step. A correction for slope compatibility is included.

The analytical investigation of the effects of web holes on the behavior of a built-up beam model (see Appendix E, Section 1) has shown that, for the hole types considered, very little increase in deflection actually took place. For the beam studied, a loss pattern, which almost disconnected the web of the end panel from both the top and the bottom flanges (Figure 1-10 of Appendix E), increased the deflection of the beam by only 4.4 percent.

4.7.8 Fatigue Criteria

Some fatigue test data on beams with rectangular web holes is provided in Ref. 7.27. The test data have shown a correlation between the corner radii of the hole, the calculated stress concentrations at the corners of the hole, and the measured fatigue lives. Fatigue life calculated based on elastic stress concentrations at the corners of the hole was found to agree reasonably well with the experimental results. The fatigue life at the corner of the hole was related to the fatigue life of a plane plate subjected to axial load (AASHTO fatigue category A).

The "British Standard 5400—Parts 1 through 10" (6.6) include in their section on fatigue the case of web holes. Diagrams which show fatigue stress concentration factors as a function of the hole dimensions and corner radii are provided. These fatigue stress concentration factors should be used with fatigue category B, as defined by the British Standards Institution.

However, as a result of the surface or edge irregularities in the case of holes created by corrosion, it is very difficult, if not impossible, to accurately determine elastic or fatigue stress concentration factors. Further discussion on this subject is provided in Chapter 5.

4.7.9 Summary of Recommendations

In order to provide some practical recommendations regarding the effects of holes on the static strength of beams, a distinction is made between small holes, which can be neglected, medium holes, which require analytical investigation for assessment, and severe holes, which must be repaired immediately. Holes located near connections, such as in the web of a beam close to a stringer connection, should be checked individually for additional effects, such as out of plane movements.

4.7.9.1 Small Holes

Holes that have little effect on the strength of a beam are defined here as small holes. Small holes can be neglected if: (1) the overall greatest internal dimension does not exceed one-tenth of the depth of web; (2) the longitudinal distance between the boundaries of two adjacent holes is at least three times the maximum internal dimension; (3) not more than one hole exists at any cross section; and (4) the hole is not located close to a concentrated load where no bearing stiffener exists.

The foregoing requirements are based on the British standards, 9.3.3 (6.6), regarding design of unstiffened openings in webs. The results of the analytical investigations support these requirements.

4.7.9.2 Medium Holes

Medium holes are defined here as those that require analytical investigations for assessment of their effects. The analysis procedure suggested in section 4.7.3 may be used.

4.7.9.3 Severe Holes

Holes that appear to be critical and are outside the range of a reliable analytical estimate are defined here as severe holes. A hole is classified as severe if: (1) its overall greatest dimensions exceed two-thirds of the depth of the web; (2) the longitudinal distance between the boundaries of two adjacent holes is less than three times the maximum internal dimension, or (3) its edge is located closer than one-half of the depth of the web from a concentrated load or the edge of a bearing where no bearing stiffener exists.

4.8 FLOOR SYSTEMS

Corrosion of floor systems is most common. It is usually aggravated by drainage from the deck which may contain deicing salts or other corrosive materials. As a result, severe loss of section may occur at certain locations, especially where poor drainage from expansion joints exists. A detailed description of the details and locations most likely to be affected by corrosion may be found in the field guidelines (Part I of this report). Usually, corrosion will occur in top flanges of stringers and floorbeams, in bottom flanges where laterals are in contact with the flanges, and in webs of floorbeams above the bottom flange, stringer flanges, or erection seat angles.

The main members of floor systems are primarily loaded in bending. Thus, for a Level I office evaluation of the effects of material loss due to corrosion, the guidelines provided in sections 4.5, 4.6, and 4.7 may be used.

If a Level II office evaluation of the capacity of a floor system is made, several reserves in strength may be taken into account, as discussed in the following.

a. *Participation of Concrete Deck Slabs.* Test results have indicated that some composite action is present in concrete deck slabs, even in the absence of mechanical shear connectors, and that fully composite interaction may be assumed for load levels up to the elastic limits (7.53). An allowance of such composite action is also permitted in Ref. 6.12 for bridge evaluation.

b. *Dispersion of Wheel Loads on Floorbeams.* The design of floorbeams assumes that the wheel loads load the floorbeams as concentrated loads. This results in an overestimation of the moments in floorbeams. A simplified approach for estimating the effects of the dispersion of wheel loads on floorbeams may be found in Ref. 5.4.

c. *Distribution of Wheel Loads on Stringers.* In many cases the AASHTO criteria for distribution of wheel loads is conservative. As an alternative, a more accurate and less conservative method for calculating the distribution of wheel loads on stringers may be used. Such a method is described in Ref. 5.10.

d. *Continuity of Stringers.* Although designed as simply supported, in many cases stringers will behave as continuous members for service load conditions. However, if an evaluation based on an ultimate strength approach is made, the stringers should be considered as simply supported.

4.9 TRUSS BRIDGES

Main truss members, top and bottom lateral members, and sway members are primarily loaded in axial tension or compression. For a Level I office evaluation of corrosion effects at the member level, the guidelines provided in sections 4.1 to 4.4 may be used. Special attention should be given to members that can be loaded in both tension and compression.

When evaluating the effects of corrosion on the overall behavior of a bridge, factors such as member criticality, redundancy, and load distributions should be considered as well. In order to investigate the effects of corrosion losses on the overall behavior of truss bridges, a detailed analysis of a simple span Warren truss model has been carried out and is reported in Appendix E. The investigation was limited to the linear elastic range of the structure. The following observations are based on the results of that investigation.

a. *Effects of Method of Analysis.* When a plane truss linear-elastic model is used for the analysis of a bridge truss, truss joints are assumed pinned and truss members are assumed to behave as axially loaded bars in tension or compression. If the truss is statically determinate, loss of section in a member will not affect stresses in the other truss members.

A plane frame model can offer a more accurate representation of the behavior of bolted or riveted joints that are actually rigid connections. Even pinned joints will usually behave as rigid or semirigid connections because of friction caused by pressure on the pin or possible corrosion at the pin. In a plane frame model, the total stresses in a member include axial effects and also bending effects in the plane of the truss. By using a plane frame model, the effects of frame action due to joint rigidity can be taken into account. These effects are relatively small for some members and negligible for others. When the cross-sectional area of truss hangers or diagonals in the truss bridge model studied was reduced, load redistribution reduced the stresses in these members by a small amount. For example, for a 50 percent section loss in a hanger, the stress in that hanger, given by the plane frame model was about 10 percent lower than the stress given by the plane truss model. When the cross-sectional area of top chord members, end posts, or bottom chord members was reduced, the plane frame model showed very little reduction in stress in these members due to load redistribution. The plane frame model also showed an increase in stress in truss members located in the immediate vicinity of the member whose cross-sectional area was reduced. The increase in stress was mainly due to bending stresses resulting from frame action. For example, a 50 percent section loss in a diagonal caused about a 2 percent increase in axial stress and a 14 percent increase in the total stress, which includes bending stress, in a nearby hanger. Truss members further away from the member whose cross-sectional area was reduced had very little change in stress.

A three-dimensional frame model can include the effects of the top and bottom lateral systems, the sway bracing system, and members of the other truss. It can also estimate the effects of out-of-plane loads such as moments in floorbeams at hanger connections. Some redistribution of loads due to three-dimensional action takes place when the cross-sectional area of a member is reduced. When the cross-sectional area of top chord members in the truss bridge model studied was reduced, the three-dimensional model showed a small reduction in stress in these members, relative to the plane frame model. For example, for a 50 percent section loss in a bottom chord member, the

stress given by the three-dimensional frame model was about 15 percent lower than that given by the plane frame model. When the cross-sectional area of diagonals, hangers, or end posts was reduced, the three-dimensional model showed that three-dimensional action has very little beneficial effect on these members. Actually, the total stress in hangers given by the three-dimensional model is larger than the stress given by the plane frame model because of out-of-plane bending at the floorbeam connection. The three-dimensional model also showed that the increase in stress in the top lateral system, the bottom lateral system, or the sway bracing system as a result of losses in a truss member can be significant. Losses in top chord members affected mainly the top lateral system, losses in bottom chord members affected mainly the bottom lateral system, and losses in diagonals and hangers affected the sway bracing system. For example, a 50 percent section loss in a bottom chord member caused an increase of about 49 percent in the maximum stress in the bottom lateral system.

b. *Effects of the Amount of Section Loss.* For amounts of section loss below about 30 percent, the effects of load redistribution due to frame or three-dimensional action on the corroded member were found to be negligible. However, the effects on the other members of the bridge were more significant. For example, a 30 percent section loss in a bottom chord member increased the maximum stress in the bottom lateral system by about 23 percent. For larger amounts of section loss, the effects of load redistribution on the corroded member and on the other members of the bridge became more and more significant. For example, a 70 percent section loss in a bottom chord member increased the stress in that member from 12 ksi to 36 ksi when three-dimensional action was taken into account. At the same time, the maximum stress in the bottom lateral system increased from 5.6 ksi to 11.8 ksi. When load redistribution is neglected the stress in the corroded bottom chord member is about 40 ksi.

c. *Effects of Load Redistribution.* Load redistribution due to corrosion losses in a truss member may result in: (1) a reduction in load in the corroded member, and (2) an increase in load in adjacent bridge members.

For amounts of section loss below 30 percent, the effect of load redistribution on the corroded member is negligible. For larger amounts of section loss a small reduction in load occurs in hangers and diagonals, because of frame action in the plane of the truss, and in the top and bottom chord members, because of three-dimensional action. Load redistribution has very little effect on the end posts.

The effect of load redistribution on bridge members adjacent to the corroded member is more significant. Frame action can increase the load in adjacent truss members and three-dimensional action may increase the load in the top or bottom lateral systems or in the bridge bracing system.

It is recommended that the effects of load redistribution on the corroded member be neglected and the effects of load redistribution on adjacent bridge members be taken into account if the section loss of the corroded member exceeds 20 percent.

d. *Member Criticality and Redundancy.* It appears that the frame action in trusses and the three-dimensional action in truss bridges can provide some reserve in strength to the top and bottom chord members, the diagonals, and the hangers. The hangers, however, are subjected to higher stresses than those assumed in design, from a combination of out-of-plane bending and direct axial tension. The stability of a truss may be governed by buckling of individual compression members or, in the case

of half-through or pony truss bridges, by lateral buckling of the top chord. In order to evaluate the effects of material loss on the lateral stability of the compression chord in half-through or pony truss bridges, the guidelines given in the AASHTO specifications, Article 10.16.12, may be used. A more accurate approach for assessing the capacity of the top chord in pony truss bridges, is given in Ref. 5.12. In many cases the load-carrying capacity of truss bridges was found to be governed by the floor system and the bottom chord tension members (6.12). The semicontinuous nature of the deck, however, can provide some additional reserve in strength to the bottom chord members. The end posts function as part of the compression chords, the legs of the end portals, and also carry the maximum shear in the truss. They have no alternative load paths. The lateral bracing systems resist lateral loads, provide bracing to the compression chord, facilitate load redistribution and, in some cases, the top and bottom laterals participate with the trusses in carrying live load.

In a Level II office evaluation, the nonlinear response range of the bridge structure may be included. In this range a higher degree of load redistribution will take place. The frame action in the plane of the truss and the three-dimensional action can have a considerable influence on the ultimate capacity of a truss bridge. As a result of the larger distortions the participation of the floor system in the load-carrying capacity of the bridge will be more significant. However, at the same time, other members may become highly overstressed and fatigue problems associated with transverse distortions will be aggravated. Therefore, counting on the reserves in strength which exist in the nonlinear range should only be allowed on a temporary basis, until appropriate repairs are made.

4.10 GIRDER BRIDGES

The main members of a girder bridge are loaded in bending. Thus, for a Level I office evaluation of corrosion effects at the member level, the guidelines provided in sections 4.5 to 4.7 will apply. When evaluating the severity of the condition created by corrosion or the effects of corrosion at the structure level, member criticality, redundancy, and effects of load redistribution must be considered as well.

In general, typical steel girder bridges are highly redundant. The girders are continuously connected to a common concrete deck and to each other with diaphragms or cross frames and bottom flange lateral bracing in the longer span bridges. In most cases, girder bridges consist of three or more girders and are load path redundant. When the girders are continuous over the supports additional redundancy is provided by the structural continuity of the bridge. Several case histories in which the inherent redundancy in girder bridges has prevented bridge collapse after severe damage in one girder are described in Ref. 7.15. A state-of-the-art report on effects of redundancy on flexural systems is presented in Ref. 7.53.

The analytical investigations carried out in this study (see Appendix E) have shown that corrosion losses in the flanges or the web of a girder do not have a significant effect on its stiffness, unless the material losses are very severe. As a result, very little load redistribution can be expected to occur while the structure behavior is still elastic.

However, when the loss of material is severe enough to cause extensive yielding or fracture, a significant load redistribution will take place. For example, when one of the girders of the

I-79 bridge over the Ohio River, which is a two-girder bridge continuous over three spans, failed, a significant redistribution of load that prevented total collapse took place (7.47). A subsequent three-dimensional analysis of the I-79 bridge has shown that the torsional stiffness provided by the concrete deck, the cross frames and the wind bracing, and the longitudinal continuity of the girders allowed the redistribution of load. Investigations of load redistribution in girder bridges can also be found in Refs. 7.30 and 7.31. In these studies a two-girder bridge model was considered. When one of the girders was subjected to an induced crack, which caused a large increase in deformations, the cross bracing and the bottom lateral bracing showed a significant contribution to load redistribution. However, as a result of the redistribution of loads the bracing members may become highly overstressed. The increased forces in the bracing members are, in turn, transmitted to the girders and may thus cause increased out-of-plane deformations in the web-to-flange connections. These deformations, even if small in magnitude, can result in large local stresses and initiate fatigue cracks.

In a Level II office evaluation, the nonlinear range of the bridge structure may be included. However, if the contribution of the bracing systems in load redistribution is taken into account, the possible adverse effects discussed above should be considered as well. Other possible reserves in strength, which may be considered in a Level II office evaluation, are related to the participation of the concrete deck in bridges designed as noncomposite, and to the method used for calculating the lateral load distribution (see section 4.8).

CHAPTER 5

EFFECTS OF CORROSION ON FATIGUE RESISTANCE

5.1 INTRODUCTION

A debate currently rages between two factions of the research community as to whether active corrosion has a detrimental effect on fatigue resistance of steel bridges. This debate was a focal point of the recent "Forum on Weathering Steel for Highway Structures" sponsored by the Federal Highway Administration (FHWA) (5.18). One faction of the research community believes that active corrosion does not have a detrimental effect on fatigue resistance. The other believes that active corrosion can reduce the fatigue resistance of steel bridges.

One research program (5.1, 5.2, 5.3) shows that, in the laboratory, active corrosion reduces the fatigue resistance of full-scale specimens by creating new, more severe fatigue initiation sites than those existing in the as-fabricated specimens.

Observing that their inventory of bridges has not exhibited fatigue cracking from heavily corroded areas, the other faction hypothesizes that the rate of corrosion on in-service bridges far out paces the rate of fatigue damage accumulation. As such, it is suggested that the corrosion process, in effect, blunts any potential initiation sites.

Without entering into this controversy, the referenced research (5.1, 5.2, 5.3) can be used to quantify the effects of corrosion on fatigue life when a corroded bridge is blast-cleaned and repainted to inhibit the corrosion process, but the corroded profile (i.e., the pitted surface) is not significantly altered. In a study of corroded rolled beams (Category A) (5.1), researchers observed a remarkably good correlation between pit depth, which can be relatively easily quantified, and the reduction of observed fatigue resistance due to pitting. Therefore, if the corroded area is to be blast-cleaned and repainted to prevent further corrosion, the evaluator can use this relationship to determine the remaining fatigue resistance of the corroded area.

If the corroded area is not retrofitted by cleaning and painting and it is allowed to continue corroding, the question of the remaining fatigue resistance of the area is questionable.

5.2 FATIGUE RESISTANCE OF RETROFITTED CORRODED AREAS

The observed reduction in Category A fatigue resistance may be separated into two parts—one attributed to loss in cross-sectional area and the other due to stress concentration generation at the pits. The procedure outlined to determine the reduction in Category A fatigue resistance because of loss of area in the plane of the pit, the stiffness loss method, yields mixed results. After careful mapping of the section loss in a localized area as suggested in this report, relatively simple and classical section property calculations can account for the first component of fatigue resistance reduction; therefore, this section will concentrate on the second component, that of stress concentration generation.

These research results suggest that a direct correlation exists between pit depth and loss of Category A fatigue resistance due to generation of stress concentrations. A pitting factor analogous to fatigue notch factors (5.3) was developed, and is defined as

$$K_p = 1.2 + 4.0 d_p \quad (60)$$

for d_p = pit depth in inches. The pitting factor represents a mean regression line through 14 data points. It can be used to reduce calculated Category A fatigue lives as follows:

$$N_p = N/(K_p)^3 \quad (61)$$

where N_p = the fatigue life reduced because of pitting, and N = the Category A fatigue life calculated including section loss but neglecting the effects of pitting.

This reduction in fatigue resistance should be applied to Category A only. This assumes that the stress concentrations inherent to Categories B through E' (due to drilled holes, welds, etc.) are greater than any generated by corrosion. This is not to say that the pit-reduced Category A fatigue resistance may not govern over an immediately adjacent Category E' cover plate, but merely that the pit reduction should not be applied to the Category E' resistance but to the Category A resistance at that location. A comparison between the pitting factor and the fatigue notch factor for a particular category suggests which condition (the one of higher magnitude) governs. The reduction in resistance due to section loss can be applied to any fatigue category.

For example, a heavily corroded cover plate end weld (AASHTO Category E) can be considered. The pitting notch

Table 2. Fatigue notch factors.

Category	Factor
A	1.00
B	1.49
C	2.35
D	2.72
E	3.51
E'	4.19

factor, K_p , which is a function of pit depth, can be compared to the fatigue notch factor for Category E. The fatigue notch factors for the various AASHTO categories are comparisons to the Category A fatigue resistance curve. For reference, fatigue notch factors for Categories A through E' are given in Table 2. The tabularized notch factors are based on the assumption that the category fatigue resistance curves all have an inverse slope of three (as assumed in the AASHTO specifications), and represent a mean resistance for use in evaluation. For the example of a cover plate end weld, the magnitude of the pitting notch factor will be compared with that of the Category E fatigue notch factor (taken from Table 2). The greater of the two governs. If the pitting notch factor is greater than the Category E fatigue notch factor, a fatigue crack would be expected to propagate from a corrosion pit prior to a crack from the weld toe of the cover plate end weld. Likewise, the converse is true. The Category E notch factor is equal to 3.51. Therefore, the deepest corrosion pit would have to be $\frac{9}{16}$ in. deep or greater, resulting in a pitting notch factor of 3.51 or greater for the pitting notch factor to govern.

CHAPTER 6

EFFECTS OF UNINTENDED FIXITIES

6.1 INTRODUCTION

6.1.1 General

Freezing of moving parts of a bridge, such as expansion devices or pinned connections, can cause the bridge structure to behave differently from that anticipated in design. This chapter includes guidelines for evaluating the effects of frozen bearings and pin and hanger assemblies on the temperature and live-load-induced stresses in the bridge structure. The guidelines apply to truss bridges, tied arch bridges, and girder bridges. Simplified analysis techniques for evaluating the effects of the flexibility of the substructure are proposed.

In conventional bridge design, structures are provided with expansion devices that allow for deformations due to temperature changes. Therefore, the design engineer is not required to perform an analysis of temperature-induced stresses in bridges with various restraint conditions. The following provides background information needed in order to perform such an analysis.

6.1.2 Temperature Effects

a. *Thermal Stresses.* Thermal stresses result from heating or cooling a body that is prevented from expanding or contracting.

The thermal stresses may be found by determining the shape and dimensions the body would assume if unconstrained and then calculating the stresses produced by forcing it back to its original shape and dimensions. Thus, if a uniform straight bar is subjected to a temperature change ΔT throughout, while held at the ends, the resulting unit stress, f_T , is

$$f_T = \alpha E \Delta T \quad (62)$$

Where α is the coefficient of thermal expansion and E is the modulus of elasticity of the material. The above relation points out that the resulting stress is independent of the bar length. A temperature change of 100 F throughout a steel bar, with a coefficient of thermal expansion of 6.5×10^{-6} , held at its ends, results in a stress of 18.85 ksi, and a temperature change of 60 F throughout a concrete bar, with a coefficient of thermal expansion of 6.0×10^{-6} and a modulus of elasticity of 3,100 ksi, held at its ends, results in a stress of 1.1 ksi.

b. *Bridge Structures.* The effect of temperature changes on bridges is more complex. Changes in temperature will cause bridge movements and induce stresses. The magnitude of these movements and stresses is a function of the temperature distribution within the structure, the temperature at the time of erection, the geometry, stiffness and material properties of the superstructure, the support conditions and the lateral stiffness of the substructure. Expansion bearings, pin and hanger assemblies, and deck expansion devices are used and designed to allow for thermal movements in a bridge and, thus, reduce the thermal induced stresses. When the expansion devices are frozen, the resulting restraint of bridge movements will induce forces and movements in both the superstructure and the substructure. In some cases where the flexibility of the substructure allows for thermal movements of the superstructure, thermal effects are not expected to be as severe as in other cases where a more rigid substructure exists. Results of a large number of recent theoretical and experimental studies on temperature effects on a variety of bridges are available (7.1, 7.9, 7.11, 7.18, 7.20–7.23, 7.28, 7.29, 7.33–7.36, 7.39, 7.40, 7.42, 7.54). They include temperature effects on abutments, approach slabs and pavements (7.9, 7.21), on straight composite and noncomposite girder bridges (7.1, 7.20–7.23, 7.33–7.35, 7.42), and on skewed bridges (7.39, 7.40).

c. *Jointless Bridges.* The reliance on the flexibility of the substructure to accommodate thermal movements has led to the design of jointless bridges, which started about 25 years ago in Tennessee (7.36, 7.54) and is becoming more and more acceptable. The experience and research work in jointless bridges (7.9, 7.28, 7.29, 7.54) is a valuable source of information regarding the behavior of an existing bridge whose expansion bearings are frozen. A typical jointless bridge uses integral abutments, with a single row of H-piles, which is flexible enough to accommodate girder translations and rotations.

6.2 UNINTENDED FIXITY OF BEARINGS

6.2.1 General

In this section guidelines for investigating the effects of frozen bearings in a bridge structure are presented.

Bridge bearings are designed to transmit the weight of the superstructure and the loads it supports to the substructure. They are also designed to accommodate changes in the geometry of the superstructure which result from live load, temperature variations, and possible foundation settlements. Some bearings are designed to allow only rotation at the end supports, while others are designed to allow both rotation and longitudinal displacement. AASHTO requires that steel spans of 50 ft or greater be provided with hinged bearings at both ends for rotation purposes and with rollers, rockers, sliding plate, or elastomeric pads at one end for expansion purposes. Spans of 50 ft or less may be arranged to slide on bearings having smooth surfaces with no provision for rotation. In design, most bearing devices are assumed to be frictionless.

In reality, however, many types of expansion bearings develop increased friction resistance due to build-up of dirt or corrosion. Field observations have shown that complete freezing of expansion bearings is not uncommon. The actual field performance of some common bearing types is discussed below.

a. *Sliding Plate Type Bearings.* Dirt and corrosion often freeze the sliding surfaces of steel bearing plates. Even if not completely frozen, bearing plates may not allow longitudinal movement because of an increased friction resistance due to build-up of dirt. In most cases, the coefficient of static friction between steel bearing plates can be as high as 0.60 to 0.80. Even when self-lubricated bronze bearing plates are used, the actual coefficient of static friction can be higher than that assumed in design due to dirt and dust which can penetrate between the plates.

b. *Rolling and Rocking Type Bearings.* The rolling type bearings are very susceptible to jamming because they can accumulate dirt easily. Debris can also trap moisture and create a corrosive environment which, together with the accumulated dirt, can lead to complete freezing of the bearing. The pins used in roller or rocker assemblies are susceptible to corrosion and freezing.

6.2.2 Description of Possible Damage Types

Bearings that are no longer able to accommodate movements of the bridge can result in damage to the bearing itself, to the bridge seats, piers, abutments and approach slabs, or to the bridge superstructure. Damage to the bearing, such as bent or broken anchor bolts, shifting of bearing elements, and cracks in bearing seats, are most common. Frozen expansion bearings can result in cracking or movement of abutments, and cracking of pier columns. Abutment movements can cause distress in approach slabs or pavements such as progressive fracturing or blow-ups. Frozen expansion bearings may also result in damage to the bridge superstructure such as localized buckling or overall member buckling in a truss bridge. Seizing of rocker bearing pins can shift the reaction of the span to the edge of the bearing shoe and cause local failure. Complete failure of one or more bearings may cause instability or collapse of the structure.

6.2.3 Case Studies

Frozen rocker bearings and structural damage such as web

cracking, web buckling and separation between the bottom flange of girders and the top bearing plate, were found on the elevated approach ramps to the Poplar Street Bridge, crossing the Mississippi River in St. Louis (4.33). A description of the investigations carried out in order to determine the causes of this damage is reported in Ref. 4.36. The investigations concluded that the freezing of the rocker bearings was the primary cause of web buckling. The seized bearing pin forced the girder to bend in order to accommodate the thermal movement and rotation. The bending of the girder lifted the bottom flange off of the bearing shoe, and shifted the reaction from the bearing shoe to behind the bearing stiffeners, thus causing the end of the girder web to buckle. The seizure of the pin-supported bearing was caused by the formation of pack rust between the contact surfaces of the pin and the supporting saddles. Salt and air pollution was thought to have contributed to the corrosion of the bearings. Temperature-induced stresses caused by a malfunctioning expansion bearing on the Perley Bridge across the Ottawa River in Canada resulted in failure of connecting angles of a girder-column connection (5.8). Freezing of the connection of a wind truss to the top of a pier on the US 190 Mississippi River Bridge in Baton Rouge, Louisiana, caused cracking of the wind truss members. The connection of the wind truss to the pier was originally designed to allow for bridge expansion. Buckling of the bottom chord in an end panel of a truss bridge with frozen bearings is described in Ref. 5.9. Frozen sliding and roller bearings of the Smith Avenue High Bridge across the Mississippi River Bridge in St. Paul resulted in shifting of stresses within the bridge. Measurements made showed that some compression members were under tension and tension members under compression during thermal cycling (4.9). Various damage to pavement joints and approach pavements and slabs is described in Ref. 7.9.

6.2.4 Effects of Frozen Bearings on a Truss Bridge

An analytical investigation into the effects of frozen bearings on a simple span truss bridge has been carried out, and is reported in Appendix E, Section 4. The following observations are based on the results of this investigation.

6.2.4.1 Temperature Effects

The magnitude of the thermal stresses resulting from freezing of expansion bearings depends on the rigidity of the substructure.

a. *Rigid Piers.* In the case of rigid bridge piers, freezing of bearings results in high temperature-induced stresses in the bearing, pier, and in members of the bridge superstructure. In a truss bridge the temperature loads resulting from freezing of expansion bearings affect mainly the bottom chord members and members of the bottom lateral system. When the expansion bearings of both trusses of the bridge are frozen, the temperature loads in all the other bridge members are negligible. Freezing of the expansion bearing of only one truss results in increased temperature loads in the bottom chord of that truss and the bottom lateral system, and also induces stresses of lower magnitude in top chord members, diagonals, and members of the top lateral system.

Most of the contribution to the longitudinal stiffness of a

truss is provided by its bottom chord. Therefore, the horizontal reaction resulting from a temperature change ΔT , when the expansion bearing is frozen and the supporting piers are rigid, may be approximated by

$$P_H = \alpha E \Delta T A_{eq} \quad (63)$$

where α is the coefficient of thermal expansion, E the modulus of elasticity, and A_{eq} is the area of an equivalent axially loaded bar having the same length as the length of the truss, L . The equivalent area A_{eq} may be calculated from

$$A_{eq} = \frac{L}{\sum_{i=1}^n \frac{\ell_i}{A_i}} \quad (64)$$

where ℓ_i and A_i are the length and the cross-section area of bottom chord member i , and n is the number of bottom chord members.

b. *Flexible Piers.* The lateral stiffness of the bridge piers at the location of the bearings has a significant effect on the magnitude of the resulting temperature loads. A simplified analysis technique for estimating the effects of the lateral stiffness of bridge piers is described below. It applies to a simple truss with one end fixed and the other end connected to a flexible pier. The procedure of analysis proposed includes the following steps:

Step 1—Calculate the lateral stiffness at top of pier, K_p , as follows:

A. For a prismatic pier (see Figure 79, A) use

$$K_p = \frac{3E_p I}{h^3} \quad (65)$$

B. For a stepped pier (see Figure 79, B) use

$$K_p = \frac{3E_p}{\sum_{i=1}^s \frac{H_i^3}{I_i}} \quad (66)$$

where: $H_i^3 = h_{ib}^3 - h_{ia}^3$, s = number of steps; and I_i , h_{ia} , and h_{ib} are defined in Figure 79. E_p is the modulus of elasticity of the pier.

Step 2—Calculate the longitudinal stiffness of the truss at the location of the bearing, K_B , from

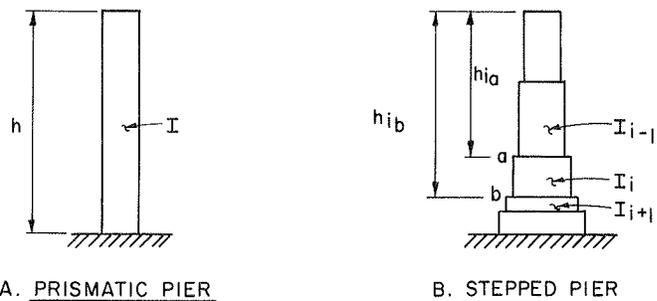


Figure 79. Bridge pier models for calculating lateral stiffness.

$$K_B = \frac{E}{\sum_{i=1}^n \frac{\ell_i}{A_i}} \quad (67)$$

where ℓ_i and A_i are the length and the cross-section area of bottom chord member i , n is the number of bottom chord members, and E is the modulus of elasticity of steel.

Step 3—Calculate the horizontal reaction, P_H , and the horizontal displacement, D , at top of pier, from:

$$P_H = \left(\frac{K_p}{\ell + \frac{K_p}{K_B}} \right) D_o \quad (68)$$

$$D = \left(\frac{1}{\ell + \frac{K_p}{K_B}} \right) D_o \quad (69)$$

where D_o is the horizontal displacement at top of pier because of a given temperature change ΔT , when the bridge is free to expand.

If both ends of the truss are connected to flexible and similar piers, the horizontal displacement of the truss D_o and its longitudinal stiffness K_B should be determined based on only half the length of the truss.

In the case of abutments, the lateral stiffness for expansion and contraction may not be the same.

c. Effects on Load Carrying Capacity. If the truss bearings are frozen in an expanded position, tension stresses are induced in the bottom chord members under normal temperature conditions. These stresses are additive to the dead load and live load stresses and, therefore, a reduction in the load-carrying capacity of the bridge may result.

Bearing frozen in a contracted position can actually reduce the live load effects on the bottom chord. However, if the resulting temperature-induced compression stress in the bottom chord is larger than the existing dead-load tension stress, buckling of the bottom chord may occur.

6.2.4.2. Live Load Effects

Live load on a simple span truss bridge causes the bottom chords to elongate as a result of the tension stresses due to bending of the trusses. If the expansion bearings are frozen, the horizontal elongation of the bottom chords is restrained and an horizontal thrust will result at the location of the bearings. This thrust will induce live load compression stresses in the bottom chord that can reduce the maximum dead load and live load tension stresses caused by bending of the truss. If the piers are rigid, a net live load compression stress may result in the bottom chord near the bearings. In most cases, however, this compression stress will not exceed the existing dead load tension stresses. The effect of the frozen expansion bearing on the stresses in the other members of the truss is negligible. If the live load capacity of the truss is governed by the capacity of the bottom chord, freezing of the expansion bearings can actually increase its load-carrying capacity. However, because the live load stresses in the other truss members are not significantly changed by freezing the expansion bearing, very little overall improvement in the live load capacity of the truss can generally be expected.

6.2.5 Effects of Frozen Bearings on a Tied Arch Bridge

An analytical investigation into the effects of frozen bearings in a tied arch bridge model has been carried out, and is reported in Appendix E, Section 5. The following guidelines are based on the results of this investigation.

6.2.5.1 Temperature Effects

When a tied arch bridge rests on rigid supports, freezing of the expansion bearings can cause high temperature stresses in its girder members. The temperature stresses induced in the other members of the tied arch are negligible. The longitudinal stiffness of the tied arch at the location of the bearing is governed by the longitudinal stiffness of the girder. Therefore, the horizontal reaction resulting from a temperature change ΔT , when the expansion bearing is frozen and the supporting piers are rigid, may be approximated by Eqs. 63 and 64, developed for a truss bridge. When Eq. 64 is used for a tied arch, ℓ_i and A_i should represent the length and the cross-section area of a prismatic segment of the girder.

The lateral stiffness of the bridge piers has a significant effect on the magnitude of the resulting temperature loads. In order to evaluate the effects of the lateral stiffness of the piers, the simplified analysis procedure described in section 6.2.2.1 may be used. The longitudinal stiffness of the tied arch, K_B , may be calculated from Eq. 67, with ℓ_i and A_i representing the length and the cross-sectional area of a prismatic segment of the girder.

If the expansion bearing of a tied arch is frozen in an expanded position, tension stresses will be induced in the girder members. These stresses are additive to the dead load and live load stresses. If the expansion bearing is frozen in a contracted position, the tension stresses in the girder members due to dead load will actually be reduced when the temperature increases.

6.2.5.2 Live Load Effects

In a tied arch bridge live load is resisted by the arch action in the rib. The girder of the tied arch prevents spreading of the ends of the rib by resisting the axial tension loads induced. When the expansion bearing is frozen and the supporting piers are rigid, the horizontal live load reactions at the ends of the rib are resisted by the piers, and the axial stress in the girder will not be affected. Therefore, if the live load capacity of the tied arch is governed by the capacity of the girder, freezing of the expansion bearing may actually increase the load-carrying capacity.

6.2.6 Effects of Frozen Bearings of a Girder Bridge

An analytical investigation into the effects of frozen bearings on a simple span girder bridge has been carried out, and is reported in Appendix E, Section 5. The following observations are based on the results of this investigation.

6.2.6.1 Temperature Effects

a. Effects of Translational Restraint. Freezing of the expansion bearing of a girder will result in an horizontal thrust at the

location of the bottom flange when the temperature changes. This thrust can affect the distribution of stresses throughout the girder. High localized thermal stresses in the areas close to the bearings and significant changes in the overall stress distribution in the girder may occur. Localized effects of translational restraint are illustrated in Appendix E, Section 6, in the form of stress contour plots and transverse stress distributions near the bearing of the girder studied. The magnitude of these stresses depends on the magnitude of the resulting horizontal reaction at the bearing. Away from the bearings, the resulting thermal stresses have a more uniform distribution along the beam. They may be regarded as a combination of axial stresses due to the horizontal thrust, assumed to be applied at the centroid of the cross section of the girder, and of bending stresses caused by the actual eccentricity of the horizontal thrust. The magnitude of the horizontal thrust caused by a given temperature change depends on the longitudinal stiffness of the girder at the location of the bearing. An approximate formulation of the longitudinal stiffness of a girder at the location of the bearing, K_B , is suggested below:

$$K_B = \frac{E A_{eq}}{L} \quad (70)$$

Equation 70 assumes that the girder may be replaced by an equivalent centrally loaded bar with a cross-sectional area A_{eq} , and a length L , equal to the length of the girder. It can be shown that the area, A_{eq} , may be calculated from

$$A_{eq} = \left(\frac{1}{A} + \frac{e^2}{I} \right)^{-1} \quad (71)$$

where A and I are the cross-sectional area and the moment of inertia of the girder, respectively, and e is the distance from the top bearing plate to the centroid of the cross section. Thus, the horizontal reaction at the location of the bearing, resulting from a temperature change ΔT , when the expansion bearing is frozen and the supporting piers are rigid, may be determined from

$$P_H = \alpha E \Delta T A_{eq} \quad (72)$$

where α is the coefficient of thermal expansion, E the modulus of elasticity, and A_{eq} is given by Eq. 71. In order to estimate the maximum localized stresses near the bearing, it is suggested that the horizontal reaction P_H be divided by the area of the bottom flange. The overall stress distribution, away from the immediate vicinity of the bearing, may be calculated from the superposition of axial stresses due to P_H , if applied at the centroid of the cross section of the girder, and bending stresses caused by the actual eccentricity of P_H .

When the elongation or contraction of a girder is restrained at the bearings, rotation due to the resulting bending stresses will occur. If the rotation at the girder ends is also restrained, additional stresses will be induced. The effects of rotational restraint are further discussed in section 6.2.6.2.

b. *Effects of Pier Flexibility.* When the supporting piers have some degree of flexibility the horizontal reaction P_H may be significantly lower. In order to evaluate the effects of the lateral stiffness of the pier, the simplified analysis procedure described in section 6.2.2.1 may be used, with the exception that the longitudinal stiffness of the girder at the location of the bearing, K_B , is to be calculated from Eqs. 70 and 71.

c. *Effects on Load-Carrying Capacity.* If the bearings of a simply supported girder are frozen in an expanded position, tension stresses and bending stresses additive to the bending stresses caused by dead load and live load will be induced under normal temperature conditions.

Bearings frozen in a contracted position will cause compression stresses and bending stresses opposite to the bending stresses caused by dead load and live load. Under these conditions the load-carrying capacity of the girder may actually increase. The load-carrying capacity of the girder will be governed by the localized compression stresses near the bearing and the magnitude of the overall compression stresses in the compression flange.

6.2.6.2 Live Load Effects

Live load on a simple span girder causes elongation of the bottom flange because of the positive moment, and rotation at the location of the bearing. If the bearing is frozen, stresses will be induced throughout the girder as a result of fixity to translation and/or rotation.

a. *Effects of Translational Restraint.* When the elongation of the bottom flange is prevented, an horizontal live load thrust will result at the location of the bearing. The effects of such an horizontal thrust on the local and overall stress distribution in the girder have been discussed in section 6.2.6.1. In order to evaluate the magnitude of the horizontal thrust caused by live load, when the expansion bearing is frozen and the supporting piers are rigid, the following procedure is suggested:

Step 1—Calculate the maximum live load horizontal displacement, D_o , at the location of the expansion bearing, when the girder is free to expand, based on the resulting rotation, θ , at the end of the girder.

$$D_o = \theta e \quad (73)$$

where e is the the distance from the top bearing plate to the centroid of the cross section.

Step 2—Calculate the longitudinal stiffness, K_B , at the location of the bearing, from Eqs. 70 and 71.

Step 3—Calculate the horizontal thrust, P_H , from

$$P_H = D_o K_B \quad (74)$$

The maximum localized stresses near the bearing may be approximated by dividing P_H by the area of the bottom flange, and the effects on the overall stress distribution, away from the immediate vicinity of the bearing, may be calculated by considering P_H as a load eccentrically applied to the cross section of the girder.

b. *Effects of Pier Flexibility.* In order to take into account the effect of the flexibility of the supporting piers on the horizontal reaction P_H , the procedure described in section 6.2.4.1 may be used. When using this procedure, K_B is to be calculated from Eqs. 70 and 72, and D_o from Eq. 73.

c. *Effects on Load-Carrying Capacity.* Freezing of the expansion bearing of a girder results in an increased bending stiffness. The horizontal live load reaction at the bearing reduces the tension stresses in the bottom flange of a simply supported girder

and may also cause some reduction in the maximum compressive stress. As a result, the load-carrying capacity of the girder may increase. It should be remembered, however, that temperature effects have to be taken into account as well. In the case of the girder studied, freezing of the expansion bearing reduced the maximum tension stress in the bottom flange from 6.51 ksi to 2.75 ksi and the maximum compression stress in the top flange from 6.51 ksi to 5.97 ksi, which represents an overall increase of about 9 percent in load-carrying capacity. The maximum localized stress near the bearing was 5.1 ksi, which is smaller than the maximum live load stress in the girder, with no horizontal restraint.

d. *Effects of Rotational Restraint.* The rotational restraint of a bearing results in a localized live load moment at the location of the bearing. As a result high localized stresses may occur. These localized effects are illustrated in Appendix E, Section 6, in the form of stress contour plots. In the case studied, the maximum live load moment caused by the rotational restraint of the bearing was 1,050 kip-in. This induced a maximum compression stress of 47.5 ksi on the interior side of the bearing, near the end of the top bearing plate and a maximum tension stress of 39.8 ksi on the exterior side of the bearing, near the end of the top bearing plate. However, the maximum moment which can occur at the bearing depends on the bending strength of the bearing itself, and the strength of its components. In most cases yielding of bolts connecting to the bottom flange or of anchor bolts will occur, which will limit the maximum moment applied to the girder. In the case studied the maximum moment which could be resisted by the bearing was about 750 kip-in. and, therefore, the maximum stresses are actually about 30 percent lower.

6.3 UNINTENDED FIXITY OF PIN AND HANGER ASSEMBLIES

6.3.1 General

Pin and hanger assemblies are used to support suspended spans in girder cantilever bridges and in cantilever truss bridges. They allow for both rotation and longitudinal movement at the span connection. Hanger plates or short eyebar links are usually used in girder cantilever bridges and built-up box members or eyebars are used in cantilever truss bridges.

Pin and hanger assemblies are critical components in a bridge structure and they are often nonredundant. The hangers are in tension and, therefore, susceptible to fatigue and fracture. Failure of pin and hanger assemblies can result in structural collapse.

The design of pin and hanger connections assumes free rotation at the pin. This assumption, however, has been found to be invalid in a very large number of cases in which a certain degree of fixity has been observed. In some cases nearly complete fixity was created by the accumulation of dirt and corrosion developed between the elements of the pin and hanger assembly. Fixity of the pin and hanger connection will cause in-plane bending stresses in the hangers and torsion forces in the pin. These loadings were not anticipated in design and they can reduce the fatigue life of the hangers or exceed the strength of the pin. In the case of girder cantilever bridges the hangers are relatively short and have a large in-plane bending stiffness. Therefore, very large bending and torsional stresses could be expected from rotational fixity. Pin and hanger assemblies are no longer a

practice of design for girder bridges. However, they are a common detail on many of the existing girder bridges. A large number of failures of pin and hanger assemblies have been reported (4.5, 4.7-4.11, 4.14, 4.16, 4.18, 4.23, 4.27, 4.29, 4.31, 5.11, 5.17).

6.3.2 Description of Possible Damage Types

The most common failure modes of a pin and hanger assembly include:

- Failure of the pin because of the increased stresses resulting from shifting of elements along the pin. This can be caused by excessive corrosion products between the hangers and web or gusset plates, or by sideways forces generated in skewed bridges. Shifting of elements along the pin may even result in the hanger coming off the pin completely.
- Failure of the pin because of cracks initiated by excessive localized wear or fretting corrosion.
- Failure of the pin because of torsion loads caused by freezing of the pin and hanger connection.
- Failure of the hanger at the net section at the pin hole. This can be due to fatigue or stress corrosion cracking.
- Failure of the hanger at the gross section, close to one of the pins, because of in-plane bending stresses caused by freezing of the pin and the hanger connection.

6.3.3 Case Studies

The majority of failures of pin and hanger assemblies occurred in girder cantilever bridges. The failure of two hanger plates supporting the ends of girders of a suspended span of the Illinois Route 157 bridge, located in St. Clair County, Illinois, is described in Ref. 4.31. The suspended span of the route 157 bridge is 60 ft long and is composed of 13 girders spaced at 5 ft 6 in. intervals. The cracking of the hanger plates occurred at the gross section near one of the pins. It was attributed to the development of frozen pin joints caused by corrosion products accumulated between the beam web reinforcement and the hanger plates. The hangers were located beneath the expansion joint and, therefore, water, salt, and other debris were directed into the pin and hanger assembly. With the joints frozen, the hanger plates were subjected to large in-plane bending stresses due to live load and temperature changes. Results of stress measurements on the hangers of the U.S. 309 bridge near Philadelphia are reported in Ref. 4.31. The U.S. 309 bridge is a girder cantilever bridge with a 100-ft suspended span composed of 6 girders. Each girder is supported by two 1 by 10 in. pin plates with a spacing between pin centers of 21 in. The measurements made indicated that, when the pin joint is frozen, cyclic stresses as high as 15 ksi were caused by live load. The measurements also indicated that the primary stresses on the pin plate cross section under live load were due to bending. In addition to the live load stresses, thermal expansion and contraction induced a daily thermal stress cycle into the pin plates. The collapse of the Mianus River Bridge on Interstate 95 in Connecticut (4.7, 4.8, 4.10, 4.11, 4.14, 4.18, 4.23) was caused by failure of a pin and hanger assembly. The Mianus River Bridge was a girder cantilever bridge with a 100-ft long suspended span. Cracking of one of the 7 in. in diameter, 7 in. long pins in the pin and hanger assembly triggered the collapse. Failure of the pin was attributed to shifting of the hanger trans-

versely on the pin, which resulted in high stresses on the edge of the pin. Two theories were advanced as to the cause of the shifting of the hanger: (1) accumulation of corrosion product from an underlying washer (4.8), and (2) a sideways force generated by the skew of the bridge (4.8). No final scientific resolution of these two theories was made. Two cracked hangers were found in the Harvard Bridge in Cambridge, Massachusetts (4.27), which is a girder cantilever bridge. The hangers, which supported the suspended span, were wrought iron eyebars that had frozen at the pins because of corrosion. The cracks occurred at the neck of the eyebars.

Relatively fewer failures of pin and hanger connections in truss bridges have been reported. Fatigue cracking of a riveted built-up hanger caused by fixity due to corrosion is reported in Ref. 5.17. Stress measurements made showed that the cracks were induced by high stresses that resulted when the corrosion restraint was overcome by joint rotation forces causing sudden release and large dynamic bending stresses.

6.3.4 Effects of Frozen Pin and Hanger Assemblies in a Cantilever Truss Bridge

An investigation of the effects of frozen pin and hanger assemblies in a cantilever truss bridge model has been carried out and is reported in Appendix E, Section 7. The following guidelines are based on the findings of this investigation.

6.3.4.1 Temperature Effects

Fixing the ends of the suspending hangers has very little effect on the longitudinal deformations of the bridge superstructure. However, relatively high bending stresses (of the order of 10 ksi, for a 100 F temperature change) may be induced in the hangers and in the truss members connecting to the hangers. The following approximate method of analysis is suggested for evaluating the magnitude of the thermal stresses induced:

Step 1—Calculate the relative horizontal displacement, D , of the ends of the hangers caused by a given temperature change, ΔT . The horizontal displacement at the lower end of the hanger, D_B , is determined by the longitudinal deformation of the suspended span and the horizontal displacement at the top of the hanger, D_T , is determined by the longitudinal deformation of the cantilever span.

$$D = D_B + D_T \quad (75)$$

Step 2—Calculate the bending stiffness of the truss members connecting to the top end of the hanger, K_T , and the bending stiffness of the truss members connecting to the lower end of the hanger, K_B , from:

$$K_T = \sum_{i=1}^n \frac{3EI_{TOP,i}}{L_{TOP,i}} \quad (76)$$

$$K_B = \sum_{i=1}^n \frac{3EI_{BTM,i}}{L_{BTM,i}} \quad (77)$$

where $I_{TOP,i}$ and $L_{TOP,i}$ are the moment of inertia and the length of truss member i connecting to the top end of the

hanger, and n is the number of members at the top joint, excluding the hanger. $I_{BTM,i}$ and $L_{BTM,i}$ apply to the joint at the lower end of the hanger.

Step 3—Determine the maximum induced moment in the hanger, M , from:

$$M = \frac{6EI}{L_H^2} \left(\frac{1}{1 + \frac{6EI}{L_H K}} \right) D \quad (78)$$

where I and L_H are the moment of inertia and the length of the hanger, respectively, and K is defined below:

$$K = \max. (K_T, K_B) \quad (79)$$

If only one end of the hanger is frozen the maximum induced moment may also be calculated from Eq. 78, with the coefficient 6 replaced by 3.

6.3.4.2 Live Load Effects

Freezing of the pin joints of a hanger has a negligible effect on the axial stresses and the horizontal displacements of the ends of the hanger caused by live load. The calculated live-load-induced bending stresses are relatively low (below 0.50 ksi). However, field measurements have shown larger live-load-induced bending stresses, caused by sudden releases of the rotational restraint at the pins (5.17). These stresses can result in fatigue cracking.

6.3.4.3 Conclusions

Freezing of the pin joints of the suspending hangers in cantilever truss bridges can cause relatively high bending stresses, which, over a period of time, may result in fatigue cracking. If detected, the pin joints should be rehabilitated to allow for proper movement.

6.3.5 Effects of Frozen Pin and Hanger Assemblies in a Girder Cantilever Bridge

An investigation of the effects of frozen pin and hanger assemblies in a girder cantilever bridge model has been carried out and is reported in Appendix E, Section 8. Other studies of the behavior of pin and hanger assemblies in girder cantilever bridges may be found in Refs. 4.31 and 5.11. Based on these sources, the following observations can be made.

6.3.5.1 Temperature Effects

Freezing of a pin and hanger assembly in a girder cantilever bridge can cause very high in-plane bending stresses in the hangers. The magnitude of these stresses can be as high as 5 ksi per 1 F temperature change. Also, very high shear stresses of the order of magnitude of 8 ksi per 1 F temperature change are induced in the pin by the torsion load resulting from hanger fixity.

6.3.5.2 Live Load Effects

The live-load-induced stresses in the hangers and pins of frozen pin and hanger assemblies are also very high. Bending stresses in hangers of about 4.0 ksi have been measured in the field. Considerably higher stresses were calculated from the analysis of a plane frame model of a girder bridge subjected to HS-20 loading, when complete hanger fixity was assumed.

6.3.5.3 Conclusions

The very high temperature and live-load-induced stresses in the hangers and the pins of pin and hanger assemblies in girder cantilever bridges indicate that corrosion-induced freezing of these devices cannot be tolerated, and repair must be done immediately.

CHAPTER 7

EFFECTS OF UNINTENDED MOVEMENTS AND PRESSURES DUE TO CORROSION

7.1 INTRODUCTION

The process of steel corrosion creates a corrosion product—rust. The volume of rust created depends on the type of the corrosion product. In most cases it will vary from two to four times the volume of steel consumed in the process (5.19). Some corrosion products may have much higher relative volumes; however, many are highly soluble in water and, if water is present, a smaller increase in volume will take place (5.19). The build-up of corrosion product in confined areas can generate pressures of up to 10,000 psi (1.2). Forces of this magnitude are obviously capable of causing movements and distortions of bridge members. This chapter presents techniques to evaluate the effects of this distortion and shifting.

7.2 EFFECTS OF DEFORMED ELEMENTS

7.2.1 Prying Action on Connectors

Build-up of corrosion product at bolted or riveted connections can lead to prying action between the connected elements. This is especially true in cases where crevice corrosion has occurred between plates connected by fasteners. In these cases edge swelling occurs, followed by swelling between fasteners. Failure of the fasteners themselves as a result of built-up corrosion product between connecting plates is rare. Once swelling between plates has occurred, some of the previous restraint of expansion is reduced.

7.2.2 Deformed Cover Plates in Tension Elements

7.2.2.1 Strength Criterion

Deformations in cover plates on tension members can change the section properties of the member and induce eccentricities and high local stresses in the plate. The section properties can change because of the offset of the deformed portions of the cover plate. The high local stresses created in the plate by the distortion process may induce localized yielding.

As was discussed earlier, the service load method does make provision for some local yielding at stress concentrations under static tension. This means that no deductions need be made from the section area because of high local stresses in distorted cover plates. Also, as discussed in section 4.1.2.2.e, tension members are self-stabilizing and the effects of the load eccentricity can be ignored. Therefore, no deductions for distortion need be made from the cover plates in statically loaded members.

7.2.2.2 Fatigue Criterion

Local damage and eccentricities should be considered when evaluating the fatigue resistance of the members involved (see section 4.1.6).

7.2.3 Deformed Cover Plates in Compression Elements

7.2.3.1 Strength and Stability Criterion

As in tension members deformed cover plates in compression elements affect the section properties and load eccentricities of the member and have high local stresses in the deformed areas of the plate. Unlike tension members, however, these effects must be considered in determining the capacity of compression elements.

Cover-plate deformations can have serious effects. The cause of failure in one truss bridge load test was failure of a compression chord at a cover plate deformed by corrosion (5.8).

a. *Service Load Method.* When using a service load approach to evaluate compression members with deformed cover plates, the deformed area of the cover plate should be deducted from the member area. This deduction allows for the reduced rigidity to compression of the bent plate relative to the axial rigidity of the other components. Obviously, deducting the deformed area of the cover plate will affect the section properties of the member and reduce its local and overall capacity. These effects may be evaluated using the guidelines given in section 4.3. In general, if the deformed area of a cover plate is limited to a short length, the effect on the buckling capacity will be negligible.

b. *Load Factor Method.* To determine the local member capacity using an ultimate strength approach, the residual plastic capacity of the member must be determined. An analysis technique for determining the strength capacity of a member with a deformed cover plate is given in Ref. 5.4.

7.3 EFFECTS OF SHIFTED ELEMENTS

7.3.1 Tension Elements

Shifting of tension elements and components of tension ele-

ments can vary the section properties and load eccentricities of these members. For a full discussion of tension members, see section 4.1 in Chapter 4.

7.3.2 Compression Elements

As in tension members, shifting of compression elements and their components can affect their section properties and induce load eccentricities. For a full discussion of compression members, see section 4.3 of Chapter 4.

7.3.3 Pin and Hanger Assemblies

Pin and hanger joints are especially susceptible to damage from movements and pressure caused by corrosion. Crevice cor-

rosion between eyebars at a pin joint, for example, can force the eyebars to spread. This changes the forces on the pin and can cause failure of retaining devices such as lomas nuts or cotter pins. If a retaining device fails, continued corrosion could push bars off the pin, leading to a catastrophic joint failure. Because of the potential for collapse, any unintended movement of a pin joint or a pin and hanger assembly requires immediate repair.

7.3.4 Bearing Elements

Bearing elements can be shifted and damaged by build-up of corrosion product. This damage could cause freezing of the bearings, as discussed in section 6.2 of Chapter 6. It could also drive the bridge off its bearings, which could cause collapse of the structure. Because of these problems, bearings that have been shifted by corrosion should be repaired immediately.

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Part III

Recommended Specification for Evaluation of Corrosion Effects in Steel Bridges

Part III contains guidelines for office evaluation of corrosion effects in existing steel bridges. It is self-contained and presented in a format suitable for consideration by the AASHTO Subcom-

mittee on Bridges and Structures. The evaluation guidelines provided are based on the findings in NCHRP Project 12-28(7).

RECOMMENDED SPECIFICATIONS FOR EVALUATION OF CORROSION EFFECTS IN STEEL BRIDGES

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FOREWORD

This appendix contains guidelines for office evaluation of corrosion effects in existing steel bridges. It is self-contained and presented in a format suitable for consideration by the AASHTO Subcommittee on Bridges and Structures. The evaluation guidelines provided are based on findings in NCHRP Project 12-28(7).

SECTION 1 INTRODUCTION

1.1 SCOPE

The guidelines presented include recommendations and simplified analysis techniques for the evaluation of common conditions created by corrosion which are not covered in the AASHTO Manual for Maintenance Inspection of Bridges. The effects of corrosion considered include effects of material loss; effects on fatigue resistance; and effects of unintended fixities, movements and pressures. The evaluation guidelines provided are based on elastic response and a service load approach.

1.2 APPLICABILITY

These guidelines are intended to complement the AASHTO Manual for Maintenance Inspection of Bridges and shall be used in conjunction with the current AASHTO Standard Specifications for Highway Bridges.

1.3 ORGANIZATION

Corrosion effects are treated independently in separate sections. In each section, effects of corrosion at the local level, the member level and the structure level are addressed, as applicable.

1.4 GENERAL APPROACH

The extent and accuracy of the office evaluation shall be related to the type, amount and accuracy of the field data available.

An initial qualitative evaluation shall be made and details or members which need to be examined in detail shall be identified. Items related to the location, nature, geometry and extent of damage and to the existing environmental conditions shall be addressed. The criticality and redundancy of the details and the members affected by corrosion shall be taken into account. Damaged members which can result in sudden structural failure shall be given immediate attention.

A quantitative evaluation of the effects of corrosion at the local, member or structure level shall be carried out using the appropriate sections of these guidelines. Residual capacity factors (RCF's), which are ratios of the capacity of deteriorated members to the capacity of the original members shall be calculated. The nominal capacity of the original member multiplied by the appropriate residual

capacity factor shall be taken to yield the remaining capacity of the member. An approximate, Level I, office evaluation shall be performed first to determine 1) whether or not there is a need for repair or for more extensive field data, or 2) if a more detailed, Level II, office evaluation is needed. A Level I office evaluation shall use an approach philosophy consistent with that used in design. In a Level II office evaluation a more accurate analysis which includes three-dimensional behavior, inelastic behavior or load redistributions in the nonlinear response range may be used to demonstrate additional strength not taken into account in the original bridge design.

SECTION 2 DEFINITIONS AND NOTATIONS

2.1 DEFINITIONS

Corrosion	= deterioration of a metal due to a reaction with its environment
Crevice Corrosion	= a form of localized corrosion occurring at locations where easy access to the external environment is prevented, such as at mating surfaces of metals or assemblies of metal and non-metal
Corrosion Fatigue	= fatigue-type cracking of metal caused by repeated stresses in a corrosive environment characterized by shorter life that would normally occur as a result of either the repeated stress alone or the corrosive environment alone
Fatigue Stress Concentration Factor	= the ratio of the fatigue strength without stress concentration to the fatigue strength with the given stress concentration
Fretting Corrosion	= a form of corrosion caused by the relative motion of two non-lubricated contacting surfaces under load
Galvanic Corrosion	= a form of corrosion caused by current flow created when two metals with different corrosive potentials are electrically connected in the presence of an electrolyte
Intermediate Member	= compression member in which failure is affected by both the tendency for buckling and compression yielding
Localized Corrosion	= corrosion attack which is limited to a certain area of a member
Long Member	= compression member in which failure occurs due to elastic instability

Pitting	= a form of localized corrosion which causes the creation of penetrations into the metal surface
Pitting Factor	= factor used to calculate the loss in fatigue resistance due to pitting
Residual Member Capacity	= the reduced nominal strength of a deteriorated member
Residual Member Capacity Factor	= the ratio of the residual member capacity to its intact capacity
Residual Capacity Factor at the Local Level	= the ratio of the residual capacity of the member at the location of the damage to the intact capacity of the member at the same location
Residual Capacity Factor at the Structure Level	= the ratio of the residual capacity of the damaged structure to its intact capacity
Rust	= the common corrosion product of iron and steel, formed by the chemical reaction of iron oxide and water
Stress Corrosion Cracking	= cracking of a metal produced by the combined action of corrosion and tensile stress
Short Member	= compression member in which failure occurs by compression yielding
Theoretical Stress Concentration Factor	= the ratio of the maximum elastic local stress to the average stress calculated based on the net section
Uniform Corrosion	= a form of corrosion characterized by a general thinning of metalwork in a uniform fashion

2.2 NOTATIONS

A	= cross-sectional area of original member (Articles 3.1.1, 3.3.1)
Ad	= remaining cross-sectional area of corroded member (Articles 3.1.1, 3.3.1)
A _{eq}	= equivalent area (Articles 5.2.2, 5.2.3, 5.2.4)
Af	= area of original flange (Article 3.6.3.1)
Afd	= remaining area of corroded flange (Article 3.6.3.1)
a	= distance center-to-center of batten plates (Article 3.4.3.1)
b	= width of element under compression (Article 3.3.1.1)
b	= length of a batten plate between rivets (Article 3.4.3.1)
b	= width of original compression flange (Article 3.6.4.2.2)
bd	= width of corroded compression flange (Article 3.6.4.2.2)
c	= distance from the neutral axis to extreme fiber in compression (Article 3.3.1.2.3)

c = width of a batten plate (Article 3.4.3.2)
 D = clear depth of girder web (Article 3.6.1.2)
 D_o = horizontal displacement of superstructure at bearing (Articles 5.2.2, 5.2.3, 5.2.4)
 dp = pit depth (Article 4.3)
 E = modulus of elasticity of steel (Articles 3.3.1, 3.6.4, 5.2.2)
 e = load eccentricity in a compression member (Article 3.3.1.2.3)
 e = distance from top of bearing plate to centroid of girder section (Article 5.2.4.2)
 F_y = specified yield point (Article 3.3.1.1)
 F_u = ultimate tensile strength (Article 3.1.5.2)
 f_a = computed axial compression stress (Article 3.6.1.7)
 I = moment of inertia of original member (Article 3.3.1.1)
 I_d = moment of inertia of corroded member (Article 3.3.1.1)
 I_{eq} = equivalent moment of inertia of a column with a variable moment of inertia (Article 3.3.1.2)
 I₁ = moment of inertia of one flange of a built-up member about its own centroidal axis normal to the plane of the web (Article 3.4.3.2)
 I₂ = moment of inertia of a pair of batten plates (Article 3.4.3.2)
 K = effective length factor in plane of buckling (Article 3.3.1.1)
 K_B = longitudinal stiffness of superstructure at bearings (Articles 5.2.2, 5.2.3, 5.2.4)
 K_f = fatigue stress concentration factor (Article 3.1.5.2)
 K_p = pitting factor (Article 4.3)
 K_s = longitudinal stiffness of substructure at bearings (Articles 5.2.2, 5.2.3, 5.2.4)
 K_t = theoretical stress concentration factor (Articles 3.1.1.2, 3.1.5.2)
 L = member unbraced length (Article 3.3.1.1)
 L = length of structure (Articles 5.2.2, 5.2.3, 5.2.4)
 l = unsupported length (Article 3.4.4, 3.6.4.2.2)
 N = category A fatigue life (Article 4.3)
 N_p = fatigue life reduced due to pitting (Article 4.3)
 P_H = horizontal reaction (Articles 5.2.2.1, 5.2.4.1, 5.2.4.2)
 P_{cr} = elastic buckling load (Article 3.3.1.2)
 P_l = force in a lacing bar (Article 3.4.2.1)
 Q_b = reduction coefficient for a built-up compression member with original batten plates (Article 3.4.3.2)
 Q_{bd} = reduction coefficient for a built-up compression member with deteriorated batten plates (Article 3.4.3.2)
 Q_c = reduction factor for the buckling load of a member with variable moment of inertia (Article 3.3.1.2)
 Q_l = reduction coefficient for a built-up compression member with original lacing bars (Article 3.4.2.2)
 Q_{ld} = reduction coefficient for a built-up compression member with deteriorated lacing bars (Article 3.4.2.2)
 r = root radius of a notch (Article 3.1.5.2)
 r = radius of gyration of original member (Article 3.3.1.1, 3.3.1.2.3)
 r_d = radius of gyration of corroded member (Article 3.3.1.1)
 RCF_l = residual capacity factor at the local level (Articles 3.1.1, 3.3.1, 3.6.1, 3.6.2)
 RCF_m = residual member capacity factor (Articles 3.1.1, 3.3.1)
 S = section modulus of original cross-section (Article 3.5.2)
 S_d = section modulus of corroded cross-section (Article 3.5.2)
 t = thickness of elements under compression (Article 3.3.1.1)
 t = thickness of batten plates (Article 3.4.3.2)

t_r = remaining thickness of a corroded cover plate (Article 3.4.4)
 t_w = web thickness of original member (Article 3.5.2)
 t_{wd} = web thickness of corroded member (Article 3.5.2)
 V = transverse shear force in a built-up compression member per plane of lacing or battens (Articles 3.4.2.1, 3.4.3.1)
 V_b = shear force in a batten plate in a built-up compression member (Article 3.4.3.1)
 α = coefficient of thermal expansion (Articles 5.2.2, 5.2.3, 5.2.4)
 γ = coefficient for calculating overall buckling stress of a built-up member (Article 3.4.2.2)
 θ = angle between a lacing bar and an axis perpendicular to the member in the plane of the lacing (Article 3.4.2.1)
 θ = rotation at the end of a girder (Article 5.2.4.2)
 ΔT = temperature change (Articles 5.2.2, 5.2.3, 5.2.4)

SECTION 3 EFFECTS OF MATERIAL LOSS

3.1 MEMBERS LOADED IN AXIAL TENSION

3.1.1 Strength

3.1.1.1 UNIFORM CORROSION

The residual capacity factor of a member with uniform corrosion losses may be expressed as follows:

$$RCF_m = \frac{A_d}{A} \quad (3-1)$$

where:

A = original cross-sectional area of member, net or gross;
 A_d = remaining cross-sectional area of corroded member, net or gross.

3.1.1.2 LOCALIZED CORROSION

Stress concentrations and eccentricities resulting from localized corrosion may be neglected when estimating static capacity of tension members with moderate deterioration. Yielding of the reduced area shall be considered as the governing limit state. Equation 3-1 shall apply when determining the residual capacity factor of the member. The original and the remaining area of the corroded member shall be calculated based on the net section.

If local yielding under working load is undesirable, diagrams of theoretical stress concentration factors, K_t, or a special analysis may be used to estimate maximum stresses. In cases of significant section loss, the possible increase in stress in members adjacent to the corroded member shall be investigated using the recommendations provided in Article 3.9.

3.1.2 Deformation

3.1.2.1 UNIFORM CORROSION

Specification, Article 10.6 shall apply when evaluating effects of increased deformations caused by uniform corrosion.

3.1.2.2 LOCALIZED CORROSION

Effects of localized corrosion on the deformation of the member under load, and on the load distribution characteristics of the structure may be neglected.

3.1.3 Stability and Slenderness

Specification, Article 10.7.5 shall apply. The reduced section properties shall be used to calculate slenderness ratios. Effects of localized corrosion on increased slenderness effects, unless severe, may be neglected.

3.1.4 Minimum Thickness of Metal

Specification, Article 10.8 shall be used as a guide for evaluating the sufficiency of the remaining metal thickness. Smaller thickness values than those specified by the Specification may be allowed, but not less than 3/16-inch.

3.1.5 Fatigue

3.1.5.1 EFFECTS OF SURFACE OR EDGE ROUGHNESS

The provisions of Section 4 shall be used for evaluating the effects of surface or edge roughness created by corrosion.

3.1.5.2 EFFECTS OF ECCENTRICITIES AND DISCONTINUITIES

Eccentricities, holes or other significant discontinuities caused by localized corrosion may be evaluated using fatigue stress concentration factors, K_t . K_t for a given notch root radius, r , may be calculated from the following equation:

$$K_t = \frac{K_t}{1 + 2 \left[\frac{K_t - 1}{K_t} \right] \left[\frac{5000}{F_u r} \right]^{\frac{1}{2}}} \quad (3-2)$$

where:

- K_t = theoretical stress concentration factor
- r = root radius of the notch, in inches
- F_u = ultimate tensile strength, in pounds per square inch

A lower bound estimate of fatigue resistance may be obtained by using theoretical stress concentration factors, K_t , and calculate elastic stresses. The increased stresses shall be evaluated using fatigue Category A.

3.2 BUILT-UP TENSION MEMBERS

3.2.1 Local Effects

The provisions of Articles 3.1.3 and 3.1.4 shall apply.

3.2.2 Overall Effects

The effect of deterioration of the connecting elements on the load sharing action among the main tension elements shall be investigated. Overall reduction in the service load capacity and fatigue resistance shall be considered. The ultimate load capacity of the member will not be affected, and evaluation at this limit state may not be required.

3.3 MEMBERS LOADED IN AXIAL COMPRESSION

3.3.1 Strength and Stability

3.3.1.1 UNIFORM CORROSION

3.3.1.1.1 Local Effects

The limiting b/t values given in Specification, Article 10.35.2, may be used as a guide to evaluate the local buckling in members with reduced thickness. If the b/t values of the corroded member exceed the values recommended by AASHTO, the AISC Specifications, Appendix C, may be used to actually evaluate the remaining local residual compressive capacity.

3.3.1.1.2 Overall Effects

Specification, Article 10.32.1 shall apply. The residual capacity factor, RCF_m , for a uniformly corroded compression member may be expressed as follows:

1. short members - $\frac{KL}{r} < 20$:

$$RCF_m = \frac{Ad}{A} \quad (3-3)$$

2. long members - $\frac{KL}{r} > \left(\frac{2\pi^2 E}{F_y}\right)^{\frac{1}{2}}$:

$$RCF_m = \left(\frac{Ad}{A}\right) \left(\frac{rd}{r}\right)^2 = \frac{Id}{I} \quad (3-4)$$

3. intermediate members - $20 \leq \frac{KL}{r} \leq \left(\frac{2\pi^2 E}{F_y}\right)^{\frac{1}{2}}$:

$$RCF_m = \left(\frac{Ad}{A}\right) \left(\frac{1 - \frac{F_y(KL)^2 / rd^2}{4\pi^2 E}}{1 - \frac{F_y(KL)^2 / r^2}{4\pi^2 E}} \right) \quad (3-5)$$

where:

K = effective length factor in plane of buckling;
 L = member unbraced length;
 I = moment of inertia of original member;
 Id = moment of inertia of corroded member;
 r = radius of gyration of original member;
 rd = radius of gyration of corroded member;
 E = modulus of elasticity of steel;
 Fy = specified yield point.

In the case of box-shape or H-shape members, Equation 3-3 may also be applied to intermediate and long members.

3.3.1.2 LOCALIZED CORROSION

3.3.1.2.1 Local Effects

Equation 3-3 may be used to determine the residual capacity factor at the local level, RCFI. For short members, RCFm may be taken as the local residual capacity factor, RCFI. For intermediate and long members, the effects of local corrosion damage on the overall stability of the member shall also be investigated.

3.3.1.2.2 Overall Effects

The effect of corrosion losses on end restraint conditions and the variation of stiffness along member length shall be investigated. The residual capacity factor of a compression member with a reduced cross-section over only a portion of its length may be expressed as follows:

1. short members:

$$RCF_m = \frac{Ad}{A} \quad (3-6)$$

2. long members:

$$RCF_m = \frac{I_{eq}}{I} = Q_c \quad (3-7)$$

3. intermediate members:

$$RCF_m = \left(\frac{Ad}{A}\right) \left(\frac{1 - \frac{F_y(KL)^2}{4\pi^2 E Q_c r^2}}{1 - \frac{F_y(KL)^2}{4\pi^2 E r^2}} \right) \quad (3-8)$$

where:

Ieq = equivalent moment of inertia of a compression member with a variable stiffness along its length;
 Qc = reduction factor equal to the ratio of the equivalent moment of inertia of the deteriorated member to the moment of inertia of the original member.

Qc and Ieq may be calculated from:

$$Q_c = \frac{P_{cr}}{\frac{\pi^2 EI}{L^2}} \quad (3-9)$$

$$I_{eq} = Q_c I \quad (3-10)$$

where:

Pcr = elastic buckling load of the deteriorated member.

The elastic buckling load of a deteriorated member may be determined from available solutions for columns with variable moment of inertia.

In the case of box-shape or H-shape members, Equation 3-6 may also be applied to intermediate and long members.

3.3.1.2.3 Eccentricity

Effects of eccentricities on short compression members may be neglected.

The eccentric ratio, ec/r , may be used to make an initial estimate of the effects of eccentricity resulting from asymmetrical deterioration. If the eccentric ratio is smaller than 0.25, the effects of eccentricity may be neglected. When the eccentric ratio is larger than 0.25, the residual capacity of the member shall be evaluated. The formulas for steel columns given in AASHTO Manual for Maintenance Inspection of Bridges, Article 5.4, or other rational analysis may be used to determine the residual capacity of the member. Effects of eccentricities due to out-of-straightness may also be evaluated by calculating the resulting moments and using the interaction formulas given in Specification, Article 10.36.

3.3.2 Deformation

Article 3.1.2 shall apply.

3.4 BUILT-UP COMPRESSION MEMBERS

3.4.1 General

Loss of lateral support resulting from deterioration of lacing bars or batten plates in built-up compression members shall be investigated. Overall reduction in the buckling strength of the member shall also be considered.

3.4.2 Deterioration of Lacing Bars

3.4.2.1 LOCAL EFFECTS

If the remaining thickness of the lacing bars is less than the values specified in Specification, Article 10.16.10, an analysis of the lacing system shall be required. The deteriorated lacing system shall be able to resist the transverse shear force, V , given by Formula 10-36 in Specification, Article 10.35.1, through-truss action. The maximum force in the lacing system, P_l , for members with single lacing, may be calculated from:

$$P_l = \frac{V}{\cos \theta} \quad (3-11)$$

and for members having double lacing:

$$P_l = \frac{V}{2 \cos \theta} \quad (3-12)$$

where:

V = transverse shear force per plane of lacing;

θ = angle between the lacing bar and an axis perpendicular to the member in the plane of the lacing.

Buckling of individual lacing bars, or of a main component with reduced lateral bracing or section loss, may be evaluated using the AASHTO column formulas. The slenderness ratio of the individual lacing bars and of the deteriorated portion of a main component between lacing bar connections shall not exceed the value given in Specification, Article 10.7, for main compression members. The remaining metal thickness shall not be less than 3/16-inch.

3.4.2.2 OVERALL EFFECTS

If the loss of section of the lacing bars is less than 25% and the resistance to local buckling is satisfactory, the effects of lacing bar losses on the overall member capacity may be neglected.

The overall residual capacity factor, RCF_m , of a built-up compression member with deteriorated lacing may be expressed in terms of reduction coefficients Q_l and Q_{ld} , as follows:

1. short members:

$$RCF_m = 1.0 \quad (3-13)$$

2. long members:

$$RCF_m = \frac{Q_{ld}}{Q_l} \quad (3-14)$$

3. intermediate members:

$$RCF_m = \frac{1 - \frac{F_y(KL)^2}{4 \pi^2 E Q_{ld}} / r^2}{1 - \frac{F_y(KL)^2}{4 \pi^2 E Q_l} / r^2} \quad (3-15)$$

For a single laced member the reduction coefficients, Q_l and Q_{ld} , may be calculated from:

$$Q_l, Q_{ld} = \left(1 + \frac{\gamma I}{A L^2 \cos^2 \theta \sin \theta} \right)^{-1} \quad (3-16)$$

where A is the undeteriorated area of the lacing bar when calculating Q_l and the deteriorated area when calculating Q_{ld} .

γ = 2.4 for single lacing and 4.8 for double lacing.

I = moment of inertia of the built-up section about an axis normal to the lacing bars.

3.4.3 Deterioration of Batten Plates

3.4.3.1 LOCAL EFFECTS

The deteriorated batten plates shall be able to resist through Vierendeel action the transverse shear force, V , given by formula 10-36 in Specification, Article 10.35.1. The maximum value of the longitudinal shear, V_b , causing bending of the battens may be calculated from:

$$V_b = V \left(\frac{a}{b} \right) \quad (3-17)$$

where:

V = transverse shear force per plane of battens;
 a = distance center-to-center of batten plates;
 b = length of a batten plate between rivets.

Local buckling of one of the components of the built-up section with reduced lateral support may be evaluated using the AASHTO column formulas.

3.4.3.2 OVERALL EFFECTS

If the loss of section of the batten plates is less than 25% and the resistance to local buckling is satisfactory, the effects of batten plate losses on the overall member capacity may be neglected.

The residual capacity factor, RCFm, of a built-up compression member with deteriorated batten plates may be expressed in terms of reduction coefficients, Q_b and Q_{bd} , as follows:

1. short members:

$$RCFm = 1.0 \quad (3-18)$$

2. long members:

$$RCFm = \frac{Q_{bd}}{Q_b} \quad (3-19)$$

3. intermediate members:

$$RCFm = \frac{1 - \frac{F_y(KL)^2}{4\pi^2EQ_{bd}}/r^2}{1 - \frac{F_y(KL)^2}{4\pi^2EQ_b}/r^2} \quad (3-20)$$

The reduction coefficients, Q_b and Q_{bd} , for a battened plated member may be calculated from:

$$Q_b, Q_{bd} = \left[1 + \frac{\pi^2 I}{L^2} \left(\frac{ab}{12I_2} + \frac{a^2}{24I_1} \right) \right]^{-1} \quad (3-21)$$

where:

I = moment of inertia of the built-up section about an axis normal to the plane of the batten plates;
 I_1 = moment of inertia of one flange about its own centroidal axis normal to the plane of the batten plates;
 I_2 = moment of inertia of a pair of batten plates:

$$I_2 = t \frac{c^3}{6} \quad (3-22)$$

where:

t = batten plate thickness;
 c = batten plate width.

Q_1 applies to original member condition and Q_{bd} applies to the deteriorated member.

3.4.4 Attached Plates

The remaining plate thickness, t_r , of stitched built-up members shall not be less than the value given by the following formula:

$$\frac{l}{t_r} = 0.52 \left(\frac{E}{F_y} \right)^{\frac{1}{2}} \quad (3-23)$$

where:

l = unsupported length.

3.5 MEMBERS LOADED IN BENDING

3.5.1 General

Provisions for the evaluation of the effects of material loss on instability failure modes, deformation and fatigue shall be as provided in Article 3.6. The effects of web holes created by corrosion shall be evaluated in accordance with Article 3.7.

3.5.2 Strength

The equations in this section shall be used if stability does not control.

3.5.2.1 UNIFORM CORROSION

The residual member capacity factors may be taken as:

- * Bending - Beams with Stocky Webs

$$RCF_m = \frac{S_d}{S} \quad (3-24)$$

where:

S = section modulus of original cross-section;
S_d = section modulus of corroded cross-section.

- * Shear

$$RCF_m = \frac{twd}{tw} \quad (3-25)$$

where:

tw = web thickness of original beam;
twd = web thickness of corroded beam.

3.5.2.2 LOCALIZED CORROSION

Effects of localized corrosion on strength, stability, deformation and fatigue shall be as specified in Articles 3.6 and 3.7.

3.6 PLATE GIRDERS

3.6.1 Uniform Web Losses

3.6.1.1 GENERAL

When evaluating the effects of web losses, the following failure modes shall be investigated:

- * web buckling due to bending
- * web buckling due to shear
- * shear yielding
- * combined bending and shear
- * web buckling due to vertical compression stresses from the compression flange
- * web crippling

3.6.1.2 WEB BUCKLING DUE TO BENDING

Web buckling due to bending may be evaluated using Specification, Article 10.34.3. If the ratio of the remaining web thickness, twd, to the clear depth of the web, D, is less than the limiting values required by AASHTO, AISC Specifications, Article 1.10.6, may be used instead. When web buckling due to bending controls:

$$RCF_l = \left(\frac{twd}{tw} \right)^3 \quad (3-26)$$

3.6.1.3 WEB BUCKLING DUE TO SHEAR

3.6.1.3.1 Girders with no Transverse Stiffeners

Web buckling due to shear in a girder with no transverse stiffeners may be evaluated using Specification, Article 10.34.4.1. If buckling due to shear controls, Equation 3-26 shall apply when determining the local residual capacity factor.

3.6.1.3.2 Interior Girder Panels

Web buckling due to shear in an interior panel of a girder with transverse stiffeners may be evaluated using Specification, Article 10.34.4.2. If buckling due to shear controls:

$$RCF_l = \left(\frac{twd}{tw} \right) \frac{0.87 + C_d [(1 - (do/D)^2)^{\frac{1}{2}} - 0.87]}{0.87 + C [1 - (do/D)^2]^{\frac{1}{2}} - 0.87} \quad (3-27)$$

Coefficients C_d and C shall apply to a corroded web and an intact web condition, respectively, as defined in Specification, Article 10.34.4.2.

3.6.1.3.3 End Girder Panels

Web buckling due to shear in an end panel of a girder with transverse stiffeners may be evaluated using Specification, Article 10.34.4.3. If buckling due to shear controls, Equation 3-26 shall apply when determining the local residual capacity factor.

3.6.1.4 SHEAR YIELDING

When shear yielding controls:

$$RCFI = \frac{twd}{tw} \quad (3-28)$$

3.6.1.5 COMBINED BENDING AND SHEAR

Specification, Article 10.34.4.4 shall apply.

3.6.1.6 WEB BUCKLING DUE TO VERTICAL COMPRESSION STRESSES FROM THE COMPRESSION FLANGE

Buckling may be evaluated using AISC Specifications, Article 1.10.2. The slenderness ratio of the corroded web shall not exceed 322 for 36,000 psi y.p. steel, or 243 for 50,000 psi y.p. steel.

3.6.1.7 WEB CRIPPLING

Where no stiffeners are provided in areas of concentrated loads web crippling may be evaluated using AISC Specifications, Article 1.10.10.

The possibility of localized buckling at the end supports of beams with no bearing stiffeners may be assessed using the following formulation:

$$t_w = \frac{b(fa)^{\frac{1}{2}}}{162.3} \quad (3-29)$$

where:

- fa = compressive stress in the web area above the bearing in ksi; it may be calculated using formula 1.10-9 in AISC Specifications, Article 1.10.10;
- b = distance in inches over which the bearing stress, fa, is assumed to be distributed.

3.6.2 Localized Web Losses

Effects of localized web losses shall be evaluated in accordance with Article 3.7.

3.6.3 Tension Flange Losses

3.6.3.1 STRENGTH

3.6.3.1.1 Uniform Corrosion

The residual capacity factor may be evaluated based on beam theory using a reduced section modulus. Equation 3-24 shall apply.

3.6.3.1.2 Localized Corrosion

Highly localized losses may be evaluated by assuming the tension flange to behave as an independent member loaded in axial tension:

$$RCFI = \frac{Afd}{Af} \quad (3-30)$$

where:

- Af = area of original flange.
- Afd = remaining area of corroded flange.

The original and the deteriorated flange area shall be calculated based on the net area.

When the tension flange losses have gradual transitions from reduced to full-section, and are uniformly distributed along the flange over a distance larger than 3.0 feet, beam theory may be used to evaluate losses below 30%. When beam theory is used, Equation 3-24 shall apply when determining the local residual capacity factor of the girder.

If the section loss is located at a point of maximum moment, the member residual capacity factor in bending, RCFm, will equal the local residual capacity factor, RCFI.

3.6.3.2 DEFORMATION

3.6.3.2.1 Uniform Corrosion

Evaluation may be based on beam theory, using a reduced moment of inertia.

3.6.3.2.2 Localized Corrosion

The effects of localized corrosion of a flange on girder deformation and shedding of load into adjacent structural members may be neglected.

3.6.3.3 FATIGUE

The tension flange may be regarded as an independent member loaded in axial tension. The provisions of Article 3.1.5 shall apply.

3.6.4 Compression Flange Losses

3.6.4.1 STRENGTH

The provisions of Article 3.6.3.1 shall apply.

3.6.4.2 STABILITY

3.6.4.2.1 General

The following instability failure modes shall be considered:

- * lateral buckling
- * vertical buckling
- * torsional buckling

3.6.4.2.2 Lateral Buckling

If lateral buckling governs, the residual capacity factor in bending for the case of uniform corrosion may be calculated as:

$$RCF_m = \frac{S_d}{S} \left(\frac{1 - 3F_y l^2 / bd^2}{\pi^2 E} \right) \quad (3-31)$$

where:

- l = length of unsupported flange between points of lateral support;
- b = width of original compression flange;
- bd = width of corroded compression flange.

3.6.4.2.3 Vertical Buckling

The provisions of Article 3.7.3 shall apply.

3.6.4.2.4 Torsional Buckling

The provisions in Article 3.3.1.1.1 shall apply.

3.7 BEAMS WITH WEB HOLES

3.7.1 General

The following possible failure modes shall be investigated:

- * increased bending and shear stresses
- * compression flange buckling
- * web buckling
- * fatigue

Stresses resulting from web holes may be divided into two:

- * local stress concentrations at the boundaries of the hole
- * change in the configuration of the overall stress field in the area affected

The stress concentrations created at the boundaries of the hole may be neglected when evaluating static strength. The provisions of Article 3.7.2.1 may be used to evaluate the effects of web holes on girder strength. The overall stresses resulting in the area of the hole may be determined using the analysis procedure described in Article 3.7.2.2. The effects of web holes on stability shall be evaluated in accordance with Articles 3.7.4 and 3.7.5, and the effects of web holes on fatigue resistance shall be evaluated in accordance with Article 3.7.6.

3.7.2 Strength

3.7.2.1 EVALUATION GUIDELINES

3.7.2.1.1 General

Web holes may be evaluated based on their size, as described in this Article. Holes located near bridge member connections shall be investigated for additional effects, such as out-of-plane movements.

3.7.2.1.2 Small Holes

Web holes may be considered small holes and may be neglected if:

1. the overall greatest internal dimension does not exceed 1/10 of the depth of the web;
2. the longitudinal distance between the boundaries of two adjacent holes is at least three times the maximum internal dimension;
3. not more than one hole exists at any cross-section;
4. the closer edge of the hole is not located within a distance equal to half the beam depth from a bearing or a concentrated load where no bearing stiffener exists.

3.7.2.1.3 Severe Holes

Web holes shall be considered severe holes and shall be repaired immediately if:

1. the overall greatest dimension exceeds 2/3 of the depth of the web;
2. the longitudinal distance between the boundaries of two adjacent holes is less than three times the maximum internal hole dimension;
3. the closer edge of the hole is located within a distance equal to half the beam depth from a bearing or a concentrated load where no bearing stiffener exists.

3.7.2.1.4 Medium Holes

All other web holes shall be considered as medium holes. Analytical investigation is needed to assess their effect. The analysis procedure in Article 3.7.2.2 may be used.

3.7.2.2 ANALYSIS PROCEDURE

The following analysis procedure may be applied to eccentric, rectangular web holes. The provisions of Article 3.7.4 may be used for holes of a different shape. The analysis procedure refers to Figure 3.7A and shall include the following steps:

Step 1: Determine the shear force V , at the location of the hole, the moment M_l at a section on the left side of the hole, and the moment M_r at a section on the right side of the hole.

Step 2: Calculate the following section properties:

A_t, A_b, I_t, I_b, y_t and y_b

- Calculate constant K from:

$$K = \left(\frac{1}{A_t} + \frac{1}{A_b} \right) \left(\frac{1}{H - y_t - y_b} \right)$$

- Calculate shear coefficients R_t and R_b . For a T-section, use:

$$R_t \text{ or } R_b = \frac{(1 + 4m)^2}{1.2 + 9.4m + 27.0 m^2 + 19.7 m^3 + 3.2n^2 m (1 + 1.1m + 0.1m^2)}$$

where:

$$m = \frac{b r_f}{h r_w} \text{ and } n = \frac{b}{h}$$

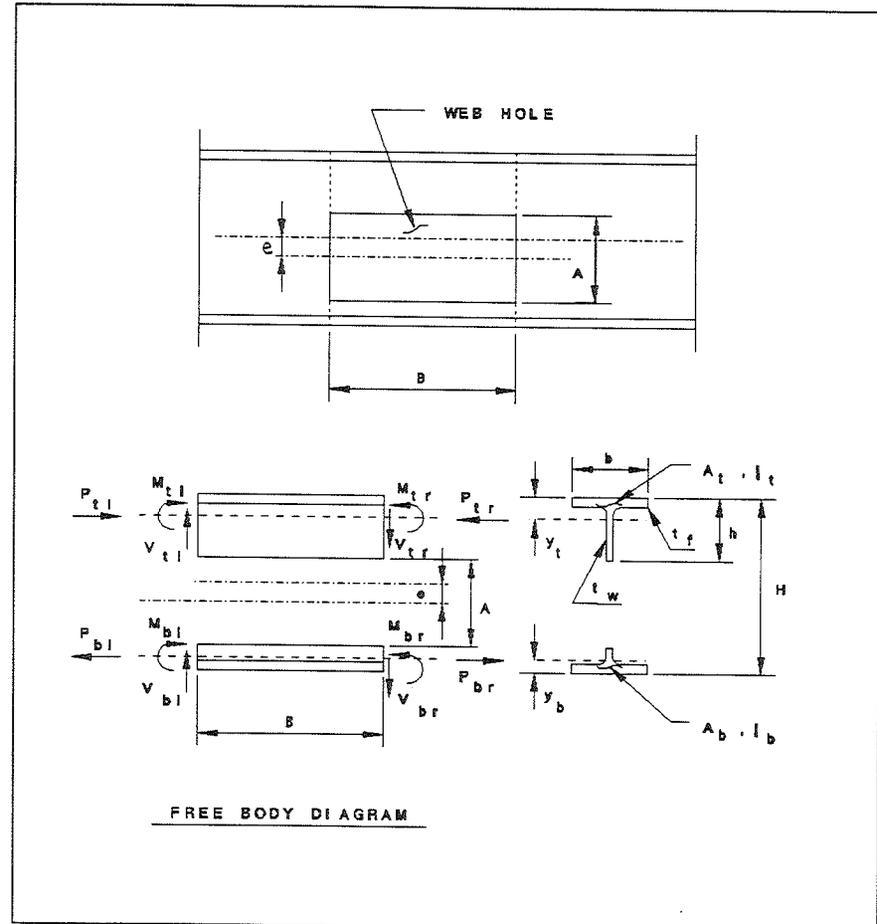


FIGURE 3.7A - Model of Analysis for Beams with Web Holes

Step 3: Find V_t ($V_t = V_u = V_v$) and V_b ($V_b = V_{bt} = V_{br}$) from the following equations:

$$V_t = V_b = V$$

$$\frac{V_t}{V_b} = \frac{\frac{B^2}{6EI_b} + \frac{2}{GA_b R_b}}{\frac{B^2}{6EI_t} + \frac{2}{GA_t R_t}}$$

where $E = 29,000$ ksi; $G = 11,500$ ksi; (G being the shear modulus).

Step 4: Find P ($P = P_u = P_v = P_{bt} = P_{br}$) from:

$$P = \frac{M_1 + \frac{B}{2} V}{K(I_t + I_b) + (H - y_t - y_b)}$$

Step 5: Find M_{ut} and M_{bt} from:

$$M_{ut} = I_t K P - \frac{B}{2} V_t$$

$$M_{bt} = I_b K P - \frac{B}{2} V_b$$

Step 6: Find M_{vt} and M_{br} from:

$$M_{vt} = M_{ut} + B V_t$$

$$M_{br} = M_{bt} + B V_b$$

Step 7: Calculate normal and shear stresses at the corners of the hole and at the flange level above and below the corners of the hole using basic equations of mechanics. Critical stresses can occur at each of these locations.

3.7.2.3 APPLICABILITY TO HOLES OTHER THAN RECTANGULAR

Provisions of Article 3.7.2.2 may be applied to holes of a different shape other than rectangular, as follows:

1. Circular holes may be replaced by a rectangular hole whose height is 0.9 the diameter of the circular hole and whose total length is 0.45 the diameter of the circular hole.

2. Elliptical holes with their major axis in the longitudinal direction of the beam may be replaced by a rectangular hole having a depth equal to the maximum depth of the actual hole and a length determined such that the assumed and the actual holes have the same area.
3. Holes of irregular shape may be replaced by a rectangular, circular or elliptical shape which best approximates the actual shape of the hole.

3.7.3 Compression Flange Buckling

3.7.3.1 VERTICAL BUCKLING

In the case of holes near the compression flange, vertical buckling of the T-section formed by the hole shall be investigated. The T-shaped flange over the hole may be treated as an isolated column with constant compression along its length. The AASHTO column formulas may be used to evaluate its susceptibility to elastic or inelastic buckling. The effective member length shall be based on the length of the hole and an effective length factor $K = 0.65$.

3.7.3.2 LATERAL BUCKLING

The effect of web holes on the lateral stability of the compression flange may be neglected when evaluating according to current AASHTO Specifications.

3.7.4 Web Buckling

3.7.4.1 LOCALIZED BUCKLING AT THE BOUNDARY OF A HOLE

The maximum width-thickness ratios of the unsupported web hole boundaries under compression shall be investigated using Specification, Article 10.35.2. If the width-thickness ratio of the unsupported portion of the web exceeds the limits recommended by AASHTO, the AISC Specifications, Appendix C, may be used.

3.7.4.2 WEB CRIPPLING

When bearing stiffeners exist, the effect of the hole shall be taken into account by deducting the missing web area. When bearing stiffeners are not present, web holes located within a distance equal to half the beam depth from the bearing shall be repaired.

3.7.4.3 OVERALL WEB BUCKLING

Evaluation of panels with web holes shall be based on classical shear buckling, not tension field action.

3.7.5 Deformation

The effects of small and medium web holes on the deformation of the beam and the shedding of load into adjacent structural members may be neglected.

3.7.6 Fatigue

The fatigue life at the corner of the hole may be related to the fatigue life of an axially loaded member. The provisions of Article 3.1.5 shall apply.

3.8 FLOOR SYSTEMS

3.8.1 For a Level I office evaluation, the provisions of Articles 3.5, 3.6 and 3.7 shall apply.

3.8.2 For a Level II office evaluation, the following structural actions may be included where appropriate:

- * participation of the concrete deck in noncomposite designs, for loads up to the elastic limit
- * dispersion of wheel loads on floorbeams
- * more accurate calculation of the wheel load distribution on stringers
- * partial stringer continuity at floorbeam connections.

If the nonlinear response range is included, some reserves in strength due to load redistribution may be identified. However, nonlinear load redistribution shall only be allowed on a temporary basis, until appropriate repairs are made.

3.9 TRUSS BRIDGES

3.9.1 For a Level I office evaluation, the provisions of Articles 3.1, 3.2, 3.3 and 3.4 shall apply. Effects of load redistribution on the corroded member shall be neglected, and effects of load redistribution on members adjacent to the corroded member shall be investigated if the section loss of a truss member exceeds 20% over most of its length. Plane frame or three-dimensional models may be needed to evaluate load redistribution effects.

When evaluating effects of corrosion at the structure level, factors such as member criticality and redundancy shall also be considered.

3.9.2 For a Level II office evaluation, the following structural actions may be included where appropriate:

- * nonlinear load redistribution due to frame and three-dimensional action
- * participation of the floor system

These actions may be used to demonstrate additional reserves in strength in top and bottom chord members and diagonals, but not in end posts and hangers. The increase in load in members adjacent to the deteriorated member, and the potential fatigue problems associated with the larger distortions shall be investigated. Nonlinear load redistribution shall only be allowed on a temporary basis, until appropriate repairs are made.

3.10 GIRDER BRIDGES

3.10.1 For a Level I office evaluation, the provisions of Articles 3.5, 3.6 and 3.7 shall apply. Load redistribution effects may be neglected.

When evaluating effects of corrosion at the structure level, redundancy shall also be considered.

3.10.2 For a Level II office evaluation, the following structural actions may be included where appropriate:

- * participation of the concrete deck in noncomposite designs, for loads up to the elastic limit
- * more accurate calculation of the lateral load distribution on girders
- * nonlinear load redistribution due to girder continuity over supports and three-dimensional action.

When the contribution of the bracing system in load redistribution is taken into account, the adverse effects of increased transverse distortions and high localized stresses at the connection of the bracing members with the girders, shall be investigated. Nonlinear load redistribution shall only be allowed on a temporary basis, until appropriate repairs are made.

SECTION 4 EFFECTS OF CORROSION ON FATIGUE RESISTANCE

4.1 GENERAL

When evaluating effects of surface roughness created by corrosion, a distinction shall be made between active corrosion areas and corroded areas which have been blast-cleaned and repainted to stop the corrosion process.

4.2 ACTIVE CORROSION

Rough edges or surfaces in areas susceptible to fatigue cracking shall be repaired so that either the progression of corrosion or the development of fatigue cracks be prevented.

4.3 RETROFITTED CORRODED AREAS

The reduction in fatigue resistance of a corroded member shall be separated into two parts, one due to loss in cross-sectional area and the other due to stress concentration generation at the pits. The loss in fatigue resistance due to section loss may be determined by taking into account the resulting increase in the average stress. If eccentricities are also created, the provisions of Article 3.1.5.2 shall apply. The loss in fatigue resistance due to the stress concentrations at the pits shall be determined using a pitting factor, K_p , defined as:

$$Kp = 1.2 + 4.0 dp \quad (4-1)$$

where:

dp = pit depth in inches

The pitting factor, Kp, shall be used to reduce calculated Category A fatigue lives as follows:

$$Np = \frac{N}{(Kp)^3} \quad (4-2)$$

where:

Np = fatigue life reduced due to pitting;

N = Category A fatigue life calculated including section loss, but neglecting the effects of pitting.

SECTION 5 EFFECTS OF UNINTENDED FIXITIES

5.1 GENERAL

This section includes provisions for determining temperature and life load stresses caused by frozen bearings and frozen pin and hanger assemblies in truss, tied-arch and girder bridges.

5.2 UNINTENDED FIXITY OF BEARINGS

5.2.1 Introduction

The following possible damage types shall be considered:

- * bearings - bend or broken anchor bolts, cracks in bearing seats or shifting of bearing elements.
- * abutments - movement or cracking
- * piers - cracking
- * superstructure - localized yielding or buckling, or overall member buckling
- * approach slabs - progressive fracturing or blow-ups.

The possibility of span instability due to damages to one or more bearings shall be investigated.

5.2.2 Truss Bridges

5.2.2.1 TEMPERATURE EFFECTS

5.2.2.1.1 Rigid Piers

When the expansion bearings of both trusses of a simple-span truss bridge are frozen, the magnitude of the temperature-induced horizontal reaction at the location of the bearing, P_H , may be determined from:

$$P_H = \alpha E \Delta T A_{eq} \quad (5-1)$$

where:

- α = coefficient of thermal expansion;
- ΔT = temperature change;
- A_{eq} = area of an equivalent axially loaded bar having the same length as the length of the truss, L.

The equivalent area, A_{eq} , shall be calculated from:

$$A_{eq} = \frac{L}{\sum_{i=1}^n \frac{l_i}{A_i}} \quad (5-2)$$

where:

- L = length of truss;
- l_i = length of bottom chord member i;
- A_i = cross-sectional area of bottom chord member i;
- n = number of bottom chord members.

Temperature-induced stresses in bottom chord members shall be computed from the axial load P_H . Temperature-induced stresses in all other truss members may be neglected. When the expansion bearing of only one truss is frozen, the thermal stresses induced in bottom chord members and bottom laterals shall be determined from analysis based on a three-dimensional truss model, or a plane truss model which includes the bottom chords and the bottom lateral system. Temperature-induced stresses in top chord members, diagonals and hangers may be neglected.

5.2.2.1.2 Flexible Piers

The following analysis procedure may be used to estimate the effects of pier lateral stiffness on the temperature loads induced in a simple-truss bridge with one end fixed and the other end connected to a flexible pier:

Step 1: Calculate the lateral stiffness at top of pier, K_p , as follows:

A. For a prismatic pier (see Figure 5.2A, A) use

$$K_p = \frac{3E_p I}{h^3} \quad (5-3)$$

B. For a stepped pier (see Figure 5.2A, B) use

$$K_p = \frac{3E_p}{\sum_{i=1}^s \frac{H_i^3}{I_i}}$$

where:

$$H_i^3 = h_b^3 - h_a^3$$

s = number of steps and I_i , h_a and h_b are defined in Figure 5.2A. E_p is the modulus of elasticity of the pier.

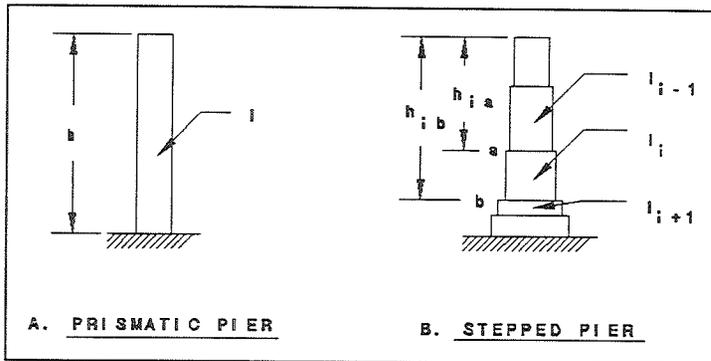


FIGURE 5.2A - Bridge Pier Models for Calculating Lateral Stiffness

Step 2: Calculate the longitudinal stiffness of the truss at the location of the bearing, K_B , from

$$K_B = \frac{E}{\sum_{i=1}^n \frac{l_i}{A_i}} \quad (5-5)$$

where l_i and A_i are the length and the cross-section area of bottom chord member i , n is the number of bottom chord members and E is the modulus of elasticity of steel.

Step 3: Calculate the horizontal reaction, P_H , and the horizontal displacement, D , at top of pier, from:

$$P_H = \left(\frac{K_p}{1 + \frac{K_p}{K_B}} \right) D_o \quad (5-6)$$

$$D = \left(\frac{1}{1 + \frac{K_p}{K_B}} \right) D_o \quad (5-7)$$

where D_o is the horizontal displacement at top of pier due to a given temperature change ΔT , when the bridge is free to expand.

If both ends of the truss are connected to flexible and similar piers, the horizontal displacement of the truss D_o and its longitudinal stiffness K_B should be determined based on only half the length of the truss.

5.2.2.2 LIVE LOAD EFFECTS

The live load-induced compression stresses in the bottom chord of a simple-span truss bridge with frozen bearings and symmetrical support conditions may be determined from a plane truss model. The effects of frozen bearings on the live load stresses in all other truss members may be neglected. The load carrying capacity of a truss may actually increase if it is governed by the capacity of the bottom chord.

5.2.3 Tied Arch Bridges

5.2.3.1 TEMPERATURE EFFECTS

5.2.3.1.1 Rigid Piers

When the expansion bearings of both arches of a tied-arch bridge are frozen and the substructure is rigid, Equations 5-1 and 5-2 may be used to calculate the magnitude of the

temperature-induced horizontal reaction at the location of the bearing. The length l_1 and the cross-sectional area A_1 shall represent the length and the cross-sectional area of a prismatic segment of the girder. Equation 5-1 may be used to calculate temperature-induced stresses along the girder of the tied arch. The temperature-induced stresses in all other bridge members may be neglected.

5.2.3.1.2 Flexible Piers

The analysis procedure in Article 5.2.2.1.2 may be used to determine the effects of pier lateral stiffness on the temperature loads induced in a tied-arch bridge. The length l_1 and the cross-sectional area A_1 shall represent the length and the cross-sectional area of a prismatic segment of the girder.

5.2.3.2 LIVE LOAD EFFECTS

The effects of the live load-induced stresses in members of the superstructure of a tied-arch bridge may be neglected.

5.2.4 Girder Bridges

5.2.4.1 TEMPERATURE EFFECTS

5.2.4.1.1 Effects of Translational Restraint

When the expansion bearings are frozen and the supporting piers are rigid, the magnitude of the horizontal reaction at the location of the bearing, P_H , may be determined from:

$$P_H = \alpha E \Delta T A_{eq} \quad (5-8)$$

where:

A_{eq} = area of an equivalent axially loaded bar having the same length as the length of the girder, L .

The equivalent area, A_{eq} , shall be calculated from:

$$A_{eq} = \left(\frac{1}{A} + \frac{e^2}{I} \right)^{-1} \quad (5-9)$$

where:

A = cross-sectional area of the girder;

I = moment of inertia of the girder;

e = distance from the top bearing plate to the centroid of the girder cross-section.

The magnitude of the localized stresses in the girder near the bearing may be estimated by dividing the horizontal reaction at the bearing, P_H , by the area of the bottom flange. The overall

stress distribution, away from the immediate vicinity of the bearing, may be calculated from the superposition of axial stresses due to P_H , if applied at the centroid of the cross-section of the girder, and bending stresses caused by the actual eccentricity of P_H .

5.2.4.1.2 Effects of Rotational Restraint

The provisions of Article 5.2.4.2.3 shall apply.

5.2.4.1.3 Effects of Pier Flexibility

The analysis procedure in Article 5.2.2.1.2 may be used to assess the effects of the lateral stiffness of the piers on the magnitude of the horizontal reaction, P_H . The longitudinal stiffness of the superstructure, K_B , shall be calculated from:

$$K_B = \frac{EA_{eq}}{L} \quad (5-10)$$

where:

L = length of girder;

Equation 5-9 shall be used to calculate the equivalent area, A_{eq} .

5.2.4.2 LIVE LOAD EFFECTS

5.2.4.2.1 Effects of Translational Restraint

When the expansion bearings are frozen and the supporting piers are rigid, the following analysis procedure may be used to calculate the magnitude of the live load-induced horizontal thrust at the location of the bearing, P_H :

Step 1: Calculate the maximum live load horizontal displacement, D_o , at the location of the expansion bearing, when the girder is free to expand, based on the resulting rotation at the end of the girder:

$$D_o = \theta e \quad (5-11)$$

where:

e = distance from the top bearing plate to the centroid of the girder section

θ = rotation of the end of the girder due to live load

Step 2: Calculate the longitudinal girder stiffness, K_B , at the location of the bearing, from Formula 5-10.

Step 3: Calculate the horizontal thrust, P_H , from:

$$P_H = D_o K_B \quad (5-12)$$

The maximum localized stresses near the bearing may be approximated by dividing P_H by the area of the bottom flange, and the effects on the overall stress distribution, away from the immediate vicinity of the bearing, may be calculated by considering P_H as a load eccentrically applied to the cross-section of the girder.

5.2.4.2.2 Effects of Pier Flexibility

The analysis procedure in Article 5.2.2.1.2 may be used to evaluate the effects of the lateral stiffness of the piers on the magnitude of the horizontal thrust, P_H . The longitudinal stiffness of the superstructure, K_B , shall be calculated from Equation 5-10 and the horizontal displacement, D_o , at the location of the bearing when the girder is free to expand, shall be calculated from Equation 5-11.

5.2.4.2.3 Effects of Rotational Restraint

The maximum moment resulting from bearing rotational fixity shall be limited by the strength of the bearing components and the strength and stiffness of the connections of the bearing to the pier and to the bottom flange of the girder. Finite element analysis shall be used to calculate maximum localized stresses in the girder where needed.

5.3 UNINTENDED FIXITY OF PIN AND HANGER ASSEMBLIES

5.3.1 Introduction

The following failure modes shall be investigated:

- * Pin failure due to increased stresses resulting from shifting of elements along the pin. The possibility of the hanger coming off the pin completely due to shifting of elements shall be considered.
- * Pin failure due to cracks initiated by excessive localized wear or fretting corrosion.
- * Pin failure due to torsion loads caused by freezing of the pin and hanger connection.
- * Hanger failure at the net section at the pin hole due to fatigue or stress corrosion cracking.
- * Hanger failure at the gross section, close to one of the pins, due to in-plane bending stresses caused by freezing of the pin and hanger connection.

5.3.2 Cantilever Truss Bridges

5.3.2.1 TEMPERATURE EFFECTS

The following analysis procedure may be used to determine the magnitude of the bending stresses induced in the suspending hangers and the truss members connecting to the hangers:

Step 1: Calculate the relative horizontal displacement, D , of the ends of the hangers caused by a given temperature change, ΔT . The horizontal displacement at the lower end of the hanger, D_B , is determined by the longitudinal deformation of the suspended span, and the horizontal displacement at the top of the hanger, D_T , is determined by the longitudinal deformation of the cantilever span.

$$D = D_B + D_T \quad (5-13)$$

Step 2: Calculate the bending stiffness of the truss members connecting to the top end of the hanger, K_T , and the bending stiffness of the truss members connecting to the lower end of the hanger, K_B , from:

$$K_T = \sum_{i=1}^n \frac{3EI_{top,i}}{L_{top,i}} \quad (5-14)$$

$$K_B = \sum_{i=1}^n \frac{3EI_{botm,i}}{L_{botm,i}} \quad (5-15)$$

where $I_{top,i}$ and $L_{top,i}$ are the moment of inertia and the length of truss member i connecting to the top end of the hanger, and n is the number of members at the top joint, excluding the hanger. $I_{botm,i}$ and $L_{botm,i}$ apply to the joint at the lower end of the hanger.

Step 3: Determine the maximum-induced moment in the hanger, M , from:

$$M = \frac{6EI}{L_H^2} \left(\frac{1}{1 + \frac{6EI}{L_H K}} \right) D \quad (5-16)$$

where I and L_H are the moment of inertia and the length of the hanger, respectively, and K is defined below:

$$K = \max. (K_T, K_B)$$

If only one end of the hanger is frozen, the maximum-induced moment may also be calculated from Equation 5-16, with the coefficient 6 replaced by 3.

If only one end of the hanger is frozen, the maximum-induced moment may also be calculated from Equation 5-16, with the coefficient 6 replaced by 3.

5.3.2.2 LIVE LOAD EFFECTS

The effects of hanger fixity on the live load-induced axial loads in the hangers and the live load bending stresses induced in truss members other than the hangers may be neglected. The possibility of sudden releases of the rotational restraint at the pins, which can result in increased stresses shall be considered when determining the magnitude of the live load-induced bending stresses in the suspending hangers. Field measurements shall be made where needed.

5.3.2.3 Frozen pin joints in suspending hangers shall be rehabilitated to allow for proper movement, so that the possible initiation of fatigue cracks at the ends of the hangers caused by temperature and increased live load stresses be prevented.

5.3.3 Girder Cantilever Bridges

5.3.3.1 The temperature-induced in-plane bending stresses in the hangers and the shear stresses induced in the pin may be determined by assuming a relative horizontal displacement of the ends of the hangers equal to the elongation or contraction of a portion of the suspended span and the cantilever span, as applicable.

5.3.3.2 The live load-induced bending stresses in the hangers and the live load-induced shear stresses in the pin shall be determined based on field measurements where needed.

5.3.3.3 Frozen pin and hanger assemblies in girder cantilever bridges shall be promptly repaired.

SECTION 6 EFFECTS OF UNINTENDED MOVEMENTS AND PRESSURES

6.1 GENERAL

This section includes provisions for evaluating effects of distortions and shifting of elements caused by corrosion products.

6.2 EFFECTS OF DEFORMED ELEMENTS

6.2.1 Deformed Cover Plates in Tension Members

The effect of localized stresses and eccentricities caused by distortions of cover plates in tension members may be neglected when evaluating static strength. The provisions of Article 3.1.5 may be used to assess the effects of deformed cover plates on fatigue resistance.

6.2.2 Deformed Covered Plates in Compression Members

The residual capacity of a compression member with a deformed cover plate may be evaluated by deducting the deformed area of the cover plate from the cross-section of the member and using the provisions of Article 3.3.1.2.

6.3 EFFECTS OF SHIFTED ELEMENTS

6.3.1 Tension Members

The effects of shifting of elements in a tension member may be neglected when evaluating member strength. The effects of shifted elements on the pin in pinned end connections shall be investigated.

6.3.2 Compression Members

The effects of shifting of compression members and their components shall be evaluated using the provisions of Article 3.3.1.2.3.

6.3.3 Pin and Hanger Assemblies

The provisions of Article 5.3 shall apply.

6.3.4 Bearing Elements

If shifting of bearing elements results in freezing of the bearing, the provisions in Article 5.2 shall apply. The stability of spans on bearings with shifted and damaged elements shall be investigated.

Part IV

Research Report

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

1.1 PROBLEM STATEMENT

Corrosion can have many detrimental effects on bridges. The most common effect of corrosion on the ability of a bridge to carry load is the general weakening of structural members through loss of metal. Very localized corrosion, such as pitting, can create stress concentrations elevating the local stress several times greater than the global stress field, which may increase the possibility of crack initiation. The accumulation of corrosion products can result in the binding of pinned joints and bearings and the distortion of bridge members and joints.

While the damage resulting from corrosion can have catastrophic effects on the performance of individual steel bridges, the monetary costs of bridge rehabilitation and replacement because of unabated corrosion on the entire transportation network are staggering and will become even more so in the future. At the present time, the bridge inspector and evaluator have no well-established means to evaluate the effects of corrosion. In some cases, the devastating effects go unheeded with tragic results, while in others, bridges are unnecessarily closed or replaced because their true load-carrying capabilities cannot be determined.

1.2 RESEARCH OBJECTIVES AND PLAN

The purpose of this study is to develop guidelines for evaluating the effects of corrosion on existing steel bridges. The study was limited to the corrosion of superstructure metalwork, steel grid bridge decks, and cables of suspension and cable-stayed bridges. The corrosion of reinforcing steel and weathering steel (e.g., ASTM A588) was not considered in this investigation. The evaluation or rating of a bridge naturally depends on interpretation of field inspection reports, and, as such, this study is concerned with guidelines for inspection and evaluation, since they must be coupled to achieve an accurate evaluation.

To achieve the objective of this investigation, six tasks were established in the research plan for this project. Each task is discussed below.

1.2.1 Task 1—Accumulation of Current Information

The objective of Task 1 was to establish present practice with regard to bridge inspection and evaluation, and to determine what performance data and current state-of-the-art information exists.

Toward this end, a literature search was made to determine

the effects of corrosion on bridges and related structures. This literature search sought to identify corrosion factors having effects on bridges such as: air pollution, acid rain, acid fog, condensation, marine environments, bird droppings, debris collection, and deicing salts. Advanced inspection and bridge analysis methods were researched through the literature as well. The search utilized the National Association of Corrosion Engineers abstracts, National Technical Information Service, and other data base sources.

Technical questionnaires, soliciting subjective narratives on corrosion effects, were sent to the Departments of Transportation of all 50 states, various bridge authorities, mass transit authorities, railroads, consulting and research engineers, and any other potentially helpful respondents. The questionnaire included: (1) identification and performance history of details sensitive to nonuniform corrosion; (2) identification and performance history of retrofits to enhance the life of corrosion-damaged details; and (3) design, inspection (visual and nondestructive evaluation (NDE)), and maintenance practices relating to non-uniform corrosion.

The in-house knowledge and unpublished reports, which represent Modjeski and Masters' historical experience in inspecting and evaluating bridges, complemented the information gathered through questionnaires, subsequent interviews, and the literature search.

1.2.2 Task 2—Analysis of Current Information and Outline of a Framework for the Development of Guidelines

Information obtained in Task 1 was evaluated for the purpose of identifying those conditions which result in greater corrosion than others and bridge details more susceptible to corrosion than others.

A framework for the development of procedures to evaluate corrosion effects on the carrying capacity of steel bridges was prepared. The procedures include sections on inspection and evaluation.

1.2.3 Task 3—Interim Report

An interim report was prepared to present the findings of Tasks 1 and 2, a research plan for Task 4, future research, and an outline of the proposed guidelines. The interim report contained a discussion of the application of the proposed guidelines for inspection and evaluation of corrosion effects on steel highway bridges.

1.2.4 Task 4—Laboratory Tests, Field Investigations, and Analytical Studies

The information gathered in Task 1 was not sufficient to prepare the comprehensive guidelines that are the ultimate objective of the study. Laboratory tests, field investigations, and analytical studies were conducted to complement the existing information.

1.2.4.1 Nondestructive Test Techniques

The literature search phase of this contract identified the present uses of nondestructive test evaluation (NDE) techniques in bridge inspection. NDE techniques presently used in bridge inspection include ultrasonic, magnetic particle, and dye penetrant methods. Radiographic inspection has been used, but rarely, and not for corrosion inspection. Of these techniques, only ultrasonic is used extensively in corrosion evaluation to measure material thickness. Additional use of NDE techniques is envisioned to facilitate gathering information for detailed analysis or to obtain information not presently obtainable. Those NDE techniques specifically identified in Task 4 as having potential applicability to bridge inspection are:

1. Ultrasonic inspection using computerized mapping techniques for the purpose of defining surface roughness and localized section loss characteristics. The techniques identified were those that can be used with easily transportable and commercially available data acquisition units (data loggers). The data would then be transferred to a computer in order to put it into a form for ready analysis. Field portability was considered desirable.
2. Ultrasonic inspection for evaluating areas not visually accessible, such as hanger and bearing pins using data acquisition techniques similar to the above.
3. Measuring devices that can be used to evaluate bearing fixity. Criteria used in establishing whether a method could be useful were: (a) ease of use in the field (the equipment must be portable and easily carried by the inspector); (b) ability to use under less than ideal conditions and without extensive surface preparation; (c) ability to perform the intended function; (d) ability of the output data to lend itself to analytical methods with a minimum of manual preparation; and (e) reasonable cost.

The approach taken was to search the technical literature for devices that would meet the criteria. Equipment manufacturers were contacted to ascertain the utility and availability of the device. Testing the equipment in the field under typical inspection conditions was considered necessary to demonstrate the feasibility of the method.

1.2.4.2 Analytical Studies

Analytical studies were conducted to provide information for use in developing guidelines for evaluating the effects of corrosion on the structural behavior of steel bridges. Unanticipated redistributions of loads were explored and simple rules of thumb for analyzing corrosion effects were developed.

The analytical studies include effects of material loss in truss bridges (see Figure 80), effects of flange and web losses in beams

(see Figures 81 and 82), effects of frozen bearings (see Figure 83), and effects of frozen pin and hanger assemblies.

Analysis models that represent bridges and bridge details found to be most susceptible to corrosion problems were created in order to facilitate the analytical investigations. They include:

1. A finite element model of a built-up riveted floorbeam with a refined mesh in its end panel, for studying web losses (see Figure 84).
2. A finite element model of a built-up riveted floorbeam with a refined mesh in its center panel for studying flange losses (see Figure 85).
3. A finite element model of a continuous welded girder with a refined mesh near a bearing, for studying effects of frozen bearings (see Figure 86).
4. A two-dimensional model of a simple truss, for studying effects of material loss and frozen bearings (see Figure 87).
5. A three-dimensional model of a simple truss bridge, for studying load redistributions due to material loss and frozen bearings (see Figure 88).
6. A two-dimensional model of a tied arch, for studying effects of frozen bearings (see Figure 89).
7. A two-dimensional model of a cantilever truss bridge, for studying effects of frozen pin-hanger assemblies (see Figure 90).
8. A two-dimensional model of a girder cantilever bridge, for studying effects of frozen pin-hanger assemblies (see Figure 91).
9. A three-dimensional finite element model of a four-girder composite bridge, for studying load redistributions due to material loss and frozen bearings.

The results of the analytical studies conducted are reported in Appendix E.

1.2.5 Task 5—Development of Guidelines

The information and data gathered in Tasks 1, 2, and 4 were used to prepare detailed guidelines for the inspection and evaluation of corrosion effects on steel highway bridges. The outline presented as a part of Task 3 was expanded and modified based on the findings of Task 4. The field inspection guidelines and the guidelines for office evaluation developed are presented in Appendixes C, D, and G, respectively. (Note: Appendixes C, D, and G of the original report as submitted by the research agency have been published herein as Parts I, II, and III.)

1.2.6 Task 6—Final Report

A final report, providing the details of the research project and the guidelines developed in Task 5, was prepared.

It includes the research findings and proposed guidelines. Further research needed for improving the ability to evaluate corrosion effects on steel bridges is detailed.

FINDINGS, INTERPRETATION, AND APPRAISAL

2.1 LITERATURE SEARCH

A review of the recent literature was performed to determine the current methods of evaluating corrosion of steel highway bridges. The search included the use of the National Technical Information Service (NTIS) and the Transportation Research Information Service (TRIS) data bases, the Applied Science and Technology Index, the Engineering Index, and various in-house documents. In addition, the Pennsylvania State Library LUIS (Library User Information Service) search, a computerized catalog, was used.

2.1.1 Corrosion and Bridge Inspection

The information gathered indicated the lack of a standardized method for evaluating the corrosion condition of steel highway bridges. The current inspection recording system does not include an analysis of the corrosion condition of bridges (6.14). Indirectly, corrosion is evaluated by accounting for loss of section in the rating calculations. The corrosion condition of a bridge is documented in reports that are often written after an inspection.

The 1967 collapse of the Point Pleasant (originally Silver) Bridge, as a result of stress corrosion cracking, prompted the initiation of the National Bridge Inspection Program (NBIP) (4.9). This program specifies minimum qualifications for inspectors and established the biennial inspection requirements (6.11). The manuals developed for this program are the 1983 AASHTO *Manual for Maintenance Inspection of Bridges* (6.2), the *Bridge Inspector's Training Manual 70* (6.7), and the *National Bridge Inspection Standards* (NBIS) (6.11). However, these manuals do not suggest a method for measuring the extent of corrosion during an inspection.

The *Bridge Inspector's Training Manual 70*, an FHWA manual, recommends that an inspector note the location, characteristics, and extent of the rusted area and that the depth of heavy pitting should be measured (6.7). The suggested tools for measuring reduced cross-sectional area are calipers, rulers, corrosion meters, and section templates.

Currently, information from the biennial inspection program is recorded on a Structure Inventory and Appraisal (SI&A) form and is filed with the FHWA (6.14). Based on the data collected, a sufficiency rating is calculated and used to compare the condition of bridges for the replacement and rehabilitation program.

The data recorded do not specifically require information concerning the condition of the protective system or amount of corrosion. The condition of the superstructure is given a numerical rating. This rating could be low enough to control the final sufficiency rating (6.14). The sufficiency rating indicates the potential for maintenance, rehabilitation or closure, but does not recognize the significance of corrosion.

Several articles have been written expressing a need for a more comprehensive bridge management program, possibly an expansion of the existing biennial inspection program (4.38,

4.41). Standard inspection procedure requires a tremendous amount of interpretation and the existing bridge program relies on the ability of a qualified inspector. A standardized method of measuring and recording inspection findings concerning corrosion will ensure uniformity when data are evaluated.

2.1.2 Corrosion Effects on Steel Bridges

A parallel literature survey was made to establish the present base of technical information related to corrosion of steel bridges. The primary goals of this survey were to identify: (1) types of corrosion damage, (2) details affected by corrosion, (3) effects of corrosion damage, and (4) factors contributing to corrosion.

A manual search included two citation index sources: Corrosion Abstracts, a compilation of numerous international journals dedicated to materials applications and corrosion, and the NTIS Materials Index. Company files were also examined, as were several unlisted trade journals and early volumes of *Materials Performance and Corrosion*. The latter are internationally circulated publications of the National Association of Corrosion Engineers. Several authorities on bridge corrosion were contacted in an effort to identify additional sources of citations. Corrosion of reinforcing bars was specifically excluded from this program. Weathering steel bridges were also excluded; however, recent literature on these structures is abundant and included here insofar as it pertains to the program goals.

Catastrophic failure, with loss of life, is probably the most publicized aspect of bridge corrosion. Collapses of the Point Pleasant Bridge over the Ohio River in 1967 (4.31, 5.21) and the Mianus River Bridge on Interstate 95 in Connecticut (4.7, 4.8, 4.10, 4.11, 4.14, 4.18, 4.23) are among the motivating forces behind the present program. The Point Pleasant Bridge, an eye-bar chain suspended structure, failed because of stress corrosion cracks at the pin hole in an eyebar. The Mianus River Bridge failure is attributed to corrosion of components of a pin-and-hanger assembly. It is hypothesized that over a relatively long period of time, accumulation of corrosion product from an underlying washer shifted the hanger transversely on the pin causing a misalignment of the hanger. This misalignment, with the hanger now bearing nearer the end of the pin, increased the stress range in the pin resulting in a fatigue crack leading to failure of the pin. The criticality of this detail and the potential effects of corrosion on the performance of the detail apparently went unappreciated during previous inspections. It is not even certain whether each of the critical hanger assemblies was inspected during previous inspections.

The Mianus River Bridge disaster sparked immediate inspections of similarly constructed bridges nationwide. While misalignments analogous to that hypothesized to explain the Mianus River Bridge failure were not uncovered, at least two other bridges were found with corrosion-related problems at pin-and-hanger assemblies. The Harvard Bridge in Cambridge, Massachusetts, was approximately 100 years old. Its hangers were wrought iron eyebars which had frozen at the pins due to corrosion (4.27). Fatigue cracks were initiated due to the redistribution of forces in these members. A major rehabilitation program was required before returning the bridge to full service. A broken hanger was found on the Yankee Doodle Bridge over the Norwalk River, on Interstate 95 in Connecticut. After blast cleaning the overlying corrosion product, 16 other hangers were found to exhibit cracks (4.16). Although the cracks were not attributed

to corrosion, it appears that corrosion had obscured serious structural damage that may well have gone undetected during a normal bridge inspection.

Other dramatic examples of undetected corrosion are also cited. In Philadelphia, severe deterioration caused failure of a main load bearing member of a Southeastern Pennsylvania Transportation Authority (SEPTA) bridge which had been hidden from view by wall and ceiling panels of a station building (4.15). In Detroit, there was a near collapse of a bridge because stirrups supporting a girder had rusted through and broken (4.19). The girder and the stirrups had been encased in concrete for protection against the exhaust fumes from locomotives passing below. Cracks were reported in beams of a Virginia bridge by a nearby work crew (4.43). These cracks were reported to be a result of bearings frozen by rust. This damage had apparently occurred within a 2-year inspection interval.

An historical bridge in St. Paul, Minnesota, provides an example of the speed in which corrosion damage can occur (4.9). Wrought iron eyebars on this structure experienced a 10 percent loss of section in only 4 years. Sliding and roller bearings were also frozen, apparently within a 2-year period. The altered behavior of this bridge was sufficient to shift members intended for tension into compression and vice versa.

Catastrophic and near catastrophic failures are also associated with other steel-supported structures. In Sweden, corrosion of a simple cotter pin was suspected of allowing a high voltage transmission line to drop onto a residential distribution line (4.17). A steel utility pole rusted through from the inside and fell over, killing a young boy in Michigan (4.50). Stainless steel hangers, supporting an indoor swimming pool roof in Switzerland, rusted through, allowing the roof to collapse (4.22). A roof over a Berlin auditorium collapsed because of stress corrosion of prestressing tendons in the structure (4.12). These examples, like the bridge collapses, exemplify the sudden and unexpected nature of the damage which corrosion can cause.

Somewhat less dramatic examples of bridge corrosion are found in which the operating authority is aware of existing corrosion damage and is able to plan major maintenance programs. Such programs have been described for the Benjamin Franklin Bridge in Philadelphia, which has experienced extensive corrosion damage to stringer beams and electrolysis damage from a direct current electric transit system operating on the bridge (4.6, 4.20). Corrosion of structural steel and regular maintenance for electrolysis damage are also cited as reasons for a rehabilitation program on the New York City elevated transit system (4.40). Significant section losses found on the floorbeams of the Williamsburg bridge in New York City caused closure of the bridge in 1988. Subsequent detailed inspections of both the Williamsburg and Manhattan bridges in New York City have found extensive corrosion losses in the floor systems and other areas exposed to deicing salts. Areas with heavy corrosion damage were evaluated and emergency repairs performed before being reopened to traffic. Repairs continued until all significant corrosion damage was remedied. Other observations of corrosion on steel bridges include footings near ground level, bearings, horizontal ledges associated with lap joints, seams along mechanically joined members, and electrical grounding cables (4.33).

Corrosion associated with paint breakdown and scale on cast iron was cited as the impetus for major repairs on the Tower Bridge in London (4.34). Here, it was noted that the majority of corrosion occurs where water collects near structural members either by seepage through the deck or as a result of pockets in

its design features. The Metro-North Commuter Railroad, in Connecticut, announced plans to rehabilitate four steel drawbridges (4.13). Corrosion damage had required reinforcement of steel girders, beams and trusses, as well as the replacement of rivets with high-strength bolts.

Recent examinations of numerous unpainted weathering steel bridges have provided significant corrosion-related observations (5.13, 5.26). Crevice corrosion, uniform corrosion, pitting, and galvanic corrosion were identified. These were prompted by deicing salt contamination, poultices formed by accumulated dirt and debris, capillarity of the corrosion product itself, and mill scale. Affected details included: pin and hanger assemblies; beams, cover plates, and weldments; cross bracing, diaphragms and stiffeners near joints; and cantilever expansion joints. Although the rust on weathering steels is of a different morphology and the pitting is more acute than for nonweathering steels, it is reasonable to expect similar attack on nonweathering steel bridges.

The angle and extent of exposure of weathering steel have also been examined in great detail (5.28, 5.30). The most severe condition for corrosion is one which promotes prolonged dampness without allowing washing of the surface. Thus, beam flanges and lower portions of their webs located near leaking joints are particularly susceptible. This is supported by the call for careful selection, design, and maintenance of expansion joints (3.36). An excellent detailed summary of the state of the art in weathering steel bridges has been assembled; reference to nonweathering steel is included for some comparisons (5.2, 5.20).

Steel pilings are another item of potential interest in bridge corrosion. Corrosion damage to piling is reported as uniform, pitting, and combinations of the two. Pit depth often exceeds $\frac{1}{2}$ in. Preferential attack at welds has also been reported (4.51). Bacteria-induced corrosion has also been observed (4.49). Preferential attack is suspected near submerged portions of piles protruding from concrete encasement (4.48).

Several studies have been reported which address the corrosion of steel piles in saltwater and brackish water (3.38, 4.30, 4.39). These considered various protective agents, including paint and metallic coatings, as well as cathodic protection. The most significant damage to piling occurs between the mean water level and the top of the splash zone due to frequent wetting and an abundance of oxygen. Corrosion is also found to concentrate just below the mean low tide level and at the mud line. This is attributed to both differential aeration and erosion caused by tidal action on silt and debris. Conventional coal tar, paint and metallic coatings can only provide 10 to 15 years of protection in these environments. Sheathing the high-water sections of piles with inherently corrosion resistant materials has provided in excess of 25 years of protection. Sheathing can be applied after corrosion begins and still provide protection. It is practical to attach it on-site. Full sheathing, if possible, offers the most corrosion protection. Cathodic protection in the form of aluminum or zinc sacrificial anodes is also effective in mitigating corrosion on piling.

Various techniques have been employed to assess the extent of corrosion damage to piles; these are summarized in Ref. 4.48. Obvious difficulties are encountered with direct measurements. Specially designed calipers and in situ ultrasonic probes have been used with some success. Several electrochemical methods are used to assess corrosion. These involve monitoring the change in electrochemical potential (polarization) of the structure as an external current is applied. The methods are called

polarization, Tafel slope extrapolation, and polarization resistance. These methods are defined in Appendix A and are described in the corrosion literature, such as Refs. 1.2 and 1.6. While these have the advantage of convenience, they are not reproducible and cannot distinguish between uniform and localized attack.

Cables are of particular importance in the integrity of suspension bridges. A complete recabbling of the Royal Gorge Bridge, after 45 years of service, was performed because of corrosion of the cables in the concrete-encased portion of the anchorages (4.21). Suspension cable corrosion was studied in great detail for bridges over the Ohio River after the Portsmouth Bridge was recabbed for the second time in its 50-year life (4.21, 4.34). This work noted that seven of the eight generally recognized forms of corrosion (see Chapter 3, Appendix C—published here in Part I) can be expected in bridge cables. The portions of cables most prone to attack are those that remain moist for extended periods. Cables in damp anchorages and poorly drained sections in the main span are cited as examples. Numerous other instances of bridges experiencing cable corrosion damage are cited in Refs. 4.21 and 4.34. Mention is made of one instance in which a gradient of 0.4 V over the length of a cable resulted in corrosion damage. The source of this gradient could be stray currents or a galvanic corrosion cell formed from material or environmental differences at the different points of voltage measurement. A checklist for bridge cable inspection is also provided in Ref. 4.34.

Recently, attention has been focused on corrosion of cables of cable-stayed bridges. A survey of many of these bridges showed significant cable corrosion (4.52). The Lake Maracaibo Bridge in Venezuela has required recabbling and the bridge in Hamburg, Germany, required cable replacement due to corrosion after only 3 years of service. Problems have been encountered because of leaking of cable sheathing. This allows water to contact the cables, leading to corrosion attack.

Some recent studies on inspection and evaluation of corroded steel bridges are reported (5.22, 5.23, 5.24). These studies were reviewed and used in developing the field and office guidelines.

Relevant references on fatigue resistance of corroded members (5.1, 5.3, 5.15, 5.16, 5.20), effects of web losses in beams (7.4, 7.5, 7.6, 7.7, 7.8, 7.10, 7.12, 7.13, 7.14, 7.17, 7.19, 7.27, 7.37, 7.41, 7.42, 7.48, 7.50), temperature effects on bridges with joint or bearing fixities (7.1, 7.9, 7.11, 7.18, 7.20, 7.21, 7.22, 7.23, 7.28, 7.29, 7.33, 7.34, 7.35, 7.36, 7.39, 7.40, 7.42, 7.54), stress concentrations (7.32, 7.38, 7.44, 7.45, 7.46), column analysis (7.3, 7.16), and load redistribution and redundancy (7.24, 7.25, 7.26, 7.30, 7.31, 7.47, 7.49, 7.51, 7.52) were also reviewed.

2.1.3 Surface Coatings

Corrosion and the environment take their toll on bridge coatings. While the original paint coating can be expected to last 15 to 20 years, subsequent repair coats are likely to provide only half that life. Paint coating literature specific to bridges teaches that paint deterioration should be assessed at individual areas of a bridge; the extent of deterioration can be indicative of underlying corrosion (3.6). The FHWA is involved in numerous cooperative programs with state highway departments to evaluate new coating systems (3.11, 3.40). Paint coating systems used by the various states have been compiled in Ref. 3.26. These are separated into primary and maintenance coating categories. The majority of coatings are oil/alkyd or zinc rich, but use of oil/alkyd is

already severely restricted because of environmental concerns. Surface preparation problems are cited as the cause for premature coating failures, but proper preparation is sometimes also hindered by environmental regulations (3.45). Other paint systems are also described (3.9, 3.17, 3.18, 3.20). Future bridge painting will probably be restricted to waterborne and high-solids epoxy systems because these have sufficiently low volatile organic compound contents to comply with tightening environmental regulations. Use of these systems will require tradeoffs in life and cost, respectively.

Bridge design and severity of its environment are significant factors for selecting a bridge paint coating (3.14, 3.15). The concept of specific utility is urged for matching coatings with the specific anticorrosion requirement (3.16). Expansion bays, drains, and scuppers require sound painting practice due to extreme exposure. Seams, edges, corners, rivets, bolts, and other protrusions often require special attention. Lower flanges and webs of beams often retain moisture and accumulate contaminants that accelerate deterioration of coatings. Some details are often difficult to cover, especially when spray painted. These include back-to-back angles, latticed structures, and railings. Direction of traffic flow and exposure to the elements are also important in determining coating requirements.

Those details subject to most rapid coating deterioration are most often those that are expected to be prone to attack by corrosion. Careful inspection of the bridge coating can thus be useful in determining the extent of underlying corrosion and in determining those areas most likely to be subject to rapid attack.

Hot-dip galvanizing is another method of bridge coating. It offers the benefit of sacrificial galvanic protection, whereby the steel substrate at small defects remains protected by the anodic nature of the zinc layer. Galvanizing, however, is limited to shop application. Several bridges have been constructed entirely of galvanized steel to avoid the high costs of maintenance resulting from corrosion (3.3, 3.12, 3.29, 3.30). These bridges have been in service for as long as 20 years with little or no breakdown in the protective galvanizing. Coating life in excess of 65 years has been projected for a painted galvanized bridge in a marine environment (3.29). Corrosion rates for galvanized coatings in a variety of environments have been studied (3.19, 3.25, 3.31, 3.34). An aluminum-zinc coating can also be used in place of zinc and is reported to give better protection than conventional galvanizing in most environments (3.41, 3.46).

Thermal spray metallizing provides an alternative to galvanizing. Zinc or an aluminum-zinc alloy are sprayed, in the molten state, onto the steel substrate. Thickness can be easily controlled and built up considerably more than in hot-dipping. Aluminum-zinc gives a harder, more abrasion resistant coating. Metallizing can be performed either in the shop or in the field and can thus be used on members too large for galvanizing and for coating repair work. Metallized coatings are presently under evaluation in Ohio and Quebec (3.25, 3.44). A zinc metallized bridge in Philadelphia has been in service since 1930 and is reported to be essentially corrosion free (3.32). Atmospheric corrosion data for metallized coatings is available (3.27).

2.1.4 Mechanisms of Corrosion

Several typical references on the general nature, theory, and mechanisms of corrosion are available (1.2, 1.5, 1.6). A number of edited sources are also available, which are dedicated to spe-

cific corrosion mechanisms (2.4, 2.5, 2.6, 2.8, 2.11, 2.13, 2.16, 2.18, 2.21). These sources include information on structural steels.

Of further interest are the factors, other than moisture, which have been identified as promoting corrosion on bridge components. They include atmospheric pollutants, roadway contaminants, electrical effects such as stray currents and voltage gradients, and bacteria.

Atmospheric pollutants have been given extensive attention in recent years. Nitrogen and sulfur compounds, resulting from automobile exhaust and industrial emissions, respectively, have been found to influence the corrosion of steel (2.10, 2.14). The effect of nitrogen and sulfur compounds is synergistic and further dependent on humidity. Carbon dioxide and gaseous ammonia can also be detrimental to steel (2.19). Environments have been classified as marine (containing high concentrations of airborne chlorides), industrial, and rural. Combinations of these categories are also considered. Numerous investigations have examined the influence of the environments at established test sites on the corrosion of steel (2.13, 2.14, 2.15). These studies include angle of exposure, time and degree of wetness, and extent of sheltering.

Extensive analysis of salt deposits found on bridges in Germany revealed that most chemical components identified could be attributed to atmospheric pollutants (2.12). Another investigation found that poultice-induced corrosion was synergistically related to chlorides and acid rain (2.1). An acid rain map was presented.

Electrical effects are not typically considered during evaluations of structural materials. Potential gradients arising from differences in the soil, thermal gradients, telluric effects, and other stray earth currents can only be analyzed in situ for a particular structure. Such an analysis can be difficult at best for a sizable bridge. References to these effects are conspicuously lacking in the bridge literature. Basic texts that discuss electrical gradient effects on corrosion are available (1.6, 1.7).

The influence of bacterial agents on corrosion is also discussed (1.6, 2.9, 2.20). Detrimental bacteria can be found in water, soil, and on surfaces exposed to the atmosphere. Their presence can often be unexpected. Bacteria-induced corrosion damage can occur at rates several times greater than those in similar, bacteria-free environments.

Advancements have been made in the field of corrosion prevention. For example, improvements have been made in the protective systems and joint details used. The development of new and inexpensive techniques to measure significant nonuniform corrosion on bridges has not progressed. As early as 1971, researchers were attempting to develop a nondestructive evaluation system for early detection of fatigue cracks (3.40). Ultrasonic and magnetic field devices showed potential, but current, typical inspection procedures have not adopted these techniques.

2.1.5 Summary of Literature Search

Review of the bridge inspection and the pertinent corrosion literature demonstrates several crucial points.

- A critical detail can be adversely affected by corrosion and go undetected until a catastrophic event occurs. Either loss of life or severe structural damage can result. Only after such a problem is recognized will similar structures be inspected for the same flaw. As evidenced by the Mianus River Bridge and the

suspension cable inspections, it is likely that similar structures will have accrued similar damage.

- The affected detail may not be considered to be directly critical; that is, sufficient redundancy may be assumed for the structure, so that perforation and loss of a beam may not be taken with immediate concern.

- Inspection methods and inspector training are inadequate for properly assessing corrosion damage and relating it to structural integrity. This is the fault of the system, not the inspector. Time and expense considerations are overriding.

The literature suggests that the type and age of a bridge and a knowledge of the actively occurring corrosion mechanisms can be used as indicators for potential catastrophic damage.

- *Type of Bridge*—Heavily detailed bridges such as trusses are more susceptible to undiscovered corrosion than simple girder-type structures. Any structure utilizing pin and hanger support is also highly suspect. Open-deck bridges, especially in heavy traffic areas, are similarly prone to extensive corrosion damage.

- *Age of Bridge*—The age of a structure will be a primary indicator of its potential for existing corrosion damage. Unfortunately, corrosion-age cannot be determined on an absolute basis. It will be influenced by several other important factors such as the type of bridge, the environment, the type of coating, and the condition of the coating. Sufficient data are available to estimate the expected life of most coatings in a number of different environments.

- *Knowledge of Corrosion Mechanisms*—Knowledge of the types of the corrosion mechanisms active on specific details of a bridge provides the third indicator of the likelihood of critical damage. If localized corrosion can be identified, an immediate and careful analysis of the affected detail should be made to determine the resultant structural impact. The literature provides ample indication of the details that may be affected. Damage from uniform corrosion should also be scrutinized. A close watch should be kept on any corrosion damage, even that considered to be only cosmetic or nuisance damage. Thorough inspection and evaluation of any potentially dangerous attack should be mandatory.

2.2 RESPONSE TO QUESTIONNAIRES

A questionnaire addressing corrosion effects and current practice was sent to the 50 state departments of transportation and the District of Columbia, Department of Public Works. In addition, responses to the questionnaire were solicited from other owners including 68 bridge authorities, railroads and turnpike commissions. A sample of the questionnaire is provided in Appendix B.

The rate of response has been generally good. To date, 67 percent of the state DOTs (and the District of Columbia) have returned the questionnaire and many included additional data. Response from the railroads and authorities has been slower as 7 of 43 (16 percent) and 12 of 25 (48 percent) have returned information respectively. The following is a synopsis of information contained in the responses. This synopsis represents the collective opinions of the respondents.

2.2.1 Bridge Inspection

The number of steel bridges that are under the various jurisdictions varies drastically. For example, the state of Hawaii has eight bridges that fall under the categories listed in the questionnaire, while the state of Pennsylvania has more than 6,000, and the Consolidated Rail Corporation (CONRAIL) maintains over 6,700. These bridges are inspected at least every 2 years, with many inspected more frequently. This complies with the inspection frequency set forth by the National Bridge Inspection Program (NBIP).

The number of inspectors on staff is roughly related to the number of bridges per state, in that the states with larger numbers of bridges have larger inspection staffs. Often, design departments supplement inspection personnel. The amount of inspection time per bridge depends on the size of structure. Generally, small structures are allocated approximately 2 hours to inspect, while large structures are allocated from 4 weeks to almost continuous inspection. Typically, two inspectors make up an inspection team.

The state inspector personnel appear to meet the National Bridge Inspection Standards (NBIS) concerning minimum requirements. All of the states responding offer a formal Bridge Inspection Training course. The railroads and authorities are free to develop their own inspection formats. It appears that the inspection personnel used by the railroads and authorities generally meet NBIS minimum requirements. Of the states that responded, 97 percent felt the Bridge Inspection Training courses offered to inspection personnel covered structural details known to be susceptible to corrosion. In addition, 33 of the 34 state respondents provide seminars or other courses for bridge inspection personnel. In general, the railroads and authorities offer similar courses for their personnel. Other than reporting inspection findings on the standard Structure Inventory and Appraisal form, methods of documenting the bridge condition varied from state to state. All of the responding states use photographs, 85 percent use written narratives, and 88 percent use a checklist. All three methods are used by 73 percent of the respondents and sketches are often added as needed. Likewise, 100 percent of the authorities and 71 percent of the railroads use written narratives, 83 percent of the authorities and 100 percent of the railroads use a checklist, and 100 percent of the authorities and 71 percent of the railroads use photographs. All three methods are used by 83 percent and 57 percent, respectively.

During an inspection, the corrosion deterioration of a bridge is estimated visually and the extent of corrosion is measured. Access for inspection is obtained by manual climbing or use of snooper-type equipment. Binoculars are commonly used. The most frequently used instruments for measuring the extent of corrosion are rulers, calipers, and micrometers. Ultrasonic thickness gages have been used by 36 percent of answering states, 33 percent of the responding authorities, and 12 percent of the railroads—most having a favorable experience. Other nondestructive evaluation methods, such as thermographic testing, x-ray testing, acoustic emission testing, and magnetic particle testing, are rarely used.

Of the responding states, two-thirds either do not have suspension bridges or the inspections are performed by consultants. Those that inspect suspension bridges appeared to perform thorough, in-depth inspections. Approximately one-third of the responding authorities inspect suspension bridges and they claim to perform thorough, visual inspections of all structural details.

2.2.2 Nature of Corrosion on Bridges

Uniform and pitting corrosion are the most common forms of corrosion observed on steel bridges. Nearly all of the components listed in the questionnaire were noted as “often” or “occasionally” having these types of corrosion. Cotter pins, welds, cables, and prestressing steel were among the group of bridge elements least likely to be afflicted by these types of corrosion. Cracking resulting from corrosion was not a frequent occurrence, as only 7 percent of all responses indicated cracking occurred often. Crevice corrosion seemed to occur most often in truss members, bearings, hanger assemblies, and gusset and connection plates. There were indications that all bridge elements, except cotter pins and piling, are susceptible to crevice corrosion. Corrosion due to dissimilar metals does not appear to be a common problem on steel bridges. A few cases were noted, but, in general, this type of corrosion “almost never” occurs.

The respondents noted no particular correlation between the forms of corrosion in bridge elements and the type or grade of steel. No correlation can be drawn, because the inspector has no previous knowledge of the grade of steel in a structure, for the most part.

A review of the case studies of steel bridge failure or closure invariably points toward a common factor, namely corrosion. One respondent stated that, “Nearly all structural failure, other than fatigue cracking, can be related to corrosion.” The most frequently stated corrosion effect which caused structural distress was frozen expansion bearing devices or hanger assemblies. Some examples of the resulting distress are cracking of substructure units or anchor bolts, and cracking of members or member distortion. In addition, loss of section has led to cases of beam web buckling, usually near bearing devices. Corrosion of fasteners has reduced the capacity of some bridges as a result of loss of clamping force. One respondent from an authority stated that crevice corrosion between plates of built-up box sections has caused concern for the integrity of the member.

According to the responses, three factors are most likely to cause corrosion: damp or wet conditions, moisture entrapment, and deicing materials. These factors cause uniform and pitting and crevice corrosion most frequently. Other factors that were suggested to have less prominent effects were industrial atmospheres, marine atmospheres (note: many states do not have bridges in this environment), debris accumulation, bird droppings, and clogged drains. One respondent felt that traffic spray and accumulations of cinders were a significant cause of uniform and pitting corrosion. The type of deicing material most used by the state DOTs and the authorities is salt. Sometimes the salt is mixed with cinders or sand. Deicing materials are rarely used by the railroads.

Of the state responses received to date, approximately 50 percent noted no difference in the severity of corrosion damage observed on bridges over either highways, railways or waterways. Of the states that determined there was a difference, 67 percent felt corrosion over waterways was most severe, 42 percent judged highways to be most severe, and one percent felt bridges over railroads were most severe. Three respondents noted that the amount of vertical clearance has significant influence on the rate of corrosion. The responses from the railroads and authorities varied and a significant conclusion could not be drawn.

When corrosion becomes extensive, maintenance is required. The most common maintenance practice is to clean and paint

corroded metal. If the deterioration is so extensive that remedial action is required, fasteners or structural members may be replaced or bolted reinforcement added. If cracking is apparent or water entrapment is a problem, holes are often drilled. Sealing of areas of crevice corrosion or leaking deck joints is often performed to prevent moisture from contacting the metalwork.

2.2.3 Protective System

The most common method of corrosion control is a comprehensive protective coating system. There are three general types of coating systems for protection of bridge metalwork: barrier protection, inhibitive primer protection, and sacrificial (galvanic) protection. Currently, there appears to be no standardization among the states, authorities, or railroads as to the type of coating system used. A movement was apparent to the use of the newer high-performance systems that use both galvanic and barrier protection. However, a number of agencies continue to use lead-based paints that provide good service and are less sensitive to the condition of the metal surface. Two of the responding railroads use a grease coating.

Corrosion has had an effect on design and maintenance practices. Several states have gone as far as to design concrete members, rather than steel members, when possible. Nonfunctioning deck joints and drainage systems have been identified as causing significant corrosion and have been modified. Continuous bridges are preferred in order to eliminate joints. In general, attempts are made to limit corrosion prone details. Maintenance practices have changed. Cleaning of bearing seats and power washing of metalworks are used. Spot painting is used as an early corrosion control measure.

Unusually rapid or unexpected localized corrosion damage has been encountered by several respondents. Most examples were either located in extremely corrosive environments or resulted from poor workmanship. The type of corrosive environment typically encountered was continual contact of moisture or salt. One case of atmospheric (industrial) corrosion was cited. Debris accumulations were also cited as a cause of rapid corrosion.

Very few reports, surveys, or compilations on the corrosive damage to bridges have been initiated by the respondents. Most information is contained in inspection reports and compilations are performed only on an informal basis.

Cathodic protection to control corrosion has received limited use except on bridge decks and some substructure units. This system has been used to protect steel piling in several states and anchor cables on floating bridges in one state.

Birds and animals are discouraged from habitation of void spaces by use of poison or installation of protective covers. Most methods have not provided much protection.

Two states and two authorities share bridges with light rail systems, but the systems have been in operation for a short time and no related corrosion was noted. One state has bridges that share substructure with railroads, but the effect is negligible.

The final question asked the respondents if there were any specific corrosion related problems they would like resolved. The responses indicated a variety of needs, but most often mentioned were a long-term environmentally safe painting system and a maintenance-free, water-tight expansion joint. Other areas of improvement noted were development of a noncorrosive deicing material and an effective means of pigeon control. One respon-

dent had a need for a quick and reasonably accurate method of measuring and reporting loss of cross-sectional area.

2.2.4 Summary of Questionnaire Responses

Review of the questionnaire responses suggests several conclusions.

1. The responses indicate there is not significant occurrence of fatigue, yielding, buckling, or other severe structural distress other than section loss due to corrosion.

2. Typical inspections occasionally include use of nondestructive evaluation methods; usually visual estimates are supplemented with mechanical measuring devices.

3. The inspection force generally meets the requirements set forth by the NBIS, but this may not be sufficient for corrosion inspection.

4. Uniform and pitting corrosion are the most common forms of corrosion and all bridge elements are susceptible to these forms of corrosion.

5. Damp or wet conditions, moisture entrapment, and deicing materials are the three factors most likely to cause corrosion on bridges.

6. There was not a standard method of corrosion control.

7. There is a general feeling among owners that the most effective means of addressing the corrosion problem is through improved coating systems and increased budgets for preventive maintenance, primarily painting.

2.3 ANALYTICAL STUDIES

The analytical studies conducted include effects of localized web losses; effects of flange losses in a beam; effects of truss member losses; effects of frozen bearings on truss, tied arch, and girder bridges; and effects of frozen pin and hanger assemblies in cantilever truss and girder cantilever bridges.

The analytical investigation of the effects of localized web losses in a beam have led to the following conclusions:

1. Web holes in a beam alter the stress distribution. When evaluating this, the resulting stress pattern in the beam shows: (a) occurrence of local stress concentrations at the boundaries of the hole, and (b) change in the configuration of the overall stress field in the area affected.

2. The stress concentrations at the boundary of the hole are highly localized and they attenuate rapidly away from the hole.

3. Small holes (2 in. by 7 in.) have little effect on the overall stress field.

4. Normal stress distributions at transverse sections through the end panel clearly indicate Vierendeel action.

5. Contour plots of resulting principal stresses in the area of the hole can indicate possible failure mechanisms.

6. Web holes, unless severe, have a negligible effect on the deformation characteristics of the beam.

The analytical study of the effects of flange corrosion losses in a beam have shown that in cases of uniform losses, beam theory with a reduced section modulus may be used to evaluate corrosion effects on strength. In cases of highly localized losses in the tension flange of a beam, effects of corrosion may be

evaluated by assuming the tension flange to behave as an independent member loaded in axial tension. For moderate flange losses (below 30 percent) beam theory may be used if the length of loss along the flange is about 3.0 ft or over and the transition from reduced to full section is not abrupt. The effects of uniform flange losses on the deformation characteristics of the beam may be determined using beam theory and the effects of localized flange losses on the deformation characteristics of the beam may be neglected.

The analytical investigation of the effects of truss member losses has led to the following conclusions:

1. Load redistribution due to corrosion losses in a truss member may result in a reduction in stress in the corroded member and an increase in stress in adjacent bridge members. The effect on the stresses in truss members further away from the corroded member is negligible.

2. Within the linear structural response range, the effects of load redistribution due to small to moderate section losses may be neglected. When the section loss of a truss member exceeds 20 percent over most of its length, the effects of load redistribution on the members adjacent to the corroded member must be considered.

3. In most cases, analysis based on simple plane truss models can provide sufficient accuracy. In cases of severe corrosion, plane frame or three-dimensional models may be needed to evaluate load redistribution effects.

4. Load redistribution in the nonlinear response range may give some additional reserve in strength to deteriorated truss diagonals, top chord members, and bottom chord members. End posts and hangers have no such additional reserve.

The investigation of the effects of frozen bearings on the temperature and live-load-induced stresses in a simple truss bridge have led to the following observations:

1. Freezing of a bearing results in temperature-induced loads in the bearing, pier, and members of the bridge superstructure. The temperature loads in the superstructure affect mainly the bottom chord members and members of the bottom lateral system. When the expansion bearings of both trusses of the bridge are frozen, the temperature loads in all the other bridge members are negligible. Freezing of the expansion bearing of only one truss results in increased temperature loads in the bottom chord of that truss and the bottom lateral system, and also induces stresses of lower magnitude in top chord members, diagonals, and members of the top lateral system.

2. The lateral stiffness of the bridge piers at the location of the bearings has a significant effect on the magnitude of the resulting temperature loads. A simplified analysis technique formulated for estimating the effects of the lateral stiffness of bridge piers is presented in Appendix D (published here as Part II).

3. If the bearings are frozen in an expanded position, tension stresses will be induced in the bottom chord members at normal temperatures. These stresses are additive to the dead load and live load stresses. Bearings frozen in a contracted position can actually reduce the live load effects on the bottom chord. However, if the resulting temperature-induced compression stress in the bottom chord is larger than the dead load stress, buckling of the bottom chord may occur.

4. Live load induces compression stresses in the bottom

chord. The effect of a frozen bearing on the live load stresses in the other truss members is negligible.

5. If the live load capacity of the truss is governed by the capacity of the bottom chord, freezing of the expansion bearings may actually increase the load-carrying capacity of the truss.

6. A plane truss model of analysis is adequate for determining temperature and live-load-induced stresses in a truss bridge with symmetrical support conditions.

The investigation of the effects of frozen bearings on the temperature and live-load-induced stresses in a tied arch bridge has shown that:

1. The temperature-induced loads in the superstructure of a tied arch bridge caused by freezing of its expansion bearings affect mainly the girder members of the bridge.

2. The magnitude of the resulting temperature load depends on the lateral stiffness of the bridge piers at the location of the bearings. A simplified analysis technique for determining these loads is presented in Appendix D (published here as Part II).

3. If the expansion bearings are frozen in an expanded position, tension stresses will be induced in the girder members at normal temperatures. These stresses are additive to the dead load and live load stresses. If the expansion bearings are frozen in a contracted position, the tension stresses in the girder members due to dead load will actually be reduced when the temperature increases.

4. In a tied arch bridge with frozen bearings and piers rigid laterally, the horizontal live load reaction at the ends of the rib will be transmitted to the piers, and the axial stresses in the girders will not be affected. If the live load capacity of the bridge is governed by the capacity of the girders, freezing of the expansion bearings may actually increase the load-carrying capacity.

5. If the substructure is not rigid enough to resist the rib live load reaction, very little change in the live distribution will occur.

The investigation of the effects of frozen bearings on the temperature and live-load-induced stresses in a girder bridge has shown that:

1. Freezing of the expansion bearing of a girder results in a horizontal thrust at the location of the bottom flange when the temperature changes. This thrust can affect the distribution of stresses throughout the girder. When the substructure is rigid laterally, high localized thermal stresses in the area close to the bearings and significant changes in the overall stress distribution in the girder may occur. Guidelines for evaluating the magnitude of these stresses and the effect of pier flexibility on these stresses are given in Appendix D (Part II herein).

2. Live load on a girder with frozen bearings also results in an horizontal thrust at the location of the bottom flange. A simplified analysis procedure for determining the magnitude of this thrust is presented in Appendix D (Part II herein).

3. Freezing of the expansion bearing of a girder results in an increased bending stiffness. The horizontal live load reaction at the bearing reduces the tension stresses in the bottom flange of a simply supported girder and may also cause some reduction in the maximum compression stress. As a result, the load-carrying capacity of the girder may increase. It should be remembered, however, that temperature effects have to be taken into account

as well. If the bearings of a simply supported girder are frozen in an expanded position, tension stresses and bending stresses additive to the bending stresses caused by dead load and live load will be induced under normal temperature conditions. Bearings frozen in a contracted position will cause compression stresses and bending stresses opposite to the bending stresses caused by dead load and live load.

4. The maximum localized live load stresses caused by the rotational restraint of a bearing are limited by the bending strength of the bearing itself and the strength of its components.

Studies of the effects of frozen pin and hanger assemblies in cantilever truss bridges have shown that:

1. Temperature-induced bending stresses occur in the hangers and in the truss members connecting to the hangers. An analysis procedure for calculating the magnitude of these stresses is given in Appendix D (Part II herein).

2. The effects of hanger fixity on the live-load-induced axial loads in the hangers and the live load bending stresses induced in truss members other than the hangers may be neglected. Determining the magnitude of the live load induced bending stresses in the suspending hangers must consider the possibility of sudden releases of the rotational restraint at the pins, which can result in increased stresses. These stresses can result over time in fatigue cracks at the ends of the hangers.

Studies of the effects of frozen pin and hanger assemblies in girder cantilever bridges have shown that:

1. Freezing of the pin and hanger assemblies in a girder cantilever bridge results in temperature-induced in-plane bending stresses in the hangers. Also, shear stresses are induced in the pin by the torsional load resulting from hanger fixity.

2. The magnitude of the in-plane bending stresses in the hangers and the shear stresses in the pin can be quite high and, if detected, frozen pin and hanger assemblies in girder cantilever bridges must be repaired as soon as possible.

2.4 LABORATORY AND FIELD STUDIES

The nondestructive test evaluation (NDE) techniques that have potential applicability to inspection of corroded bridge components include: (1) ultrasonic inspection using computerized mapping techniques for the purpose of defining surface roughness and localized section loss characteristics; (2) ultrasonic inspection for evaluating areas not visually accessible, using data acquisition techniques similar to the ones used in the above; and (3) measuring devices that can be used to evaluate bearing or pin hanger fixity.

Several types of ultrasonic equipment for thickness measurement of bridge elements have been identified and investigated. They include the Krautkramer-Branson DME Ultrasonic Thickness Meter, Panametrics Models 22DL and 26 DL, and Cygnus Instruments Model 1.

Two devices have been identified as having potential use for evaluating bearing fixity. The first of these is simply a listening device such as an amplified stethoscope which amplifies the noise generated by a moving bearing. The second device is a tilt sensor which can provide quantitative data to the inspector by measur-

ing the degree of change in the position of the elements attached to the bearing.

A detailed description of the various types of ultrasonic equipment and devices for measuring bearing fixity including advantages and disadvantages, costs, and a list of manufacturers is given in Appendix F.

The laboratory and field tests conducted have shown that the ultrasonic method is a viable technique to provide the engineer with quantitative corrosion data. The method is most useful when a flat, uncorroded surface is available upon which to place the transducer. Corroded surfaces, however, present a difficult situation, which may or may not be measurable. A small probe should be used and the surface cleaned as much as possible, with power tools if available. Even at that, the data readings will be highly variable and might seem to indicate thinner metal than actually occurs. Ultrasonics on corroded surfaces should be used only by an inspector who thoroughly understands the techniques and limitations of the method; otherwise, false information will be the result. Detailed corrosion measurements are most readily obtained when the location of the corroded area is known. If the location of the corroded area is not known, random measurements are needed to locate the area prior to performing profile measurements.

Ultrasonics is also a useful tool to evaluate the hidden surfaces of bearing and hanger pins, but only if a central hole is present for the transducer to scan the surface. Attempts to use ultrasonics on the end of the pin to locate surface corrosion were unsuccessful. It might be possible to locate grooving corrosion on the pin using back reflection techniques whereby the sound wave is reflected from the irregular surface of the groove and then back to the receiving transducer; however, this technique would not be capable of providing quantitative information on the depth of corrosion. On the other hand, ordinary pulse echo ultrasonics using a flaw detector would pick up the presence of circumferential cracking emanating from the groove. Adequate acoustic contact between the transducer and metal surface is the most difficult problem in using ultrasonics. The transducer used to scan the hole must be specially produced to fit the hole. Transducers made to fit a small hole will probably fit a large hole, but not vice versa. A transducer made to fit a 1-in. diameter hole appears to be the smallest available at the present time.

The field testing of acoustic methods of monitoring bearing fixity has yielded inconclusive results. This method, however, may still have merit and should be looked at in more detail. More experience is necessary to establish whether the sounds emitted by a moving bearing can be distinguished from other noise on the bridge.

Field testing of the use of tilt sensors has shown that the tilt meter has potential use for qualitatively determining the rotation of bridge components where installation is possible. A recording device would be useful for unattended monitoring through various temperature cycles. Improvements in the installation procedure and in the instrument's sensitivity would be effective in making the tilt meter more useful in field applications.

A detailed description of the laboratory and field tests conducted is included in Appendix F.

CHAPTER 3

APPLICATIONS**3.1 INTRODUCTION**

Based on the information gathered in Task 1 and on the laboratory tests, field investigations and analytical studies conducted, guidelines for field inspection, office evaluation and non-destructive test evaluations were developed.

3.2 FIELD INSPECTION GUIDELINES

Field inspection guidelines for corrosion inspection of bridges are included in Appendix C (published here as Part I). These guidelines give corrosion inspectors background information on corrosion, the locations and forms of corrosion attack on bridges, and a suggested framework for corrosion inspection reports.

3.3 OFFICE EVALUATION OF CORROSION EFFECTS

Techniques for office evaluation of corrosion effects in bridges are presented in Appendix D (Part II herein). They include simplified analysis methods suitable for routine office evaluation. The office guidelines contain the necessary background for assessing localized and overall effects of corrosion on bridge structures. Effects of material loss, unintended fixities and distortions caused by corrosion are treated in detail.

Appendix G (Part III herein) summarizes the findings of Appendix D (Part II) in a guide specification format.

3.4 GUIDELINES FOR NONDESTRUCTIVE TEST EVALUATIONS (NDE)

NDE techniques which may be applied to corrosion inspection were identified. An evaluation of the applicability of these techniques is given in Appendix F. It applies to: (1) ultrasonic measurement of surface profile, (2) acoustic methods of monitoring bearing fixity, and (3) use of a tilt meter to monitor bearing fixity.

CHAPTER 4

CONCLUDING REMARKS AND SUGGESTED FURTHER WORK**4.1 CONCLUDING REMARKS**

- Corrosion is the major cause of deterioration of steel bridges. The results of this deterioration range from progressive weakening of a bridge structure over a long period of time to sudden bridge collapse.

- Corrosion damage must be carefully appraised and evaluated. In some cases immediate repair or closure is necessary while in other cases the conditions created by corrosion may be

tolerated. In all cases, however, the likely progression of corrosion should be considered.

- The present inspection procedures do not pay adequate attention to the types of corrosion attack on bridges. Also, in most cases, the amount of deterioration is not adequately determined. Bridge inspectors should be given additional instructions regarding inspection with respect to corrosion. Using corrosion inspection forms such as those presented in Appendix C (Part I herein) can guide inspectors in the appraisal of corrosion damage on bridges.

- The AASHTO *Manual for Maintenance Inspection of Bridges* and the AASHTO *Standard Specifications for Highway Bridges* are primarily concerned with good quality materials and bridge structural behavior consistent with its design. A corrosion damaged structure may behave differently from the "design" structure and different failure modes may govern its capacity. Therefore, additional criteria must often be considered when evaluating the capacity of a deteriorated structure. The office guidelines presented in Appendix D (Part II herein) include recommendations and analysis techniques aimed at helping the engineer evaluate remaining structural resistance and making him aware of the variety of potential failure modes.

4.2 SUGGESTED FURTHER WORK

- The field investigations conducted have pointed out the need for a more systematic approach for preventing corrosion problems in bridges. It is suggested that further work be done to develop guidelines for design and detailing to be used in the initial stages of bridge design. These guidelines should make the engineer aware of potential corrosion problems and help him select adequate materials and details to avoid future corrosion problems.

- Some forms of corrosion, such as stress corrosion, are very difficult to detect and can result in sudden failure. It is suggested that more work be done to develop means of prediction and detection of these forms of corrosion which could be applicable to bridges.

- At this time the effects of active corrosion on fatigue resistance are not completely understood. More research is needed in this area.

- The emphasis in the office guidelines developed is on an initial, Level I, evaluation, based on elastic methods of analysis and mainly on a service load approach. Additional work is needed in order to provide more detailed guidelines for a Level II evaluation that would include the nonlinear range of structural response.

- The office guidelines include simplified methods of analysis suitable for routine office evaluation. These methods are based on a service load approach. Additional work would be required to formulate analysis techniques consistent with a load and resistance factor approach.

- In many cases of corrosion damage repair is necessary. It is suggested that further work be done to provide guidance for the design of such repairs. For example, recommendations can be made as to what would be the most efficient way to retrofit web holes in girders and how to design the retrofit.

- The laboratory and field investigations have shown that various nondestructive evaluation (NDE) techniques can be used for corrosion inspection. However, further work is needed for developing these techniques to a level acceptable for practical use.

NOTES



Figure 80. Photographs show truss bridge losses.



Figure 81. Photographs show flange losses.



Figure 82. Photograph shows web losses.



Figure 83. Photograph shows frozen bearings.

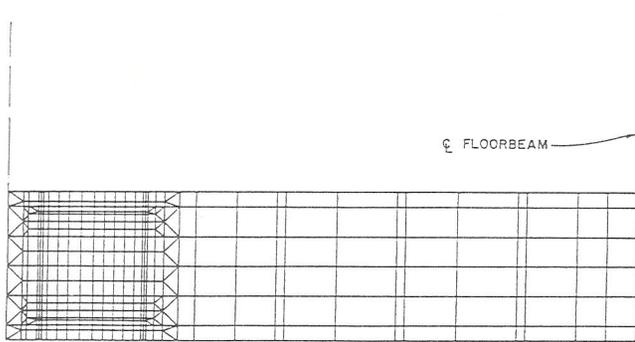
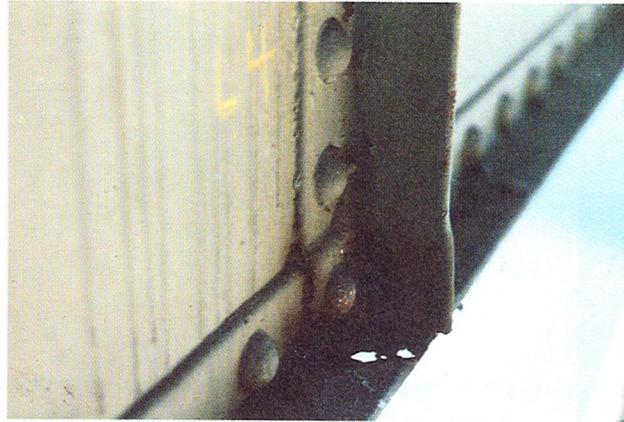
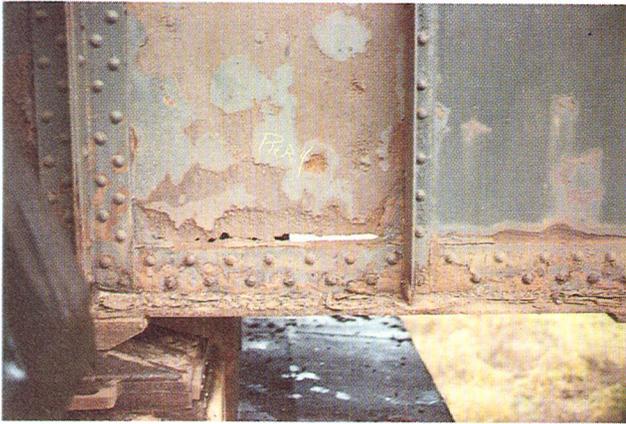


Figure 84. Model of analysis for studying web losses. Top photograph shows floorbeam elevation; bottom diagram illustrates finite element idealization for web losses.

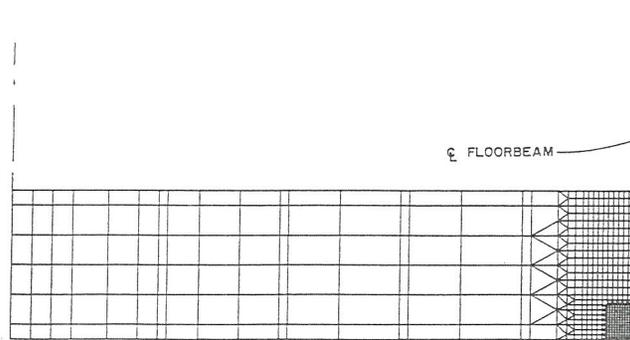


Figure 85. Model of analysis for studying flange losses. Top photograph shows floorbeam elevation; bottom diagram illustrates finite element idealization for flange losses.

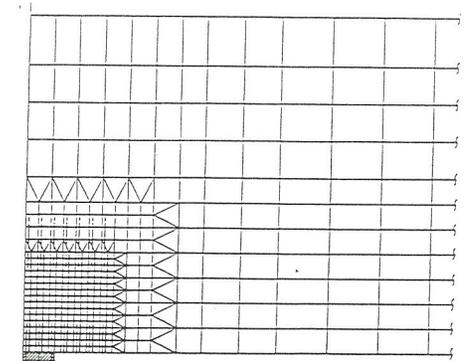


Figure 86. Model of analysis for studying web losses and frozen bearings. Top photograph shows girder elevation; bottom diagram illustrates finite element idealization for web losses and frozen bearing.

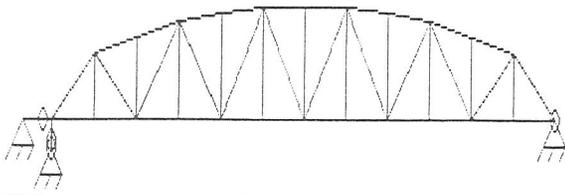
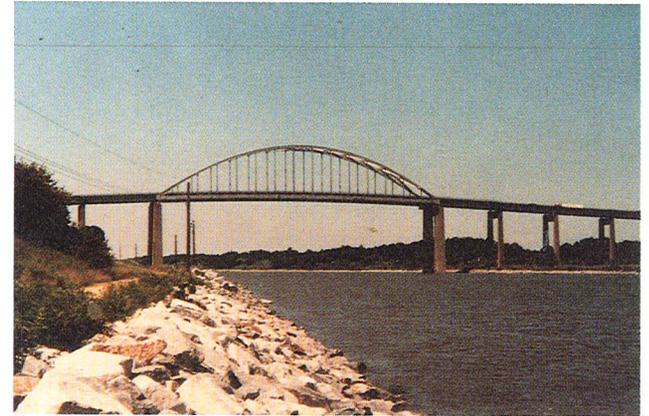


Figure 87. Two-dimensional model for truss analysis. Top photograph shows elevation of truss bridge; bottom diagram illustrates model for truss analysis.

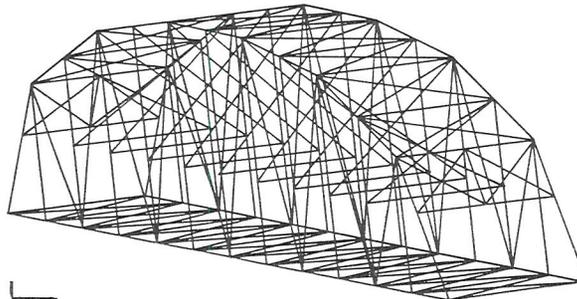


Figure 88. Three-dimensional model for truss analysis. Top photograph shows truss bridge. Bottom diagram illustrates 3-dimensional model for analysis.

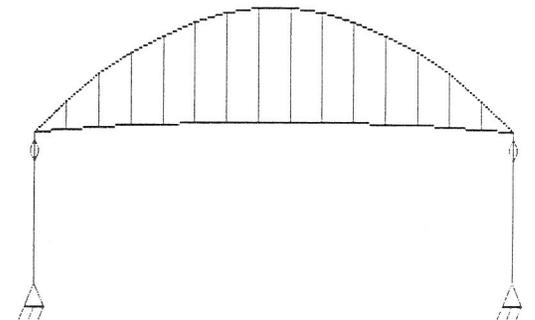


Figure 89. Two-dimensional model for tied arch analysis. Top photograph shows elevation of tied arch bridge; bottom diagram illustrates model for analysis.

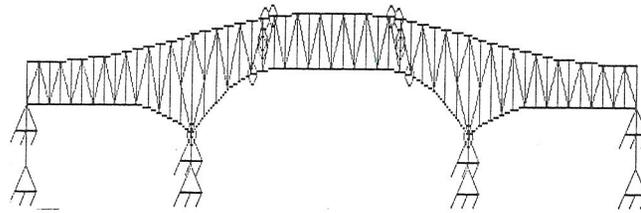
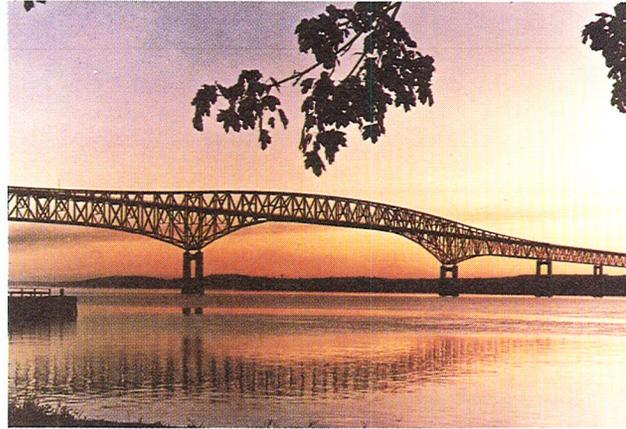


Figure 90. Two-dimensional model for cantilever truss analysis. Top photograph shows elevation of truss bridge; bottom diagram illustrates model for analysis.

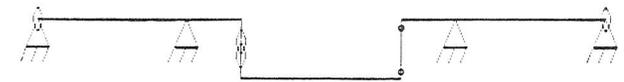
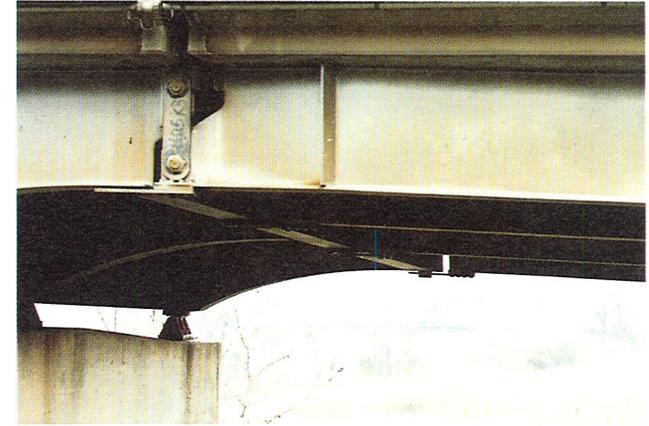


Figure 91. Two-dimensional model for girder cantilever bridge analysis. Top photograph shows elevation of girder cantilever bridge; bottom diagram illustrates model for analysis.

APPENDIX A

GLOSSARY

- acid rain (acid precipitation)**—Precipitation with increased acidity due to the interaction of atmospheric moisture with air pollutants, notably oxides of sulfur and nitrogen.
- acoustic emission testing**—The detection of high-frequency stress waves generated by the rapid release of strain energy that occurs in a material during crack growth, plastic deformation, or phase transformation.
- anode**—The element in a corrosion cell that loses electrons and corrodes.
- cathode**—The element in a corrosion cell that receives electrons and is protected from corrosion.
- cathodic protection**—Corrosion protection by electrochemically causing the metal to act as the cathode in a corrosion cell.
- chemical species**—Any of the chemical elements, ions, or compounds.
- concentration cell**—A corrosion cell in which current flow (and, thus, corrosion) is caused by difference in concentration of a component in the electrolyte.
- corrosion**—Deterioration of a metal due to a reaction with its environment.
- corrosion cell**—A corrosion "system" consisting of an anode, a cathode, and an electrolyte.
- corrosion fatigue**—Fatigue-type cracking of metal caused by repeated or fluctuating stresses in a corrosive environment characterized by shorter life than would normally occur as a result of either the repeated or fluctuating stress alone or the corrosive environment alone.
- corrosion rate**—The rate at which corrosion attack proceeds, commonly expressed in inches penetration per year (ipy) or mils penetration per year (mpy).
- crevice corrosion**—A form of localized corrosion occurring at locations where easy access to the external environment is prevented, such as at the mating surfaces of metals or assemblies of metal and nonmetal.
- deposit attack**—A type of crevice corrosion that occurs where foreign material such as roadway debris or bird droppings accumulate on a metal surface.
- dissimilar metal corrosion**—See **galvanic corrosion**.
- electrolysis**—Corrosion caused by exposure to stray electrical currents.
- electrolyte**—A substance containing ions that travel from anode to cathode in a corrosion cell (on bridges, the electrolyte is usually water).
- erosion corrosion**—A form of corrosion caused by fluid flow over a surface which continually wears away the protective corrosion product, accelerating the corrosion rate.
- fretting corrosion**—A form of corrosion caused by the relative motion of two nonlubricated contacting surfaces under load. The motion wears away protective corrosion products on the surfaces, accelerating the corrosion rate.
- galvanic corrosion**—A form of corrosion caused by current flow created when two metals with different corrosive potentials are electrically connected in the presence of an electrolyte.
- galvanic series**—A list of metals arranged in order of their corrosion potential in a specific environment.
- galvanizing**—A form of surface protection by sacrificial zinc plating applied to iron or steel.
- general corrosion**—See **uniform corrosion**.
- hydrogen embrittlement**—A loss of ductility of a metal resulting from absorption of hydrogen.
- intergranular corrosion**—A form of corrosion that attacks the boundaries of the grains comprising a metal, leading to the metal's disintegration.
- ion**—A charged atom of an element (the charge comes from gain or loss of electrons compared to the atom's usual state). An anion is a negatively charged ion. A cation is a positively charged ion.
- linear polarization (polarization resistance)**—A technique of measuring uniform corrosion rate, utilizing the fact that the slope of the polarization curve at 10 to 20 mV polarization is generally linear and is proportional to the corrosion rate.
- localized corrosion**—Corrosion attack which is limited to a certain area of a member.
- magnetic testing**—The use of magnetic lines of force to detect defects in metals by observing the changes in the force lines at the defects.
- metal ion concentration cell**—A concentration cell in which the corrosion reaction is driven by differences in concentration of metal ions in the electrolyte.
- mill scale**—An oxide layer on metals (usually iron or steel) produced by metal rolling, welding, or heat treatment.
- optical scanning**—Use of light wave interference techniques to measure minute depths of roughness caused by wear, impact, stress, strain, or corrosion.
- oxygen concentration cell**—A concentration cell in which the corrosion reaction is driven by differences in concentration of oxygen ions in the electrolyte.
- pack rust**—A form of crevice corrosion in which rust accumulates between two metal surfaces. This can produce enough pressure to distort the surrounding metal.
- pickling**—An electrochemical process used to remove mill scale and corrosion products from metal surfaces.
- pitting**—A form of local corrosion which causes the creation of penetrations into the metal surface.
- polarization**—The deviation from the open circuit potential (corrosion potential) of an electrode resulting from the passage of current through it.
- preferential attack**—See **selective leaching**.
- profilometer**—A device used to measure the depth and/or width of surface irregularities.
- radiographic testing**—A method of nondestructive evaluation that is based on differential absorption of penetrating radiation, either electromagnetic radiation of very short wavelength or particulate radiation, by the object being inspected.
- rust**—The common corrosion product of iron and steel, formed by the chemical reaction of iron oxide and water.
- sacrificial protection/sacrificial anodes**—Sacrificial metals that are attached to or plated on other metals to be protected. The sacrificial metal acts as the anode in the corrosion reaction and corrodes, protecting the other metal, which acts as a cathode.
- selective leaching**—A form of corrosion that attacks one component of an alloy, changing the material properties of the alloy.
- stray current corrosion**—Corrosion resulting from direct current flow through paths other than the intended circuit.

stress corrosion cracking—Cracking of a metal produced by the combined action of corrosion and tensile stress.

surface preparation—Cleaning of a metal surface prior to coating, using pickling, blasting, brushing, or other techniques.

Tafel slope—The slope of the linear portion of a polarization curve plotted using semilogarithmic coordinates, where the polarization is proportional to the applied current.

telluric current—A natural electric current flowing near the earth's surface.

thermographic testing—A method using heat sensing devices or substances to detect irregular temperatures. Thermography is the mapping of areas of equal temperatures over a test surface.

ultrasonic testing—A nondestructive method using high-frequency sound waves that are introduced into a material to detect flaws or thickness.

underfilm corrosion—A type of crevice corrosion that attacks a metal surface beneath its paint coating.

uniform corrosion—A form of corrosion characterized by a general thinning of metalwork in a uniform fashion.

weather resistance—A general term used to describe the ability of a material to resist attack from its environment. It is often used to describe corrosion resistance.

weld decay—A type of intergranular corrosion that attacks metals in zones affected by heat during welding.

x-ray testing—A form of radiographic testing using electromagnetic radiation of very short wavelength.

APPENDIX B THROUGH APPENDIX G

(See Note under Appendix I, p. 140)

APPENDIX H

REFERENCES AND BIBLIOGRAPHY

1. CORROSION OF STEEL

- 1.1 AMERICAN SOCIETY FOR TESTING AND MATERIALS, G15-86, "Standard Definitions of Terms Relating to Corrosion and Corrosion Testing." (1986)
- 1.2 FONTANA, M. G., *Corrosion Engineering*, McGraw-Hill, Inc., New York (1986).
- 1.3 NATIONAL ASSOCIATION OF CORROSION ENGINEERS, *Forms of Corrosion—Recognition and Prevention*. Dillon, C. P. Ed., Houston, Texas (1982).

- 1.4 PLUDEK, V. R., *Design and Corrosion Control*. Halsted Press, New York (1977).
- 1.5 SCHWEITZER, P. A., Ed., *Corrosion and Corrosion Protection Handbook*. Marcel Dekker, Inc., New York (1983).
- 1.6 UHLIG, H. H. and REVIE, R. W., *Corrosion and Corrosion Control*. 3rd Ed., John Wiley & Sons, Inc., New York (1985).
- 1.7 VAN DELINDER, L. S., Ed., *Corrosion Basics*. National Association of Corrosion Engineers, Houston, Texas (1984).

2. CORROSION TYPES AND FACTORS AFFECTING RATES

- 2.1 BABOIAN, R. and TURCOTTE, R. C., "Development of Poulitce Corrosion Tests for Automobiles." *Materials Performance* (Dec. 1985) pp. 13–17.
- 2.2 COBURN, S. K., Ed., *Atmospheric Factors Affecting the Corrosion of Engineering Metals*. ASTM STP 646, American Society for Testing and Materials, Philadelphia (1978).
- 2.3 *Corrosion in Natural Environments*. ASTM STP 558, American Society for Testing and Materials, Philadelphia (1974).
- 2.4 CRAIG, H. L., ET AL., Eds., *Corrosion—Fatigue Technology*. ASTM STP 642, American Society for Testing and Materials, Philadelphia (1978).
- 2.5 CRAIG, J. R., Ed., *Stress-Corrosion—New Approaches*. ASTM STP 610, American Society for Testing and Materials, Philadelphia (1976).
- 2.6 CROOKER, T. W. and LEIS, B. N., Eds., *Corrosion Fatigue: Mechanics, Metallurgy, Electrochemistry, and Engineering*. ASTM STP 801, American Society for Testing and Materials, Philadelphia (1983).
- 2.7 DEAN, S. W. and RHEA, E. C., Eds., *Atmospheric Corrosion of Metals*. ASTM STP 767, American Society for Testing and Materials, Philadelphia (1982).
- 2.8 DEAN, S. W., ET AL., Eds., *Environment-Sensitive Fracture: Evaluation and Comparison of Test Methods*. ASTM STP 821, American Society for Testing and Materials, Philadelphia (1984).
- 2.9 DEXTER, S. C., Ed., *Biologically Induced Corrosion*. National Association of Corrosion Engineers, Houston (1986).
- 2.10 ERICSSON, P. and JOHANSSON, L. G., "The Role of NO₂ in the Atmospheric Corrosion of Different Metals." 10th Scandinavian Corrosion Congress, Stockholm (1986) pp. 43–48.
- 2.11 *Galvanic and Pitting Corrosion—Field and Laboratory Studies*. ASTM STP 576, American Society for Testing and Materials, Philadelphia (1976).
- 2.12 GROSS, H., "Examination of Salt Deposits Found Under German Painted Steel Bridge Decks." *Materials Performance* (Oct. 1983) pp. 28–33.
- 2.13 *Hydrogen Embrittlement Testing*. ASTM STP 533, American Society for Testing and Materials, Philadelphia (1974).
- 2.14 JOHANSSON, L. G., "The Corrosion of Steel in Atmospheres Containing Small Amounts of SO₂ and NO₂." 9th International Congress on Metallic Corrosion, Toronto (June 3–7, 1984) Vol. 1, pp. 407–411.
- 2.15 KOBELT, P. S., "New Alloy for Reduced Bridge Maintenance." *Public Works*, Vol. 101, No. 2 (Feb. 1986) p. 96.
- 2.16 *Localized Corrosion—Cause of Metal Failure*. ASTM STP

- 516, American Society for Testing and Materials, Philadelphia (1972).
- 2.17 LOGAN, H. L., *The Stress Corrosion of Metals*. John Wiley & Sons, Inc., New York (1966).
- 2.18 *Materials Evaluation Under Fretting Conditions*. ASTM STP 780, American Society for Testing and Materials, Philadelphia (1982).
- 2.19 NAEEMI, A. H. and ALBRECHT, P., "Atmospheric Corrosion Resistance of Weathering Steel." 9th International Congress on Metallic Corrosion Proceedings, Vol. 1, Toronto, Canada (June 1984).
- 2.20 PEABODY, A. W., *Control of Pipeline Corrosion*. National Association of Corrosion Engineers, Houston (1967).
- 2.21 UGIANSKY, G. M. and PAYER, J. H., Eds., *Stress Corrosion Cracking—The Slow Strain-Rate Technique*. ASTM STP 665, American Society for Testing and Materials, Philadelphia (1979).
- ### 3. CORROSION, BRIDGE AND COATINGS DETAILS
- 3.1 ASSHO, "Guide for Bridge Painting." Washington, D. C. (1962) 7 pp.
- 3.2 ABATE, R. F., "The Importance of Bridge Details in the Safety of Bridge Design." Presented at the ASCE Structures Congress, New Orleans (1986).
- 3.3 AMERICAN HOT-DIP GALVANIZERS ASSOCIATION, "Performance Report on Hot-Dip Galvanizing: Highways No. 1." (Mar. 1985).
- 3.4 AMERICAN IRON AND STEEL INSTITUTE, *Handbook of Corrosion Protection for Steel Pile Structures in Marine Environments* (1981) pp. 3-51, 121-130, and 185-198.
- 3.5 ANON, "Repainting Steel Bridges." *Better Roads*, Vol. 55, No. 5 (May 1985), pp. 30-32.
- 3.6 APPLEMAN, B. R., "Maintenance Repainting of Structural Steel: Chemistry and Criteria." *J. Protective Coatings and Linings* (Aug. 1984) pp. 20-29.
- 3.7 COBURN, S. K., "Cost-Effectiveness of Hot-Dip Galvanizing for Exposed Steel." *NCHRP Synthesis of Highway Practice 112* (1984) pp. 3-7.
- 3.8 "The Corrosion Protection of Steel Structures." *Steel Construction—A Supplement to Civil Engineering* (Nov. 1985) pp. 42-43.
- 3.9 DEANIN, R. D. and FRONDISOU-YANNAS, S., "Coatings for Hand-Cleaned Structural Steel." *Proc., Second World Congress—Coating Systems for Bridges*, University of Missouri-Rolla (Oct. 26-27, 1982) pp. 63-71.
- 3.10 EDWARDS, A., "Wax Barrier Coatings for Bridges." *Paint and Resin* (Feb. 1985) pp. 22-23.
- 3.11 FRONDISOU-YANNAS, S., "Coating and Corrosion Costs of Highway Structural Steel." *Report No. FHWA-RD-79-121*, Federal Highway Administration, Washington, D. C. (Mar. 1980) 103 pp.
- 3.12 GAUSMAN, R., "FHWA and New Materials for Painting Highway Structures." *J. Protective Coatings and Linings* (Mar. 1986) pp. 32-37.
- 3.13 GUTTMAN, H., "Zinc Coated Highway Bridges in the Province of Quebec—An Update." Presented at The American Hot-Dip Galvanizers Association Seminar of Performance of Galvanized Products, Chicago, Illinois (Sept. 17, 1980).
- 3.14 HARE, C., "Factors in Selection of Anti-Corrosive Coating Systems for Bridge Maintenance." *Proc., Second World Congress—Coating Systems for Bridges*, University of Missouri-Rolla (Oct. 26-27, 1982).
- 3.15 HARE, C. H., "Factors Involved in Selection of Anti-Corrosive Coating Systems for Bridge Maintenance—Part 1." *American Paint and Coatings J.* (Feb. 21, 1983) pp. 40-47.
- 3.16 HARE, C. H., "Factors Involved in Selection of Anti-Corrosive Coating Systems for Bridge Maintenance—Part 2." *American Paint and Coatings J.* (Feb. 28, 1983) pp. 40-49.
- 3.17 HARE, C. H., "Specific Utility in the Design of Coating Systems for Steel Bridges." *J. Protective Coatings and Linings* (Oct. 1984) pp. 18-25.
- 3.18 HARE, C. H. and FERNALD, M. G., "Modified Wax Barrier Coatings—Their Use on Bridge Structures (Part 1)." *American Paint and Coatings J.* (Feb. 20, 1984) pp. 39-43.
- 3.19 HARE, C. H. and FERNALD, M. G., "Modified Wax Barrier Coatings—Their Use on Bridge Structures (Part 2)." *American Paint and Coatings J.* (Feb. 27, 1984) pp. 37-44.
- 3.20 HAYNIE, F. H., ET AL., "Effects of Air Pollutants on Weathering Steel and Galvanized Steel: A Chamber Study." *Atmospheric Factors Affecting the Corrosion of Engineering Metals*, ASTM STP 646, S. K. Coburn, Ed. American Society for Testing and Materials (1978) pp. 30-47.
- 3.21 HENDRY, C. M., "Paint Research Institute Paint Corrosion Inhibitor Research." *New Concepts for Coating Protection of Steel Structures*, ASTM STP 841. D. M. Berger and R. F. Wint, Eds., American Society for Testing and Materials (1984) pp. 113-117.
- 3.22 IGETOFT, L., "Relation Between Surface Cleanliness and Durability of Anti-Corrosive Paint Systems on Structural Steel." *Proc., Second World Congress—Coating Systems for Bridges*, University of Missouri-Rolla (Oct. 26-27, 1982).
- 3.23 KEANE, J. D., "Protective Coatings for Highway Structural Steel." *NCHRP Report 74* (1969).
- 3.24 KOBELT, P. S., "Coating System for Extended Bridge Life." *Public Works*, Vol. 101, No. 1 (Jan. 1986) p. 86.
- 3.25 LALLY, A. and MILEK, W. A., "Bridge Construction Details." *AISC Eng. J.* (Jan. 1969).
- 3.26 LEGAULT, R. A. and PEARSON, V. P., "Kinetics of the Atmospheric Corrosion of Galvanized Steel." *Atmospheric Factors Affecting the Corrosion of Engineering Metals*, ASTM STP 646, S. K. Coburn, Ed., American Society for Testing and Materials (1978) pp. 83-96.
- 3.27 LEYLAND, D. S., "Coating System Used by State Highway Departments." *J. Protective Coatings and Linings* (May 1985) pp. 20-29.
- 3.28 LONGO, F. N. and DURMANN, G. J., "Corrosion Prevention with Thermal-Sprayed Zinc and Aluminum Coatings." *Atmospheric Factors Affecting the Corrosion of Engineering Metals*, ASTM STP 646, S. K. Coburn, Ed., American Society for Testing and Materials (1978) pp. 97-114.
- 3.29 LOUREIRO, R. A. and ENGLLOT, J. M., "Program and Methodology for Replacing Rivets, Lacing Bars and Tie Plates on Long Span Steel Bridges." *Proc., 5th Annual International Bridge Conference*, Pittsburgh, Pa. (1988).
- 3.30 LWIN, M. M., "Zinc-Plus-Paint System for Corrosion Protection of a Steel Bridge." *New Concepts for Coating Protec-*

- tion of Steel Structures, ASTM STP 841, D. M. Berger and R. F. Wint, Eds., American Society for Testing and Materials (1984) pp. 53–62.
- 3.31 MCCLURE, W. R., "The Nation's First Galvanized Steel Bridge Has Already Paid for Itself Through Savings in Maintenance After Eleven Years of Service." St. Joe Zinc Company, Pittsburgh, Pennsylvania (1978).
- 3.32 MELVIN, J. S., "Evaluation of Some Materials and Coatings for Utility Pole Line Hardware." *Corrosion*, Vol. 17 (Apr. 1961) pp. 14–19.
- 3.33 *Metallizing With Zinc*. The Platt Bros. & Co., Waterbury, Connecticut.
- 3.34 MONCARZ, P. D., OSTERAAS, J. D., and WOLF, J., "Designing for Maintainability." *Civil Engineering*, Vol. 56, No. 5 (June 1986) pp. 62–64.
- 3.35 OCCASIONE, J. F., BRITTON, T. C., and COLLINS, R. C., *Atmospheric Corrosion Investigation of Aluminum-Coated, Zinc-Coated, and Copper-Bearing Steel Wire and Wire Products: A Twenty-Year Report*. ASTM STP 585A, American Society for Testing and Materials (1984).
- 3.36 "Ohio Department of Transportation Continues Metallizing Program." *Metallize*, The Platt Bros. & Co., Waterbury, Connecticut (Sept. 1986).
- 3.37 PURVIS, R. L. and BERGER, R. H., "Bridge Joint Maintenance." *Public Works*, Vol. 14, No. 1 (Jan. 1983) pp. 30–32.
- 3.38 PURVIS, R. L. and BERGER, R. H., "Bridge Joint Maintenance." *Transportation Research Record 899* (1983) pp. 1–10.
- 3.39 ROSS, R. W. and ANDERSON, D. B., "Protection of Steel Piling in Marine Splash and Spray Zones—The Metallic Sheathing Concept." *Proc.*, 4th International Congress on Marine Corrosion and Fouling, Antibes (June 1976).
- 3.40 SKIPSEY, E. T., "Greater Protection from Paint Systems." *Civil Engineering Incorporating Consulting Engineer International* (Sept. 1985) pp. 40–42.
- 3.41 SMITH, L., "FHWA Coatings Research Program." *J. Protective Coatings and Linings* (Oct. 1984) pp. 26–37.
- 3.42 STAVROS, A. J., "Galvalume Wire." *Wire J. International* (July 1982) pp. 66–71.
- 3.43 STEEL STRUCTURES PAINTING COUNCIL, *Systems and Specifications—Steel Structures Painting Manual*. Vol. 2, 2nd Ed. (1982) pp. 27–32.
- 3.44 "Throgs Neck Bridge Metallizing Project." *Metallize*, The Platt Bros. & Co., Waterbury, Connecticut (No Date).
- 3.45 WOODS, A. L., "The Evolution of Bridge Paints in California." *J. Protective Coatings and Linings* (Oct. 1984) pp. 38–43.
- 3.46 ZOCCOLA, J. C., ET AL., "Atmospheric Corrosion Behavior of Aluminum-Zinc Alloy - Coated Steel." *Atmospheric Factors Affecting the Corrosion of Engineering Metals*, ASTM STP 646, S. K. Coburn, Ed. American Society for Testing and Materials (1978) pp. 165–184.
- control Methods." *Better Roads*, Vol. 53, No. 11 (Nov. 1983) pp. 18–22.
- 4.3 ANON, "Better Roads 1984 Bridge Inventory." *Better Roads*, Vol. 54, No. 11 (Nov. 1984) pp. 18–20.
- 4.4 ANON, "Better Roads 1985 Bridge Inventory." *Better Roads*, Vol. 55, No. 11 (Nov. 1985) pp. 23–25.
- 4.5 ANON, "Bridge Crack Raises Inspection Questions." *Engineering News-Record* (Sept. 6, 1984) pp. 10–11.
- 4.6 ANON, "Bridge Redeck Job Gets Lift." *Engineering News-Record* (Mar. 10, 1983) p. 14.
- 4.7 ANON, "Collapse Review Continues." *Engineering News-Record* (Sept. 22, 1983) pp. 11–12.
- 4.8 ANON, "Collapse's Cause Questioned." *Engineering News-Record* (Feb. 28, 1985) p. 12.
- 4.9 ANON, "Eyebars Span is Condemned." *Engineering News-Record*, Vol. 213, (Sept. 27, 1984).
- 4.10 ANON, "Failed Pin Assembly Dropped Span." *Engineering News-Record* (July 7, 1983) pp. 10–12.
- 4.11 ANON, "Fallen Span Had Design Ills." *Engineering News-Record* (Aug. 4, 1983) pp. 12.
- 4.12 ANON, "Fluctuating Stress, Corrosion Deteriorated Berlin Hall's Roof." *Engineering News-Record* (No Date Available).
- 4.13 ANON, "Four Drawbridges Undergo Rehabilitation." *Passenger Transport* (Sept. 8, 1986) p. 4.
- 4.14 ANON, "Girder Flaws Eyed in Span Collapse." *Engineering News-Record* (July 14, 1983) pp. 10–11.
- 4.15 ANON, "Hidden Bridge Decay Found." *Engineering News-Record* (Nov. 29, 1984).
- 4.16 ANON, "More Hanger Cracks Found in Connecticut Bridge." *Engineering News-Record* (Sept. 13, 1984) p. 11.
- 4.17 ANON, "Power Lines Shock Swedes." *Engineering News-Record* (Dec. 22, 1983).
- 4.18 ANON, "Rusted Parts Studied in Fatal Bridge Collapse." *The Philadelphia Inquirer* (ca. July 1, 1983).
- 4.19 ANON, "Rusted Supports Snap, Drop Old Bridge Span." *Engineering News-Record* (Nov. 10, 1983).
- 4.20 ANON, "Special Fasteners, Rehab Methods for Transit Bridge Track." *Railway Track and Structures* (Dec. 1984) pp. 20–25.
- 4.21 ANON, "Splicing in New Anchorages." *Engineering News-Record* (May 10, 1984).
- 4.22 ANON, "Stainless Steel Hangers Suspected." *Engineering News-Record* (May 16, 1985).
- 4.23 ANON, "State Blamed for Failure." *Engineering News-Record*, Vol. 213, No. 1 (July 5, 1984) p. 27.
- 4.24 ATKINSON, J., "Spanning the Gap." *American City & County*, Vol. 99, No. 2 (Feb. 1984) pp. 41–42, 45–47.
- 4.25 "Better Targeting of Federal Funds Needed to Eliminate Unsafe Bridges." *General Accounting Office (GAO) Report B-201433*, Washington, D. C. (Aug. 11, 1981) 131 pp.
- 4.26 "Bridges and Highways—Serious Deterioration but Being Attacked Aggressively." *Civil Engineering* (Nov. 1978) pp. 70–73.
- 4.27 BRODERICK, R. F., "Harvard Bridge Eyebars Failures." Proceedings of the Failure Analysis Program at the International Conference and Exposition of Fatigue, Corrosion Cracking, Fracture Mechanics and Failure Analysis: Salt Lake City, Utah (Dec. 2–6, 1985).
- 4.28 CHEYNEY, R., "Decaying Bridges Teach Valuable Lessons." *Public Works*, Vol. 101, No. 8 (Aug. 1986) pp. 50–52.

4. CASE STUDIES AND EXAMPLES

- 4.1 ANON, "A New Era in Bridge-Rebuilding." *Engineering News-Record* (Jan. 5, 1984) pp. 36–39.
- 4.2 ANON, "Aging Bridges Saved with Modern Corrosion Con-

- 4.29 "Collapse of US-35 Highway Bridge, Point Pleasant, West Virginia, December 15, 1967." *Report NTSP-HAR-71-1*, National Transportation Safety Board, Washington, D.C. (1970).
- 4.30 ESCALANTE, E., ET AL., "Corrosion and Protection of Steel Piles in a Natural Seawater Environment." *NBS Monograph 158* (June 1977).
- 4.31 FISHER, J. W., *Fatigue and Fracture in Steel Bridges—Case Studies*. John Wiley & Sons, Inc. (1984) pp. 1–40.
- 4.32 "Frozen Bearings Crack Bridge." *Engineering News-Record* (Aug. 21, 1975) p. 8.
- 4.33 "Highway Corrosion Problems." *Corrosion*, Vol. 17 (Apr. 1971) pp. 22–37.
- 4.34 HILL, G. V., "Tower Bridge; Keeping Corrosion at Bay." *Materials Performance* (Apr. 1985) pp. 78–79.
- 4.35 HOPWOOD, T. and HAVENS, J. H., "Corrosion of Cable Suspension Bridges." *Research Report UKTRP-84-84*, Kentucky Transportation Research Program, University of Kentucky (May 1984) 70 pp.
- 4.36 HSIONG, W., "Repair of Poplar Street Complex Bridges in East St. Louis." *Transportation Research Record 664: Bridge Engineering, Volume 1*, Transportation Research Board, National Research Council, Washington, D.C. (1978).
- 4.37 HSIONG, W., "Repair of Poplar Street Complex Bridges in East St. Louis." *Transportation Research Record 664*, National Research Council, Washington, D.C. (1978) pp. 110–119.
- 4.38 KLINE, E. S. and CORBETT, W. D., "Upgrading Field-Aged Alkyd Bridge Coating Systems." *Public Works*, Vol. 101, No. 11 (Nov. 1986) pp. 42–44, 82–83.
- 4.39 KUMAR, A., ET AL., "Corrosion Control of Pilings in Seawater: Buzzards Bay, MA." Paper No. 345, *Corrosion/84*, New Orleans Hilton, New Orleans, Louisiana (Apr. 2–6, 1984).
- 4.40 LOSHINSKY, M., "What's New in Aerial Structures Renovation and Replacement." *APTA 1982 Rapid Transit Conference*, Cleveland, Ohio (June 14–17, 1982).
- 4.41 MCGOGNEY, C. H., "Bridge Safety: Finding Future Flaws Today." *American City & County*, Vol. 100, No. 7 (July 1985) pp. 28–30.
- 4.42 OTAOLA, J., JR., "Replacing Corroded Cables on a Cable-Stayed Bridge." *Civil Engineering* (Sept. 1982) pp. 78–80.
- 4.43 PARSONS, D., "Beam Cracks Close 39th Street Bridge." *Daily Press*, Newport News, Hampton, Virginia (June 15, 1978).
- 4.44 "Recabling Adds Life to Bridge." *Engineering News-Record* (Jan. 17, 1980) pp. 66–69.
- 4.45 ROBINSON, R., "Weathering Steel: Industry's Stepchild." *Civil Engineering*, Volume 58, No. 10 (Oct. 1988) pp. 42–45.
- 4.46 RUSSEL, J. R., "No Simple Answers to Bridge Questions." *Better Roads*, Vol. 54, No. 11 (Nov. 1984) pp. 12–13.
- 4.47 SMALLOWITZ, H., "Bridging the Gaps in Bridges." *Civil Engineering*, Vol. 56, No. 11 (Nov. 1986) pp. 43–45.
- 4.48 STRATFULL, R. F. and SEIM, C., "Corrosion of Steel Piles at the Richmond-San Rafael Bridge." California Department of Transportation (June 1979).
- 4.49 "Survey of Steel Piling Performance in Marine Environments." British Steel Corporation (1983).
- 4.50 TITTSWORTH, J. and ELDRIDGE, E., "Rusty Utility Pole Falls, Kills Boy, 11, on Bicycle." *Detroit News* (Oct. 9, 1984).
- 4.51 WAGNER, J. and FITZGERALD, J. H., "Cathodic Protection for Wharf Foundation Piles at the Port of Anchorage, Alaska." Paper No. 254, *Corrosion/79*, Atlanta Hilton, Atlanta, Georgia (Mar. 12–16, 1979).
- 4.52 WATSON, S. and STAFFORD, D., "Cables in Trouble." *Civil Engineering*, Volume 58, No. 4 (Apr. 1988) pp. 38–41.

5. INSPECTION AND EVALUATION OF STEEL BRIDGES

- 5.1 ALBRECHT, P. and GOUPING, X., "Fatigue Strength of Long-Term Weathered Rolled Beams." Maryland Department of Transportation (Mar. 1988).
- 5.2 ALBRECHT, P. and NAEEMI, A. H., "Performance of Weathering Steel in Bridges." *NCHRP Report 272*, Transportation Research Board, National Research Council, Washington, D.C. (June 1984).
- 5.3 ALBRECHT, P. and SIMON, S., "Fatigue Notch Factors for Structural Details." *J. Structural Division*, American Society of Civil Engineers, Vol. 107, No. ST 7 (July 1981).
- 5.4 AMERICAN SOCIETY OF CIVIL ENGINEERS, "Repair and Strengthening of Old Steel Bridges." 345 East 47th Street, New York, New York (1979).
- 5.5 ANON, "Inspection Image Still Tarnished." *Engineering News-Record*, Vol. 213, No. 24 (Dec. 13, 1984) pp. 28–34.
- 5.6 ANON, "Inspections Unify Bridge Strategies." *Better Roads*, Vol. 54, No. 11 (Nov. 1984) pp. 16–42.
- 5.7 ANON, "Other Countries Move to Improve Inspections." *Engineering News-Record*, Vol. 213, No. 24 (Dec. 13, 1984) p. 34.
- 5.8 BAKHT, B. and CSAGOLY, P. F., "Bridge Testing." *Research Report No. 79-10*, Ministry of Transportation and Communications of Ontario, Ontario, Canada (1979).
- 5.9 BAKHT, B., "Testing of Two Pinned-Connected Truss Bridges." *Research Report No. 86-06*, Ministry of Transportation and Communications of Ontario, Ontario, Canada (1986).
- 5.10 BAKHT, B. and MOSES, F., "Lateral Distribution Factors for Highway Bridges." *J. Structural Engineering*, American Society of Civil Engineers, Vol. 114, No. 8 (Aug. 1988) pp. 1785–1803.
- 5.11 BELLENOIT, J. R., YEN, B. T., and FISHER, J. W., "Stresses in Hanger Plates of Suspended Bridge Girders." *Transportation Research Record 950*, Transportation Research Board, National Research Council, Washington, D.C. (1984) pp. 20–23.
- 5.12 CSAGOLY, P. F. and BAKHT, B., "Lateral Buckling of Pony Truss Bridges." *Research Report No. 199*, Ministry of Transportation and Communications of Ontario, Ontario, Canada (1975).
- 5.13 CULP, J. D. and TINKLENBERG, G. L., "Interim Report on Effects of Corrosion on Bridges of Unpainted A588 Steel and Painted Steel Types." *Research Report No. R-1142*, Research Laboratory Section, Testing and Research Division, Michigan Transportation Commission, Lansing, Michigan (June 1980).
- 5.14 FISHER, J. W., BELLENOIT, J. R., DANIELS, J. H., and YEN, B. T., "High Cycle-Fatigue Behavior of Steel Bridge

- Details—A Final Report." *Fritz Engineering Laboratory Report 386-13*, Lehigh University (1982).
- 5.15 FISHER, J. W., HAUSAMMANN, H., SULLIVAN, M. D., and PENSE, A. W., "Detection and Repair of Fatigue Damage in Welded Highway Bridges." *NCHRP Report 206* June 1979) p. 62.
- 5.16 FISHER, J. W., MIKI, C., SLUTTE, R. G., MERTZ, D. R., and FRANK, W., "Fatigue Strength of Steel Pipe - Base Plate Connections." *Engineering Structures*, Vol. 5 (Apr. 1983).
- 5.17 FISHER, J. W., YEN, B. T., WANG, D., and MANN, J. E., "Fatigue and Fracture Evaluation for Rating Riveted Bridges." *NCHRP Report 302* (1987).
- 5.18 "Forum on Weathering Steel on Highway Bridges, Summary Report." FHWA, U.S. Department of Transportation, *Publication No. FHWA-PS-89-016* (June 1989).
- 5.19 GRIMM, C. T. and YURA, J. A., "Shelf Angles for Masonry Veneer." *J. Structural Engineering*, Vol. 115, No. 3 (Mar. 1989) pp. 509-525.
- 5.20 HEIDERSBACH, R., "Corrosion Performance of Weathering Steels." 66th Annual Meeting of the Transportation Research Board, Washington, D. C. (Jan. 1987).
- 5.21 HOPWOOD, T. and HAVENS, J. H., "Inspection, Prevention and Remedy of Suspension Bridge Cable Corrosion Problems." *Research Report UKTRP-84-15*, Kentucky Transportation Research Program, University of Kentucky (May 1984).
- 5.22 KAYSER, J. R. and NOWAK, A. S., "Capacity Loss Due to Corrosion in Steel-Girder Bridges." *J. Structural Division*, American Society of Civil Engineers, Vol. 115, No. 6 (June 1989) pp. 1525-1537.
- 5.23 KAYSER, J. R. and NOWAK, A. S., "Evaluation of Corroded Steel Bridges." *Proc.*, Structures Congress Related to Bridges and Transmission Line Structures, American Society of Civil Engineers, Orlando, Florida (Aug. 1987) pp. 35-36.
- 5.24 KAYSER, J. R. and O'NEIL, M. A., "Inspection of Steel Bridges for Corrosion." *Proc.*, 4th International Bridge Conference, Pittsburgh, Pennsylvania (June 1987) pp. 154-157.
- 5.25 LEDOUX, F. S., LEVINE A. S., and KAMP, R. N., "New York State Department of Transportation Bridge Inspection and Rehabilitation Design Program." *Transportation Research Record 899* (1983) pp. 35-43.
- 5.26 MCCRUM, R. L., ARNOLD, C. J., and DEXTER, R. P., "Current Status Report—Effects of Corrosion and Unpainted Weathering Steel Bridges." *Research Report No. R-1255*, Research Laboratory Section, Testing and Research Division, Michigan Transportation Commission, Lansing, Michigan (Feb. 1985).
- 5.27 MCGOGNEY, C. H., "Bridge Safety: Finding Future Flaws Today." *American City & County*, Vol. 100, No. 7 (July 1985) pp. 28-30.
- 5.28 MCKENZIE, M., "The Corrosion Performance of Weathering Steel in Highway Bridges." *TRRL Report No. 857*, Transport and Road Research Laboratory, Crowthorne, Berkshire, England (1978).
- 5.29 OUT, J. M. M., FISHER, J. W., JEN, B. T., "Fatigue Strength of Weathered and Deteriorated Riveted Members." *USDOT/OST/P-34/85/016*.
- 5.30 REED, F. O. and KENDRICK, C. B., "Evaluation of Weathering Effects on Structural Steel." Office of Transportation Laboratory, California Department of Transportation, Sacramento, California (Mar. 1982).
- 5.31 SCHELLING, D. R. and CHUNG, C. FU, "Comparison of Bridge Rating Methods." *J. Structural Division*, American Society of Civil Engineers, Vol. 110, No. 7 (July 1984) pp. 1447-1466.

6. DESIGN CODES AND GUIDELINES

- 6.1 AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS, *AASHTO Manual for Bridge Maintenance*. Washington, D.C. (1976) 251 pp.
- 6.2 AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS, *Manual for Maintenance Inspection of Bridges*, 1983 As amended by Interim, 1988, Washington, D.C.
- 6.3 AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS, *Standard Specifications for Highway Bridges*. Thirteenth Edition, 1983, As amended by the Interim Specifications—Bridges, 1988, Washington, D.C.
- 6.4 AMERICAN INSTITUTE OF STEEL CONSTRUCTION, *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings*. AISC, New York (1978).
- 6.5 AMERICAN RAILWAY ASSOCIATION, *Specifications for Steel Railway Bridges* (1950).
- 6.6 BRITISH STANDARDS INSTITUTION, "British Standard 5400—Parts 1 through 10." London (1978-1984).
- 6.7 FEDERAL HIGHWAY ADMINISTRATION, U.S. Department of Transportation, *Bridge Inspector's Training Manual 70*. Washington, D.C. (1979).
- 6.8 FEDERAL HIGHWAY ADMINISTRATION, U.S. Department of Transportation, *Inspection of Fracture Critical Members*. Washington, D.C. (1986).
- 6.9 KORETZKY, H. P., PATEL, K. R., and WASS, G., "Pennsylvania Structure Inventory Record System: SIRS." *Transportation Research Record 899* (1983) pp. 43-52.
- 6.10 MOSES, F. and VERMA, D., "Load Capacity Evaluation of Existing Bridges." *NCHRP Report 301*, Transportation Research Board, National Research Council, Washington, D.C. (Dec. 1987).
- 6.11 "National Bridge Inspection Standards." Code of Federal Regulations, Title 23, Chapter 1, Part 650, Subpart C (May 1, 1983) 3 pp.
- 6.12 ONTARIO MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, *Ontario Highway Bridge Design Code*, Toronto, Ontario, Canada (1983).
- 6.13 PENNSYLVANIA DEPARTMENT OF TRANSPORTATION, "Structure Inventory Record System Coding Manual." (Sept. 1981).
- 6.14 U. S. DEPARTMENT OF TRANSPORTATION, Federal Highway Administration, "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges." (1979).

7. BRIDGE ANALYSIS

- 7.1 BAKHT, B. and JAEGER, L. G., "Bearing Restraint in Slab-on-Girder Bridges." *J. Structural Engineering*, American

- Society of Civil Engineers, Vol. 114, No. 12 (Dec. 1988) pp. 2724–2740.
- 7.2 BJORHOVDE, R. and CHAKRABARTI, S. K., "Tests of Full-Size Gusset Plate Connections." *J. Structural Engineering*, American Society of Civil Engineers, Vol. 111 No. 3 (Mar. 1985) pp. 667–684.
 - 7.3 BLEICH, F., *Buckling Strength of Metal Structures*. McGraw-Hill Book Company, New York (1952).
 - 7.4 BOWER, J. E., "Design of Beams with Web Openings." *J. Structural Division*, American Society of Civil Engineers, Vol. 94 No. ST3 (Mar. 1968) pp. 783–807.
 - 7.5 BOWER, J. E., "Elastic Stresses Around Holes in Wide-Flange Beams." *J. Structural Division*, American Society of Civil Engineers, Vol. 90 No. ST3 (June 1964) pp. 85–101.
 - 7.6 BOWER, J. E., "Experimental Stresses in Wide-Flange Beams with Holes." *J. Structural Division*, American Society of Civil Engineers, Vol. 92 No. ST5 (Oct. 1966) pp. 167–186.
 - 7.7 BOWER, J. E., "Ultimate Strength of Beams with Rectangular Holes." *J. Structural Division*, American Society of Civil Engineers, Vol. 94 No. ST6 (June 1968) pp. 1315–1337.
 - 7.8 BROWN, C. J., YETTRAM, A. L., and BURNETT, M., "Stability of Plates with Rectangular Holes." *J. Structural Engineering*, American Society of Civil Engineers, Vol. 113 No. 5 (May 1987) pp. 1111–1116.
 - 7.9 BURKE, M. P., "Bridge Approach Pavements, Integral Bridges, and Cycle-Control Joints." *Transportation Research Record 1113*, Transportation Research Board, National Research Council, Washington, D.C. (1987) pp. 54–65.
 - 7.10 CHAN, P. W., and REDWOOD, R. G., "Stresses in Beams with Circular Eccentric Holes." *J. Structural Division*, American Society of Civil Engineers, Vol. 100 No. ST1 (Jan. 1974) pp. 231–248.
 - 7.11 CHURCHWARD, A. and SOKAL, Y. J., "Prediction of Temperatures in Concrete Bridges." *J. Structural Division*, American Society of Civil Engineers, Vol. 107, No. ST11 (Nov. 1981) pp. 2163–2176.
 - 7.12 CLAWSON, W. and DARWIN, D., "Strength of Composite Beams at Web Openings." *J. Structural Division*, American Society of Civil Engineers, Vol. 108 No. ST3 (Mar. 1982) pp. 623–640.
 - 7.13 CLAWSON, W. and DARWIN, D., "Tests of Composite Beams with Web Openings." *J. Structural Division*, American Society of Civil Engineers, Vol. 108 No. ST1 (Jan. 1982) pp. 145–162.
 - 7.14 COWPER, G. R., "The Shear Coefficient in Timoshenko's Beam Theory." *Transactions*, American Society of Mechanical Engineers, Series E, Vol. 33 (June 1966) pp. 334–340.
 - 7.15 CSAGOLY, P. F. and JAEGER, L. G., "Multi-Load-Path Structures for Highway Bridges." *Transportation Research Record 711*, National Research Council, Washington, D.C. (1979) pp. 34–39.
 - 7.16 DALAL, S. T., "Some Non-Conventional Cases of Column Design." American Institute of Steel Construction, *Engineering J.*, Volume 6, No. 1 (Jan. 1969) pp. 28–39.
 - 7.17 DARWIN, D. and DONAHEY, R. C., "LRFD for Composite Beams with Unreinforced Web Openings." *J. Structural Engineering*, American Society of Civil Engineers, Vol. 114 No. 3 (Mar. 1988) pp. 535–552.
 - 7.18 DILGER, W., BEAUCHAMP, J. C., CHEUNG, M. S., and GHALI, A., "Field Measurements of Muskwa River Bridge." *J. Structural Division*, American Society of Civil Engineers, Vol. 107, No. ST11 (Nov. 1981) pp. 2147–2161.
 - 7.19 DOUGLAS, T. R. and GAMBRELL, S. C. JR., "Design of Beams with Off-Center Web Openings." *J. Structural Division*, American Society of Civil Engineers, Vol. 100 No. ST6 (June 1974) pp. 1189–1203.
 - 7.20 EMANUEL, J. H. and HULSEY, J. L., "Temperature Distribution in Composite Bridges." *J. Structural Division*, American Society of Civil Engineers, Vol. 104, No. ST1 (Jan. 1978) pp. 65–78.
 - 7.21 EMANUEL, J. H., and LEWIS, D. B., "Abutment-Thermal Interaction of a Composite Bridge." *J. Structural Division*, American Society of Civil Engineering, Vol. 107, No. ST11, *Proc. Paper 16631* (Nov. 1981) pp. 2111–2126.
 - 7.22 EMANUEL, J. H. and TAYLOR, C. M., "Length-Thermal Stress Relations for Composite Bridges." *J. Structural Division*, American Society of Civil Engineers, Vol. 111, No. 4 (Apr. 1985) pp. 788–804.
 - 7.23 EMANUEL, J. H., and WISCH, D. J., "Thermal Response of a Continuous Two-Span Composite Bridge Structure." *Transportation Research Record 711*, Transportation Research Board, National Research Council, Washington, D.C. (1979) pp. 40–46.
 - 7.24 FENG, Y. S., and MOSES, F., "Optimum Design, Redundancy and Reliability of Structural Systems." *Computing and Structures*, 24(2) (1986) pp. 239–254.
 - 7.25 FRANGOPOL, D. M. and CURLEY, J. P., "Effects of Damage and Redundancy on Structural Reliability." *J. Structural Division*, American Society of Civil Engineers, Vol. 113, No. 7 (July 1987) pp. 1533–1541.
 - 7.26 FRANGOPOL, D. M. and GOBLE, G. G., "Development of a Redundancy Measure for Existing Bridges." *Bridge Evaluation, Repair and Rehabilitation*, A. S. Nowak, and E. Absi, Eds., Univ. of Michigan, Ann Arbor, Michigan (1987) pp. 365–375.
 - 7.27 FROST, R. W. and LEFFLER, R. E., "Fatigue Tests of Beams with Rectangular Web Holes." *J. Structural Division*, American Society of Civil Engineers, Vol. 97 No. ST2 (Feb. 1971) pp. 509–527.
 - 7.28 GREIMANN, L. F., and WOLDE-TINSAE, A. M., "Design Models for Piles in Jointless Bridges." *J. Structural Engineering*, American Society of Civil Engineers, Vol. 114, No. 6 (June 1988) pp. 1354–1371.
 - 7.29 GREIMANN, L. F., YANG, P. S., and WOLDE-TINSAE, A. M., "Nonlinear Analysis of Integral Abutment Bridges." *J. Structural Engineering*, American Society of Civil Engineers, Vol. 112, No. 10 (Oct. 1986) pp. 2263–2280.
 - 7.30 HEINS, C. P. and HOU, C. K., "Bridge Redundancy: Effects of Bracing." *J. Structural Division*, American Society of Civil Engineers, Vol. 106 No. ST6 (June 1980) pp. 1364–1367.
 - 7.31 HEINS, C. P., and KATO, H., "Load Redistribution of Cracked Girders." *J. Structural Division*, American Society of Civil Engineers, Vol. 108 No. ST8 (Aug. 1982) pp. 1909–1915.
 - 7.32 HEYWOOD, R. B., *Designing by Photoelasticity*. Chapman and Hall, Ltd. (1952).

- 7.33 HULSEY, J. L., and EMANUEL, J. H., "Environmental Stresses in Flexibly Supported Bridges." *Transportation Research Record 664*, Bridge Engineering, Vol. 1, National Research Council, Washington, D.C. (1978) pp. 262-270.
- 7.34 HUNT, B. and COOKE, N., "Thermal Calculations for Bridge Design." *J. Structural Division*, American Society of Civil Engineers, Vol. 101, No. ST9, *Proc.*, Paper 11545, (Sept. 1975) pp. 1763-1781.
- 7.35 IMBSEN, R. A. and VANDERSHAF, D. E., "Thermal Effects in Concrete Bridge Superstructures." *Transportation Research Record 950*, Second Bridge Engineering Conference, Vol. 2, Washington, D. C. (1984) pp. 101-113.
- 7.36 LOVEALL, C. L., "Jointless Bridge Decks." *Civil Engineering* (Nov. 1985) pp. 64-67.
- 7.37 NARAYANAN, R. and DER-AVANESSIAN, N. G. V., "Design of Slender Webs Having Rectangular Holes." *J. Structural Engineering*, American Society of Civil Engineers, Vol. 111 No. 4 (Apr. 1985) pp. 777-785.
- 7.38 PETERSON, R. E., *Stress Concentration Design Factors*. Wiley, New York (1975).
- 7.39 RAHMAN, F. and GEORGE, K. P., "Thermal Stress Analysis of Continuous Skew Bridge." *J. Structural Division*, American Society of Civil Engineers, Vol. 105, No. ST7, *Proc.*, Paper 14695 (July 1979) pp. 1525-1541.
- 7.40 RAHMAN, F. and GEORGE, K. P., "Thermal Stresses in Skew Bridge by Model Test." *J. Structural Division*, American Society of Civil Engineers, Vol. 106, No. ST1, *Proc.*, Paper 15112 (Jan. 1980) pp. 39-58.
- 7.41 REDWOOD, R. G., BARANDA, H., and DALY, M. J., "Tests of Thin-Webbed Beams with Unreinforced Holes." *J. Structural Division*, American Society of Civil Engineers, Vol. 104, No. ST3 (Mar. 1978) pp. 577-595.
- 7.42 REDWOOD, R. G. and POUMBOURAS, G., "Analysis of Composite Beams with Web Openings." *J. Structural Engineering*, American Society of Civil Engineers, Vol. 110 No. 9 (Sept. 1984) pp. 1949-1958.
- 7.43 REYNOLDS, J. C. and EMANUEL, J. H., "Thermal Stresses and Movements in Bridges." *J. Structural Division*, American Society of Civil Engineers, Vol. 100, No. ST1, *Proc.*, Paper 10275 (Jan. 1974) pp. 63-78.
- 7.44 ROARK, R. J., *Formulas for Stress and Strain*. 4th ed., McGraw-Hill Book Company (1965).
- 7.45 ROARK, R. J. and YOUNG, W. C., *Formulas for Stress and Strain*. McGraw-Hill, New York (1982).
- 7.46 SAVIN, G. N., *Stress Concentrations Around Holes*. Pergamon Press (1961).
- 7.47 SCHWENDEMAN, L. P. and HEDGREN, A. W., "Bolted Repair of Fractured I-79 Girder." *J. Structural Division*, American Society of Civil Engineers, Vol. 104 No. ST10 (Oct. 1978) pp. 1657-1670.
- 7.48 SEGNER, E. P. JR., "Reinforcement Requirements for Girders with Web Openings." *J. Structural Division*, American Society of Civil Engineers, Vol. 90 No. ST3 (June 1964) pp. 147-164.
- 7.49 SEONG, C. K., WARD, B. A., YEN, B. T., and FISHER, J. W., "Behavior of Truss Bridges as Three Dimensional Structures." *Proc.*, 1st International Bridge Conference, Pittsburgh, Pennsylvania (June 1984) pp. 203-207.
- 7.50 SUBCOMMITTEE ON BEAMS WITH WEB OPENINGS OF THE STRUCTURAL DIVISION, "Suggested Design Guides for Beams with Web Holes." *J. Structural Division*, ASCE, Vol. 97, No. ST 11 *Proc.*, Paper 8536 (Nov. 1971) pp. 2707-2728.
- 7.51 SWEENEY, R. A. P., "Importance of Redundancy in Bridge-Fracture Control." *Probabilistic Design, Redundancy, and Other Bridge Papers, Transportation Research Record 711*, Washington, D.C. (1979) pp. 23-30.
- 7.52 TANG, J. P. and YAO, J. T. P., "Evaluation of Structural Damage and Redundancy." *Effects of Damage and Redundancy on Structural Performance*, D. M. Frangopol, Ed., ASCE, New York, N.Y. (1987) pp. 47-62.
- 7.53 TASK COMMITTEE ON REDUNDANCY OF FLEXURAL SYSTEMS OF THE ASCE-AASHTO COMMITTEE ON FLEXURAL MEMBERS OF THE COMMITTEE ON METALS OF THE STRUCTURAL DIVISION, "State-of-the-Art Report on Redundant Bridge Systems." *J. Structural Engineering*, American Society of Civil Engineers, Vol. 111, No. 12 (Dec. 1985) pp. 2517-2529.
- 7.54 WASSERMAN, E. P., "Jointless Bridge Decks." *Engineering J*, American Institute of Steel Construction, Vol. 24, No. 3 (1987) pp. 93-100.
- 7.55 YAMAMOTO, K., AKIYAMA, N., and OKUMURA, T., "Elastic Analysis of Gusseted Truss Joints." *J. Structural Engineering*, American Society of Civil Engineers, Vol. 112 No. 12 (Dec. 1985) pp. 2545-2564.

APPENDIX I

NOTE

Appendixes B, E, and F contained in the report as submitted by the research agency are not published herein. Their titles are listed here for convenience of those interested in the subject area. Qualified researchers may obtain copies on loan by written request to the NCHRP, Transportation Research Board, 2101 Constitution Avenue, NW., Washington, D.C. 20418. The titles are: Appendix B—Steel Bridge Inspection Questionnaire; Appendix E—Description of Analytical Studies; Appendix F—Description of Laboratory and Field Studies.

Appendixes C, D, and G—entitled Field Inspection Guidelines, Office Evaluation of Corrosion Effects, and Recommended Specifications for Evaluation of Corrosion Effects in Steel Bridges, respectively, have been published here as Parts I, II, and III, respectively.

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