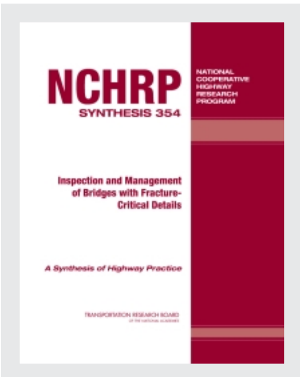

Inspection and Management of Bridges with Fracture-Critical Details

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP SYNTHESIS 354

Inspection and Management of Bridges with Fracture-Critical Details

A Synthesis of Highway Practice

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SUBJECT AREAS
Bridges, Other Structures, and Hydraulics and Hydrology and Maintenance

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TRANSPORTATION RESEARCH BOARD

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Finally and most importantly, Robert J. Connor and Hussam Mahmoud are forever indebted to their dear friend Dr. Robert J. Dexter who died suddenly in November 2004, during the preparation of the final version of this document. It is no overstatement to say that Robert was one of the kindest men we have had the privilege of knowing, both personally and professionally. We are better people for having known him and will cherish our memories of him for as long we live.

FOREWORD

By Staff
Transportation
Research Board

Highway administrators, engineers, and researchers often face problems for which information already exists, either in documented form or as undocumented experience and practice. This information may be fragmented, scattered, and unevaluated. As a consequence, full knowledge of what has been learned about a problem may not be brought to bear on its solution. Costly research findings may go unused, valuable experience may be overlooked, and due consideration may not be given to recommended practices for solving or alleviating the problem.

There is information on nearly every subject of concern to highway administrators and engineers. Much of it derives from research or from the work of practitioners faced with problems in their day-to-day work. To provide a systematic means for assembling and evaluating such useful information and to make it available to the entire highway community, the American Association of State Highway and Transportation Officials—through the mechanism of the National Cooperative Highway Research Program—authorized the Transportation Research Board to undertake a continuing study. This study, NCHRP Project 20-5, "Synthesis of Information Related to Highway Problems," searches out and synthesizes useful knowledge from all available sources and prepares concise, documented reports on specific topics. Reports from this endeavor constitute an NCHRP report series, *Synthesis of Highway Practice.*

This synthesis series reports on current knowledge and practice, in a compact format, without the detailed directions usually found in handbooks or design manuals. Each report in the series provides a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems.

PREFACE

This synthesis may be useful to bridge owners and consulting engineers engaged in the design, inspection, and management of bridges with fracture-critical details, as a guide to present specifications and engineering judgment. It focuses on the inspection and maintenance of bridges with fracture-critical members (FCMs), as defined in the *AASHTO LRFD Bridge Design Specifications*. The objectives of this report were to survey and identify gaps in the literature; determine practices and problems with how bridge owners define, identify, document, inspect, and manage bridges with fracture-critical details; and identify specific research needs. Among the areas examined were: inspection frequencies and procedures; methods for calculating remaining fatigue life; qualification, availability, and training of inspectors; cost of inspection programs; instances where inspection programs prevented failures; retrofit techniques; fabrication methods and inspections; and experience with FCM fractures and problems details.

This synthesis report of the Transportation Research Board contains information obtained from a survey distributed to bridge owners and consultant inspectors (72 state, provincial, and international departments of transportation and agencies), a literature search, and targeted interviews. Useful responses were received from 34 states and three Canadian provinces.

Robert J. Connor, Purdue University, West Lafayette, Indiana, Robert Dexter, University of Minnesota, Minneapolis, and Hussam Mahmoud, Lehigh University, Bethlehem, Pennsylvania, collected and synthesized the information and wrote the report, under the guidance of a panel of experts in the subject area. The members of the topic panel are acknowledged on the preceding page. This synthesis is an immediately useful document that records the practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As progress in research and practice continues, new knowledge will be added to that now at hand.

CONTENTS

INSPECTION AND MANAGEMENT OF BRIDGES WITH FRACTURE-CRITICAL DETAILS

SUMMARY

This report is focused on inspection and maintenance of bridges with fracture-critical members (FCMs). The *AASHTO LRFD Bridge Design Specifications* (*LRFD Specifications*) defines an FCM as a "component in tension whose failure is expected to result in the collapse of the bridge or the inability of the bridge to perform its function." Note that the FCM can refer to a component such as a flange of a girder and does not necessarily include the whole "member." Approximately 11% of the steel bridges in the United States have FCMs. Most of these (83%) are two-girder bridges and two-line trusses, and 43% of the FCMs are built-up riveted members.

The objectives of this synthesis project were to:

- Survey the extent of and identify gaps in the literature;
- Determine best practices and problems with how bridge owners define, identify, document, inspect, and manage bridges with fracture-critical details; and
- Identify research needs.

The information was obtained from the literature, from a survey of bridge owners and consultant inspectors, and from targeted interviews. Thirty-four states and three Canadian provinces responded to the survey. Information was gathered regarding how bridge owners define, identify, document, inspect, and manage bridges with fracture-critical details. Specific inquiries were made about the following issues:

- Inspection frequencies and procedures;
- Methods for calculating remaining fatigue life;
- Qualification and training of inspectors;
- Available and needed training;
- Experience with FCM fractures and problem details;
- Examples of where the inspection program prevented failures;
- Cost of inspection programs;
- Retrofit techniques, including emerging technologies;
- Nondestructive evaluation/nondestructive testing;
- International experience and practice;
- Fabrication methods and fabrication inspection; and
- Needed research related to fracture-critical bridges (FCBs).

This report will be useful to owners and consulting engineers engaged in the design, inspection, and management of bridges with fracture-critical details as a guide to present specifications and typical engineering judgment.

During the 1970s, the material, design, fabrication, shop inspection, and in-service inspection requirements were improved for steel bridges in general. In 1978, special provisions were implemented for FCMs, primarily in reaction to bridge collapses. These requirements were successful in transforming the industry and the design of modern bridges, so that fatigue and fracture are very rare in bridges built in the last 20 to 25 years (i.e., since 1980). Unfortunately, approximately 76% of all FCMs presently in inventory were fabricated before 1978.

The focus on inspection and maintenance is appropriate, because it was found that the extra fabrication and material costs were small in comparison with the additional costs of "hands-on" fracture-critical inspection mandated since 1988 by the FHWA's National Bridge Inspection Standards. The approximate initial cost premium for new bridges with FCMs is on the order of 8% of the cost of fabricated steel. There is also a hidden initial cost in some cases where more expensive superstructure designs have been used than are necessary to maintain an acceptable reliability level because of restrictions or more subtle prejudice against bridges with FCMs.

The major impact on life-cycle costs is the additional mandate for hands-on, in-service inspection of FCMs. The cost of the fracture-critical inspection is typically two to five times greater than inspections for bridges without FCMs. Inspections of closed sections such as tub girders and tie members are extremely expensive, because inspectors must get inside. Interestingly, what constitutes a fracture-critical inspection is often subject to interpretation and disagreement. The frequency of fracture-critical inspections is actually not currently specified in the National Bridge Inspection Standards and varies up to every 5 years, is typically every 2 years, but often is every year or more frequently if there are specific problems.

These hands-on inspections have revealed numerous fatigue and corrosion problems that otherwise might have escaped notice. Twenty-three percent of survey respondents indicated that they found significant cracks that could have become much worse, possibly averting collapses. Similar experiences may be found in the literature. The information from these inspections is useful in bridge management; that is, in prioritizing bridges for rehabilitation and replacement. Furthermore, states that have centralized teams that do all of the fracture-critical inspections report numerous advantages with this approach.

FCMs are nonredundant. The *LRFD Specifications* define redundancy as "the quality of a bridge that enables it to perform its design function in the damaged state." Note that these definitions are not clear about what type of damage, load type, magnitude, distribution on the bridge, dynamic amplification, and load factors are supposed to be resisted by the damaged structure.

Nonredundant is a broader term than FCM because nonredundant also includes:

- Substructures;
- Members that may be inherently not susceptible to fracture, such as compression members, but still could lead to collapse if damaged by overloading, earthquakes, corrosion, fire, terrorism, ship or vehicle collisions, etc.; and
- Members made of materials other than steel.

Substructures such as piers are often nonredundant and have contributed to most of the major collapses of both steel and concrete bridges.

In addition to prevention of collapse in the event of fracture, redundancy of the superstructure is important for several other reasons of varying levels of importance. The first is the need to more easily redeck the bridge, which can often be readily overcome by adding an additional line of stringers. It must be recognized that events other than fracture can also damage and completely destroy members of the superstructure. These are compelling reasons to have redundancy.

These reasons for redundancy (other than fracture) could be used to encourage redundancy outright instead of indirectly by penalizing FCMs. For example, redundancy is encouraged in Sections 1.3.2 and 1.3.4 of the *LRFD Specifications*. Load factors are modified based on the level of redundancy, and it is stated that multiple-load-path and continuous structures could be used unless there are compelling reasons to do otherwise.

In the AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating* (*LRFR*) *of Highway Bridges* (*LRFR Manual*), redundancy is reflected in system factors that reduce the capacity of each member in nonredundant systems. The system factors are calibrated so that nonredundant systems are rated more conservatively at approximately the same level of reliability associated with new bridges designed by the *LRFD Specifications,* called the "inventory" level in former rating procedures.

Redundancy is related to system behavior rather than individual component behavior. Redundancy is often discussed in terms of three types:

- Internal redundancy, also called member redundancy, can occur when a nonwelded member is comprised of multiple elements, and a fracture that formed in one element cannot propagate directly into the adjacent elements.
- Structural redundancy is external static indeterminacy and can occur in a two or more span continuous girder or truss.
- Load-path redundancy is internal static indeterminacy resulting from having three or more girders or redundant truss members. One can argue (and show analytically) that the transverse members such as diaphragms between girders can also provide load-path redundancy.

In preparing this synthesis, it was found that internal and structural redundancy are often neglected by agencies and designers. Retrofits are described, which have been used to add all three forms of redundancy to bridges with FCMs.

Ultimately, it is the target level of reliability that designers and raters of bridges should strive to achieve, rather than focusing exclusively on redundancy. Redundancy has a major impact on the risk of collapse and this impact is accounted for appropriately for all types of structures in both the *LRFD Specifications* and the *LRFR Manual.* Using the LRFD and LRFR procedures, it is possible to achieve the target level of reliability without redundancy in a bridge that is more conservatively designed by only approximately 17%. For example, a nonredundant bridge designed for an HS-25 loading would have greater reliability than a redundant bridge designed for HS-20 loading.

There are various classifications of superstructure types having FCMs and consequently there is wide disagreement. For example, twin box girders would be expected to perform even better than twin I-girders owing to the torsional capacity of the intact box girder and the alternate load paths available within each box girder; however, most agencies consider these as FCMs. Unfortunately, the *LRFD Specifications* are not clear about what types of superstructures have FCMs.

Common assumptions that fractures in certain superstructure types will lead to collapse are generally too simplistic. Many bridges have had a full-depth fracture of an FCM girder and did not collapse, usually owing to the alternative-load-carrying mechanism of catenary action of the deck under large rotations at the fracture. It is apparent that other elements of these two-girder bridges, particularly the deck, are sometimes able to carry the loads and prevent collapse in these fracture-critical bridges (FCBs). Indeed, these alternate load paths were so robust in some of these failures that there was little or no perceptible deformation of the structure.

The capacity of damaged superstructures (with the FCM "damaged" or removed from the analysis) may be predicted using refined three-dimensional analysis. However, there is a strong need to clarify the assumptions, extent of damage, load cases and factors, and dynamic effects in these analyses. The commentary of the *LRFD Specifications* states that

> The criteria for refined analysis used to demonstrate that part of a structure is not fracture critical has not yet been codified. Therefore, the loading cases to be studied, location of potential cracks, degree to which the dynamic effects associated with a fracture are included in the analysis, and fineness of the models and choice of element type should all be agreed upon by the owner and the engi-

> neer. The ability of a particular software product to adequately capture the complexity of the problem should also be considered and the choice of software should be mutually agreed upon by the owner and the engineer. Relief from full factored loads associated with the Strength I Load Combinations of Table 3.4.1-1 should be considered as should the number of loaded design lanes versus the number of striped traffic lanes.

NCHRP Report 406: Redundancy in Highway Bridge Superstructures, also gives practical requirements for estimating the residual capacity of the damaged superstructure.

This same type of analysis is being used to evaluate older structures, as well to better direct resources for maintenance and replacement; for example, if a bridge is not really fracture-critical then it may not be necessary to replace it as soon.

Interestingly, other countries do not make a distinction between FCM and non-FCM. Scanning tours have noted that three-dimensional refined analysis is more often used in the design and evaluation of bridges in Europe and that the inspection interval is often based on risk. There are some in the United States who believe that fracture-critical inspection should be less frequent for more modern bridges as a result of the much better detailing, materials, and fabrication used in their construction. It has been suggested that the inspection interval should be based on the level of truck traffic, fatigue details, and other risk factors.

Overall, training for inspectors appears to be adequately available through existing courses provided by the National Highway Institute. None of the agencies responding to the survey identified any problems with current educations strategies. However, one area that could require additional effort is related to the documentation and archiving of previous failures and problems. Failures known to have occurred in certain states were not reported in some of the survey responses. Hence, a better method of tracking, archiving, and disseminating such information appears to be needed.

Although training seems adequate related to inspection, many engineers have noted how the education and training related to fatigue and fracture is not. Engineers are reportedly not learning lessons from fatigue and fracture incidents because of inadequate understanding. There is concern that they are not able to predict future problem details. Better education in this area in engineering programs, as well as short courses for practicing engineers, could lead to improved new bridge designs and better educated engineers available to participate in maintenance and inspection programs.

CHAPTER ONE

INTRODUCTION

BACKGROUND

The inspection and maintenance of all types of bridges is critical to the safety of the public and often critical to the economy of a region. A bridge as defined in the *AASHTO LRFD Bridge Design Specifications* (*LRFD Specifications*) (*1*) is "a structure, including supports, erected over a depression or an obstruction . . . having an opening measured along the center of the roadway of more than 20 feet" (6 m). Failure of a bridge as a result of inadequate inspection and maintenance can lead to loss of life as well as incalculable user costs. To minimize the potential for such problems, significant public funds are spent inspecting and maintaining our nation's bridges. These funds should only be expended if

- There is a payoff in lower future maintenance costs or
- The reliability (a measure of the relative safety) of the bridge is inadequate and the inspection and maintenance will sufficiently improve the reliability.

SCOPE AND OBJECTIVE

The focus of this report is on the inspection and maintenance of bridges with fracture-critical members (FCMs). The *LRFD Specifications* (*1*) define an FCM as a "component in tension whose failure is expected to result in the collapse of the bridge or the inability of the bridge to perform its function." Slight variations of this definition can be found in the AASHTO/AWS-D1.5 *Bridge Welding Code* (*2*), the AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating* (*LRFR*) *of Highway Bridges* (*LRFR Manual*) (*3*), and FHWA's National Bridge Inspection Standards (NBIS) (*4*). Note that by this definition substructures and superstructure members made of concrete and other materials can also be classified as FCMs, although in practice, typically, only steel superstructure members are so classified.

Although the definition indicates that failure of an FCM may lead to collapse, it is not clear what loading would be necessary to cause the collapse. The definition leaves much to engineering judgment, and consequently there is disagreement about what type of members should be classified as FCMs, as is discussed in detail in chapter two.

The *LRFD Specifications* note that designers are required to clearly indicate FCMs in the contract documents. Note that although the term "member" is often used to refer to an entire girder, the definition of an FCM refers only to the critical tension element of a member. In the case of an 'I' or box girder, the tension flange and the web plate(s) would be FCMs, as well as the weld of the tension flange to the web. However, the compression flanges and the weld of the compression flanges to the webs would not be FCMs. According to the *LRFD Specifications,* longitudinal attachments welded to the FCM and greater than 4 in. (100 mm) in length in the longitudinal direction are also considered as FCMs. Bridges containing FCMs are commonly referred to as fracture-critical bridges (FCBs), although there are large parts of the bridge that are not fracture-critical.

FHWA has proposed that states create FCM inspection plans. There is therefore much interest in the current practices of various states for inspecting and managing FCBs. However, before the development of such a plan, a more comprehensive and unambiguous method of classifying bridges and members as fracture-critical must be developed to ensure consistent application of such inspection standards. There is also a need for other information related to FCMs that may be used to update the *LRFR Manual* and the current NBIS. Finally, for obvious reasons, many owners are also concerned about terrorist threats against FCBs.

Therefore, NCHRP initiated this synthesis project to focus on inspection and management of bridges with FCMs. The objectives of this synthesis project were to

- Survey and identify gaps in the literature;
- Determine practices and problems with how bridge owners define, identify, document, inspect, and manage bridges with fracture-critical details; and
- Identify research needs.

The scope of the project included studying

- Inspection frequencies and procedures;
- Methods for calculating remaining fatigue life;
- Qualification and training of inspectors;
- Available and needed training;
- Experience with FCM fractures and problem details;
- Examples of where inspection programs prevented failures;
- Cost of inspection programs;
- Retrofit techniques, including emerging technologies;
- Nondestructive evaluation (NDE);

- International experience and practice (largely from scanning tours);
- Fabrication methods and fabrication inspection; and
- How owners are dealing with fracture-critical details that cannot be easily inspected.

ORGANIZATION

The following chapter (chapter two) presents results from the literature review, including a brief summary of the development and impact of specifications related to fatigue and fracture of steel bridges, the performance of bridges in the event of fractures, and classification of FCMs. Additional detail on these subjects is also presented in Appendix A, along with details on fatigue life prediction, NDE, repair and retrofit techniques, and the impact of high-performance steel (HPS). In general, the literature review confirmed that research conducted to evaluate the robustness of bridges deemed to be fracture-critical demonstrated that most bridges have considerable reserve strength. Studies evaluated robustness using different methods including case histories of actual fractures that occurred in service, field testing, finite-element simulation, and reliability theory. An annotated bibliography of the most relevant research is included in Appendix D.

Chapter three reports on the results of the survey of bridge owners and consultant inspectors, and discusses targeted interviews. The survey was sent to 72 state DOTs, agencies, and provinces, such as those in Canada. Useful replies were received from 40 agencies. (It should be noted that more than 40 surveys were returned; however, a few surveys were not filled out or the agency did not have any FCBs in their inventory.) One of the objectives of this survey was to gather information on fracture-critical structures and how bridge owners define, identify, inspect, and manage FCBs. Information related to structural failures was gathered along with repair and retrofit policies and strategies. In addition, input was solicited from owners on their perceived research needs related to FCBs. Useful information was gained related to inspection, failures, classification of FCBs, and costs associated with field inspection.

Chapter four includes discussion of the findings, and chapter five provides the conclusions and proposes future research needs.

CHAPTER TWO

BACKGROUND

DEVELOPMENT OF SPECIFICATIONS FOR STEEL BRIDGES

In 1967, the Point Pleasant Bridge over the Ohio River (constructed in 1928 and also known as the Silver Bridge) collapsed, resulting in 46 deaths (Figure 1). The collapse was the result of a brittle fracture of one of the nonredundant eyebars supporting the main span (*5–7*). As discussed later in this chapter, although there is disagreement about what should be classified as an FCM, there is no doubt that this eyebar was an FCM. There are several reasons why this catastrophe was extraordinary and is not likely to be repeated. The small flaw in the eyebar may have been caused by stress-corrosion cracking (SCC) (*5,8*), which is discussed further in Appendix A. Stress-corrosion cracking should not occur in modern bridge steel; however, in this instance, the eyebar steel was 1928 vintage, heat-treated AISI 1060 steel, which is substantially different than today's steel. The fracture toughness of the eyebar was also marginal and a relatively small crack led to the brittle fracture of the eyebar, which in turn led to the collapse of the bridge. This collapse was the catalyst for many changes in material specifications, design, fabrication, shop inspection, in-service inspection, and maintenance of steel bridges.

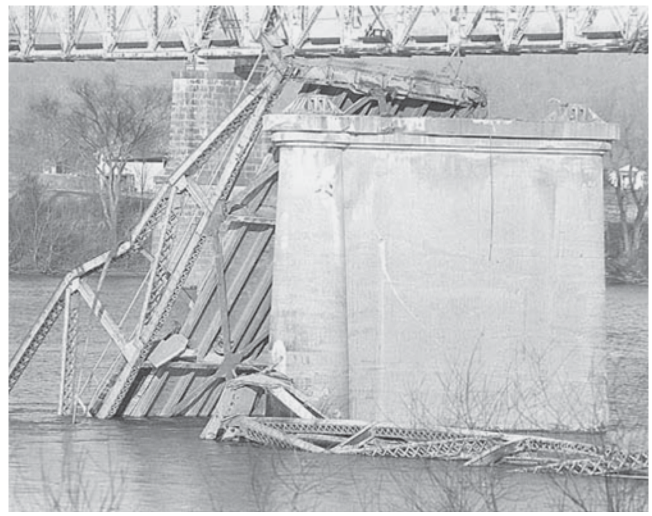


FIGURE 1 Collapse of Point Pleasant Bridge.

Material, Design, and Fabrication Specifications, and Effect of Bridge Design Date

In 1974, in part as a result of the Point Pleasant Bridge collapse, mandatory Charpy V-notch (CVN) toughness requirements were initiated for welds and base metal to ensure adequate resistance to fracture; that is, fracture toughness (*9,10*). The greater the CVN at a particular temperature, or the lower the temperature at which the CVN is required, the larger the critical crack size that can be tolerated at lowest anticipated service temperature without fracture. The present CVN requirements (for non-FCM) are essentially the same as the CVN requirements implemented in 1974. The CVN requirements were the result of significant debate and some compromise during their development, which is discussed further in Appendix A.

Presently, material selection, design, and fabrication of steel bridges are governed by

- *AASHTO LRFD Bridge Design Specifications* (*1*) and
- AASHTO/AWS-D1.5, *Bridge Welding Code* (*2*).

In addition to CVN requirements, these provisions restrict the choice of details as well as control weld flaws and other crack-like defects. These provisions have reshaped industry practices and result in an acceptably low probability of fatigue cracking and brittle fracture in new bridges.

However, many older steel bridges built before the implementation of modern fatigue design provisions in the mid-1970s possess poor fatigue details, such as cover plates that can develop fatigue cracks (Figure 2) (*11*), which if not repaired, can grow and lead to fracture of the member and possible collapse of part or all of the bridge.

Other factors that make these older bridges susceptible to fracture include:

- Marginal fracture toughness of the steel and weld metal;
- Detailing, fabrication quality, and shop inspection below modern standards;
- Severe corrosion problems, especially at open or failing expansion joints;
- Higher traffic volumes and truck weights than the bridge was originally designed to handle.

In light of these factors, periodic in-service inspection is particularly important for older bridges to provide an opportunity to detect cracks and corrosion before they grow to a critical size. In 1970, partly in reaction to the collapse of the

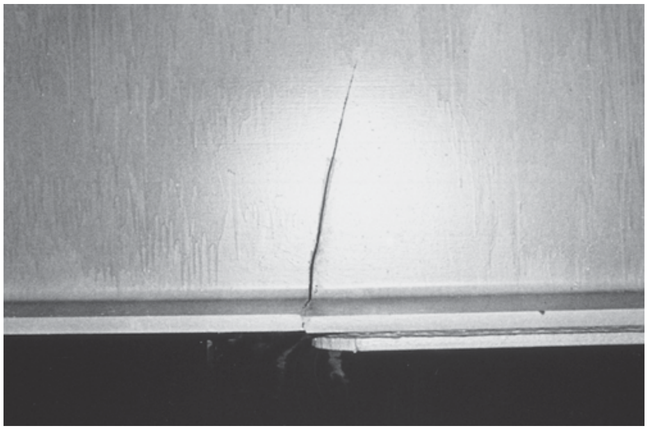


FIGURE 2 Development of fatigue crack at cover plate ends on the multibeam Yellow Mill Pond Bridge in Connecticut in 1976. (*Courtesy:* John W. Fisher.)

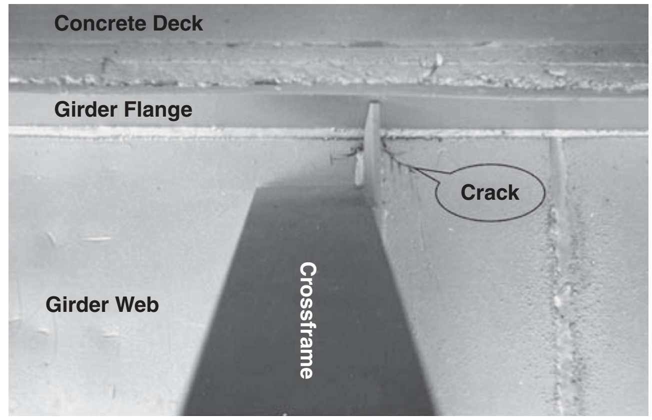


FIGURE 3 Typical web-gap fatigue cracking.

Point Pleasant Bridge over the Ohio River, the NBIS (*4*) were established. Title 23, Code of Federal Regulations, Part 650, Subpart C sets forth the NBIS for all bridges of more than 20 ft (6 m) span on all public roads. Section 650.3 specifies inspection procedures and frequencies, indicates minimum qualifications for personnel, and states reporting, inventory, load posting, and inspection recordkeeping requirements. The current NBIS mandates a 2-year inspection interval for all highway bridges carrying public roads.

However, modern steel bridges are not nearly as susceptible to fracture as older bridges (*12,13*). As a result, the ways modern bridges are managed could possibly be evaluated differently than older bridges. This could be studied further, with considerable potential benefits.

For example, problems with severe corrosion have been reduced. In the last 20 years, durability of weathering steel and coating systems has improved. Expansion joints have been improved if not eliminated through the use of continuous jointless bridges (*14*).

In addition, there have been few if any cases where weld defects or low-toughness steel has been an issue for modern steel bridges, owing primarily to improvements in details, fabrication practices, and fracture toughness of the steel and weld metal (*12,15*). If spontaneous fracture from weld defects is ruled out, then fracture can only occur if preceded by fatigue (*15*). Therefore, in this case, it is essentially sufficient to control fatigue to prevent fracture (*12,15*).

Distortion-induced fatigue cracking, discussed further in Appendix A, continued as a fatigue problem in typical plate girder bridges designed before the mid-1980s (*11,12, 15,16*). A common example of distortion-induced fatigue cracking is web-gap cracking, which occurs in the gap when a connection plate is not attached to a flange and is subject to out-of-plane distortion (Figure 3). This problem was corrected in 1985 by a change in AASHTO specifications that mandated the attachment of the connection plate to both flanges.

Hence, it is important to distinguish three different age ranges of steel bridges:

1. Steel bridges built before the implementation of modern fatigue design provisions in the mid-1970s.
2. Steel bridges designed after the mid-1970s, but before 1985, which have fewer fatigue problems but remain susceptible to distortion-induced fatigue.
3. Modern steel bridges designed after 1985 that should not be susceptible to fatigue at all.

Fatigue is virtually unheard of in modern steel bridges as a result of improved design specifications, more fatigue-resistant details, and improvements in shop inspection (*15,16*). Fatigue problems that have occurred in modern bridges were typically the result of unintended behavior or a design error that is not consistent with the intent of present specifications and usually manifest in the first few years of the life of the bridge. Although the bridges that are referred to herein as "modern" are less than 20 years old, there is substantial confidence that these structures will continue to perform with few problems resulting from fatigue.

As a final note, in changing from load factor design (LFD) to LRFD, two-girder bridges will be designed more conservatively than before. According to Dr. Dennis Mertz, in calibrating the LRFD specifications, loads were increased slightly to compensate for improved and less conservative distribution factors. However, the distribution factors for two-girder bridges were always reasonably accurate, so they did not get the benefit of improved distribution factors that multigirder bridges did. This should be kept in mind when considering the reliability of these two-girder and two-line truss systems.

Additional Material, Fabrication, and In-Service Inspection Requirements for FCMs and Cost Impact

FHWA led the development of a fracture control plan (FCP) to provide a higher level of safety for FCBs. In the broad sense, an FCP includes everything that affects the potential for fracture—in-service inspection and maintenance as well as design, fabrication, and shop inspection (*17*). The idea is that trade-offs can be made between components of the plan without compromising reliability. For example, better toughness could be required to compensate for relaxed in-service inspection standards, because better toughness would lead to a larger critical crack size that would be easier to see from a distance and that would take longer to develop. Efforts to make FCBs more conservative were largely the result of experiences with cracking in tied arches, mostly owing to large fabrication defects because of difficulty associated with welding A514 steel (*11*).

The American Iron and Steel Institute initiated a research project to develop an improved FCP for fabrication of nonredundant structures. This work (*9,17–21*) ultimately resulted in the 1978 publication of the AASHTO Guide Specification for Fracture Critical Non-Redundant Bridge Members (*22*). A key element was more stringent CVN requirements for base metal and weld metal for FCMs.

This Guide Specification has now been withdrawn. The CVN requirements for base metal, including the greater requirements for FCMs, are now included in the *AASHTO LRFD Bridge Design Specifications* (*1*), as well as the ASTM and AASHTO specifications for the steel (*23,24*). Most of the remaining material from the Guide Specification is now included in Section 12 of AASHTO/AWS-D1.5, "AASHTO/AWS Fracture Control Plan (FCP) for Non-Redundant Members" (*2*). Note that in AASHTO/AWS-D1.5 the definition of an FCP is narrower, including only fabrication and shop inspection—not base metal selection, in-service inspection, and maintenance. Therefore, these provisions will be referred to as a fabrication FCP. (Unfortunately, yet another completely different meaning for the term "fracture control plan" has arisen and that is the plan- or elevation-view drawing identifying all FCMs for use in in-service inspection.)

The differences between the provisions for the fabrication of FCMs in Section 12 and the provisions for non-FCMs elsewhere in AASHTO/AWS-D1.5 are primarily more strict fabrication and shop inspection requirements to control weld flaws and other crack-like defects in FCMs. For example, transverse groove welds are required to be inspected in the shop with both radiographic testing (RT) and ultrasonic testing (UT), whereas only RT is required for non-FCMs.

The fabrication FCP and the more stringent CVN requirements result in an even lower probability of brittle fracture in new FCMs than for typical non-FCM members. Note that this additional fracture reliability does not apply to older FCMs designed before implementation of the FCP in 1978 (*22*). The survey (described in chapter three) indicated that approximately 75% of FCBs in present inventory were designed before the FCP.

However, the fabrication provisions and the CVN requirements for the materials of the fabrication FCP increase costs. One major bridge fabricator reported that the approximate increase in initial costs for new FCMs relative to non-FCMs is on the order of 8% of the cost of fabricated steel.

In 1983, the Mianus River Bridge on I-95 in Connecticut (built in 1957) collapsed, killing three persons (Figure 4). Packout corrosion in a nonredundant pin and hanger assembly pushed one of the plates partly off the pin, eventually leading to a fatigue crack and collapse of the suspended span (between two cantilevers) (*16,25,26*). This event can be further attributed to poor maintenance, because a clogged drain was partly responsible for the packout corrosion. Throughout the country there are numerous other bridges with similar pin and hanger details; however, this type of suspended span is rarely if ever used in new designs. As with the eyebar of the Point Pleasant Bridge, this bridge collapse demonstrated that these pin and hanger assemblies are also clearly FCMs. (Also similar to the Point Pleasant Bridge collapse, the Mianus River Bridge collapse was the result of extraordinary circumstances, in this case corrosion, and not just fatigue or fracture.)

In part because of this failure, the NBIS were revised in 1988, requiring among other things a hands-on inspection of FCMs. This requirement significantly increases life-cycle costs relative to non-FCMs, which may be inspected from the ground, through, in most cases, binoculars (*27–29*). This requirement is particularly onerous for box girders, because it requires the inspectors to enter the boxes, which significantly increases costs. The frequency and extent of inspection are not clear in the current NBIS. Consequently, there is disagreement on what constitutes a fracture-critical inspection and how often it is done. (When the word inspection is

FIGURE 4 Collapse of Mianus River Bridge. (*Courtesy:* John W. Fisher.)

used throughout the rest of this report, it is intended to mean fracture-critical, hands-on field inspection, unless otherwise noted.)

These increased life-cycle costs for FCMs are significantly greater than the approximately 8% increase in initial materials and fabrication costs for FCMs discussed earlier. According to the survey (and as described later in chapter three), many owners believe that inspection costs associated with FCBs consume a large portion of their entire inspection budget. Owners were asked to estimate the relative increase in costs when inspecting an FCB relative to inspection and non-FCBs. There was substantial variation in the response; however, most agencies indicated increases of between 200% and 500%. The most common reasons indicated for these increases were:

- Specialized access equipment such as a snooper (Figure 5), manlift (Figure 6), or rigging required for hands-on inspection (*30*).
- Traffic control to close lanes to permit the access equipment to be placed on or below the bridge (see Figure 5).
- Additional employee-hours required to conduct a detailed hands-on inspection.
- More frequent use of nondestructive testing (NDT) (described in Appendix A).
- Greater frequency of inspection for FCBs.

These hands-on inspections have revealed numerous fatigue and corrosion problems that otherwise might have escaped notice. Many of these problem details are discussed in Appendix A. Twenty-three percent of respondents to the survey indicated that they found significant cracks that could have become much worse, possibly averting collapses (see chapter three). Similar examples may be found in trade magazines [e.g., see Zettler (*31*)].

FIGURE 5 Snooper used for hands-on inspection from bridge deck.

FIGURE 6 Hands-on inspection from manlift.

Primarily because of these increased life-cycle costs, there is a general reluctance to design new FCBs. Fewer FCBs have been proposed since the fabrication FCP went into effect in 1978 (*22*). FCMs, such as steel pier caps and cross girders, are still frequently designed, although usually only if they cannot be avoided. In some circumstances, bridge designs with FCMs, such as tied arches, two-girder bridges, and trusses, may be the most efficient and cost-effective structural system. Although the more stringent CVN requirements, the fabrication FCP, and the additional inspection requirements for FCMs are beneficial, if they are overly conservative for modern bridges they can become an obstacle to the savings gained in using more cost-effective designs.

International scanning tours for bridge management (*32*) and fabrication (*33*) have noted that Europe does not have special policies for FCMs. A risk-based approach, coupled with more rigorous three-dimensional analysis techniques, is used to ensure that a sufficient level of structural reliability is provided. Consequently, steel bridge designs that would be considered fracture-critical in the United States are still commonly built without prejudice. However, they have also had failures of what we would consider FCBs.

The following is from the fabrication scanning tour report (*33*):

> Perhaps the most significant design-related observation of the scan team was the rest of the industrialized world's more liberal view of the importance of redundancy. Two-girder bridges, as well as other structure types considered nonredundant and fracture critical in the United States, are not discouraged and, in fact, are used extensively as safe and cost-effective bridge designs. Kawada Industries cited redundancy studies it performed to demonstrate adequate redundancy of its two-girder systems with widely spaced, mid-depth cross beams. No special design, fabrication, or inspection requirements for such bridges were apparent. The U.S. design philosophy for nonredundant bridges should be reconsidered, based upon these observations and improvements in steel toughness.
>
> Twin-girder railroad bridges are common in Germany. The single-cell box girder, commonly used for elevated roadways in

urban areas of Japan, would be classified as fracture critical in the United States, but has provided excellent performance.

Other interesting findings from the scanning tours were that the inspection frequency is risk-based in Europe and that the inspectors' qualifications are commensurate with the complexity of the bridge.

REDUNDANCY AND COLLAPSE OF STEEL BRIDGES

Definition of Redundancy and Contrast to Indeterminacy and FCMs

The *AASHTO LRFD Bridge Design Specifications* define redundancy as "the quality of a bridge that enables it to perform its design function in the damaged state." In *NCHRP Report 406* (*34*), Ghosn and Moses defined superstructure redundancy as "the capability of a bridge superstructure to continue to carry loads after the damage or the failure of one of its members," and this definition is also used in the AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating* (*LRFR*) *of Highway Bridges* (*3*). Even though it has to do with potential performance in the event of damage, redundancy is a quality of the undamaged structure.

Note that these definitions are not clear about what load type, magnitude, distribution on the bridge, dynamic amplification, and load factors are supposed to be resisted by the damaged structure. Ghosn and Moses attempted to set requirements for the residual capacity of the damaged superstructure (*34*).

The definitions of redundancy are also not clear regarding the type and extent of damage. For example, in a bolted or riveted built-up member it is likely that a fracture would be limited to only one tension element, because it cannot propagate directly into neighboring elements. However, a ship collision could destroy the entire member.

Ultimately, it is the target level of reliability that designers and engineers who rate bridges should strive to achieve and the focus should not exclusively be on redundancy. Redundancy has a major impact on the risk of collapse and this impact is accounted for appropriately for all types of structures in both the *LRFD Specifications* and the *LRFR Manual,* as discussed here. Using the LRFD and LRFR procedures, it is possible to achieve the target level of reliability without redundancy in a bridge that is more conservatively designed.

Structural redundancy and structural indeterminacy are often confused and used interchangeably, although they are really two separate issues. Structural indeterminacy simply refers to whether or not the forces in a structure can be determined with statics. A structure that is indeterminate, although possibly providing alternate load paths, would not meet the definition of redundant if it were to collapse, because the members in the alternate load path did not have sufficient capacity to carry the redistributed loads. It is also true, but less obvious, that there are determinate structures that can be shown to meet the definition of redundant by developing new alternative load paths—and even a few examples (discussed later) of determinate structures that have demonstrated that they meet the definition of redundant by surviving a significant fracture in service.

As defined in the introduction, FCMs are nonredundant; however, nonredundant is a broader term because it also includes

- Substructures;
- Members that may be inherently not susceptible to fracture, such as compression members, but still could lead to collapse if damaged by overloading, earthquakes, fire, terrorism, ship or vehicle collisions; and
- Members made of materials other than steel.

Substructures such as piers are often nonredundant and therefore earthquakes, scour, vehicle collisions (Figure 7), and ship collisions (Figure 8) have led to most of the major collapses of both steel and concrete bridges. For example, an article published in 2002 just after the collapse of the Interstate 40 bridge over the Arkansas River in Oklahoma, listed seven major bridge disasters in the United States up to that time (*35*). The two FCBs discussed previously, the Point Pleasant Bridge and the Mianus River Bridge, were 28% of the list. The remaining 72% were the result of substructure failure.

- The Arkansas River Bridge, Sunshine Skyway (Florida) (1980), and the Queen Isabella Causeway (2001) bridge collapses were caused by ship collisions.
- The Schoharie Creek Bridge in New York (1987) and Arroyo Pasajero Bridges in California (1995) collapses were caused by scour.

FIGURE 7 Example of vehicle–bridge collision causing collapse owing to nonredundant substructure. (*Courtesy:* Robert Sweeney.)

FIGURE 8 Collapse of Queen Isabella Causeway Bridge in Texas in 2001 when pier was struck by a barge.

In addition to prevention of collapse in the event of fracture, redundancy of the superstructure is important for several other reasons. The first is the need to more easily redeck the bridge. Also, as discussed earlier, events other than fracture can also damage and completely destroy members of the superstructure. For example, the fascia girder of the I-610 Bridge over the Houston Ship Channel was struck twice by ships, once in December 2000 and once in May 2001. The highly redundant multigirder bridge withstood each collision, although both times the bridges had to be closed for repairs. These are compelling reasons to have redundancy. (Note that periodic inspection is not really helpful in finding this type of damage from collisions or other extreme events because the damage is usually immediately obvious to the public. A determination to close and repair the bridge can be made quickly in these cases. This is an important point because in these cases it reflects on the maximum live load the damaged structure is likely to experience in the brief period before closure. Periodic inspection may be more helpful in finding fatigue cracks and fractures because they are often not immediately obvious.) These are compelling reasons for redundancy.

These reasons for redundancy (other than fracture) could be used to encourage redundancy outright instead of indirectly by penalizing FCMs. For example, in the *LRFD Specifications,* redundancy is encouraged in Sections 1.3.2 and 1.3.4. Load factors are modified based on the level of redundancy, and it is stated that multiple-load-path and continuous structures should be used unless there are compelling reasons to do otherwise.

In the *LRFR Manual,* redundancy is reflected in system factors that reduce the capacity of each member in nonredundant systems. The system factors are calibrated so that nonredundant systems are rated more conservatively at approximately the same level of reliability associated with new bridges designed using *LRFD Specifications,* called the "inventory" level in former rating procedures. Redundant systems are rated at a reduced reliability level corresponding approximately to the traditional "operating" level. The system factor for the most nonredundant bridge types is 0.85. This means that a nonredundant bridge designed for 1.17 (the inverse of 0.85) times the design load has approximately equal reliability to a redundant bridge designed for 1.0 times the design load.

According to Ghosn and Moses (*34*), a redundant superstructure has at least one alternate load path and is capable of safely supporting the specified dead loads and live loads and maintaining temporary serviceability of the deck following failure of a main load-carrying member. They recognized that redundancy is related to system behavior rather than individual component behavior. The specifications generally ignore the interaction between members and structural components (i.e., system behavior) in a bridge, however.

Redundancy is often discussed in terms of three types (*28, 29,34*):

- Internal redundancy, also called member redundancy, exists when a member is comprised of multiple elements and a fracture that formed in one element cannot propagate directly into the adjacent elements. Examples include girders with composite deck (the deck remains intact in the event of a girder fracture as in Figure 9); riveted or bolted built-up girders, tie girders, or tension members of a truss (Figure 10); split box sections with longitudinal bolted splice (Figure 11); the bracing system, laterals and cross frames within a box member (Figure 12); bolted continuous plates or shapes to give a member redundancy (Figures 13–15), and single-cell concrete boxes with multiple post-tensioning strands. Note that it must be shown that the damaged member (several possible cases considered separately with one element fractured or removed) can survive the prescribed loads to be internally redundant.

FIGURE 9 Example of bridge deck acting as catenary with hinge at fracture location in end span of the approach spans of the Hoan Bridge in Wisconsin—two of the three girders had full-depth fractures in December 2000.

- Structural redundancy is external static indeterminacy and can occur in a two or more span continuous girder or truss. Note that only part of an end span between the fracture and the pier may be supported by structural redundancy and that the part of the end span at the abutment could theoretically collapse. However, internal redundancy of the deck on a composite girder could be sufficient to maintain stability of the end span, especially when combined with the structural redundancy, as in the Hoan Bridge shown in Figure 9.
- Load-path redundancy is internal static indeterminacy arising from having three or more girders or redundant truss members. One can argue that the transverse members such as diaphragms between girders can also provide load-path redundancy (see Figure 12).

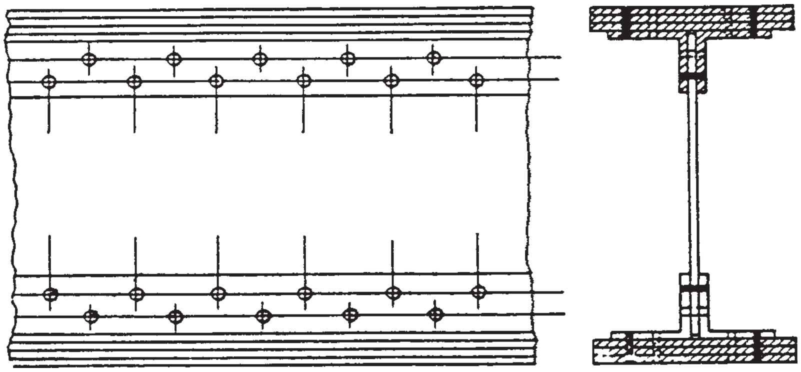
Note that the *LRFR Manual* (*3*) and the *Bridge Inspector's Reference Manual* (*29*) state that "in the interest of conservatism" internal and structural redundancy should be neglected, meaning that load-path redundancy is the only redundancy that matters. As shown by in-service behavior of fractured FCBs discussed in the next section, neglecting all but load-path redundancy is clearly oversimplifying and possibly overconservative.

Examples of Behavior of FCBs That Experienced Major Fractures

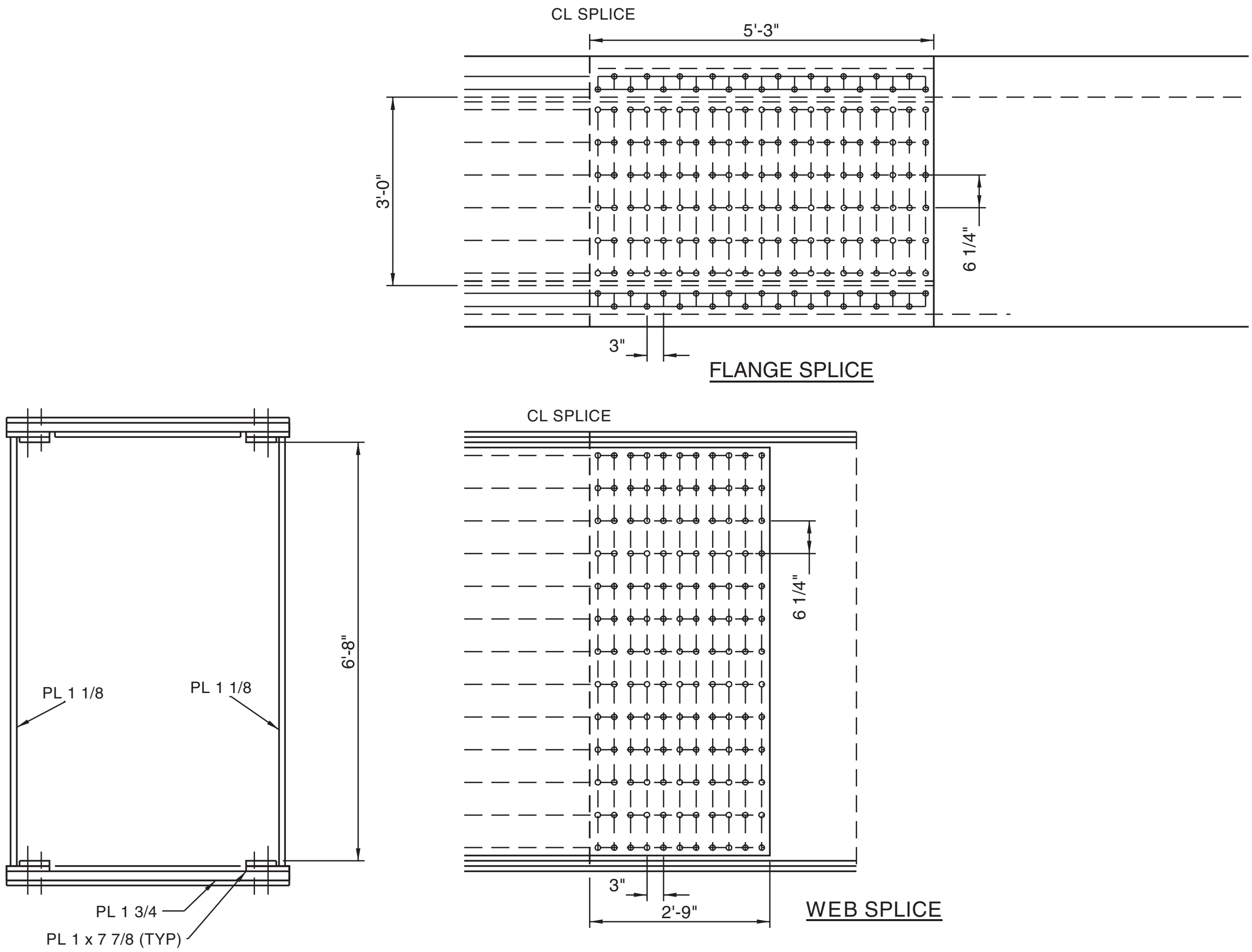
Two examples of FCB collapses, the Point Pleasant Bridge (constructed in 1928, Figure 1) and the Mianus River Bridge (constructed in 1957, Figure 4), have been discussed. These are the only two examples of collapses of major steel bridges as a result of fracture in the superstructure. As explained previously, there were circumstances other than just fatigue and fracture that were the root cause in both of these failures. On the other hand, there are numerous examples of bridges with members that would traditionally have been classified as FCMs that have fractured, but the bridge did not even partially collapse.

Two-girder FCBs that have experienced either partial or full-depth fractures but did not collapse include:

- The 1976 full-depth fracture of the US-52 bridge over the Mississippi River in St. Paul, Minnesota (called the Lafayette St. Bridge) (see Figure 16) (*11,36*). (It should be noted that during the course of this synthesis, it was mentioned that the bridge remained stable



(a)



(b)

FIGURE 10 Examples of internally redundant members: (*a*) riveted built-up girder and (*b*) bolted built-up tie girder proposed for Blennerhassett Arch Bridge. (*Courtesy:* Michael Baker Jr., Inc.)

because it leaned on an adjacent bridge). However, interviews with Donald Fleming, former Bridge Engineer of the Minnesota Department of Transportation (DOT) and John W. Fisher, both of whom were directly involved with the failure investigation, confirmed that this was not the case. The bridge did sag 6.5 in. (165 mm), but was not supported by the adjacent bridge.

- The 1977 full-depth fracture of the I-79 bridge at Neville Island in Pittsburgh, Pennsylvania (Figure 17) (*11,15*).
- The May 2003 fracture of the US-422 Bridge near Pottstown, Pennsylvania. The entire bottom flange and approximately 9 in. (230 mm) of the web fractured (*37*).

It is apparent that other elements of these two-girder bridges, particularly the deck, along with the floorbeams, cross-

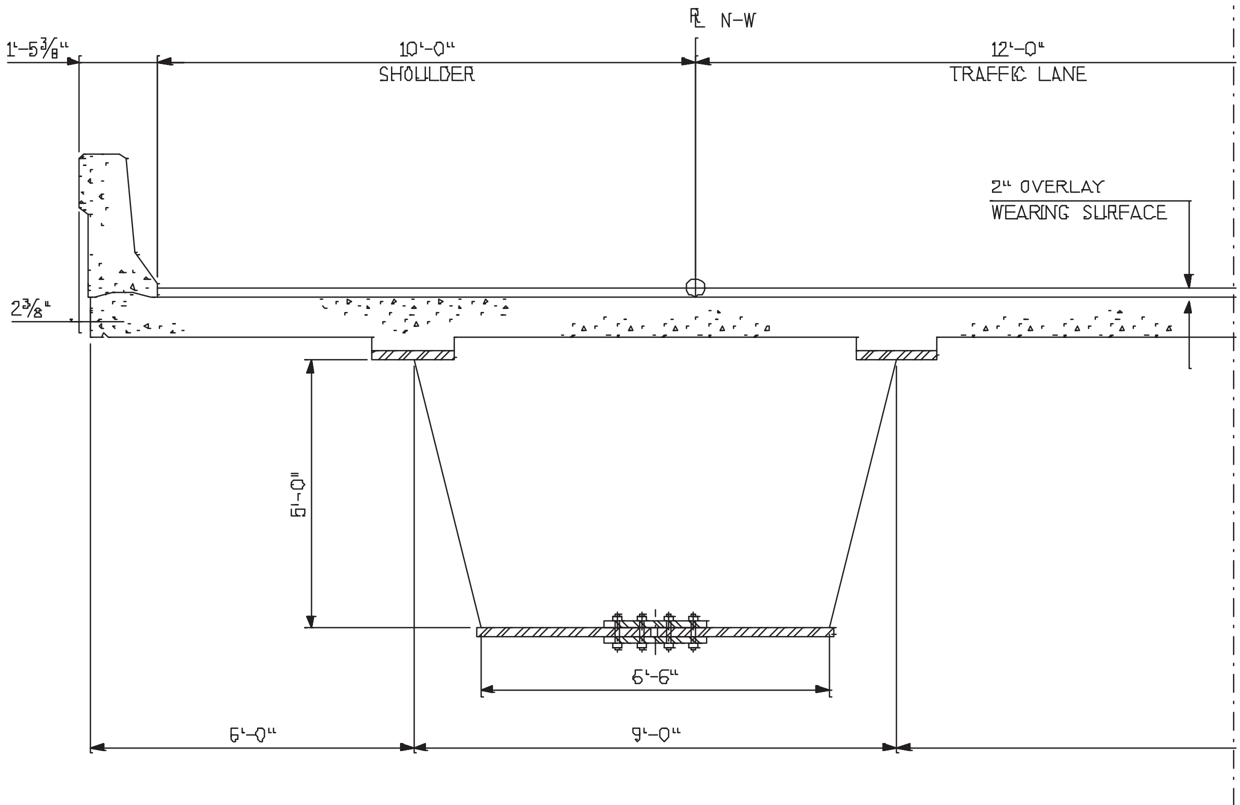


FIGURE 11 Splitting a box section with a bolted longitudinal splice to give it internal redundancy. (*Courtesy:* HNTB.)

frames, and stringers, are sometimes able to carry the loads and prevent collapse. These alternate load paths were so robust in the I-79 and US-422 fractures that there was little or no perceptible deformation of the structure. For example, when a tugboat pilot discovered the I-79 fracture, the bridge was still providing a serviceable roadway. In the case of the Lafayette St. Bridge, displacements of 2.5 in. (63 mm) were noticed relative to the adjacent bridge 48 days before the fracture was discovered, growing to 6.5 in. (230 mm) as the crack length increased over that time.

The less obvious nature of the damage in the case of a fracture as opposed to the other causes such as collisions discussed previously has implications for the loading to evaluate the damaged member (residual capacity). A lower level of residual capacity would be required for members damaged by these other more obvious causes because the bridge is likely to be closed within hours after the event. However, if a fracture goes unnoticed for an extended period, the probability of larger permits or illegal loads increases significantly.

As explained in chapter three, some agencies even classify three-girder bridges as FCBs. The Hoan Bridge fracture, shown in Figure 9, is an example of a three-girder bridge end span (which is viewed as most critical owing to inadequate con-

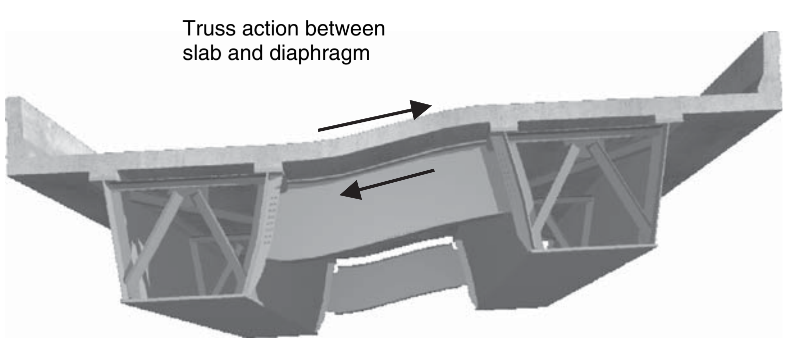


FIGURE 12 Schematic of twin composite tub girder superstructure showing internal redundancy provided by bracing system and possible alternative load path provided by slab and diaphragm. (*Courtesy:* HNTB.)

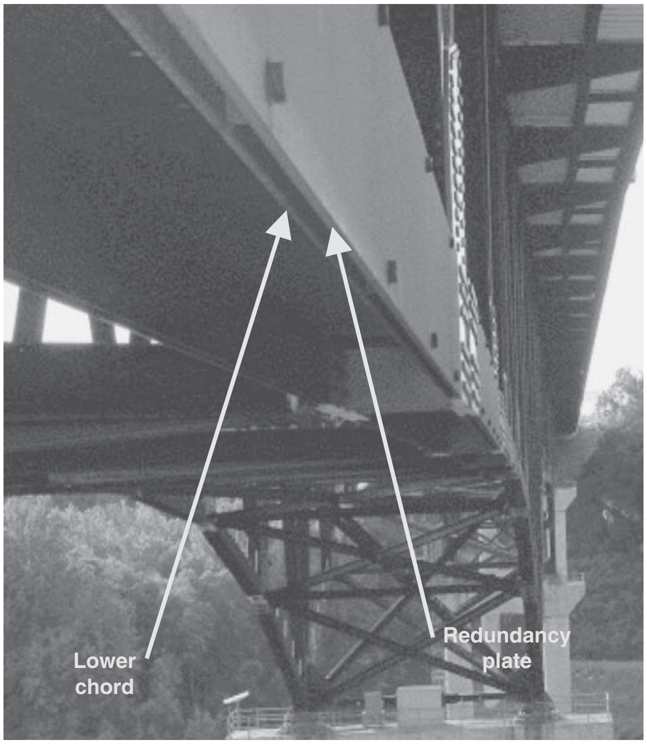


FIGURE 13 Redundancy plate bolted to lower chord of SR-33 bridge near Easton, Pennsylvania. (*Courtesy:* HNTB.)

tinuity at the joint), with two out of three girders and the web of the third girder fractured (*38*).

These fractures in actual bridges are the most valuable data available to judge the necessity of special provisions for FCMs. These full-scale tests are much more valuable than laboratory tests or numerical simulations, because the former are subject to idealizations and assumptions. Although not sufficient to prove that two-girder bridges should not be classified as having FCMs, these incidents do show that under some circumstances they do not meet the definition of an FCM.

The survey (see chapter three) revealed several other examples of FCBs that had experienced major fractures but had not collapsed. These were usually noticed in an inspection, but had occurred at an earlier, unknown time. Similar accounts can be found elsewhere (*31,39*).

Although not caused by a fracture, a train derailment on a nonredundant truss bridge, shown in Figure 18, is another example of valuable in-service behavior of a full-scale bridge. Note that several diagonals, hangers, and upper chord braces are completely severed, but the truss did not collapse even though a significant portion of the live load remained on the bridge.

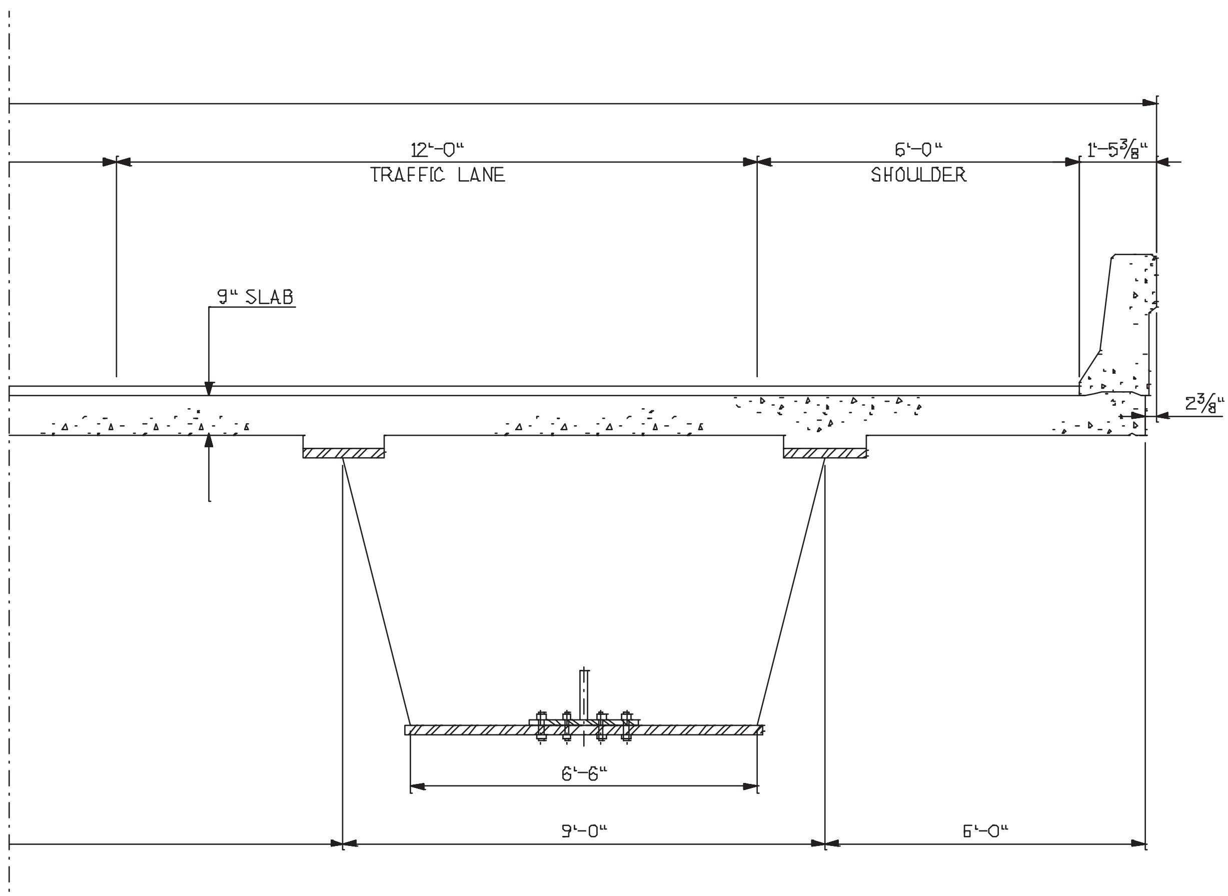


FIGURE 14 Tee section bolted to continuous lower flange of box section to provide redundancy. (*Courtesy:* HNTB.)

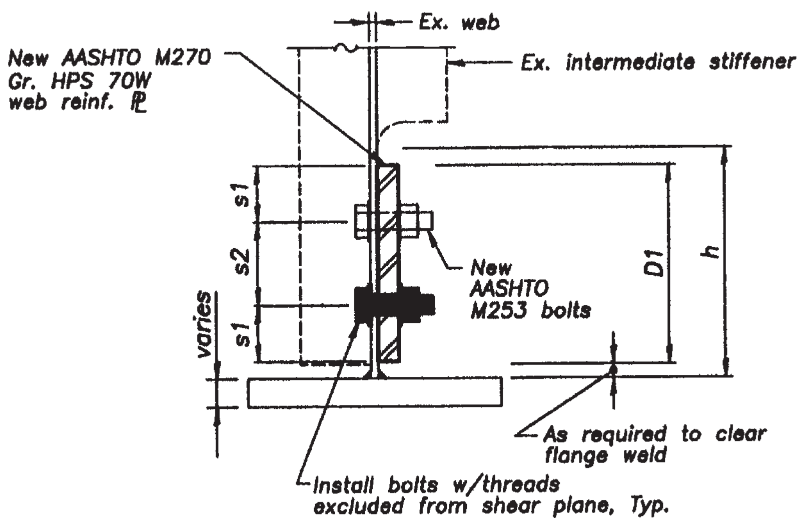


FIGURE 15 Retrofit redundancy plate bolted to web of existing two-girder superstructure in Poplar Street Bridge complex in East St. Louis. (*Courtesy:* Wiss, Janney, Elstner Associates.)

Other interesting field tests have been performed on I-40 bridges over the Rio Grande in Albuquerque, New Mexico (*40*). The two-girder bridges, which had spans ranging from 131 to 163 ft and were classified as nonredundant fracture-critical, were built in 1963. The girders were 10 ft deep and spaced at 30 ft on center. A torch cut was used to simulate a fracture of four different lengths to one of the girders in the bridge, the last of which was nearly full depth. Idriss et al. studied the redistribution of loads, the loading the bridge can withstand in the damaged condition, and the potential for collapse. The bridges were loaded with a truck that was 95% of New Mexico legal load and roughly equivalent to HS-18.35.

Idriss et al. (*40*) also reported that under dead and live loads and when the truck was located above the crack, the flange only deflected 1.2 in. (28 mm). There was no sign of yielding and no significant change in strains experienced by the other instrumented members until the bottom flange was completely severed. This suggests that load redistribution did not occur until the bottom flange was completely severed. They also reported that most of the load was redistributed through the damaged girder and stringer deck system to the interior supports. In general, the load was redistributed from the damaged girder to the diagonal bracing, diaphragms, stringers, deck, floorbeams, and remaining girder.

Reliability Studies of Redundancy

A brief summary of selected research studies focusing on redundancy is presented here. More complete summaries of

FIGURE 16 View of cracked girder in two-girder span of Lafayette Street Bridge in St. Paul, Minnesota, as an example of a bridge that is sufficiently redundant to avoid collapse despite a fracture of the tension flange and the web of one girder.

FIGURE 17 View of cracked girder in two-girder span of I-79 Bridge at Neville Island in Pittsburgh as an example of a two-girder bridge that is sufficiently redundant to avoid collapse despite a fracture of the tension flange and the web of one girder.

FIGURE 18 Example of train derailment on a fracture-critical truss bridge that severed several members but did not collapse. (*Courtesy:* Robert Sweeney.)

the individual articles and additional articles can be found in the annotated bibliography in Appendix D.

A large number of studies have attempted to characterize the reliability of bridge designs with varying redundancy. In these reliability studies, the degree of redundancy of a system was examined by reviewing the difference between reliability indices. Ghosn and Moses (*34*) studied redundancy in highway bridge superstructures by examining the difference between the safety indices they defined and those of bridges that have been known to perform as desired. Kritzler and Mohammadi (*41*) used the same approach in which they compared the safety reliability index of a redundant structure considering all failure paths and the safety index of the exact same structure with no alternative load path. A reliability approach was also used by Moses (*42*) for the evaluation of bridge safety and remaining life.

Frangopol and Curley (*43*) recognized the need for the development of a better understanding and definition of redundancy in various types of bridges. They defined the term *R* as the redundant factor for a bridge, which is the reserve strength between component(s) damage and system collapse. The redundancy factor was later used in other studies [e.g., Frangopol and Curley (*44*), Frangopol and Nakib (*45*), Frangopol and Yoshida (*46*)] to investigate redundancy of systems.

Ghosn and Moses (*34*) included both a reliability approach and a direct system factor approach to evaluate the degree of redundancy of an existing bridge or when designing a new bridge. In the reliability approach, relative reliability was calculated and a level of redundancy is satisfied if obtained values of the relative reliabilities are greater than or equal to specified values. In the direct system redundancy approach, adequate load factor ratios (system reserve ratios) are required to satisfy a minimum level of redundancy. A bridge is then considered adequate if system reserve ratios are greater than or equal to specified values.

A drawback of the reliability approach is that it requires measures of statistical variation that are often not available. In these examples, estimates are made and the results are still insightful, but a great deal of additional data would be required to use this approach as a practical tool.

It is important to note that it is the reliability of the system that is important. As discussed previously, redundancy affects this reliability but not as much as might be assumed. A two-girder bridge designed for HS-25 for example might have greater reliability than a multigirder bridge designed for HS-20 (*34*). Therefore, one should not place too much emphasis on redundancy and lose sight of the important goal, system reliability.

Numerical Simulations of the Residual Capacity of Fractured Bridges

Finite-element analysis is increasingly being used to simulate the after-fracture behavior and residual capacity of FCBs. These analyses provided insight about the secondary load paths in FCB systems after an FCM is severed or otherwise removed from the model. In some cases they are used to get a waiver from FHWA on FCM design requirements for a new bridge. However, this type of analysis and associated waiver of the FCM provisions is presently being done on a case-by-case basis, and the analysis requirements, loads, and failure criteria are not always clear. In other cases, they are used to evaluate the residual capacity of existing FCBs.

Section 6.6.2 of the *AASHTO LRFD Bridge Design Specifications* discusses the fracture limit state. The commentary of this section states that:

> The criteria for refined analysis used to demonstrate that part of a structure is not fracture critical, has not yet been codified. Therefore, the loading cases to be studied, location of potential cracks, degree to which the dynamic effects associated with a fracture are included in the analysis, and fineness of the models and choice of element type should all be agreed upon by the owner and the engineer. The ability of a particular software product to adequately capture the complexity of the problem should also be considered and the choice of software should be mutually agreed upon by the owner and the engineer. Relief from full factored loads associated with the Strength I Load Combinations of Table 3.4.1-1 should be considered as should the number of loaded design lanes versus the number of striped traffic lanes (*1*).

Heins and Hou (*47*) used an analytical two-girder and three-girder space frame model to study the effect of bracing members in bridge structures on the load distribution of two-girder and multigirder systems after the development of a crack in one of the girders. Heins and Hou found that when one crack develops and no bracing is used, the deformation increases by 40% for the two-girder system and 10% for the three-girder system. However, if bracings are considered, the deformation increases by 10% for the two-girder system, whereas almost no increase in the deformation is noticed in the three-girder system. Heins and Kato (*48*) also studied the effect of bracing on load distribution in a two-girder bridge system when one of the girders is damaged and found that the deformation of the cracked girder was substantially reduced when bracing was used.

Ghosn and Moses (*34*) developed recommendations for the residual capacity of fractured bridges to demonstrate sufficient redundancy. Load factors LF_1 and LF_d should be calculated using a three-dimensional finite-element analysis. LF_1 is the multiple of side-by-side HS-20 trucks that the structure can carry in addition to unfactored dead loads (using elastic analysis) before the first member reaches the resistance predicted by the design specifications. LF_1 is typically on the order of 3.8, depending on the ratio of live load to dead load

LF_d is the residual capacity of the damaged structure and is calculated by performing a nonlinear analysis of the damaged structure (with the FCM removed) under the effect of the unfactored dead load and incrementing the multiple of side-by-side HS-20 truck loads until the system collapses. Redundancy is considered adequate if the ratio of LF_d to LF_1 is greater than 0.5. This means that the damaged structure should be able to support approximately 1.9 times the side-by-side HS-20 loading. (Figure 19 shows a schematic of the loading of a damaged girder in terms of multiples of side-by-side HS-20 trucks.)

Note that this requires a greater residual LF_d if the bridge is originally over-designed, which does not seem logical. *NCHRP Report 406* indicated that bridges that do not meet the required load factor ratios could still provide a high level of system safety (*34*).

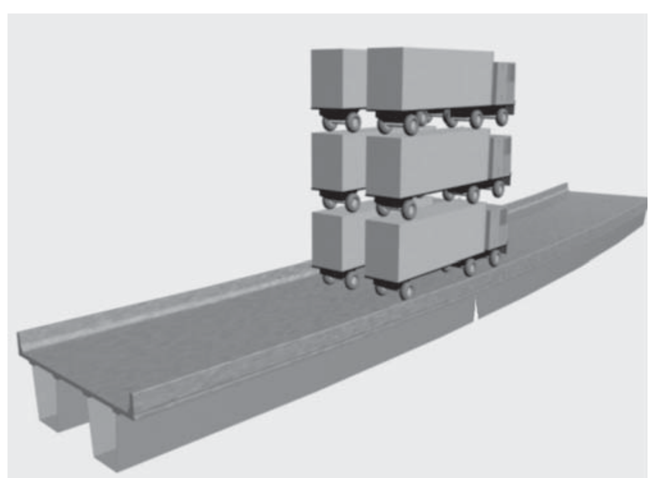


FIGURE 19 Schematic of multiple HS-20 loads on damaged superstructure. (*Courtesy:* HNTB.)

HNTB, as part of Milwaukee Transportation Partners, used these recommendations in a redundancy analysis for twin curved box girders designed for the Marquette Interchange in Wisconsin to demonstrate to FHWA that waiving the fracture-critical inspection requirements was justified (*49*). The boxes were going to be fabricated as FCMs anyway. (Recall previously in this chapter that the fabrication part of the additional cost of FCMs is relatively small compared with the in-service inspection part.)

A three-dimensional shell element model was used for elastic incremental analyses, manually updating the mesh to account for nonlinear effects. The reserve capacity of the undamaged bridge was found to be 4.9 times HS-25 side-by-side trucks for the pier section of the outside box and 3.4 times HS-25 side-by-side trucks for the midspan section of the outside box. The residual capacity of the damaged midspan section was found to be 3.35 times HS-25 side-by-side trucks, above the 0.5 times the original reserve capacity recommended by Ghosn and Moses (*34*).

A dynamic amplification factor of 1.8 was conservatively estimated using a single-degree-of-freedom impact model and assuming 5% of critical damping. Based on this, it was determined that the structure might have to withstand dynamic stresses that are equivalent to 2.68 times HS-25 side-by-side trucks in addition to the static dead load effect. (The 2.68 level included the anticipated dynamic part of the dead load and both the static and dynamic part of one set of HS-25 trucks as the live load.) Because the residual capacity was greater than 2.68, the bridge is considered safe for the temporary dynamic loading.

Lai (*50*) developed a three-dimensional finite-element model and used a static incremental study to determine the degree of redundancy of a tied arch bridge. He found that after the fracture of the one of the ties, the structure was capable of carrying its own weight plus 1.3 times HS-20 truck loading without catastrophic collapse.

Michael Baker Jr., performed an analysis for the proposed Blennerhassett Tied Arch Bridge in West Virginia (*51*). The tie girder section is a bolted built-up box section as shown in Figure 10*b.* Because the section is a bolted built-up section, it has internal redundancy as discussed previously in this chapter. In this case, a fatigue crack, fracture, corrosion failure, ductile rupture, or other failure of any one of the four plates cannot propagate directly into any of the other three plates.

The designer justified using the inelastic capacity of the residual cross section because *AASHTO LRFD Specifications* Articles 4.5.2.1 and 4.5.2.3 permit inelastic response for extreme events, and the fracture of one of the plates in the cross section was considered an extreme event (*51*). The analysis of the Blennerhassett Bridge did not include the effect of plastic redistribution of forces, which, if included, indicate greater capacity.

The tie girder was modeled using elastic beam elements. Elastic analysis is always conservative, even when used with inelastic cross-section capacity, provided the sections are sufficiently ductile. The analysis involved a 14-ft (4.2-m)-long segment of the tie girder representing a fractured residual section. The centroid of the C-shaped residual section is not collinear with the centroid of the uncracked tie girder, creating some additional bending moment. The capacity of the residual section is then compared with the elastic moments from this model with the full-factored *AASHTO LRFD Specifications* HL-93 load and impact [1.25 DC + 1.50 DW + 1.30 (LL+I)]. Although no dynamic amplification effects were included, the Strength I factored loading is very conservative and would amount to a large multiple of HS-20 trucks.

The capacity of the cross section is obtained from a model that relates moment to curvature and strain. Nominal cross-section strains of less than 1%, approximately six times the yield strain of the HPS 70W steel, were considered acceptable. This level of allowable nominal strain allows a margin for localized concentrated strains near holes and other discontinuities, and was supported by nonlinear finite-element analysis and testing of tension and flexural members made from similar HPS 70W at the University of Minnesota (*52,53*). The residual cross section; that is, a cross section with either a web or a flange missing, was shown to be able to continue to carry the load after such an event. Therefore, the tie girder was shown to meet the definition of redundant.

This same type of analysis is being used to evaluate older structures, as well to better direct resources for maintenance and replacement; for example, if a bridge is not really fracture-critical then it may not be necessary to replace it as soon. For example, as part of a study being conducted on the I-35W truss bridge in Minneapolis, Minnesota, complex three-dimensional finite-element models of the bridge are being developed. Fracture-critical members are removed from the model and the resulting redistribution of loads accurately is being studied. Adequacy of connections, individual components, and overall stability are being assessed.

PRESENT CLASSIFICATION OF STEEL BRIDGE SUPERSTRUCTURES AS FRACTURE-CRITICAL

Common assumptions that fractures in certain superstructure types will lead to collapse may be too simplistic, as shown by the bridges traditionally classified as FCBs that did not collapse, as discussed earlier in this chapter.

Table 1 is a chart assembled by the California DOT (Caltrans) showing the varying definitions of different superstructure types as "fracture-critical" in four related documents. There appears to be substantial disagreement, as was also found in the survey conducted as part of this project (see chapter three). Note that only the book (in the left-hand column) considers two-box-girder systems to be FCBs. In the case

TABLE 1
VARIOUS DEFINITIONS OF COMMON FRACTURE-CRITICAL BRIDGE SUPERSTRUCTURES

Bridge Inspection and Structural Analysis (*27*)	*Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges* (*3*)	*Inspection of Fracture Critical Bridge Members*, Report FHWA-IP-86-26 (*28*)	*Bridge Inspectors Refere nce Manual* (*29*)
Two-girder system (simple and continuous span) • Suspended span with pin and hanger system • Suspended span • Welded plate girder • Riveted or bolted plate girder	One- or two-girder systems, including single boxes with welding Suspended spans with two girders	Two-girder systems (single span and end span of continuous span units) • With fix hanger suspended span • With suspended span • Welded plate girders • Riveted or bolted plate girders	• Suspended span with two girders • Simple span two-girder bridge with welded partial length cover plates on the bottom flange • Continuous span two-girder system with cantilever and suspension link arrangement and welded partial length cover plates • Simple span two-girder system with lateral bracing connected to horizontal gusset plates that are attached to webs
Truss system (simple and continuous span) • Eyebar truss • Welded truss • Riveted truss • Three deck truss with pin and hanger assembly	Two-truss systems	Truss system (simple and continuous spans) • Eyebar truss • Welded truss • Truss with suspended span • Riveted truss	• Simple span truss with two eyebars or single member between panel points
Suspension bridge • Eyebar chain • Cable	Suspension systems with two eyebar components	Suspension bridge • Eyebar chain • Cable	• Bar chain suspension bridge with two eyebars per panel

(*continues*)

TABLE 1 (*Continued*)

Tie arch • Welded tie box girder • Riveted tie box girder	Welded tie arches	Tie Arch • Two welded tie box girder • Two riveted tie box girder	Welded tie arches with box shape tie girder
Steel pier cap • Welded box or plate girder • Riveted box or plate girder • Two column steel bent	Steel pier caps and cross girders	Steel pier cap • Welded box or plate girder • Riveted plate girder	• Single welded I-girder or box girder pier cap with bridge girders and stringers attached by welding
Longitudinal box beam • Single welded box • Single riveted box	Single boxes with welding	Longitudinal box beam • Single welded box • Single riveted box	• Simple span single welded box girders with details such as termination of longitudinal stiffeners or gusset plate
Anchor for cable stayed bridge			
Pin and hanger connections when used in suspended span configuration in nonredundant systems	Pin and hanger connections on two- or three-girder systems		
Two or less box girders			
Three or more box girders if spacing is large (should be determined by structural engineer)			

Courtesy: Tom Harrington, Caltrans.

of the AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating* (*LRFR*) *of Highway Bridges* (*3*) [identical to the predecessor *Manual for Condition Evaluation of Bridges* (*54*)] and the *Bridge Inspectors Reference Manual* (*29*), only welded tie members for arches or single box girders are considered FCMs, whereas riveted or bolted built-up tie members or single box girders, such as those shown in Figure 10, would not be FCMs. This would appear to be giving credit to the internal redundancy of the bolted or riveted built-up members (but these manuals note elsewhere that internal redundancy should be neglected).

In Table 1, not all two-I-girder bridges are considered FCBs in some of the documents. The *LRFR Manual* and the reference manual include all two-I-girder bridges as FCBs. However, in *Inspection of Fracture Critical Bridge Members* (*28*), continuous two-girder bridges are not considered fracture-critical except for the end span. This FHWA report gives credit to the structural redundancy of the continuous spans.

Unfortunately, the *AASHTO LRFD Bridge Design Specifications* are not clear about what types of superstructures are to be classified as FCBs. The fatigue and fracture limit state requirements are specified in Article 6.10.5. Special provisions for box sections are included in Section 6.11.5, which also states that:

> For single box sections, box flanges in tension shall be considered fracture-critical, unless analysis shows that the section can support the full dead and appropriate live load after sustaining a complete fracture of the flange at any point.

The commentary of this section states that:

> There may be exceptions where box flanges of single-box sections subject to tension need not be considered fracture-critical. For example, continuously braced top flanges in regions of negative flexure where there is adequate deck reinforcing to act as a top flange in such cases, adequate shear connection must also be provided.

The section was recently amended to include the following:

> Unless adequate strength and stability of a damaged structure can be verified by refined analysis, in cross sections comprised of two box sections, only the bottom flanges in the positive moment region should be designated fracture-critical. Where cross sections contain more than two box girder sections, all of the tension flanges should be considered non-fracture-critical.

Therefore, the redundancy of two box or tub girders is sometimes even in question. Twin tub girders would be expected to perform even better than twin I-girders owing to the torsional capacity of the intact tub girder and the alternate load paths available within each tub girder. Unfortunately, this built-in redundancy shown by these structures is not explicitly recognized in the *LRFD Specifications.* As will be discussed in chapter three, the results of the survey revealed that different agencies classify tub girders differently when determining if the bridge is fracture-critical or not.

CHAPTER THREE

RESULTS OF SURVEY

This chapter summarizes the findings of the survey. One of the objectives of this survey was to gather information on fracture-critical structures and how bridge owners define, identify, inspect, and manage FCBs. Information related to structural failures and how owners have addressed or developed retrofit policies and strategies was also collected. In addition, what owners see as the most relevant research needs related to FCBs was solicited.

Specifically, the survey was intended to collect data in varying levels related to the following:

- How FCMs are presently defined, documented, and managed;
- Inspection frequencies and procedures;
- Methods for calculating remaining fatigue life;
- Qualification and training of inspectors;
- Available and needed training;
- Locally owned bridges;
- Experience with FCM fractures and problem details;
- Examples of where an inspection program prevented failures;
- Cost of inspection programs;
- Retrofit techniques used;
- NDE methods used;
- As-built versus as-designed;
- Fabrication methods and fabrication inspection; and
- Impact of staff turnover.

BACKGROUND

A detailed questionnaire intended to identify and characterize specific issues related to FCBs was developed and distributed to all state and Canadian provincial DOTs and various other transportation authorities within the United States. The survey was divided into four parts. Part I (General) collected general information related to FCBs and was the screening portion of the survey to determine whether participants should continue on to the other three sections. Part II (Inspection and Classification) was intended to identify the policies and approaches used to inspect FCBs. In addition, this section also requested specific data related to the classification and number of FCBs in an owner's inventory. Part III (Failures) requested specific details about each owner's experiences with respect to problems with both FCBs and non-FCBs. Owners were requested to provide information about all types of failures, whether they were the result of fatigue and fracture or for other reasons (e.g., impact, overload, or corrosion). This section also differentiated between failures that occurred before and after the FCP was initiated (around 1975 for most agencies) and before and after the FCB inspection program became regulation (around 1988 for most agencies). Part IV (Retrofit Procedures) contained a series of questions that sought to acquire information about how owners deal with failed bridges and/or subsequently develop procedures to improve redundancy. This section also requested that individuals provide opinions on future research needs related to FCBs.

Overall, the response to the survey was reasonably thorough. After it was sent out, several owners indicated that they did not have the ability to retrieve the data requested with respect to the breakdown of the structure types that were classified as FCBs in their inventory. This was the result of limitations of the software that they used to manage their bridge databases. Most however were able to provide the detailed data, which are included in Appendix B. A list of survey respondents is included in Appendix C.

Two states, Pennsylvania and Oklahoma, provided additional documentation that described their general inspection procedures. [More information can be found on the website of each state (http://www.dot.state.pa.us/ and http://www.okladot.state.ok.us/).] According to the document provided, the procedures used by Oklahoma are based somewhat on those of the Pennsylvania DOT. That document provides guidance on how to prioritize FCBs for inspection criteria and intervals based on remaining fatigue life, fatigue detail, material properties, and so forth.

In addition, issues related to FCBs were discussed with several colleagues, the project panel, AASHTO T-14 at the July 2003 meeting in Baltimore, and participants at the FHWA FCB workshop held in Orlando in November 2004. Many of the comments from these conversations are also reported here as anecdotes.

SUMMARY OF RESPONSES TO PART I—GENERAL

In this section, agencies were asked to provide their definition of an FCB. For the most part, the responses were consistent and nearly all agencies referred to or quoted the definition provided by the *AASHTO LRFD Bridge Design Specifications.* However, although the definition provided was consistent, how it is applied to various structure types was much more variable. A question was posed in tabular format asking the respondent to classify various types of bridges as fracture-critical or non-fracture-critical. The question and responses are summarized in Table 2. Most respondents indicated that their answers were based both on general policy where applicable and their own opinion.

As is apparent from the summarized data, there is considerable inconsistency in how owners apply the definition of FCBs. The only structural configurations that all respondents classified the same were a two-girder system (fracture-critical) and multisteel tub girder bridges (non-fracture-critical).

The respondents were generally more conservative in their application of the definition than the AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating* (*LRFR*) *of Highway Bridges* (*3*) [identical in this respect to the predecessor *Manual for Condition Evaluation of Bridges* (*54*)] and the *Bridge Inspector's Reference Manual* (*29*).

Interestingly, truss bridges with two truss lines were classified as non-fracture-critical by a few owners. In discussing this issue with one of the respondents, they indicated that they were primarily referring to riveted trusses because individual members were internally redundant and fracture of the entire tension chord was very unlikely. When asked if they would consider riveted two-girder bridges as non-fracture-critical if there were riveted cover plates, they indicated that they might if there was more than one cover plate, but that this would have to be considered on a case-by-case basis. Therefore, it appears that internal member redundancy is sometimes equated with load-path redundancy, or it at least lowers an owner's concern regarding load-path redundancy.

TABLE 2
SURVEY RESPONSES TO QUESTION 3 (How would you categorize the following bridges?)

Description	Fracture-Critical Yes	Fracture-Critical No
Two-girder bridges	38	0
Three-girder bridges	9	28
Three-girder bridges with girder spacing	10	21
Multigirder bridges with girder spacing	3	32
Truss bridges	34	3
Two-girder bridges fabricated using HPS 70W	31	1
Truss bridges fabricated using HPS 70W	28	2
Single-steel t ub" girder bridges	32	5
Twin-steel "tub" girder bridges	22	12
Multisteel "tub" girder bridges	0	34
Other (post-tensioned, timber, steel cross girders, etc.)	13	3

The greatest scatter in the responses appeared to be related to three-girder bridges and twin-steel "tub" girders. As indicated earlier, because respondents classified the bridge type based on their opinion owing to the lack of a formal policy by their agency, it is clear that different results would be obtained from different people. Interestingly, this implies that the same structural configuration may be classified as fracture-critical in one portion of a state and non-fracture-critical in another.

As indicated in Table 2, agencies were also asked to identify and classify other types of bridges as fracture or non-fracture-critical. The responses were limited to this question. However, approximately one-third of the agencies indicated that there are other bridge types they classify as fracture-critical. Some of these included timber bridges, post-tensioned concrete, and steel-tied arch bridges. Suspension bridges were also typically classified as fracture-critical owing to the main cables.

SUMMARY OF RESPONSES TO PART II—INSPECTION AND CLASSIFICATION

Part II of the survey provided considerable information as to the number of bridge types in the inventories of the responding agencies. Specific structural configurations were identified by most agencies. Unfortunately, not all agencies provided data for this set of questions, because they were not able to extract the information from their databases.

As can be seen, approximately 11% percent of the steel bridge inventory is classified as fracture-critical, based on the agencies that responded to this question. The percentage ranged from approximately 10% to up to 30%, depending on the agency. However, the majority of these bridges (about 75%) were built before 1975 (around the time the FCP was introduced by AASHTO). This is most likely for several reasons, including increases in quality control and the more stringent (and hence more costly) material and fabrication costs demanded by the FCP. In addition, the lack of apparent redundancy of FCBs was an obvious undesirable feature and led many agencies to move away from building such structures. Because most of the FCBs in the U.S. inventory were designed before 1975, these bridges contain fatigue details known to be poor and those that are susceptible to out-of-plane distortion. It should also be noted that the "modern" fatigue design provisions were introduced into the specifications beginning around 1973. Hence, most of the bridges built before 1975 were designed using the older, nonconservative fatigue design provisions. Fortunately, the loading used for fatigue design and the analytical models were, in most cases, quite conservative.

FCBs designed beginning in the early 1980s were detailed to minimize out-of-plane distortion cracking and minimize the use of low-fatigue resistance details (D, E, and E′). In addition, these bridges were built with material having improved fracture toughness requirements and shop inspection.

The two most common types of fracture-critical structural systems are not surprisingly two-girder bridges and trusses. The combination of these two structural systems comprises approximately 83% of all FCBs. About one-half of these structures (41%) are riveted structures, the remaining being fully welded or welded and bolted.

Inspection Issues

Many agencies (60%) require that inspectors successfully complete the National Highway Institute fracture-critical training course and indicated that experience with various NDT techniques is helpful. NDT (discussed in detail in Appendix A), such as magnetic-particle testing, dye-penetrant testing, or ultrasonic testing, was required when warranted. These techniques were also used depending on the bridge's age, average daily truck traffic (ADTT), stress level, and condition. No information on the use of special devices such as boroscopes for inspection of details that cannot be accessed was provided. Several states rotate field inspectors and shop inspectors and report that the cross transfer of knowledge is beneficial.

Many respondents are concerned about shrinking budgets and staff, personnel turnover, and lost expertise. Many agencies are also concerned about contracting out inspections to consultants; however, no firm examples to warrant such concern were provided. These concerns are usually coupled with the perceived need for improved documentation. Concerns have been expressed about locally owned bridges and bridges less than 20 ft (6 m) in span not receiving any or enough attention from skilled inspectors. Consultants are often hired to do this work and results are often reported to be inadequate. A local municipality was contacted as well as two local consultants and there does not appear to be a unified approach to the inspection of local bridges. Some, depending on size, location, and use, are inspected more rigorously than others. Some states may override the decisions of the local governments. Some state agencies have expressed the desire to be able to review the quality of local inspectors to ensure that they are adequately trained and performing inspections consistent with required standards.

Although the inspectors' training is considered to be adequate, many engineers have noted how the training in fatigue and fracture is not adequate. Engineers are reportedly not learning lessons from fatigue and fracture incidents because of a lack of understanding. There is concern that they are not able to predict future problem details. Better education in this area in engineering programs as well as short courses for practicing engineers could lead not only to better new bridge designs but also to more qualified and knowledgeable engineers to participate in maintenance and inspections.

In addition, approximately 65% of respondents indicated that special procedures were followed when inspecting FCBs. No details were provided as to what these special procedures included, however, it can be assumed that they involve a more thorough inspection requiring a greater level of effort.

Some of the following cost data were briefly discussed in chapter two. Costs associated with bridge inspection take up a considerable portion of each agency's budget. It is believed by many owners that inspection costs associated with FCBs consume a large portion of the budget dedicated to inspection. Questions were asked to determine if this belief is consistent with actual practice. There was considerable variation in the data obtained. This is partly because the survey did not clearly indicate to respondents what cost comparisons should be included. However, for a given agency, it is reasonable to assume that the person completing the survey compared the same items for each type of inspection. Therefore, although the response from different agencies cannot be compared, relative increases indicated from an individual survey are comparable.

Owners were asked to estimate what, if any, additional costs are incurred when inspecting an FCB. Surprisingly, the answers ranged from 0% to 6,000%, with most agencies indicating increases of from 200% to 500%. It should be noted that only two owners mentioned that there was none or negligible increases in costs associated with inspecting FCBs. (Some agencies did not reply to these questions.) The individuals who indicated that there were significant additional costs provided solid reasons for these increases, the most common of which were as follows:

- Additional costs associated with the use of specialized access equipment such as a snooper, manlift, or rigging. In many cases, non-FCBs can be inspected from the ground with binoculars. FCBs require "arm-length" access.
- Additional costs associated with traffic control to close lanes to permit the access equipment to be placed on or below the bridge. Indirect costs associated with lane closures were estimated by one agency to be $11,000 per lane per hour of closure. Thus, if inspection required two lanes to be closed for 3 h, there would be a cost of $66,000 to the motoring public.
- Increased costs associated with additional employee-hours required to conduct a detailed hands-on inspection.
- Additional costs associated with needs to more frequently perform NDT.
- Many states inspect FCBs at greater frequency than non-FCBs. This in itself may raise costs assuming that there are no other increases.

As stated, some agencies inspect FCBs more frequently than non-FCBs. As part of the survey, owners were asked to provide the intervals at which inspections are conducted. There was moderate variation in the responses, with intervals ranging from 2 to 5 years. Certainly, the condition of a given structure has a significant influence on the interval between inspections. Based on discussions with some owners, intervals of 6 months or even less are sometimes used in unusual cases where warranted. However, some agencies did not distinguish between the FCB and non-FCB bridges when determining the interval between inspections.

The survey revealed that there are differences in how owners inspect FCBs. When asked if the entire bridge is inspected or just the FCMs in detail, there appeared to be no clear consensus. Apparently, in the training course, Michael Baker, Jr., has indicated that if there are FCMs the entire bridge should be subjected to hands-on inspection. Some owners see no increase in costs by inspecting the entire bridge in greater detail, whereas others indicated that only the FCMs are included to reduce costs. Some owners noted that hands-on inspection is encountering significant problems in non-FCBs as well.

Inspectors reported that variance of as-built conditions when compared with what is shown on the plans is a problem. Documentation of the as-built details could be very useful. Examples include SR-422 in Pennsylvania (*37*), the Hoan Bridge (*38*), and other bridges with shelf–plate details for lateral bracing that were not built as shown in the plans and were not the same quality welding as expected. This and other problem details are discussed further in Appendix A.

In conversations with owners and bridge engineers it is often expressed that inspection intervals and the level of scrutiny should be flexible; being determined by the states and different for different bridge situations. Many individuals have expressed a desire to see such levels based on risk, which might include ADTT and the type of fatigue details. On the other hand, public and private inspectors want a cookbook procedure, owing to inadequate knowledge, not getting paid to make judgments, and concerns about liability if they do make judgments. A proper balance must be found between these two needs and the safety of the bridge. For example, John W. Fisher, Professor Emeritus of Lehigh University, expressed similar views during a personal interview conducted in June 2004. Dr. Fisher also stated that efforts related to inspection of a given bridge should be a function of the material used in the fabrication of the bridge. For example, a bridge constructed of HPS 70W material will not need to be inspected as often as bridges made from other steel, at least with respect to issues of fatigue and fracture. He suggests the following inspection scenario for new bridges. After construction, a bridge should be inspected every 2 years for the first 4 years to identify any critical issues, which are usually manifest early in the life of a bridge. If the condition of the bridge is acceptable after 4 years, then the inspection interval could be increased to 5 years for the next 10 to 15 years. The inspection frequency should be reconsidered and the inspection interval decreased, if needed, every 15 years.

Follow-up questions were asked of some agencies to determine how many states have centralized teams, including engineers and NDE technicians, that perform all the FCB inspections in the state. This is common in low population states such as Wyoming, because there are a limited number of personnel anyway. However, many of the larger states, such as Texas and Minnesota, also have centralized statewide teams. They noted the advantages of working with a snooper and continuity in having the same team do the inspections repeatedly. Other larger states assign inspections to regional divisions that do the non-FCB inspections as well—or there is a mix of some inspections of major or troublesome FCBs done by centralized teams, whereas inspections of smaller, benign FCBs are done by regions.

Approximately 20% of responding agencies have done more rigorous analysis to determine which members or portions of members are actually in tension and hence considered fracture-critical. The objective of this question was to ascertain if owners actually analyze a structure to determine which members, if they were to fail, would lead to the collapse of the bridge. For example, there may be many tension members in a large truss that are subject to tensile dead and live load stresses. However, through more advanced analysis, it can usually be shown that there may only be a few of these critical members that if lost as a result of fracture would lead to collapse. One example is Texas, which has its centralized team do this type of analysis in advance of inspecting all of their FCBs.

Unfortunately, this question does not seem to have been worded clearly enough to ensure that this is the level of analysis being referred to. Thus, it seems that some agencies indicating "analysis" is performed were not referring to the advanced analysis described previously. Nevertheless, it appears that a very small percentage of owners would perform this level of analysis and only in large critical structures.

In addition to the cost, inspectors and owners see many advantages to the hands-on fracture-critical inspections. They reported finding numerous problems with fatigue and corrosion that might not otherwise have been discovered. There are also reports of finding these problems in non-FCMs; therefore, hands-on inspection is good for all members of all bridges. These hands-on inspections are also reported to be useful for the purpose of bridge management; that is, for prioritizing bridges as part of an overall bridge replacement and rehabilitation program.

Inspection and Failures

The following question (no. 13) was asked related to inspection of FCBs and whether or not it has prevented any failures: Has the inspection of a fracture-critical bridge(s) ever identified a condition that has clearly prevented a fracture and the subsequent collapse of the structure?

Respondents were informed that the objective of this question was to identify specific cases or examples whereby the additional inspection efforts dedicated to FCBs prevented a failure that would have occurred had the inspection not been carried out. However, case examples that would not warrant a "yes" were provided. For example, the discovery of typical out-of-plane distortion cracks, which usually take years to become critical, would not warrant a response of "yes." Furthermore, inspections that found fractured members would also not warrant a response of "yes" because the inspection did not prevent the fracture and the bridge did not fail.

Interestingly, approximately 30% of the agencies that replied to this question answered "yes." However, in describing the specific case where this had occurred, the fracture of the member had already taken place. Thus, the respondent should have answered "no." Note that these responses also indicate cases where FCMs had fractured, and bridges did not collapse, and therefore would add to the few cases described in chapter two (see Figures 16 and 17).

When one of the owners who completed the survey was asked why they answered "yes" to the question, they indicated that although the fracture had occurred, the bridge did not collapse. In addition, there was no indication of any sagging of the structure. Hence, because the inspection found the fracture before any loss of structural integrity, the structure could be repaired before there was any more significant damage. (It should be noted that the fracture in question occurred in May 2003 under moderate temperatures. Had the fracture occurred in the winter during a cold period, it could have been much worse and significant damage introduced. Furthermore, the forensic investigation revealed that the fracture had occurred less than a few days to a maximum of one week before being discovered. Therefore, it is almost purely coincidental that the fracture was found before any additional damage occurred. Ironically, the owner had developed retrofit drawings and was about to let the construction contract to retrofit the bridge to prevent this type of problem at the time the failure occurred.)

Nevertheless, when the responses were closely reviewed and adjusted to properly answer the question, the percentage of "yes" respondents drops to 23%, which is still significant because failures were prevented.

Journal articles have reported on instances of fractures that were found during inspections; for example, the Edgewood Road Bridge in Cedar Rapids, Iowa (*31*). This bridge was being inspected by a private firm and a large fracture in the top flange of the twin-box structure was discovered at several locations. Although contracting inspections out worries many bridge owners, this firm was apparently doing a good job. Fractures in the Paseo Bridge on I-35 in Kansas City, Missouri, were noticed only when an 8 in. (200 mm) gap opened at the expansion joint (*39*).

Only 5% of those responding indicated that the use of HPS would influence how they view FCBs. Most believed that if a member fails, it does not matter what type of steel was used and that owing to the lack of redundancy inherent in the bridge, there was no advantage. There appeared to be little recognition of the significantly superior toughness of HPS in decreasing the potential for fracture of a given member. In addition, most agencies (79%) did not feel that eliminating poor fatigue details, say less than a Category C fatigue detail, would influence their decisions with respect to inspection frequency or the level of detail in new bridges. Some states base their inspection frequency on the types of fatigue details that are on the bridge and use this (and other data) to determine how often the bridge should be inspected and with what level of detail.

With respect to fatigue, it is commonly observed that by using simplified structural analysis methods the calculations for many bridges indicated no remaining fatigue life or even "negative" fatigue life. These calculations imply that fatigue cracking should be observed presently or in the near future on these bridges. However, such bridges typically show no signs of fatigue-related problems. (This does not include cracking from secondary stresses, such as web-gap cracking, because this type of cracking is not explicitly considered in fatigue rating calculations.) When asked about their agency's policy regarding cases when this inconsistency occurs, results were that one of the owners responding had a formal policy regarding this common problem. Most indicated that they do a more rigorous analysis or field instrumentation on a case-by-case basis. Even then, these efforts are mostly limited to use on larger or critical bridges. However, other factors, such as route, ADTT, existing condition, type of steel, and age also influence the decision to use the more advanced methods. Weigh-in-motion or advanced three-dimensional analyses are used by approximately 10% and 32% of the agencies, respectively, as needed. Interestingly, field instrumentation and load testing was used by approximately 45% of the agencies at one time or another. However, this is somewhat misleading, because an agency may have only used field instrumentation once in 10 years. Thus, although nearly half of those responding have indicated they have used field instrumentation to improve fatigue life predictions at one time, it is not used very often.

SUMMARY OF RESPONSES TO PART III—FAILURES

This section collects information relating only to bridge failures. Cracking associated with fatigue that did not result in fracture was not included as a failure. For example, out-of-plane distortion cracks were not to be counted as failures. However, fractures that resulted from a fatigue crack were to be included. Respondents were asked to distinguish between failures that occurred before and those that occurred after the implementation of the FCB inspection program. Failures in FCBs and non-FCBs were to be identified separately. Furthermore, owners were asked to distinguish between failures that were the result of impact, scour, and so forth, and those caused by fatigue or fracture. Unfortunately, the data collected related to these questions were not as complete as was desired and were very difficult to quantify.

The results of this section revealed that there have been very few failures of FCBs that were the result of fatigue or fracture. As discussed chapter two, with the exception of the Silver Bridge (1967) (Figure 1) and the Mianus River Bridge (1983) (Figure 4), no bridges have completely collapsed as a result of brittle fracture caused by fatigue or a flaw in the past 45 years. As discussed in chapter two, the failure of the Silver Bridge led to the development and implementation of the national bridge inspection program in the United States. Interestingly, the failure of the Mianus River Bridge in 1983 occurred after the implementation of that program. As discussed in chapter two, however, the failure of the Mianus River Bridge did lead to the development and implementation of the FCB inspection program, which was put into practice in 1988.

As mentioned earlier, respondents were asked to distinguish between all failures (fracture-critical and non-fracture-critical) that occurred before and after the implementation of the FCB inspection program. Unfortunately, many agencies did not indicate when the failure occurred. Based on the limited responses, it appears that most were not because the bridge was fracture-critical and could not have been prevented through inspection. Note that this is not to say that FCBs have not failed catastrophically for reasons such as overloads, scour, or impact. (A bridge posted for 3 tons that has collapsed because a 30-ton truck attempted to cross it did not fail simply because it was fracture-critical.) Rather, the data suggest that there are very few failures that have been caused by fatigue or brittle fracture in the absence of overloads, impact, scour, or corrosion.

Owners were asked if any special or formal investigative procedures were implemented when a failure had occurred to identify the cause of the fracture. Few of the agencies indicated that any formal procedures exist, although limited investigations were conducted on most failures. In other cases, the girder was repaired and no formal investigation was conducted. Therefore, failure investigations appear to be conducted at various levels on a case-by-case basis.

Another interesting observation from the survey data is related to the lack of owners documenting and archiving reports or data related to failures. Although many failures throughout the United States over the past 30 years are generally known, interestingly many states did not include these failures when replying to this section of the survey. It is assumed that the individual who responded had no personal knowledge of the failure or that the individuals who did either did not get the survey, left the organization, or simply forgot. This also implies that not all owners keep a centralized database of failures within their jurisdiction that can be easily accessed. Thus, new employees may not be adequately informed of previous problems on a given structure or when a problem previously studied arises.

The issue of individuals retiring or leaving state DOTs was reviewed in *NCHRP Synthesis of Highway Practice 313: State DOT Outsourcing and Private-Sector Utilization* (*55*). A primary objective of that report was to provide guidance on the outsourcing of major program responsibility of state transportation agencies. The outsourcing of the decision-making process and issues associated with procuring and administering outsourced activities are also discussed.

SUMMARY OF RESPONSES TO PART IV—RETROFIT PROCEDURES

This section was intended to identify any standard practices that have been developed to improve the redundancy of FCBs. Approximately 92% of the agencies responding to this question have not developed such policies.

In addition, agencies were asked to identify any research needs related to FCBs. A summary of the "yes" votes for each of the suggested potential research topics is summarized here. The top three are highlighted in bold.

- Develop guidelines related to advanced structural analysis procedures to better predict service load behavior in FCBs (8).
- **Develop advanced fatigue-life calculation procedures taking into account the lack of visible cracks for FCBs (9).**
- **Field monitoring for FCBs (10).**
- Crack arrest capabilities of bridge steel (3).
- Establish evaluation procedures for advanced large deformation and member loss (7).
- **Develop advanced analyses techniques and procedures to investigate alternate load paths, redundancy, and bridge collapse (10).**
- Develop retrofit procedures to add redundancy (1).

Although some examples were provided, owners were also asked to suggest other potential research topics. Only a few agencies suggested additional research topics. Three owners indicated that inspection frequency and extent for fracture-critical (and all) bridges should be risk-based and related to ADTT and fatigue details. FCBs on very low ADTT roads should not need the same frequency of inspection as those on busy Interstates. Two states indicated that they already conduct inspections as a function of ADTT. For these states, if the ADTT is less than 1,000 (150 for the other state), they are not required to perform the detailed inspections associated with FCBs.

Another potential topic mentioned was that the loading of the structure should be checked when investigating the potential for collapse. *NCHRP Report 406* (*34*), discussed in chapter two, provides a procedure that has been used successfully in practice. One respondent suggested that some guidance as to the extent of damage, analysis methods, magnitude of live load, impact, and so forth, be specified so that a designer can determine if there is the potential for collapse. It should be noted that the 2005 AASHTO LRFD is to have substantial additional commentary on how to analyze and address the loading and analysis issues for FCBs.

The results of the survey also suggest that the application of the AASHTO definition of FCMs and FCBs needs clarification, because there is considerable variability in the classification of structures.

One owner suggested that research be conducted to evaluate and identify issues related to overhead sign bridges and high-mast lighting towers because these are often fracture-critical structures. They further indicated that these structures actually give them more problems than bridges; however, there are no standardized inspection and fabrication specifications for these structures.

SUMMARY OF SURVEY RESULTS

The results of the survey can be summarized as follows:

- Owners use a consistent definition for FCBs that is in agreement with that provided by AASHTO. However, how they establish which bridges are fracture-critical is much more variable. A bridge that is determined to be fracture-critical in one state may not be identified as such in another state. Often, the decision is based on engineering judgment.
- Additional costs associated with the inspection of FCBs can be considerable, both in terms of direct dollar costs and the additional indirect costs to the public as a result of lane closures and traffic delays. Most agencies reported inspection costs that were 2 to 5 times greater for FCBs than for non-FCBs. Increases in inspection costs of 10 to 50 times have been reported for some structures.
- There are two documented collapses of FCBs where the structure catastrophically failed. One is the Point Pleasant Bridge that failed before any initiatives related to the FCP and NBIS. The second is the Mianus River Bridge, which failed before the implementation of the FCB inspection program. Other failures that were the result of unreasonable overloads should not be directly attributed to the circumstance of the bridge being fracture-critical. Other failures have occurred as a result of scour or impact; but again, these failures are independent of bridge type.
- There have been several fractures of bridges identified as FCBs over the last 30 years that have occurred without collapse or resulting in fatality. The apparent adequacy of alternate load paths within all of these structures has provided substantial redundancy, although they were not designed as such.
- Owners identified the following as the most important areas for future research related to FCBs:
  - Develop load models, criteria for extent of damage, and guidelines related to advanced structural analysis procedures to better predict service load behavior in FCBs and the behavior after fracture of an FCM, including dynamic effects from the shock of the fracture and, if necessary, large deformations.
  - Develop advanced fatigue-life calculation procedures taking into account a lack of visible cracks for FCBs.
  - Provide field monitoring for FCBs.
  - Develop rational risk-based criteria for inspection frequency criteria and level of detail based on ADTT, date of design, and fatigue detail categories present.
  - Evaluate fracture-critical issues related to sign, signal, and light supports. (Respondents indicated that these structures give them more problems than bridges).

CHAPTER FOUR

DISCUSSION

RESULTS OF FRACTURE CONTROL PLAN—25 YEARS LATER

It has been approximately 25 years since the introduction of the FCP, as previously found in the AASHTO/AWS-D1.5 (*2*). Subsequent to the FCP, there have been relatively few FCBs designed (e.g., two-girder bridges and two-truss systems), when compared with before the plan. This is primarily because of the added costs placed on fracture-critical structures in terms of materials, fabrication, and in-service inspection. Nevertheless, there have been some notable long-span FCBs built. Some states continue to use steel cross girders and twin tub girders. For example, Texas has built approximately 190 FCBs in the last 20 years. Among bridges designed after the FCP, the fracture toughness, detailing, and weld quality has been much better than before, resulting in members with greater reliability than non-FCMs. Approximately 11% of steel bridges have FCMs; however, 76% of these were built before the FCP. Most of these, 83%, are two-girder bridges and two-line trusses, and 43% of the FCMs are riveted.

Based on the results of the survey and personal communications with industry and academic experts, there do not appear to have been any failures (certainly none where there was loss of life) of any structures built after the implementation of the FCP. However, it must also be noted that there have been only two such catastrophic bridge failures identified in the past 40 years that are attributed to fracture. Hence, it is difficult to measure the success of the FCP, and the NBIS requirements for hands-on inspection introduced in 1988, in preventing catastrophic failure of bridge structures, because at least one of the two (Mianus River) could have been prevented through routine inspection and better maintenance of drainage. Fabricators question the need for much of the testing, noting that the procedures become routine and automatic after building FCMs for years.

Nonredundant is a broader term than FCM because nonredundant also includes:

- Substructures;
- Members that may be inherently not susceptible to fracture, such as compression members, but still could lead to collapse if damaged by overloading, earthquakes, fire, terrorism, ship or vehicle collisions, and so forth; and
- Members made of materials other than steel.

Interestingly, extreme loadings of substructure elements such as piers have led to most of the spectacular collapses of both steel and concrete bridges.

In addition to prevention of collapse in the event of fracture, redundancy of the superstructure is important for several other reasons. The first is the need to more easily redeck the bridge. Also, events other than fracture can damage and completely destroy members of the superstructure. These are compelling reasons for redundancy. These reasons for redundancy (other than fracture) should not be used to justify unnecessary requirements for FCMs, a subset of nonredundant members. The two sets of bridge members, nonredundant members and the subset FCMs, should be addressed separately as appropriate.

These reasons for redundancy (other than fracture) should be used to encourage redundancy outright instead of indirectly by penalizing FCMs. For example, in Sections 1.3.2 and 1.3.4 of the *LRFD Specifications,* redundancy is encouraged. Load factors are modified based on the level of redundancy and it is stated that multiple-load-path and continuous structures should be used unless there are compelling reasons to do otherwise.

In the *AASHTO Manual for Condition Evaluation and Load Resistance Factor Rating* (*LRFR*) *of Highway Bridges,* redundancy is reflected in system factors that reduce the capacity of each member in nonredundant systems. The system factors are calibrated so that nonredundant systems are rated more conservatively at approximately the level of reliability associated with new bridges designed by the *LRFD Specifications,* called the “inventory” level in former rating procedures. Redundant systems are rated at a reduced reliability level corresponding approximately to the traditional “operating” level.

Redundancy is related to system behavior rather than individual component behavior and is often discussed in terms of the following three types:

- Internal redundancy, also called member redundancy, exists when a member is comprised of multiple elements and a fracture that formed in one element cannot propagate directly into adjacent elements.
- Structural redundancy is external static indeterminacy and can occur in a two-or-more-span continuous girder or truss.

- Load-path redundancy is internal static indeterminacy arising from having three or more girders or redundant truss members. One can argue that the transverse members such as diaphragms between girders can also provide load-path redundancy.

Internal and structural redundancies are often neglected, but this is clearly oversimplifying and possibly overconservative. Retrofits are described that have been used to add redundancy to bridges with FCMs.

Ultimately, it is the target level of reliability that designers and raters of bridges should strive to achieve; they should not focus exclusively on redundancy. Redundancy has a major impact on the risk of collapse and this impact is accounted for appropriately for all types of structures in both the *LRFD Specifications* and the *LRFR Manual.* Using these LRFD and LRFR procedures, it is possible to achieve the target level of reliability without redundancy in a bridge that is more conservatively designed, by only approximately 17%. For example, a nonredundant bridge designed for an HS-25 loading would have greater reliability than a redundant bridge designed for HS-20 loading.

The cost premium for higher toughness is lower than it was 30 years ago. Even ordinary bridge steel typically has much greater than minimum specified notch toughness, except perhaps for Zone 3. The new HPS provide a toughness level that far exceeds the minimum requirements. This has significantly altered the economic factors that were considered in setting the current AASHTO toughness requirements. There is a reasonable argument for increasing the notch toughness requirements. Extremely large cracks, greater than 14 in. (350 mm), could be tolerated in mild steel if the temperature shifts were zero. An increase could be combined with some loosening of the definition of FCMs that would, for example, allow two-girder bridges. If full advantage were taken of the toughness of HPS, research suggests it would be possible to:

- Eliminate special in-service inspection requirements for fracture-critical structures for HPS,
- Reduce the frequency and need for hands-on fatigue inspections for HPS,
- Eliminate of the penalty for structures with low redundancy for HPS.

IDENTIFYING FRACTURE-CRITICAL BRIDGES

Assuming that the FCP is serving its purpose, the literature, survey, and several failures appear to suggest that it is not the FCP that needs to be revisited. Rather, how individual bridges are characterized as fracture-critical in the first place may require further refinement. There are varying classifications of superstructure types as having FCMs and consequently there is wide disagreement. For example, twin box girders would be expected to perform even better than twin I-girders owing to the torsional capacity of the intact box girder and the alternate load paths available within each box girder; however, most agencies consider these FCMs. These structures contain four webs and are fully composite with the concrete deck and unlikely to collapse catastrophically. It seems unreasonable to consider these structures fracture-critical. However, the *AASHTO LRFD Bridge Design Specifications* do not specifically address what types of superstructures have FCMs.

Common assumptions that fractures in certain superstructure types will lead to collapse are too simplistic. Numerous bridges have had a full-depth fracture of an FCM girder and did not collapse, usually owing to the alternative load carrying mechanism of catenary action of the deck under large rotations at the fracture. It is apparent that other elements of these two-girder bridges, particularly the deck, are sometimes able to carry the loads and prevent collapse in these FCBs. These alternate load paths were so robust in some of these failures that there was little or no perceptible deformation of the structure. For example, the I-79 Neville Island Bridge over the Ohio River, which would have been classified as fracture-critical, did not collapse and actually carried traffic for a considerable period. Therefore, was this bridge actually fracture-critical? Furthermore, it was fabricated before the implementation of the FCP; therefore, it serves as an example of the robustness of structures that were not fabricated to the higher requirements.

There are many other examples of bridges classified as fracture-critical or containing FCMs that have exhibited complete fractures in girders, cross girders, hangers, and other members that continued to perform until the failure was discovered, sometimes by accident. Therefore, the question becomes, should these bridges or members have been classified as fracture-critical in the first place because collapse did not occur?

Clearly, one of the greatest research needs is related to how to determine if a bridge should be considered fracture-critical. This finding is consistent in two ways with the suggestions noted on the survey. First, there was considerable variation in how individual agencies classify bridges (i.e., as fracture-critical or non-fracture-critical). Second, those individuals who identified areas of needed research selected topics related to this area as being the most needed.

The capacity of damaged superstructures (with the FCM "damaged" or removed from the analysis) may be predicted using refined three-dimensional analysis. However, there is a strong need to clarify the assumptions, extent of damage, load cases and factors, and dynamic effects in these analyses. The commentary of the *AASHTO LRFD Bridge Design Specifications* states that:

> The criteria for refined analysis used to demonstrate that part of a structure is not fracture critical has not yet been codified.

> Therefore, the loading cases to be studied, location of potential cracks, degree to which the dynamic effects associated with a fracture are included in the analysis, and fineness of the models and choice of element type should all be agreed upon by the owner and the engineer. The ability of a particular software product to adequately capture the complexity of the problem should also be considered and the choice of software should be mutually agreed upon by the owner and the engineer. Relief from full factored loads associated with the Strength I Load Combinations of Table 3.4.1-1 should be considered as should the number of loaded design lanes versus the number of striped traffic lanes.

NCHRP Report 406 (*34*) gives practical requirements for the residual capacity of the damaged superstructure.

This same type of analysis is being used to evaluate older structures as well to better direct resources for maintenance and replacement; for example, if a bridge is not really fracture-critical then it may not be necessary to replace it as soon.

Other countries do not make a distinction in FCMs; however, scanning tours have noted that three-dimensional refined analysis is more often used in the design and evaluation of bridges in Europe and that the inspection interval is often based on risk. Some in the United States believe that fracture-critical inspection should be less frequent for the modern bridges as a result of the much better detailing and materials, and perhaps should be based on truck traffic, fatigue details, and other risk factors.

ROLE OF INSPECTION FOR FRACTURE-CRITICAL BRIDGES

There is no doubt that the hands-on fracture-critical inspections have revealed numerous fatigue and corrosion problems that otherwise might have escaped notice. Many of these problem details are discussed in Appendix A. Twenty-three percent of the survey respondents (see chapter three) indicated that they had found significant cracks and corrosion that could have become much worse; and in doing so, possibly averting collapses. Similar examples may be found in trade magazines [see Zettler (*31*)]. However, agencies also report finding these problems on non-FCBs when hands-on inspections are performed. Therefore, such inspections are good for all steel bridges, not just FCBs.

However, owners spend a major portion of their budget on efforts associated with the inspection and maintenance of all structures. Fracture-critical inspections consume a large fraction of that budget for a comparatively few structures. The cost of the fracture-critical inspection is typically two to five times greater than inspections for bridges without FCMs. Inspections of closed sections such as tub girders and tie members are extremely expensive because inspectors must get inside such sections. What constitutes a fracture-critical inspection is subject to interpretation and disagreement. The frequency of fracture-critical inspections is actually not specified and varies up to every 5 years, is typically every 2 years, but often is every year or more frequently if there are specific problems. It is not known if efforts beyond what is normally put forth during the inspection of non-FCBs are actually warranted; however, even if there are only a few cases of such efforts preventing a major failure, they are most likely worthwhile.

Questions do arise with respect to inspection requirements for newer bridges, which were built using superior steels (especially using any type of HPS), subjected to advanced NDT techniques, and fabricated using higher-quality welding procedures than used in the past. In addition, those fabricated using the FCP should be of superior quality than bridges built before the introduction of the FCP. Modern steel bridges are also built with a composite deck slab and are inherently more capable of carrying redistributed loads through alternate paths. Two-girder curved-girder bridges, especially those built since the early 1980s, almost always contain heavy transverse cross frames capable of carrying a significant load from one girder to another.

In addition, based on the survey responses and information received during interviews of recognized experts in fatigue and fracture, it was found that there is a strong sense that the inspection interval should in some way be related to risk, ADTT, and age of the structure. Possibly, bridges built after the implementation of the FCP or with HPS can be inspected at greater intervals. Furthermore, bridges with low ADTT could be inspected at a frequency somehow related to traffic data. The approach could be similar to that used in the aircraft industry, where airframes are inspected based on hours of flight, not just years of service. It is recognized that issues related to corrosion, settlement, scour, and so forth, which can only be quantified through regular inspection, are not related to ADTT. However, these issues are common to all nonredundant bridges and not just FCMs.

Research in risk-based inspection seems justified because the payoff can be substantial. For example, there would be significant savings if it could just be shown that bridges can be inspected every 3 years instead of 2. The potential savings should be considered when developing the scope and budget for such a research project to ensure that reliable data are collected and safe recommended changes to the inspection program established.

It must be emphasized that before inspection intervals are adjusted, it is critical that the potential ramifications be thoroughly understood. It is possible that some components need to be inspected on the schedule currently in place. For example, the existing inspection intervals might need to remain unchanged to ensure that problems with elements such as deck slabs and bearings do not go undetected. It was also suggested by a few of the designers who were interviewed that it might be important to inspect the deck, for example, more frequently than the steel superstructure. Another example would be a bridge susceptible to scour, where it might be required to inspect the substructure at a greater frequency than the superstructures. Nevertheless, it is clear that before changes are made there are many factors that must be considered.

Education and Training of Inspectors

Overall, training of inspectors seems to be adequately available through the existing courses offered by the National Highway Institute. None of the responding agencies identified any problems with the current educations strategies. However, one area that appears to need additional effort is related to the documentation and archiving of previous failures and problems. As discussed in the previous chapter, failures known to have occurred in certain states were not always reported in the replies to the survey. Therefore, a better method of tracking such information appears to be needed.

Education and Training of Engineers

Although not evident from the surveys, discussions with industry leaders have also revealed that a general knowledge gap exists with respect to fatigue and fracture design, evaluation, and behavior in the engineering community. This is the result of several factors. First, few academic institutions offer any substantial material in undergraduate coursework on fatigue and fracture. Therefore, recent graduates must learn on the job and often simply follow cookbook specification approaches with little understanding of the spirit of the specifications. Second, there has been a major shift in the experience level in U.S. DOTs over the last several years. Many of the most experienced engineers are retiring and with their departure will be lost the years of experience acquired during the period when most of the issues with fatigue and fracture were foremost (1970–1990).

To address this, several owners and engineers expressed the need for additional training. Although current National Highway Institute courses seem adequate for training inspectors and providing overall guidance, a more in-depth course appears to be needed for bridge designers. For example, the Pennsylvania DOT has sponsored the development of a short course on fatigue and fracture design for its engineers and consultants. In addition, FHWA is currently (2005) considering the development of a similar course designed to address this need that would be taught in strategic locations across the United States.

CHAPTER FIVE

CONCLUSIONS

The fracture control plan (fabrication provisions and Charpy V-notch requirements) and new fatigue specifications have necessitated substantial changes in the design and fabrication of bridges. As a result, modern bridges are much less susceptible to fatigue, corrosion, and fracture than bridges designed before 1975 (and 1985 for web-gap cracking).

In the United States, the fatigue-cracking problem in steel bridges is essentially confined to bridges designed before 1975, expect for web-gap cracking in bridges designed up to 1985. Approximately 11% of steel bridges have fracture-critical members (FCMs) and 76% of these were built before the implementation of the fracture control plan. Most of these (83%) are two-girder bridges and two-line trusses, and 43% of the FCMs are riveted.

There are numerous examples of FCMs where one girder in a two-girder bridge has fractured but the bridge does not even partially collapse. It is apparent that the deck deflects and begins to act as a catenary to carry the load. Other countries do not have additional provisions for FCMs and their associated costs and continue to use fracture-critical designs with no apparent problems. European countries use three-dimensional analysis to design and assess their bridges, and the inspection interval is often based on risk.

Very few traditional FCM bridges (e.g., two-girder bridges or two-truss systems) are now being built. In some cases, these systems are potentially more efficient; however, there is additional initial cost and added inspection cost.

Ultimately, designers and raters of bridges strive to achieve the target level of reliability. Redundancy has a significant impact on the risk of collapse, and this impact is accounted for appropriately for all types of structures in both the *AASHTO LRFD Bridge Design Specifications* and the *Manual for Condition and Load and Resistance Factor Rating* (*LRFR*) *of Highway Bridges.* It is possible to achieve the target level of reliability without redundancy in a bridge that is more conservatively designed. For example, a nonredundant bridge designed for an HS-25 loading would have greater reliability than a redundant bridge designed for HS-20 loading.

There could be a useful distinction between the subset of FCMs and the encompassing set of all nonredundant members, which includes substructures; members that may be inherently not susceptible to fracture, such as compression members, but still could lead to collapse if damaged by overloading, earthquakes, fire, terrorism, ship or vehicle collisions, and so forth; and members made of materials other than steel. Substructures such as piers are often nonredundant and have been responsible for most of the spectacular collapses of both steel and concrete bridges.

The cost premium for higher toughness is lower than it was 30 years ago. Even ordinary bridge steel typically has much greater than minimum specified notch toughness, except perhaps for Zone 3. The new high-performance steels (HPS) provide a toughness level that far exceeds the minimum requirements. This has significantly altered the economic factors that were considered in setting the current AASHTO toughness requirements. However, there is a reasonable argument for increasing the notch toughness requirements. Extremely large cracks, greater than 14 in. (350 mm), could be tolerated in mild steel if the temperature shift was zero. An increase could be combined with some loosening of the definition of FCMs that would, for example, allow two-girder bridges. If full advantage were taken of the toughness of HPS, research suggests it would be possible to:

- Eliminate special in-service inspection requirements for fracture-critical structures for HPS,
- Reduce frequency and need for hands-on fatigue inspections for HPS, and
- Eliminate the penalty for structures with low redundancy for HPS.

The capacity of damaged superstructures (with the FCM "damaged" or removed from the analysis) may be predicted using refined three-dimensional analysis. However, there is a strong need to clarify the assumptions, extent of damage, load cases and factors, and dynamic effects in these analyses. *NCHRP Report 406: Redundancy in Highway Bridge Superstructures* gives practical requirements for the residual capacity of the damaged superstructure. This type of analysis and associated waiver of the FCM provisions is presently being done on a case-by-case basis. This same type of analysis is being used to evaluate older structures as well to better direct resources for maintenance and replacement; for example, if a bridge is not really fracture-critical then it may not be necessary to replace it as soon.

Owners are not consistent in classifying bridges as fracture-critical. In the *LRFR Manual* (which is identical to the earlier

Manual for Condition Evaluation of Bridges) and the *Bridge Inspector's Reference Manual,* it is stated that twin box or tub girders are not considered FCM. It is also stated that:

- Only welded tie members for arches or single-box girders are considered FCMs, whereas riveted or bolted built-up tie members or single-box girders would not be FCMs. This would appear to be giving credit to the internal redundancy of the bolted or riveted built-up members.
- Not all two-I-girder bridges are considered FCMs; in the *Bridge Inspector's Reference Manual,* even simple-span two-I-girder bridges are not FCMs unless they have cover plates, shelf plates, or a suspended span.

The *AASHTO LRFD Bridge Design Specifications* are not clear about what types of superstructures have FCMs.

It has been conclusively determined that hands-on, fracture-critical inspections have revealed numerous fatigue and corrosion problems that otherwise might have escaped notice. Twenty-three percent of survey respondents indicated that they found significant cracks and corrosion that could have become much worse, possibly averting collapses. However, transit agencies also reported finding these problems on non-FCMs when hands-on inspections are done. Therefore, inspections such as these are good for all steel bridges, not just FCMs.

The cost of a hands-on, in-service inspection for FCMs is estimated to be two to five times that of such inspections of non-FCMs and may not always be required. Inspection frequency of existing bridges could be based on risk factors such as truck traffic, type of details, and date of design; that is, a distinction could be made between (1) bridges designed before 1975, (2) bridges designed between 1975 and 1985, and (3) modern bridges (designed after 1985) with only Category C details or better.

Training of inspectors seems to be adequately available through the existing National Highway Institute courses. One area needing additional training is fatigue and fracture for design engineers. Also, additional effort is required for national documentation and archiving of previous failures and problems.

Recently, FHWA proposed that the states create FCM inspection plans. However, survey data suggest that before the development of such a plan, a more comprehensive method of classifying bridges as fracture-critical could be developed to ensure consistent application of such inspection standards.

Innovative retrofits have been developed and implemented successfully to improve the redundancy of FCBs, including using post-tensioning and bolted redundancy plates.

Owners identified the following as the most important areas for future research as related to FCBs:

- Develop load models, criteria for the extent of damage, and guidelines related to advanced structural analysis procedures to better predict service load behavior in FCM bridges and the behavior after fracture of an FCM, including dynamic effects from the shock of the fracture and, if necessary, large deformations.
- Develop advanced fatigue-life calculation procedures, taking into account a lack of visible cracks for fracture-critical bridges.
- Investigate field monitoring for fracture-critical bridges.
- Develop rational risk-based criteria for inspection frequency criteria and level of detail based on average daily truck traffic, date of design, and fatigue detail categories present.
- Evaluate fracture-critical issues related to sign, signal, and light supports.

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APPENDIX A

Background Discussion on Fatigue, Fracture, Nondestructive Evaluation, and Repair and Retrofit

OVERVIEW OF FATIGUE

Fatigue is considered a serviceability limit state for bridges. This is because fatigue cracks do not typically compromise structural integrity and are more of a maintenance issue. However, as was recognized by the Task Committee on Redundancy of Flexural Systems (*A1*), fatigue is the most common cause of reported damage in steel bridges.

The fatigue design and assessment procedures outlined in this appendix are included in the AASHTO specifications for bridges (*A2*). As a result, steel bridges that have been built in the last two decades have not and should not have any significant problems with fatigue and fracture (*A3*). However, bridges designed before the modern specifications will continue to be susceptible to the development of fatigue cracks and to fracture.

Detailing rules are perhaps the most important part of the fatigue and fracture design and assessment procedures. These rules are intended to avoid notches and other stress concentrations, as well as the use of details known to be very fatigue sensitive. They also often result in details that have improved resistance against brittle fracture as well as fatigue. Modern steel bridges are also detailed in a way that appears much cleaner than those built before the 1970s. There are fewer connections and attachments in modern bridges and the connections use more fatigue-resistant details, such as high-strength bolted joints.

In bridges there are usually a large number of cycles of significant live load and fatigue will almost always precede fracture. Therefore, controlling fatigue is practically more important than controlling fracture. Usually, the only measures taken in design that are primarily intended to ensure fracture resistance are those to specify materials with minimum specified toughness values [such as a Charpy V-notch (CVN) test requirement]. As explained in chapter three, toughness is specified so that the structure is resistant to brittle fracture despite manufacturing defects, fatigue cracks, and/or unanticipated loading. However, these material specifications are less important for bridges than the *S–N* curves and detailing rules.

Nominal Stress *S–N* Curves

The established method for fatigue design and assessment of steel bridges in the United States is the nominal stress approach. The nominal stress approach is based on *S–N* curves, where *S* is the nominal stress range and *N* is the number of cycles until the appearance of a visible crack. Details are designed based on the nominal stress range in the connecting members rather than the local "concentrated" stress at the detail. The nominal stress is usually obtained from standard design equations for bending and axial stress and does not include the effect of stress concentrations of welds and attachments. AASHTO (*A2*) has seven *S–N* curves corresponding to seven categories of weld details (A through E'), as shown in Figure A1.

The fatigue design procedure is based on associating the weld detail under consideration with a specific category. The effects of the welds and other stress concentrations, including the typical defects and residual stresses, are reflected in the ordinate of the *S–N* curves for the various detail categories. Consequently, the variability of fatigue life data at a particular stress range is typically about a factor of 10.

The AASHTO *S–N* curves in Figure A1 are also used throughout North America for a variety of other welded structures, including the American Institute for Steel Construction *Manual of Steel Construction* (*A4*), the American Railway Engineering and Maintenance-of-Way Association *Manual for Railway Engineering* (*A5*), the American Welding Society (AWS D1.1) Structural Welding Code (*A6*), and the Canadian Standards Association (CSA S16-2001) *Limit States Design of Steel Structures* (*A7*).

The *S–N* curves can be represented by the following power law relationship:

$$N = \frac{A}{S^3} \tag{1}$$

where *N* is the number of stress cycles, *S* is the nominal stress range, and *A* is a constant particular to the detail category, as given in Table A1.

In the nominal stress range approach, each detail category also has a constant-amplitude fatigue limit (CAFL), which is given in Table A1. The CAFL is the stress range below which no fatigue cracks occurred in tests conducted with constant-amplitude loading.

The approach to designing and assessing bridges for fatigue is empirical and is based on tests of full-scale members with welded or bolted details. Such tests indicate that:

- The strength and type of steel have only a negligible effect on the fatigue resistance expected for a particular detail (*A8–A10*).
- The welding process also does not typically have an effect on the fatigue resistance (*A11,A12*).

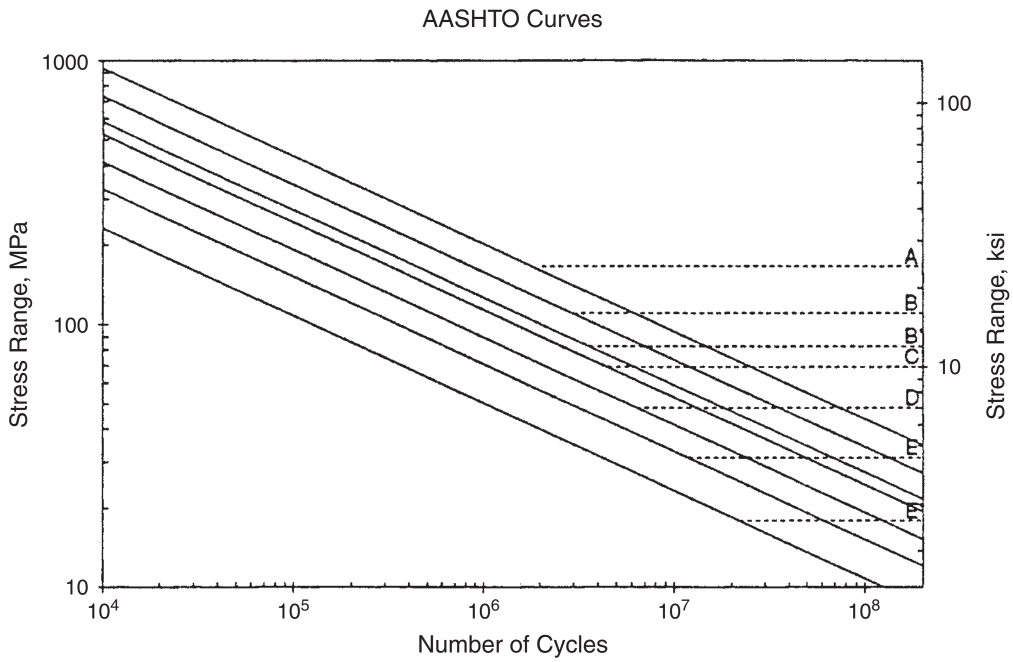

FIGURE A1 Lower-bound *S–N* curves for seven primary fatigue categories from AASHTO specifications. Dotted lines are the constant-amplitude fatigue limits and indicate detail category.

- The primary effect of constant-amplitude fatigue loading can be accounted for in the live-load stress range (*A8–A10*); that is, the mean stress is not significant. (The reason that the dead load has little effect is that, locally, there are very high residual stresses.)

It is worth noting that when information about a specific crack is available, a fracture mechanics crack growth rate analysis should be used to calculate remaining life (*A13–A16*). However, in the design stage, without specific initial crack size data, the fracture mechanics approach is not any more accurate than the *S–N* curve approach (*A17*). Therefore, there will be no further discussion of the fracture mechanics crack growth analysis.

Effect of Corrosion on Fatigue Resistance

Many bridge owners are concerned about the influence of corrosion on the fatigue and fracture performance of both fracture-critical and non-fracture-critical bridges (FCBs and non-FCBs). However, the concern is greater with respect to fracture-critical structures, as indicated by several responses to the survey and as discussed in chapter three.

TABLE A1
PARAMETERS FOR *S–N* CURVES

AASHTO Category	Coefficient A (MPa^3)	CAFL (MPa)
A	81.9×10^{11}	165
B	39.3×10^{11}	110
B	20.0×10^{11}	83
C	14.4×10^{11}	69
D	7.21×10^{11}	48
E	3.61×10^{11}	31
E’	1.28×10^{11}	18

CAFL = constant-amplitude fatigue limit.

The full-scale fatigue experiments upon which the fatigue rules are based were carried out in moist air and therefore reflect some degree of environmental effect or corrosion fatigue. Hence, the lower-bound *S–N* curves in Figure A1 can be used for the design of details with a mildly corrosive environment (such as the environment for bridges, even if salt or other corrosive chemicals are used for deicing) or provided with suitable corrosion protection (galvanizing, other coating, or cathodic protection). Some design codes for offshore structures have reduced fatigue life by approximately a factor of two when details are exposed to seawater (*A18,A19*).

At the relatively high stress ranges at which most accelerated tests are conducted, the effect of seawater is clearly detrimental. However, there is evidence that the effect of corrosion in seawater is not so severe for long-life, variable-amplitude fatigue of welded details. Full-scale fatigue experiments in seawater at realistic service stress ranges do not show significantly lower fatigue lives, provided that corrosion is not so severe that it causes pitting or significant section loss (*A20*). At relatively low stress ranges near the CAFL typical of service loading, it appears that the build-up of corrosion product in the crack may actually increase crack closure and retard crack growth, at least enough to offset the increase that would otherwise occur owing to the environmental effect. The fatigue lives seem to be more significantly affected by the stress concentration at the toe of welds than by the corrosive environment.

Severely corroded members may be evaluated as Category E details (*A21*), regardless of their original category (unless of course they were Category E′ or worse to start with). However, pitting or significant section loss from severe corrosion can also lower that fatigue strength (*A21,A22*).

Loading for Fatigue Design

It is important to note that the load producing fatigue cracking is comprised of the entire load spectrum crossing the bridge. The load that produces a fracture, on the other hand, is typically the largest load from the load spectrum. Bridges experience what is known as long-life, variable-amplitude loading [i.e., very large numbers of random amplitude cycles greater than the number of cycles associated with the CAFL (*A23*)].

If the percentage of stress ranges exceeding the CAFL is greater than approximately 0.01%, the history of *N* variable stress ranges can be converted to *N* cycles of an effective stress range that can then be used just like a constant-amplitude stress range in *S–N* curve analysis. Typically, Miner's rule (*A24*) is used to calculate an effective stress range from a histogram of variable stress ranges. Theoretically, this effective constant-amplitude stress range results in approximately the same fatigue damage for a given number of cycles as the same number of cycles of the variable-amplitude service history. If the stress ranges are counted in discrete "bins," as in a histogram, the effective stress range, S_{Re} (*A23*), can be calculated as:

$$S_{\mathrm{Re}} = \left(\sum_i \left(\alpha_i S_{ri}^3 \right) \right)^{1/3} \tag{2}$$

where α_i = number of stress cycles with stress range in the bin with average value S_{ri} divided by the total number of stress cycles (*N*).

In the AASHTO specifications (*A2*), the stress range from the fatigue design truck (i.e., the HS15) represents the effective stress range. No additional safety factor is used for Miner's rule, because it is relatively accurate for truck loading on bridges. For large numbers of cycles, the AASHTO specification has another check that involves comparing the stress range from the fatigue design truck with one-half of the CAFL. This check is actually intended to compare the CAFL with a stress range that is twice that produced by the fatigue design truck (i.e., dividing the resistance by two is the same as multiplying the load by two). Although somewhat confusing in application, the intent is to ensure that almost all of the stress ranges should be below the CAFL, but that occasionally the stress range can exceed the CAFL with no significant effect.

Fatigue Life Prediction Methodology for Existing Bridges

The 1990 AASHTO *Guide Specifications for the Fatigue Evaluation of Existing Steel Bridges* (*A25*) can be used for the fatigue evaluation of steel bridges. The purpose of the guide is to provide procedures for calculating the remaining fatigue life of existing steel bridges using concepts of probabilistic limit states. The fatigue evaluation procedures in the guide were adopted from and are identical to the proposed procedures presented in *NCHRP Report 299: Fatigue Evaluation Procedures for Steel Bridges* (*A26*). The effects of repeated loading on the fatigue life of an existing bridge are defined in terms of the remaining life of the structure. This means that the effect of exceeding the allowable fatigue stress is a reduction of the remaining fatigue life of the structure rather than immediate failure.

There are two levels for which the remaining fatigue life can be calculated, the remaining safe life and the remaining mean life. The remaining safe life provides a much higher level of safety. The safe life has an exceedence probability of 97.7% for redundant members (the same probability inherent in using the basic design *S–N* curves). The mean remaining life, however, is the best estimate of the actual remaining fatigue life of the detail under consideration. The mean life has an exceedence probability of 50%.

The safe life is used for design and for a first screening analysis. The impact of reaching a safe life equal to zero should be relatively minimal; it may mean having to repair just a small percentage of the details on a bridge. In this case, the calculated mean life may be used to determine when the cracking will be so pervasive that half the details will be cracking, requiring extensive retrofitting or possibly replacement of the bridge. If the remaining life that is estimated is not satisfactory, there are four options: (1) recalculate the fatigue life more accurately (possibly using load testing), (2) restrict truck traffic on the bridge, (3) repair or modify the detail, and/or (4) perform more frequent inspections of the detail.

The procedures in *NCHRP Report 299* (*A26*) are the best available procedures for predicting the number of years before substantial fatigue cracking *may* occur at a detail. It is difficult to accurately predict all of the variables that the bridge will experience over its remaining life, such as past, present, and future truck volumes and stress ranges, which can also vary with changing truck weights over time.

Although calculations are by far the most common and favored methods of estimating remaining fatigue life in a bridge, they are also the least accurate. Field instrumentation and monitoring has consistently demonstrated that measured in-service stress range histograms result in the most accurate estimates of remaining life. Analysis methods rely on approximate load models and simplified structural analysis models, both of which are typically conservative. Measurements made in the field reflect actual site conditions and traffic patterns when recorded over a sufficient period of time. Because the stress range is directly measured at the detail in question, there is no error introduced through analytical simplifications.

Because in nearly all cases field measurements indicate that in-service stresses are less than predicted, costly retrofits can be eliminated or reduced in number. The resulting savings will almost always far exceed the cost of the additional efforts associated with the field instrumentation. Despite this potential for savings, most agencies do not use this approach, as will be discussed in the review of the survey results.

OVERVIEW OF FRACTURE

Fracture may be defined as rupture in tension or rapid extension of a crack, leading to gross deformation, loss of function or serviceability, or complete separation of the component (*A4,A14,A27*). Because the scope of this synthesis is limited to practical information, there are many interesting aspects of fracture that are not discussed. However, there are several good texts that can serve as a starting point for more in-depth studies (*A14,A15,A27*).

It is important to rapidly assess the potential for fracture whenever there is a crack in any tension element (tension member or tension flange of flexural member). However, the occurrence of fatigue cracks does not necessarily mean that the structure is in danger (*A13,A27*). In some redundant structures, a fatigue crack may stop propagating with no intervention at all as a result of redistribution of stresses (*A28*). Usually, however, a fatigue crack will propagate and eventually cause a fracture if not repaired in a timely manner (*A13,A15,A27*). The development of a fatigue crack in a fracture-critical member (FCM) should, in most cases, warrant closure of one or more lanes, posting the bridge for cars only, or result in complete closure of the bridge until repairs can be made. Fracture is the rupture in tension or rapid extension of a crack leading to gross deformation, loss of function or serviceability, or complete separation of the component.

Details that have good fatigue resistance, Category C and better, are usually also optimized for resistance to fracture. Detailing rules to avoid fracture are very similar to the common sense rules to avoid fatigue. For example, intersecting welds should always be avoided owing to the probability of defects and excessive constraint. Intersecting welds, or even welds of too close proximity, have caused brittle fractures [e.g., the Hoan Bridge in Wisconsin (*A29*) and the SR-422 bridge in Pennsylvania (*A30,A31*)]. Weld backing bars must usually be removed to achieve the needed resistance to both fatigue and fracture. Stress concentrations such as reentrant corners should be avoided and instead transition radii that are ground smooth and flush should be provided. This commonly occurs at copes in floorbeams at connections designed for vertical shear only.

The ends of butt welds are always a potential location of defects. It is important to use run-out tabs and to later grind the ends of the weld flush or to a radius. Fillet weld terminations should not be ground, for this will expose a very thin ligament near the weld root that will tear easily.

Fracture Assessment Procedures

A fracture assessment is made using fracture mechanics principles (*A13–15,A32,A33*). In the absence of other established procedures, an often used reference is the 1999 British Standard, BS 7910, "Guide on Methods for Assessing the Acceptability of Flaws in Metallic Structures" (*A32*). There is presently no comparable U.S. standard; however, BS 7910 is used by some U.S. industries (primarily the oil and gas industries).

A fracture assessment requires some knowledge of the fracture toughness of the steel and/or weld metal, which may be estimated from the Charpy energy or CVN (*A15,A33*). Unlike fatigue, the susceptibility to fracture is strongly dependent on the type of material and even the particular heat of the steel or lot of weld metal (*A14,A15,A34,A35*). The CVN may be obtained from mill reports for the steel and from the certifications for the weld metal, if good records exist. Lacking such specific information, for bridges built since 1974 when minimum Charpy requirements were first included in the ASTM A709 specification for bridge steels, the CVN for the steel plate and weld metal may be assumed to be at least as large as the minimum specified values. For older structures, or for cases when the assumed minimum values are not sufficient, it may be necessary to drill a core and get a sample of the steel, then make and test some Charpy specimens. A minimum of three specimens should be tested from each plate or structural shape. The fracture assessment will not be discussed further because it is documented elsewhere and is not the emphasis of this report.

Causes for Fracture Other Than Fatigue

As mentioned previously, fracture in bridges is almost always a result of fatigue cracking. However, if a detail and material are particularly susceptible to fracture, failure will usually occur as the loads are applied for the first time during or soon after construction. An example is the fracture that occurred when the Caltrans Workers Memorial Bridge (previously known as the Bryte Bend Bridge) was under construction in 1970. The fracture, shown in Figure A2, was attributed to use of low-toughness A514 steel (*A36*) at a point where a transverse bracing member was welded along the edge of the primary tension flange of a tub girder, creating a stress concentration at the reentrant corner. A514 steel, marketed under the name of T1 steel, is quenched and tempered with a minimum specified yield stress of 690 MPa.

It is also possible that fracture can occur in service directly without apparent fatigue crack growth. For example, at poor details that are highly constrained, such as the intersection point of two or three welds, fracture may occur in service directly from small crack-like weld discontinuities, such as the fractures that originated at shelf plate details in the Hoan Bridge in Milwaukee in December 2000 (*A29*) (Figure A3) or the SR-422 failure (*A30,A31*). Repair and retrofit of these shelf-plate details

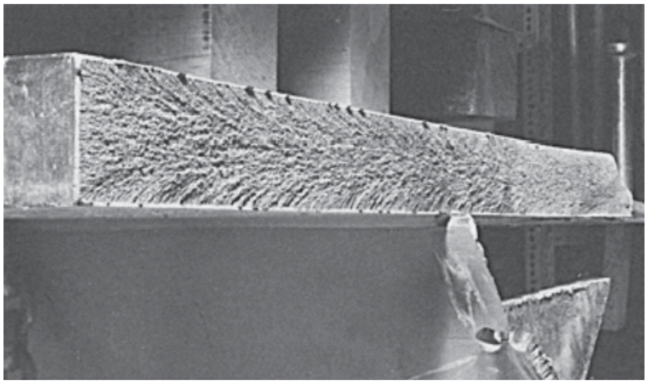

FIGURE A2 Brittle fracture of flange of tub girder of Bryte Bend Bridge in Sacramento, California, while under construction.

are discussed in special directives from FHWA and are not discussed further in this report.

Subcritical crack propagation in bridge elements may also occur by stress corrosion cracking (SCC), although this is a concern only for very-high-strength steels; that is, steel with yield strength much greater than 100 ksi. SCC involves electrochemical dissolution of metal along active sites under the

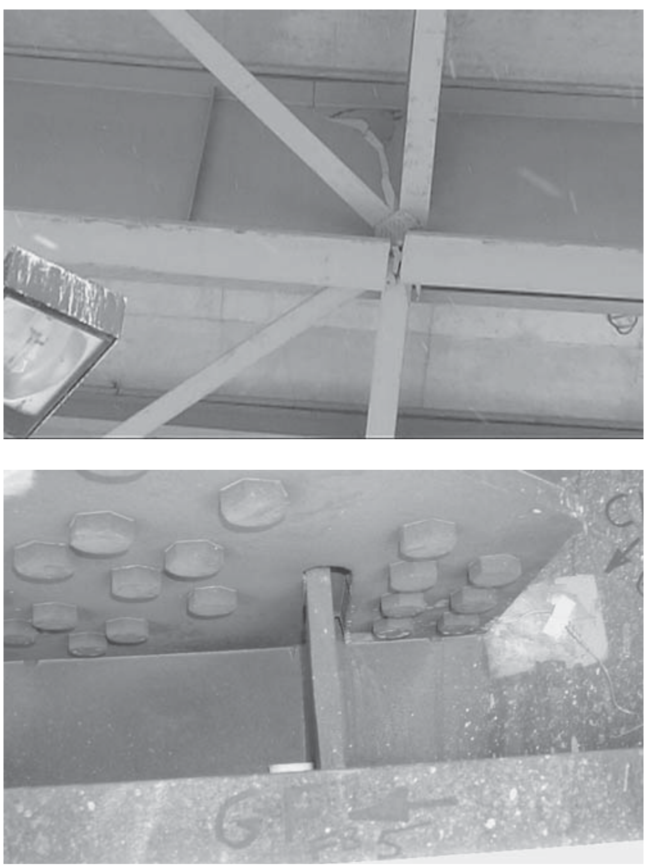

FIGURE A3 Fractured girder of the Hoan Bridge in Milwaukee (top) and view of critical shelf plate detail featuring intersecting welds.

influence of tensile stress (*A14,A15*). As hydrogen is liberated in the process, this failure mechanism may also involve hydrogen cracking. SCC can be distinguished from fatigue cracking by examining the fracture surface under a light microscope. SCC has occurred in prestressing cables (*A37*) and in A490 bolts that were out of specification (*A38*).

Specification of Charpy V-Notch for Fracture Resistance

Fracture behavior depends strongly on the type and strength level of the steel or filler metal. The fracture resistance of each type of steel or weld metal varies significantly from heat to heat, or from lot to lot. Although fracture toughness can be measured directly in fracture mechanics tests (*A14–A16*), the usual practice is to characterize the toughness of steel in terms of the impact energy absorbed by a CVN specimen (*A15*). Because the Charpy test is relatively easy to do, it will likely continue to be the measure of toughness used in steel specifications.

Because it is not directly related to the fracture toughness, CVN energy is often referred to as notch toughness. The notch toughness is still very useful, however, because it can often be correlated to the fracture toughness and then used in a fracture mechanics assessment (*A15,A32,A33*). Figure A4 shows a plot of the CVN energy of A709 Grade 50 (350 MPa yield strength) structural steel at varying temperatures. These results are typical for ordinary hot-rolled structural steel.

The fracture limit state includes phenomena ranging from the brittle fracture of low-toughness materials at service load levels to ductile tensile rupture of a component. The transition between these phenomena depends on temperature, as reflected by the variation of CVN with temperature as shown in Figure A4. The transition is a result of changes in the underlying microstructural fracture mode.

Brittle fracture on the so-called lower shelf in Figure A4 is associated with cleavage of individual grains on select crystallographic planes. Brittle fracture may be analyzed with linear-elastic fracture mechanics theory because the plastic zone at the crack tip is very small. At the high end of the temperature range, the so-called upper shelf, ductile fracture is associated with the initiation, growth, and coalescence of microstructural voids, a process requiring much energy. The net section of plates or shapes fully yields and then ruptures with large slanted shear lips on the fracture surface.

Transition-range fracture occurs at temperatures between the lower and upper shelves and is associated with a mixture of cleavage and shear fracture. Large variability in toughness at constant temperature and large changes of temperature are typical of transition-range fractures.

AASHTO specifications for bridge steel and weld filler metal require minimum CVN values at specific temperatures

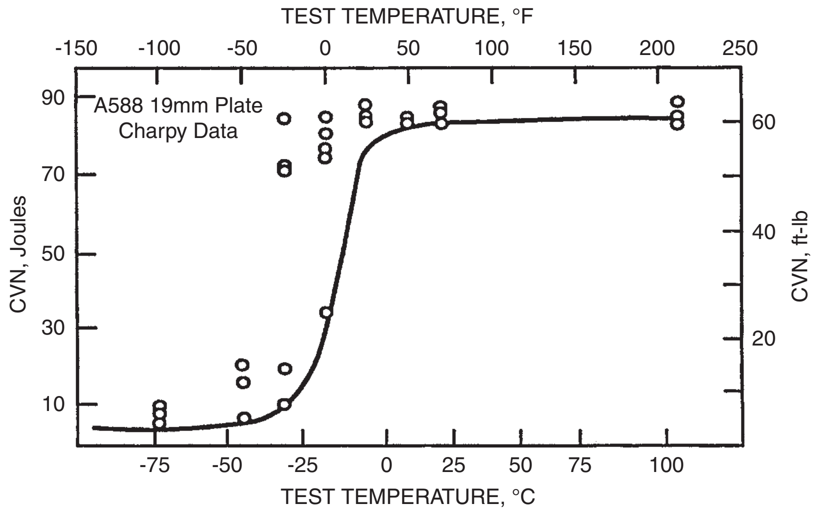

FIGURE A4 Charpy energy transition curve for A709 Grade 50 (350 MPa yield strength) structural steel.

(*A25*). As shown in Figure A4, the typical lower-shelf CVN is about 10 J. Therefore, when a minimum CVN of 20 J or more is specified at some temperature, the most important result of such a specification is that the lower shelf of the Charpy curve will start at a temperature lower than the specified temperature. This indicates that the lower shelf of a structure loaded statically or at intermediate strain rates such as traffic loading on a bridge is even lower, a phenomena known as the temperature shift (*A15*). Because of the temperature shift, the temperature at which the CVN requirement is specified may be greater than the lowest anticipated service temperature.

If the material is not on the lower shelf at service temperature, brittle fracture will not occur as long as large cracks do not develop. It essentially does not matter what the specified CVN value is as long as it is at least 20 J. Usually, an average from three tests of 34 J (25 ft-lbs) or 27 J (20 ft-lbs) is specified at a particular temperature. The greater the value of the average CVN requirement, the more certain that the material is well above the lower shelf; however, there may be a greater premium to be paid with diminishing increases in certainty.

Table A2 lists, as an example, values of impact energy presently required for base metal in FCMs for the three temperature zones into which the United States is divided. [For nonfracture-critical members (non-FCMs), typically only 20 J is required at the same temperatures.] The complete table is found in the *LRFD Specifications* (*A2*). For example, 50-mm thick flange plates of grade 345W steel for an FCB to be built at a site where the lowest anticipated service temperature (LAST) is −15°C must have the minimum impact energy for Zone 1, which is 34 J at 21°C.

History of Development of the Fracture Critical Plan

The fracture toughness requirements are based on a correlation between the fracture toughness in terms of the stress-

TABLE A2
MINIMUM CHARPY IMPACT TEST REQUIREMENTS FOR FRACTURE-CRITICAL MEMBERS

ASTM A709 Steel Grade	Plate Thickness (mm) and Joining Method[a]	Zone 1 LAST = −18°C (Joules at °C)	Zone 2 LAST = −34°C (Joules at °C)	Zone 3 LAST = −51°C (Joules at °C)
250F	Up to 100 M & W	34 at 21	34 at 4	34 at −12
	Up to 50 M & W	34 at 21	34 at 4	34 at −12
345F, 345WF	Over 50 to 100 M	34 at 21	34 at 4	34 at −12
	Over 50 to 100 W	41 at 21	41 at 4	41 at −12
HPS-485WF	Up to 100 M & W	48 at −23	48 at −23	48 at −23
	Up to 65 M & W	48 at −1	48 at −18	48 at −34
	Over 65 to 100 M	48 at −1	48 at −18	48 at −34
690F, 690WF	Over 65 to 100 W	68 at −1	68 at −18	Not permitted

[a]M = mechanically fastened; W = welded.

intensity factor and the CVN, including a "temperature shift." The temperature shift accounts for the differences in strain rate between the Charpy impact test and the traffic loads. For example, the time it takes to reach the yield strain in bridges from an overloaded truck is about one second versus one millisecond in the Charpy test (*A15*). Several independent studies verified the temperature shift (*A39–A41*).

For example, several girders were tested at the U.S. Steel Research Laboratory that contained fatigue cracks initiated at the ends of cover plate details (*A41*). A more extensive study was done at Lehigh University that examined an increased number of specimens fabricated from grade 250, 345, and 690 steels (*A39*). In both studies, the fatigue cracks were grown at temperatures and loading rates that closely resemble those encountered in bridges, and the cracked girders were subjected to periodic overloads until fracture occurred.

The temperature shift was the subject of some disagreement in the development of notch toughness requirements in the early 1970s. The prevailing opinion in the debate was that ordinary hot-rolled mild structural steels had sufficient notch toughness. The specified CVN values are the typical minimum values that could be consistently achieved. Having the specification was very important to screen out unusually brittle materials.

Roberts and Krishna (*A39*) and Roberts et al. (*A40*), among others, argued that the fracture-critical specification should require a level of dynamic toughness sufficient to arrest pop-in-type fractures occurring from local brittle zones. Hartbower (*A42*) proposed an alternate fracture control plan that called for the CVN test to be done at the same temperatures as the LAST for the bridge service site; that is, without exploiting the benefit of the temperature shift. This would mean that CVN would be required at −34°C for Zone II (most of the country). To achieve notch toughness at these lower temperatures, the steel would require additional processing, such as normalization. This increased processing would significantly increase the cost of the steel, which is a major part of total bridge costs.

Ultimately, the temperature shift concept was retained, largely out of economic necessity, and the fracture control plan placed a strong emphasis on defect control to prevent the pop-in-type fracture events. Therefore, the CVN requirements represented a compromise that allowed the continued use of hot-rolled steel rather than normalized steel.

The good fracture-resistance track record of mild steel shows that requiring normalized steel would have been unnecessary. Experience in service and in full-scale experiments has verified the temperature shift and the adequacy of the notch-toughness requirements for mild steels. Some premature cracks and fractures occurred in tied arches fabricated with A514 steel. A514 steel is quenched and tempered steel with a minimum specified yield stress of 100 ksi. This steel is much different than mild steel and should be avoided for FCMs. Fractures that occurred during construction in the box girders of the Bryte Bend Bridge in California also occurred in A514 steel. Consequently, higher CVN requirements are implemented for the higher strength steel, as shown in Table A2.

Again a compromise was reached, which resulted in the slightly increased CVN for FCMs at the same temperature shift. At the same time, more stringent fabrication rules were applied for FCMs to control the possibility of initial defects. Most importantly, the stress range for fatigue design was arbitrarily dropped by about one category, which resulted in a decrease in the allowable stress range of approximately 25% in most cases. A 25% decrease in the stress range should essentially double the fatigue life. It is important to realize that this decrease in the allowable stress range for FCMs in the Standard Specifications has been dropped in the most recent *LRFD Specifications* (*A2*).

The Bridge Welding Code D1.5 calls for higher impact energies in weld metal than in base metal, a reasonable requirement given that the welds create stress raisers and high-tensile residual stresses. Given that the cost of filler metal is relatively small in comparison with the overall cost of materials, it is well worth specifying high-toughness filler metal to mitigate the effects of high-tensile stresses induced by welding. For FCMs, 34 J is required at −29°C for weld metal, regardless of the temperature zone. For non-FCMs, weld metal is required that can give 27 J at −29°C for Zone 3, and at −18°C for Zones 1 and 2.

The somewhat higher toughness requirements shown in Table A1 for thick plates that are to be welded is the result of the higher degree of constraint and plane–strain behavior at temperatures higher than would be the case for thinner plates. This is more of a problem in welded members, because welding increases the potential for initial flaws and defects that can initiate brittle fracture. Another feature of the specification requires higher toughness for steels with yield strength greater than 345 MPa.

The AASHTO material toughness specifications provide the minimum level of toughness required to sufficiently minimize the risk of brittle fracture. The existing AASHTO fracture control plan has generally done a good job of preventing fracture failure in bridge structures since design using its provisions. A few brittle fractures have still occurred without noticeable fatigue crack growth in previously designed bridges, however, although the steel met the modern CVN toughness requirements (i.e., those required by the fracture control plan or general AASHTO toughness requirements). In most such cases, the fracture can be traced back to one or more aspects of the fracture control plan related to controlling the size of defects. If all aspects of the fracture control plan are properly implemented, the risk of brittle fracture is minimized by the current specifications and fracture will be a rare event in service. However, in the ensuing decades, very few FCBs were constructed, in part because the fabrication costs were significantly greater.

High-Performance Steel

The recently developed high-performance steels (HPS) are much tougher but also more expensive than the ordinary Grade 250, 345, or 345W steel. The first grade to be fully integrated into the AASHTO specifications is the HPS 485W. Two additional steels are currently being developed, an HPS 690 W Cu-Ni and an HPS 345W. Both of these steels are much tougher than the ordinary Grade 250, 345, and 345W steels. The main reason for developing HPS has been to improve weldability, but a large gain in toughness has been a desirable by-product. Development of HPS in the United States has been done under a joint research and development program by FHWA, U.S. Navy, and the American Iron and Steel Institute (*A43*).

In contrast to ordinary bridge steel, Grade HPS 485W shows upper shelf behavior at test temperatures down to approximately −20°C. Even at the extremely cold service temperature for Zone 3 (LAST = −51°C), this steel exhibits toughness in the upper transition region; without the benefit of the temperature shift, the CVN energy still exceeds 150 J. Likewise, high dynamic and crack arrest toughness can be expected of this steel. Clearly, HPS 485W steel provides a level of toughness far exceeding the current requirements, which are based on preventing brittle fracture.

In current research underway at the Turner–Fairbank Highway Research Center of FHWA, the fracture toughness of HPS are being determined as a function of temperature, loading rate, and plate thickness (*A44,A45*). At the same time, the fracture resistance of fatigue-cracked I-girders fabricated from HPS 485 is being determined. The test girders are cyclically loaded until a crack grows to the desired length; the girder is then cooled to −34°C and subjected to a typical design overload. If the girder does not fracture, the fatigue crack is grown larger and the same overload is reapplied. The HPS 485W steel girder resisted the full design overload until approximately 50% of the tension flange area was lost to fatigue, meaning that the net section of the cracked tension flange had yielded.

In contrast, the full-scale tests discussed previously (*A39, A41*) on girders made of Grade 345 steel fractured when the net section of the cracked tension flange reached a stress of approximately 60% of the yield stress, meaning that the fracture was brittle. In contrast, the fracture mode for the Grade HPS 485W indicates a large amount of plastic deformation before failure.

The HPS grade steels provide a toughness level that far exceeds the minimum requirements cited in Table A2. Although HPS 485W costs more than ordinary grade 345W steel, the advantages afforded by higher strength more than offset the difference in material costs. This has significantly altered the economic factors that were considered in setting the current AASHTO toughness requirements. The cost premium for higher toughness is lower than it was 30 years ago.

Consideration of Increasing Toughness Requirements for Bridge Steel

Fortunately, today's scrap-based steel typically has improved toughness relative to the steel of the 1970s. This is the result of: (1) a reduction in carbon, (2) a reduction in impurities such as phosphorous and sulfur, as well as (3) an increase in alloying elements such as nickel and chromium, which enhance toughness. Nickel and chromium and other beneficial alloys just happen to be in the scrap that is derived primarily from automobiles and other sheet metal, which tends to have greater alloying than structural steel made from iron ore.

Because steel today has greater toughness than the steels of the 1970s is perhaps the best argument for increasing the notch toughness requirements for FCMs. This would allow designers to take advantage of the superior toughness characteristics of HPS (*A45*). In other words, because the CVN specifications were essentially just below what was consistently attainable in the 1970s, and because the toughness has generally improved over time, it is rational to raise the bar somewhat to ensure that the bridge steel represents the best practice of today. This could probably be done without forcing the mills to use any additional processing and therefore would not significantly affect the cost of steel. Even though the argument can be made that the increased toughness is not absolutely necessary, it is always better to have greater toughness if there is no cost penalty.

If there was no temperature shift, extremely large cracks, greater than 350 mm, can be tolerated in mild steel. If the notch toughness requirements were increased, the increased toughness could also be used to offset the decrease in reliability associated with no longer decreasing the stress range for FCMs in the *LRFD Specifications* (*A2*). To make it more palatable, this increase could be combined with some loosening of the definition of FCMs that would, for example, allow two-girder bridges that are known to be redundant, as discussed earlier.

Research currently underway at FHWA's Turner–Fairbank Highway Research Center is studying ways of taking advantage of the benefits of higher toughness as in most modern plate steel, especially HPS. These benefits include:

- Elimination of special in-service inspection requirements, such as hands-on or arms-length inspection for fracture-critical structures for HPS.
- Reduction in the frequency of inspections for the superstructure components of HPS bridges.
- Elimination of the penalty for structures with low redundancy for HPS.

Ultimately, the greatest benefit might be achieved by providing a solid foundation for structural innovation. The current U.S. practice of providing a high level of structural redundancy prevents engineers from considering other structural systems that can result in more efficient, lower-cost bridges. For example, two-girder and tied-arch bridges are rarely built today in the United States, even though experience has shown that in many situations they are very economical. A higher factor of safety against fracture will increase the reliability level of low-redundancy systems, thereby reducing this barrier to innovation.

Although higher toughness makes bridges more tolerant to longer cracks, it does not significantly increase fatigue life. Fatigue cracks grow according to a power law; therefore, most of the fatigue life is spent growing the crack while it is very small. Additional fracture toughness, greater than the minimum specified values, will allow the crack to grow longer before the member fractures. But, at that late stage, the crack is growing so rapidly that relatively few cycles are needed to reach the end of the life.

Ultimately, decisions to specify toughness beyond the minimum required level must be made based on cost. The current AASHTO steel toughness requirements were developed using the cost factors that existed in the 1970s and considering the state of the art in steel production at that time. Modern steel processing practice has made it more economical to produce high-toughness steels such as A709 HPS 485W. In addition, the Grade 250, 345, and 345W steels produced today have typical CVN that far exceed the minimum requirements. The current AASHTO specifications for material toughness may need to be reevaluated to take maximum advantage of the higher-toughness steels used today.

INSPECTION AND NONDESTRUCTIVE TESTING

Periodic in-service inspection provides a final safety net to detect cracks before they grow to a critical size. Because of the repetition of details in a bridge, when one crack is found, it is very likely that similar details may also be cracked. Therefore, the first and most urgent step in planning repairs and retrofits is to thoroughly inspect the bridge for other cracks, usually visually but often followed up with nondestructive evaluation such as magnetic-particle testing (MT) or dye-penetrant testing (PT), especially for FCMs. The focus of inspection for fatigue cracks should be on details similar to the one that cracked on elements of the bridge in tension, with a priority for elements with high live-load stress ranges and FCMs (*A46–A48*).

Elements for which the applied stress remains in compression need not be inspected closely for fatigue cracks, because complete fracture is not possible. Owing to welding residual stresses, a crack can still occur in a structural element that undergoes cyclic loading even if the applied stress remains in compression. However, these cracks will usually arrest as they grow away from the welds as the tensile residual stress field either decreases or is relieved by the cracking (*A4*).

Inspection is primarily performed visually. The survey noted that many agencies inspected non-FCBs from the ground with binoculars, whereas the FCBs were visually inspected hands on. Nondestructive testing (NDT) is used some for FCMs, in particular if there is an indication that is clearly not a crack from the visual inspection. NDT methods presently used in service for bridges include but are not limited to the following: PT, MT, and ultrasonic testing (UT). Radiographic testing and eddy-current testing are usually only used in the shop and therefore are not discussed here.

Dye-Penetrant Testing

PT is used to detect surface discontinuities only. A penetrating liquid dye, either visible or fluorescent, is placed on the surface of the member and will enter any discontinuities. After a period of time, up to 30 min for extremely fine, tight discontinuities, the excess dye is removed and the area is allowed to dry. A developer is then applied, pulling the residual wet dye from the discontinuities as shown in Figure A5. Penetrant inspection is inexpensive, simple, and easy to learn. However, inspectors need to be properly trained in noting the difference between real and false indications that often occur from a slight weld undercut and other discontinuities that are not significant.

Magnetic Particle Testing

MT involves the use of magnetic field lines to determine whether surface or near surface cracks exist by the disruption of the lines. This disruption of lines results from a discontinuity in the member; for example, a crack. The material can either be magnetized through direct magnetization or by placing a magnetic field (indirect magnetization) on the member. Once the field is established, magnetic particles (typically in the form of a powder) are placed on the inspection surface. Discontinuities are exposed when they are trapped in the leakage of the magnetic field and the location, shape, and size of a crack can accurately be determined. Figures A6–A8 depict this process and the required equipment.

MT can be conducted very quickly and, compared with other NDT methods, it is relatively cost-effective in terms of

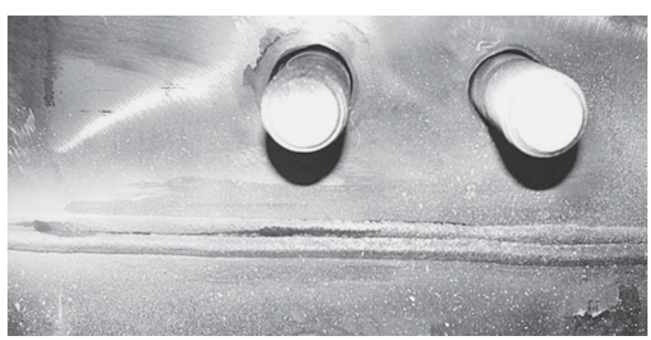

FIGURE A5 Example of indication from dye penetrant inspection.

FIGURE A6 Magnetic particle testing—Placement of magnetic field.

FIGURE A7 Magnetic particle testing—Application of magnetic particles.

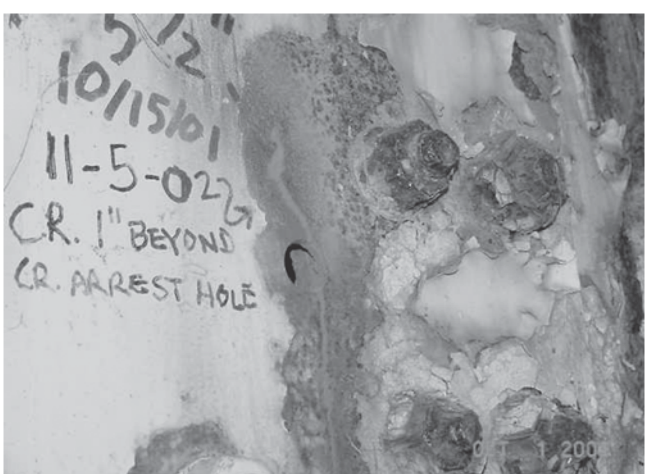

FIGURE A8 Magnetic particle testing—Indication.

equipment and procedures. In contrast to PT, MT can reveal shallow cracks below the surface, is very accurate, requires less time, and may be more economical after the equipment is obtained. This procedure is favored among many inspectors.

Ultrasonic Testing

UT inspection is another commonly used NDT method in practice. By using high-frequency sound waves, surface and subsurface discontinuities can be detected. As the sound waves travel through the material and reflect back, the presence and location of any discontinuities, which also cause reflections, in the member can be detected. This information is then displayed on a cathode ray tube screen for interpretation.

Advantages in using this type of test are the ability to detect small internal discontinuities, accuracy, and nearly instantaneous test results. The primary disadvantage to this test is that highly trained and experienced technicians are needed to operate and accurately interpret this type of test results. The international fabrication scanning tour noted that automated UT, providing a permanent record, is often used outside the United States.

Because small, possibly innocuous, discontinuities can be detected with UT, acceptance criteria are presented in AWS D1.5. These acceptance criteria are workmanship standards; that is, they represent the typical quality level easily achievable by good welders. The AWS D1.5 UT acceptance criteria are not based on the effect that the rejectable discontinuities might have on the resistance to fatigue and fracture; they are typically more strict than necessary.

The AWS standards are for new fabrication and were not intended for existing structures. It is a misapplication to apply these workmanship standards to an evaluation of existing bridges. If there is no impact on fatigue and fracture, an owner will be far more reluctant to take out of service or repair (at owner expense) a structure that is found to have poor workmanship than the owner would be when the component is still in the fabrication shop and the repairs would be at the fabricator's expense.

Notwithstanding that it is a misapplication, the AWS D1.5 criteria have frequently been used to assess the UT of butt welds in service. On many occasions, an indication with a rejectable decibel (dB) rating will be cored out of a large groove weld for destructive examination and characterization of the actual flaw. Figure A9 shows the results of many of these investigations, as conducted by Dr. Eric Kaufmann at Lehigh University's Advanced Technology for Large Structural Systems (ATLSS) Center, plotted in terms of the dB rating versus the actual flaw size. Also shown are the D1.5 rejection limits for various thickness butt welds. It can be seen that the AWS criteria are very conservative. For typical plating thickness, the AWS criteria will reliably screen out defects of only a few millimeters in width.

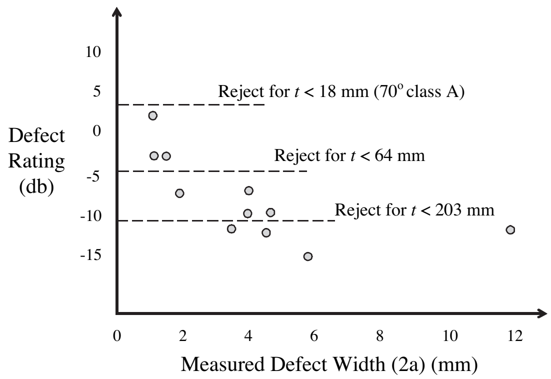

FIGURE A9 Actual defect size from destructive examinations of cores from welds with UT indications of various dB rating. (Data from Lehigh University.)

Established fracture mechanics principles can be used to define acceptable initial crack sizes that will not propagate to the critical size in the lifetime of a structure (*A15,A32*). These types of calculations are referred to as "fitness-for-purpose" calculations, indicating that although a component may have rejectable discontinuity, it can be proven that the component is fit for its defined purpose (lifetime and anticipated loading).

RETROFIT METHODS

This section presents commonly used repair and retrofit techniques for fatigue-critical details, as well as retrofit techniques to improve the redundancy of fracture-critical bridges (FCBs). A distinction is made between a repair and a retrofit: a repair is intended to arrest the propagation of a fatigue crack, whereas a retrofit is intended to either (1) upgrade the fatigue resistance and prevent the occurrence of fatigue cracking, or (2) create an alternative load path in the event of fracture to make an FCM into a non-FCM.

Issues related to repair and retrofit of fatigue cracks will be discussed first followed by some discussion related to retrofit of FCBs. One of the best sources for information related to retrofit strategies is *Fatigue and Fracture in Steel Bridges* by John W. Fisher (*A13*). Although out of print, it remains an excellent source for material on retrofitting fatigue- or fracture-damaged bridges.

Repair and Retrofit of Fatigue Cracks

Many different methods are used for the repair of fatigue cracks and retrofit of fatigue-prone details. The choice of method depends on the circumstances of the fatigue cracking and may also depend on the availability of certain skills and tools from local contractors who would perform the repairs. Repair and retrofit techniques can be placed in three major categories: (1) surface treatments, (2) repair of through-thickness cracks, and (3) modification of the connection or the global structure to reduce the cause of cracking.

Surface Treatments

Weld toe surface treatments include grinding, gas tungsten arc or plasma remelting of the weld toe, and impact treatments. These techniques can be used as "weld improvement" retrofit methods; that is, for increasing the fatigue strength of uncracked welds. With any of these treatments, the improvement in fatigue strength can be attributed to one or a combination of the following:

- Improvements in the weld geometry and corresponding reduction in the stress concentration,
- Elimination of some of the more severe discontinuities from which the fatigue cracks propagate, or
- Reduction of tensile residual stress or the introduction of compressive residual stress (*A49,A50*).

The easiest and lowest cost of these treatments is hammer peening, which is very effective and commonly used. Some of the methods, including hammer peening, can also be used for the repair of shallow surface cracks up to 3 mm deep.

A relatively new process, known as Ultrasonic Impact Treatment, has been the subject of several recent studies (*A51,A52*). The process, developed in Russia, is similar to air hammer peening, but applies the treatment at a very high frequency, up to 35 kHz. This technique uses sound waves to excite a peening device that introduces compression into the steel at the toe of fillet welds. Fatigue testing done on treated and untreated specimens cut from the plate girder concluded that, for the conditions tested, Ultrasonic Impact Treatment altered the performance of a Category C detail by imparting to it the fatigue strength of a Category B detail. Further research is being conducted to address the effect Ultrasonic Impact Treatment will have, if any, on the fatigue threshold of details and how different fatigue stress ranges, welding processes, and quality control affect the results.

Once either treatment is applied to the welds that have already been in service, the remaining fatigue life is at least as good as the life of the original detail when it was new. In other words, there is no remaining effect of prior fatigue loading cycles. In most cases, these treatments result in fatigue strength of the treated detail that is at least one fatigue "category" greater than the original detail; that is, the next greatest *S–N* curve in the AASHTO set of *S–N* curves can be used to predict the residual life of the repaired detail. Surface treatments only affect the weld toes; therefore, fatigue cracks may still develop from the weld roots.

Hole Drilling

Hole drilling is perhaps the most widely used repair method for fatigue cracks or retrofit method for fatigue-critical details. It is often used as a temporary measure to arrest a propagating crack, followed eventually by more extensive repairs. It is rare that any repair scheme such as repair welding or modifying the detail does not begin with drilling the crack tips. For retrofit, hole drilling is often used to isolate a detail or to intercept potential cracks before they can propagate far into main elements. By properly tensioning a high-strength bolt in the hole it can be considered a Category B detail. A hole by itself is basically a Category D detail.

For repairs, the hole drilling method requires placing a hole at the tip of the crack, essentially blunting the tip of the crack, thus removing the high-stress concentration associated with the sharp tip. However, the hole needs to be a specific diameter to be successful in arresting the crack. Typically, a hole diameter of 4 in. (200 mm) is recommended because this has proven to be effective. When a more refined estimate of the required hole size is necessary, relationships have been developed to define the size of the hole needed to arrest the crack (*A53,A54*). Appropriate checks on the net section capacity of the member should be made.

The fatigue resistance of a hole can also be increased by the cold expansion of the hole, which upsets the material around the perimeter of the hole and introduces beneficial compressive residual stress around the hole. This technique is widely used in aluminum airframes. Once the hole is drilled, a tapered mandrel (also referred to as a drift pin) slightly larger than the hole can be forced through the hole by hitting the pin with a hammer. As the pin passes through the hole, the hole plastically deforms creating the compressive field around the hole. This method will not work when the hole used contains a crack. If the hole has a crack entering it, the hole is forced open by the pin as the pin is driven into the hole because the cracked edge is compliant (flexible) and the hole does not provide sufficient constraint to induce the compressive stresses at the edge. However, if the hole is drilled ahead of the crack tip, the hole may be cold expanded.

Adding Doubler and Splice Plates

Another technique that can be used to repair through-thickness cracks is by the addition of doubler plates, or doublers. Doubler plates add material to the cross section to either increase or make up for the cracked cross-sectional area. Doubler plates may be bolted (Figure A10) or welded (Figure A11). From a fatigue-resistance standpoint, bolted doublers are always better than welded ones, because a high-strength bolted connection can be considered an AASHTO Category B detail, whereas a welded connection will be Category E or worse. It is therefore usually recommended that only bolted doubler plates be used for permanent repair or retrofit on bridges.

The philosophy of doublers for fatigue crack repair is to add cross-sectional area, which in turn reduces stress ranges. For instance, if a fatigue crack grows across the full depth of the bridge girder, there are two ways that it can be repaired. First, a vee and weld repair can be specified, but the base metal that is weld repaired will have at best a Category D fatigue resistance (*A54–A56*). To ensure that the weld repair will have adequate fatigue resistance, doubler plates can be added after the weld repair to decrease the stress range.

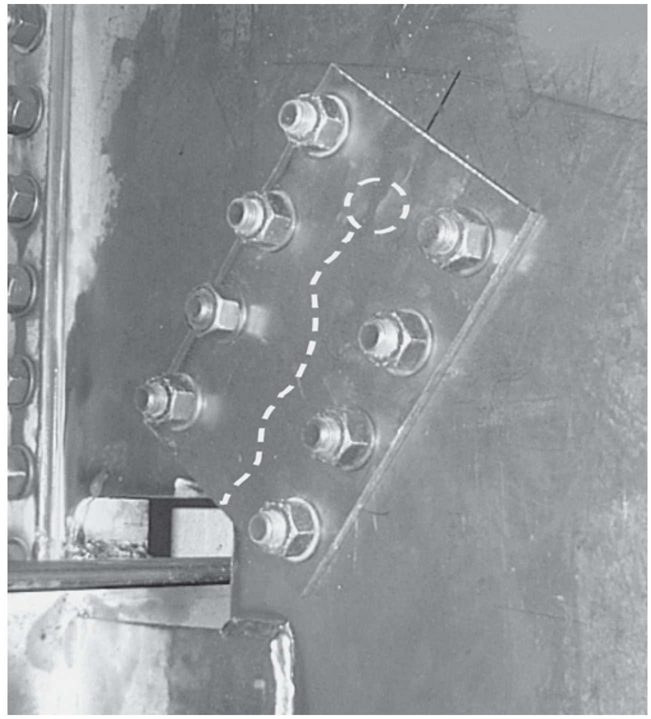

FIGURE A10 Bolted doubler plate repair. Dotted line represents crack line beneath doubler plate and circle is the hole drilled at crack tip to intercept further growth.

The one problem with this repair is the alignment of the two sides of the crack before the weld repair. As can be seen in Fig-

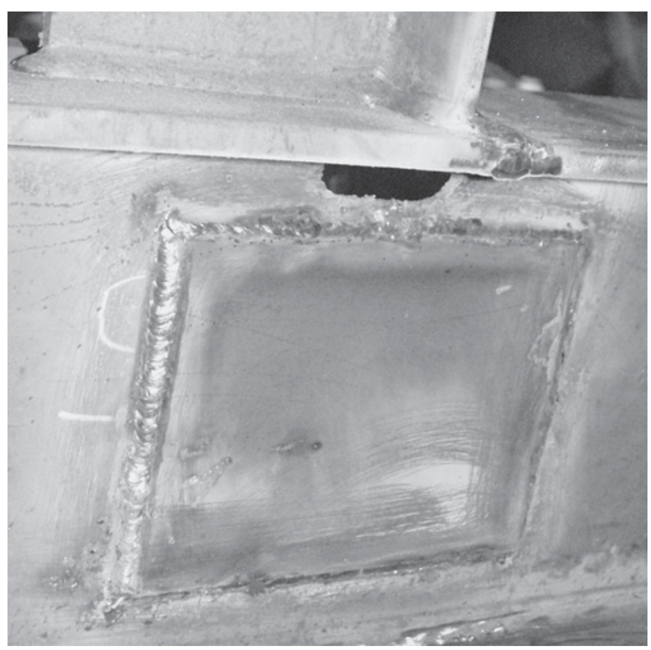

FIGURE A11 Welded doubler plate detail. (Note that corners should be rounded.)

ure A12, the cracked surface usually develops buckles, making their alignment difficult. In this case, the second option would be to use thicker doublers and bolt them to the girder. The thicker plates add enough cross section assuming the crack will not be weld repaired, and bolting them together then ensures that any buckles can be straightened out. This technique is particularly useful when a full-depth crack forms in a bridge girder. The doubler plates are then meant to make up for the lost cross section from the crack. Doublers are also typically used to restore a section that has been heavily damaged by corrosion.

Retrofit of Fracture-Critical Members to Make Them non-Fracture-Critical Members

There are a few methods that have been developed that were identified in the literature to improve the redundancy of FCBs. Some of these were illustrated in the main body of the report. Others will be summarized briefly here.

Prestressing Strands

On some bridges, the addition of prestressing strands or rods has been used to supplement tension members. An example is the Girard Point Bridge that carries I-95 over the Schuylkill River near Philadelphia. The bridge is a double-deck cantilevered truss that has a very long suspended center span. Stainless steel rods were added to supplement the primary hanger members in the event that the existing hanger truss member were to fracture. The members are preloaded so that the additional rods carry a portion of the dead load and so that if a hanger were to fracture, there would be minimal "snap" as the full load was dynamically transferred to the rods. Similar systems have been incorporated to other tension members in other trusses such as diagonals and chords.

Another related retrofit technique is to string high-strength strands along the bottom face of the bottom flange of a fracture-critical plate girder. The strands are then preloaded and a portion of the dead load transferred to the newly added strands. Thus, in the event the existing tension flange fractures, there are additional tension members (strands) with sufficient capacity available to carry both dead and live loads. In addition, depending on the level of post-tensioning applied, the dead-load stresses (as well as live-load stresses) can be reduced in the existing bottom flange.

Another approach that used post-tensioning was on the Hazard, Woodhead, Dunlavy, and Mandell Street tied arch bridges in Houston, Texas. In these bridges, the 2 ft × 2 ft tie girder was internally post-tensioned and encased in concrete. (As part of the strategy to improve redundancy the tie was not encased in concrete.) Four post-tensioning strands are in each tie. However, because the tie is encased in concrete, it is inaccessible for future inspection. These recently built steel tied arch bridges span 224 ft (68 m) over the freeway and carry two lanes of traffic, two bicycle lanes, a utility parapet in each direction, and sidewalks outside of each arch for a total width of 60 ft (18 m). One of these spans is shown in Figure A13. Note the shallow tie girder.

Post-tensioning of the arch tie provided redundancy and virtually eliminated tension in the tie, which allayed concerns about the history of problems with tie beams on other tied arch bridges, but necessitated passing the arch tie flanges through the junction with the arch rib.

FIGURE A12 Full-girder-depth fatigue crack of Lafayette Street Bridge in St. Paul, Minnesota.

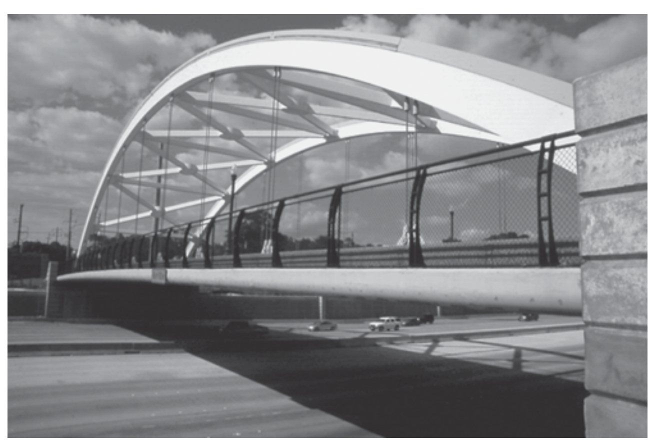

FIGURE A13 Post-tension tied arch bridge in Houston, Texas.

Bolted Redundancy Plates

Another technique that has been used on some bridges is the addition of bolted redundancy plates. This retrofit consists of bolting plates or angles to existing tension members. The primary function is to provide an additional component(s) so that if the tension flange of an existing member were to fail at some location along the length, the added component would assume the full dead and live loads of the tension flange. For example, the two-girder approach spans on the Poplar Street Bridge in East St. Louis, Missouri, have been retrofit by bolting thick HPS plates that would take the place of the tension flange along the web just above the tension flange, as shown in Figure 15 in chapter two of this report (page 17).

Because the added components are bolted to the existing member, there is no direct path for the fracture to travel into the added component. Hence, the member becomes internally redundant as with a riveted built-up member. For example, in a riveted tension flange comprised of several plates it has been observed that the cover plate can fully fracture, although the other elements of the member remain and the member continues to take the entire load. When a retrofit plate is added to a bridge with no live load, it also reduces the live-load stress range in the existing member.

The technique has also been used in new construction on a large truss bridge that carries SR-33 over the Lehigh River near Easton, Pennsylvania. On this new bridge, redundancy plates were bolted alongside of selected tension chords that were identified to be critical, as shown in Figure 13 in chapter two of the report (page 16). The plates were fully spliced at the panel points and nominally connected to the member along the length. An advantage of redundancy plates used in the design is that the components share both dead and live loads throughout the life of the bridge.

Although these techniques add internal member redundancy, they do not add overall structural load path redundancy. In other words, in the unlikely event that the entire lower chord failed only one lower chord remains.

Use of Composite Construction

The SR-33 Bridge near Easton, Pennsylvania, incorporated an additional measure to increase redundancy. The top chord is fully composite with the concrete deck (see Figure A14). Traditionally, in truss bridges, only the floorbeams and stringers are made composite with the deck. The deck is cast-in-place reinforced concrete supported by steel stringers and transverse floorbeams. It is the only composite truss in the state of Pennsylvania and possibly the United States. The structure is a four-span continuous haunched steel deck truss that is composite with the reinforced concrete deck. The main river span is 181 m and the depth of the trusses varies from 11 m to 22 m.

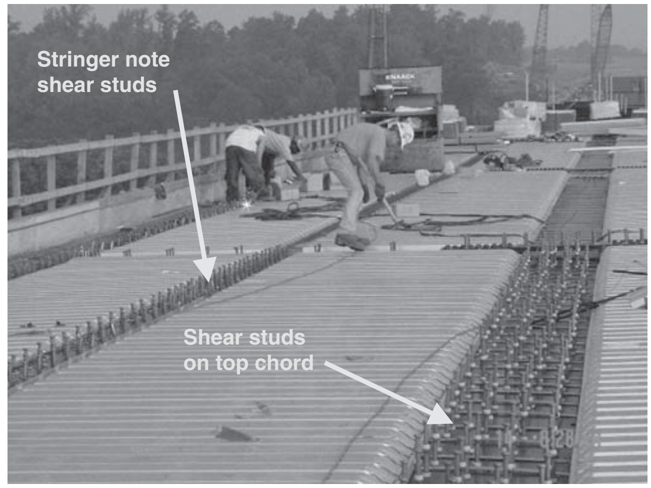

FIGURE A14 Shear studs being installed on top chords, floorbeams, and stringers on the SR-33 bridge near Easton, Pennsylvania.

Controlled load testing and long-term monitoring of the bridge confirms that there is excellent load distribution between trusses and that stresses in the upper chord in the negative moment regions are very small owing to the composite action (*A56*). Hence, it is believed that the choice to make the deck composite with the upper chord added substantial redundancy.

Installation of Additional Girders

This retrofit technique is only applied when a structure is to be widened. In cases where the existing structure is functionally obsolete and additional or wider lanes are required, an additional exterior girder is sometimes added. If the new girder is adequately attached to the existing girders, full load sharing can be realized. Thus, a two-girder bridge can become a four-girder bridge and be removed from the list of FCBs in an owner's inventory. It is emphasized that the new girder must be sufficiently attached to the existing girder and deck.

This technique was employed on the Pennsylvania Turnpike near the Valley Forge Interchange northwest of Philadelphia. In this example, a two-girder bridge was widened by adding parallel girders adjacent to the existing riveted girders. In effect, a two-girder cross section was converted into a four-girder cross section. Although the girder spacings are unequal and the distance between the two original girders may still be considerable, the structure as a system should not be considered fracture critical as long as the connections between the girders and to the concrete deck are adequate.

The additional girders can also be used to retrofit cross girders. For example, two girders were used to provide redundancy to existing steel cross girders carrying multi-girder composite spans, as shown in Figure A15. The bridge carries I-95 just north of Philadelphia. The retrofit required

FIGURE A15 Additional support girders are used to provide redundancy to steel cross girders on I-95 bridge north of Philadelphia, Pennsylvania.

the addition of new steel support columns to carry the loads from the new cross girders to the footings. The retrofit was installed as a preemptive strategy, because no problems with the cross girders have been observed.

Pin and Hanger Retrofits

There are two common strategies used to retrofit pin and hanger bridges. Both will be discussed here. It is important to note that although this system improves the redundancy of the given pin and hanger assembly, it does not necessarily add load path redundancy to the entire structure. However, for the approaches that actually remove the pin and hanger and replace the detail with a full moment connection, the case could be made that, if sufficient diaphragms are present, there may be sufficient alternate load paths.

Addition of Supplemental "Catcher" Systems

This form of retrofit is typically used on pin and hanger systems. In the typical application, an additional group of components are added to "catch" the suspended girder should the existing pin and hanger system fail. A typical installation of this system is shown in Figure A16.

Removal of Pin and Hanger Assembly

In this approach, the entire pin and hanger assembly is removed and replaced with a new short section of girder that is attached to existing portions of the girders with full moment splices. The girders are then made continuous for live load and even some proportion of dead load. Field instrumentation conducted on the bridge in Figure A17 confirmed that after the retrofit, the bridge behaved as a typical continuous multispan bridge (*A57*).

The ability of the structure to behave as a continuous multispan bridge, primarily in the negative moment regions, must be adequately checked. During construction, either false work or strong backs are required to ensure that the bridge is stable. The process can be completed with a live load on the bridge. Figure A17 illustrates a two-girder bridge where the pin and hanger were removed and replaced.

DEVELOPMENT OF FRACTURE CONTROL PLAN

This section reviews the history of the development of the fracture toughness requirements for steel and weld filler metal, which are a central part of the bridge fracture control plan contained in D1.5. The requirements appear to have served their purpose; that is, there have been no catastrophic

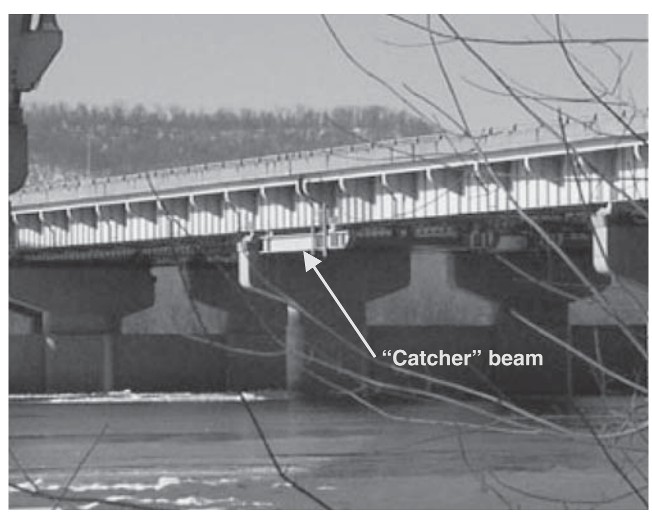

FIGURE A16 Catcher system as used on a typical pin and hanger bridge. (*Courtesy:* Modjeski and Masters, Inc.)

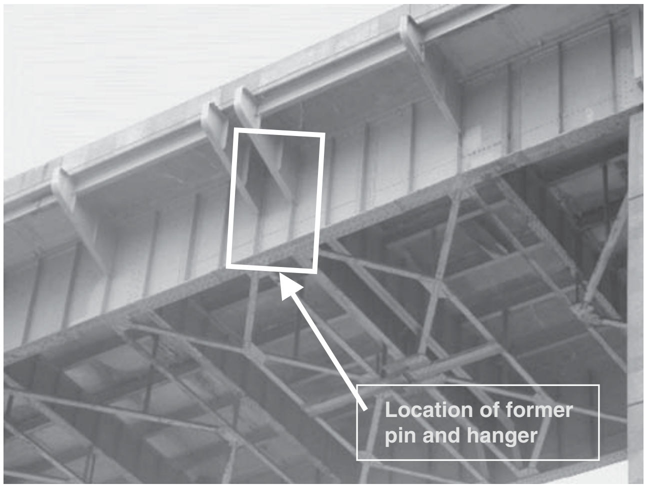

FIGURE A17 Typical complete girder splice installed to replace a pin and hanger connection in a two-girder bridge.

fracture problems with bridges since their implementation. However, many things have changed since the development of these specifications. There are fundamental differences in the steel and the way it is produced. Some of these differences may have some impact on the temperature shift and other assumptions in the original development of the toughness requirements.

The difference between the fracture control plan provisions and the provisions for non-FCMs elsewhere in AASHTO/AWS D1.5 is primarily that there are more strict fabrication and shop-inspection requirements to control weld flaws and other crack-like defects. For example, both radiographic testing and ultrasonic testing are required on all groove welds for fracture control elements. In addition, CVN requirements for welds and base metal are increased for fracture control elements. The provisions result in an even lower probability of brittle fracture in new FCMs than for typical non-FCMs.

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APPENDIX B

Survey Questionnaire

QUESTIONNAIRE

NCHRP SYNTHESIS TOPIC 35-08

INSPECTION AND MANAGEMENT OF BRIDGES WITH FRACTURE CRITICAL MEMBERS

Please complete and return this questionnaire by March 19, 2004

Name: ____________________
Title: ____________________
Agency: ____________________
Address: ____________________
City: ____________________
State: ____________________
Zip: ____________________
Phone: ____________________
Fax: ____________________
E-mail: ____________________

Please return the completed questionnaire to:

Robert J. Connor
Lehigh University
117 ATLSS Drive
Bethlehem, PA 18015
Phone: (610) 758-6103 Fax: (610) 758-5553
Email: rjc3@lehigh.edu

We would greatly appreciate discussing any matters related to this survey with you on the telephone. If you have any questions, comments, or can provide any additional information that is not identified in the questionnaire, please do not hesitate to call myself at 610-758-6103 or Robert J. Dexter at 612-624-0063.

Background

During the past three decades, special emphasis has been placed on the evaluation, inspection, and rehabilitation of bridges containing fracture-critical members (FCMs). FCMs are steel members in tension or with tension elements whose failure would probably cause a portion of or the entire bridge to catastrophically collapse. Bridges containing FCMs are commonly referred to as fracture-critical bridges (FCBs).

This survey is being conducted as part of NCHRP Synthesis Topic 35-08: Inspection and Management of Bridges with Fracture Critical Members. One of the objectives of this survey is to gather information on fracture-critical structures and how bridge owners define, identify, inspect, and manage FCBs.

Notes for Completing this Survey

A copy of this survey should be given to all departments within your agency that have significant experience with FCBs (e.g., designers, inspectors, district bridge engineers). <u>Multiple responses from a single agency are welcomed and encouraged.</u>

It would be advantageous to provide a copy to the individuals within the agency with the most experience and familiarity with the history of the bridge inspection and maintenance program of the agency.

In responding to all of the questions below, if you do not have exact information or if it would be very difficult to obtain, please feel free to estimate the approximate answer and indicate that it is an estimate. It is better to have an estimate than no information at all. Also, please provide sketches, drawings, photographs, inspection reports and any other useful information if available. If you have such material in digital format, please send it via e-mail if possible. If this additional information is not digital in format, you can send this to us by regular mail. If you request, we will return it to you right away.

QUESTIONNAIRE

PART I—GENERAL

1. How does your agency define a fracture-critical bridge?

   Nearly all agencies responded that they either use the AASHTO or NBIS definitions.

2. Are different criteria used for different structural systems (e.g., are trusses identified using different criteria than box girders)?

   33 No
   7 Yes

   If yes, please describe the criteria:

   Results indicated the criteria are developed for specific projects on case-by-case basis. Some rely on a ranking system to prioritize inspection procedures based on age, average daily traffic, type of material, and type of fatigue details on the bridge.

3. How would you categorize the following bridges (place an 'X' in the box to select the type of bridge)?

CHAPTER 10. Description	Fracture Critical	
	Yes	No
Two girder bridges	38	0
Three girder bridges	9	28
Three girder bridges with girder spacing > ______ ft	10	21
Multi-girder bridges with girder spacing >_______ft	3	32
Truss bridges	34	3
Two girder bridges fabricated using HPS 70W	31	1
Truss bridges fabricated using HPS 70W steel	28	2
Single steel "tub" girder bridges	32	5
Twin steel "tub" girder bridges	22	12
Multi-steel "tub" girder bridges	0	34
Other (post-tensioned, timber, steel cross girders, etc.)	13	3

   Are these answers your opinion or your agency's written policy?

   Engineering judgment in conjunction with established policies; e.g., NBIS and AASHTO.

4. Does your agency identify suspension bridges as fracture critical?

   8 No
   25 Yes

If yes, what elements are considered fracture critical?

____

____

5. Does your agency currently have any fracture-critical bridges in its inventory?

1 No
39 Yes

If you answered "yes" to question 5, please continue and complete the survey. If you answered "no," please return the survey to the address listed above. We thank you for your time. (ID = insufficient data.)

PART II—INSPECTION AND CLASSIFICATION

1. How many steel bridges are in your inventory? 85,723
   a. How many are fracture critical? 11.8%
      How many fracture-critical bridges were built before 1975? 76%
      How many fracture-critical bridges were built before 1985? 70% (ID)
   b. How many are non-fracture critical? 80%
      How many non-fracture-critical bridges were built before 1975? 67%
      How many non-fracture-critical bridges were built before 1985? 81%

2. Please provide a breakdown of the approximate number of each type of structure identified as fracture critical in your inventory. Some common types of bridges often identified as fracture critical are listed below:

16.4% Welded two-girder system (with bolted or welded field splices)
1.0% Welded three-girder system (with bolted or welded field splices)
<1% Other welded girder systems considered fracture critical

8.6% Riveted two-girder system
<1% Riveted three-girder system
<1% Other riveted girder systems considered fracture critical

30.9% Riveted truss bridges with two truss lines
3.8% Welded truss bridges with two truss lines (with bolted or welded field splices)
1.6% Welded and bolted truss bridges with two truss lines

28% Bridges with steel pier caps
1.5% Bridges with pin and hanger systems
6.1% Bridges with steel cross girders

<1% Bridges with single steel tub girders
<1% Bridges with twin steel tub girders
1.1% Bridges with multi-steel tub girders

7.5% Other steel bridges considered fracture critical

Please describe: Steel cross girders, cable supported, tied arch.

3.1% Other non-steel bridges (e.g., wood or post-tensioned concrete) considered fracture critical. Please describe:

____

____

3. Does your agency require any special inspection procedures for fracture-critical bridges?

11 No
29 Yes

If yes, please describe below:

Examples given: hands-on, arms length at defined intervals.

4. Does your agency require any special training or qualifications for individuals inspecting fracture-critical bridges?

17 No
23 Yes

If yes, please describe below:

Examples given: NHI, in-house training, FCB inspection team lead must have experience and in most cases be a PE.

5. What is the maximum interval (in years) between inspections of typical non-fracture-critical bridges permitted by your agency?

Two years typically, 5 years maximum for in-depth.

6. What is the maximum interval (in years) between inspections of typical fracture-critical bridges permitted by your agency?

Two years typically, 5 years maximum for in-depth.

7. Are there any special methods or nondestructive testing techniques required by your agency to inspect fracture-critical bridges?

25 Yes

If yes, please briefly describe:

Dye penetrant, UT, magnetic particle testing.

15 No

The next two questions make a distinction between cursory inspection and detailed "hands-on" inspection. Your agency may not make such a distinction, so the answers would be the same.

8. What is the maximum interval (in years) between detailed "hands-on" inspections of typical *non-fracture-critical* bridge permitted by your agency?

5

9. What is the maximum interval (in years) between detailed "hands-on" inspections of typical *fracture-critical bridge* permitted by your agency?

5

10. If fracture-critical bridges are inspected at a greater frequency than non-fracture-critical bridges, is the entire bridge inspected more frequently or just the fracture-critical members?

Most agencies answered no to this question and that interval is based on a case-by-case basis. Some agencies indicated that they only inspected the FCM, while others indicated the entire bridge is inspected since crews and equipment are on site.

11. Is there any significant increase in the costs associated with the inspection of fracture-critical bridges?

13 No
26 Yes

If you answered "yes" to question 11, what are the reasons for the increased costs (e.g., increased frequency of inspection, increased cost per inspection)?

Traffic control, additional time on site to perform in-depth inspection, added equipment costs such as snooper, testing equipment, special FCB inspection team, etc.

Can you attempt to estimate at the absolute or percentage increase in cost? (e.g., 10%, 15%, . . . etc.)

Increase in cost due to frequency of inspection __________
Increase in cost per inspection __________
Other factors (please describe): ______________________________

12. Is there any special or advanced analysis performed to identify which specific members are fracture critical or if the structure is actually fracture critical (e.g., 3-D structural analysis, etc.)?

32 No
8 Yes

If yes, please describe below:

Some owners indicated that 3-D modeling has been conducted on a case-by-case basis, but none had a defined policy.

13. Has the inspection of a fracture-critical bridge(s) ever identified a condition that has clearly prevented a fracture and the subsequent collapse of the structure? (The objective of this question is to identify specific cases or examples whereby the additional inspection efforts dedicated to FCBs prevented a failure that would have occurred had the inspection not been carried out. For example, the discovery of typical out-of-plane distortion cracks, which usually take years to become critical, would not warrant a response of "yes" for this question. Furthermore, inspections which found fractured members would also not warrant a response of "yes" for this question since the inspection did not prevent the fracture and the bridge did not fail.)

27 No
8 Yes

14. If you answered "yes" to question 13, please describe the problem(s) in detail. Bridge name and/or inventory number, location, year built:

Questions 13 and 14 did not provide information intended. Question 13 was not worded clearly enough to determine if inspection program found immediate problems. Only a few owners replied to both questions. Overall, most who replied felt inspection permitted them to find and correct a problem before failure occurred.

15. If a bridge normally identified as fracture critical had all fracture-critical members fabricated from HPS 70W steel, would there be any changes to the inspection methods or interval that you would recommend?

Nearly all owners answered "no" to this question. However, the reason given for a response of "no" was almost always because FHWA/NBIS does not allow a change in inspection requirements if HPS 70W is used.

16. If a bridge normally identified as fracture critical was fabricated so that the worst fatigue category used was Category C would there be any changes to the inspection methods or interval that you would recommend? (Most multi-beam bridges built in the last twenty years do not have details worse than Category C.)

Nearly all owners answered no to this question.

17. It is commonly observed that by using simplified structural analysis methods the calculations for many bridges indicate no remaining fatigue life or even "negative" fatigue life. These calculations imply that fatigue cracking should be observed presently or in the near future on these bridges. However, such bridges typically show no signs of fatigue-related prob-

lems. (Please note this does not include cracking from secondary stresses, such as web gap cracking, as this type of cracking is not explicitly considered in fatigue rating calculations.)

What is your agency's policy regarding cases when this inconsistency occurs?

One state relied on a fracture mechanics approach to evaluate potential for fracture. Interestingly, many states do not check fatigue life on FCBs. Others, if they check fatigue, will retrofit the bridge or post the bridge. Overall, none of the agencies that replied indicated a defined procedure to address this issue.

18. Are more rigorous methods ever used such as:

a. Weigh-in-motion studies to better characterize loading

Yes 4

If yes, how many bridges per year _____.

No 36

b. Advanced analysis, such as grid or 3-D frame analysis

Yes 13

If yes, how many bridges per year _____.

No 27

c. Field instrumentation and/or controlled load testing or monitoring

Yes 18

If yes, how many bridges per year _____.

No 22

Please describe other methods and provide procedures as necessary.

19. What criteria are used to identify bridges that will be evaluated using the more rigorous methods for a fatigue evaluation? Owners indicated that these additional efforts are undertaken on a case-by-case basis.

Influencing factors are structure size, age, average daily traffic, type of fatigue details on the bridge, overall condition (including fatigue problems) of the bridge.

20. In performing a fatigue evaluation, does your agency use a specific procedure for fracture-critical bridges?

9 Yes
30 No

21. If you answered "yes" to question 19, what is your level of confidence in these procedures?

_____ Low
_____ Medium
_____ High

Very few agencies answered this question. Those who replied indicated low-to-medium confidence.

PART III—FAILURES

The failures of interest in this section are *only* related to fractures. Cracking associated with fatigue that did not result in fracture should not be included in this group. However, fractures that have occurred as a result of a fatigue crack should be included. *For all failures cited, please provide a description of the bridge, failure, inventory number of the bridge, and any other information you can provide.* (ID = insufficient data.)

1. Has your agency ever experienced the failure of any *non-fracture-critical bridges* the cause of which could be *attributed to any reason,* such as fracture of a member, scour, vehicle impact, . . . etc?

   8 No
   31 Yes

2. If you answered "yes" to question 1, were there any fatalities? (Please provide information for each failure.)

   20 No
   13 Yes

   If yes, how many? ID

3. Has your agency ever experienced the failure of any *fracture-critical bridges* the cause of which was attributed to reasons *other than* fatigue or fracture of a member, such as, scour, impact, . . . etc?

   24 No
   16 Yes

   If yes, how many were prior to the implementation of the fracture-critical bridge inspection program (1975 for most agencies)? ID

   Bridge name and/or inventory number, location: ______

   If yes, how many were after the implementation of the fracture-critical bridge inspection program? ID

   Bridge name and/or inventory number, location: ______

4. If you answered "yes" to question 3, were there any fatalities? (Please provide information for each failure.)

   11 No
   2 Yes

   If yes, how many? ID

5. Has your agency ever experienced the failure of any fracture-critical bridges the cause of which was clearly attributed to fatigue or fracture of a member?

   35 No
   4 Yes

   If yes, how many? ID

   If yes, how many were prior to the implementation of the fracture-critical bridge inspection program (1975 for most agencies)? ID

   Bridge name and/or inventory number, location: ______

If yes, how many were after the implementation of the fracture-critical bridge inspection program? ID

Bridge name and/or inventory number, location: ____________________

If you answered yes to question 5 were there any fatalities?

ID No
ID Yes

If yes, how many? ____________ (Please provide information for each failure.)

6. Has your agency ever experienced a brittle fracture in a member of a *fracture-critical bridge* in which the bridge *did not collapse* catastrophically but remained standing? (For example, a girder may have fractured with damage only resulting in significant deflections or local buckling of other members.)

29 No
10 Yes

If yes, how many? ID

If yes, how many were prior to the implementation of the fracture-critical bridge inspection program (1975 for most agencies)? ID

Bridge name and/or inventory number, location: ____________________

If yes, how many were after the implementation of the fracture-critical bridge inspection program? ID

Bridge name and/or inventory number, location: ____________________

7. Have existing small fatigue cracks ever been observed to propagate significantly (greater than 6 inches) between regularly scheduled inspections?

29 No
11 Yes

If yes, how many were prior to the implementation of the fracture-critical bridge inspection program (1975 for most agencies)? ID

Bridge name and/or inventory number, location: ____________________

If yes, how many were after the implementation of the fracture-critical bridge inspection program? ID

Bridge name and/or inventory number, location: ____________________

8. Have fatigue cracks ever resulted in fracture of a fracture-critical member?

34 No
4 Yes

If yes, how many were prior to the implementation of the fracture-critical bridge inspection program (1975 for most agencies)? ID

Bridge name and/or inventory number, location: ____________________

If yes, how many were after the implementation of the fracture-critical bridge inspection program? ID

Bridge name and/or inventory number, location: ____
____

9. If you answered "yes" to question 8, were tests conducted to determine the material properties of the steel?

 1 No
 5 Yes

10. If you answered "yes" to question 8, has it been confirmed through a detailed investigation that the fractures were initiated by the fatigue crack?

 1 No
 4 Yes

11. What was determined to be the specific cause of the fracture?

 There were very few responses to this question. Where additional information was given, fatigue, impact from ships, or constraint-induced fracture originating at intersecting weld (Hoan-like failure) were identified as the causes.

12. How long (in inches) was the fatigue crack at the time of fracture? (Please attach sketches, photographs, etc., as required.)

 ID

13. Would you be willing to speak with us over the telephone to discuss these failure(s) further?

 6 No
 15 Yes

PART IV—RETROFIT PROCEDURES

1. Has your state or agency developed any policies for retrofit of bridges identified as fracture critical to improve redundancy?

 30 No
 7 Yes

2. Please provide copies of these policies. (Please attach documents as required.)

 ID

3. Have the retrofits been implemented or been scheduled to be implemented in the field on any bridges to date?

 13 No
 15 Yes

4. Can your agency identify any research needs related to fracture-critical steel bridges?

 6 No.
 6 Yes—if yes, please describe below. Some examples are provided below that can simply be checked.
 8 Develop guidelines related to advanced structural analysis procedures to better predict service load behavior in fracture-critical bridges.
 9 Develop advanced fatigue-life calculation procedures taking into account lack of visible cracks for fracture-critical bridges.
 10 Field monitoring for fracture-critical bridges.
 3 Crack arrest capabilities of bridge steel.

7 Establish evaluation procedures for advanced large deformation and member death.
10 Develop advanced analyses techniques and procedures to investigate alternate load paths, redundancy, and bridge collapse.
1 Retrofit procedures to add redundancy.

Others you may wish to suggest:

ID—See responses to question 5 of this section.

5. Can your agency suggest any changes in the current practice of identifying, inspecting, or retrofitting fracture-critical steel bridges?

_____ No
_____ Yes

If yes, please describe below.

- Develop methods to establish inspection intervals and criteria based on ADT/ADTT, age, detail types, existing condition, etc.
- Develop fracture-critical inspection procedures for high-mast lighting towers, overhead sign bridges. These structures should be considered as fracture critical according to one respondent. The individual indicated that these structures are more problematic than their fracture critical bridges.

Survey Completed—Thank You for Your Time

APPENDIX C

List of Survey Respondents

U.S. State Departments of Transportation

Alaska
Arizona
Arkansas
California
Colorado
Delaware
Georgia
Hawaii
Illinois
Indiana
Iowa
Kansas
Kentucky
Maryland
Michigan
Minnesota
Mississippi
Missouri
Nebraska
Nevada
New Jersey
New Mexico
New York
North Carolina
Oklahoma
Oregon
Pennsylvania
Tennessee
Texas
Virginia
Washington
West Virginia
Wisconsin

Canadian Provinces

Alberta
Quebec
New Brunswick

APPENDIX D
Annotated Bibliography

1. Idriss, R.L., K.R. White, and D.V. Jauregui, "After-Fracture Response of a Two-Girder Steel Bridge," *Natural Hazards Mitigation,* Presented at the Structures Congress '93, Irvine, Calif., Apr. 19–21, 1993.

Idriss et al. (*1*) developed a computer model to study the after-fracture response of I-40 two-girder bridges. Field test data (*2*) were used to validate or modify the analytical model. The main focus of the analysis was to investigate the way in which the load distributes, the maximum load the bridge could withstand, and the potential for the bridge to collapse after a full-depth crack was introduced in one of the bridge girders. The analysis showed that the structure remained stable after the crack was introduced. The stability of the structure was attributed to a large surge in the tension force of the lateral bracing, which helped in stiffening and stabilizing the structure.

2. Idriss, R.L., K.R. White, C.B. Woodward, and D.V. Jauregui, "After-Fracture Redundancy of Two-Girder Bridge: Testing I-40 Bridges Over Rio Grande," *Conference Proceedings 7,* Fourth International Bridge Engineering Conference, San Francisco, Calif., Aug. 28–30, 1995, pp. 316–326.

Idriss et al. (*2*) conducted field testing on I-40 bridges over the Rio Grande in Albuquerque, New Mexico. The bridges were built in 1963 and were classified by AASHTO as nonredundant fracture critical. Similar to the intention of the analytical model (*1*), the main focus of the authors was to study the redistribution of loads, the load capacity the bridge can withstand, and the potential for collapse. The bridges were loaded in the positive moment region with a truck that was 95% of New Mexico legal load, and roughly equivalent to HS-18.35.

The truck load was placed on the bridge four different times as four different levels of damage were introduced in one of the girders in the bridge. The fourth level of damage was such that a 6-ft crack was introduced in one of the 10-ft girders. The crack stretched to include the bottom flange, which was totally severed (only 4 ft of the web and the top flange were left to carry the bridge).

Idriss et al. reported that under dead and live loads, and when the truck was located above the crack, the flange deflected 13/16 in. There was no sign of yielding, with no significant change in strains experienced by the gauged members until the bottom flange was completely severed. In other words, load redistribution did not occur until the bottom flange was completely severed. Another important finding was that most of the load was redistributed through the damaged girder and stringer deck system to the interior supports. In general, the load was redistributed through the damaged girder, the diagonals, the stringers, the deck, and the floorbeams.

3. Idriss, R.L. and K.R. White, "Secondary Load Paths in Bridge Systems," *Transportation Research Record 1290,* Transportation Research Board, National Research Council, Washington, D.C., 1991, pp. 194–201.

Idriss and White (*3*) studied the secondary load path in a four-girder bridge using finite-element analysis. The focus of the study was to investigate the distribution of the loads and the secondary load paths along which a load is transmitted when damage occurs in the given set of proportionate structures. Three finite-element analyses were conducted and the results were compared with experimental results from an AASHTO road test conducted in the early 1960s on four bridges.

The load-deflection response showed good agreement between the finite-element model and the experimental results up to the load causing the first plastic hinge. Beyond that point, the actual structure showed much greater deflection than the model. The analysis was also conducted with three types of defects modeled at midspan in one of the two exterior girders. The first defect was a 50% loss in the flange section for a distance of 5 ft 6 in. at about the centerline of the girder, whereas the second defect was a 100% loss. The third defect included a crack at midspan in the bottom flange extending upward through the full depth of the web plate. The first sign of an excessive yielding with a plastic hinge was observed in the defective girder. The more extensive the damage in the girder, the more widespread and extensive the yielding in the slab and diaphragms. When a near-full-depth crack was modeled at midspan, the maximum capacity of the bridge was 1.5 to 3 times an HS-20 truck loading.

4. Sweeney, R.A.P., "Importance of Redundancy in Bridge-Fracture Control," *Transportation Research Record 711,* Transportation Research Board, National Research Council, Washington, D.C., 1979, pp. 23–30.

Sweeney (*4*) discussed the importance of redundancy in controlling the fracture of bridges. He characterized welded structures to be generally not "fail-safe" in which, if cracks starts to run in a weld, it will not stop unless it runs out of material, weld, or driving force. Riveted structures however are internally member redundant because most members are built up of several components. Also, the holes provided in a riveted structure serve as crack stoppers at the interface between components. Using examples of bridges that had failed, Sweeney emphasized that designers of welded nonredundant structures must ensure that no brittle fracture will occur during the life of a bridge. Sweeney also mentioned that fires have shown that trusses have considerable redundancy or an alternate load path that will prevent collapse by brittle fracture.

5. Lai, L.L.-Y., "Impulse Effect on Redundancy of a Tied-Arch Bridge," *Computing in Civil Engineering,* Proceedings of the First Congress held in conjunction with A/E/C Systems '94, Washington, D.C., June 20–22, 1994.

Lai (*5*) studied the redundancy of a tied arch bridge using a three-dimensional finite-element model. In a tied arch bridge the tie is always in tension and is classified as a fracture-critical member. In other words, if one of the ties fractures, the bridge will not be able to carry the load and will eventually collapse. Lai used a static incremental study to determine the degree of redundancy of the structure and found that after the fracture of the one of the ties, the structure can carry its own weight plus 1.3 times HS-20 truck loading without catastrophic collapse. Lai also considered the dynamic and impulse effect on the behavior of the bridge after the fracture of one of the ties. The dynamic analysis of the structure with unfractured ties showed that the first four modes of vibrations are 1.57, 1.82, 3.2, and 3.31 Hz, respectively. With one of the ties fractured, the natural frequencies changed to 1.29, 1.69, 2.73, and 2.89 Hz, respectively. Clearly, the reduction in the structural stiffness resulted in a reduction in the natural frequencies. When considering the impulse loading, the maximum deflection of the fractured structure was calculated to be twice as much as that when using static incremental analysis.

6. Pandey, P.C. and S.V. Barai, "Structural Sensitivity as a Measure of Redundancy," *Journal of Structural Engineering,* Vol. 123, No. 3, 1997, pp. 360–364.

Pandey and Barai (*6*) presented a definition of redundancy based on response sensitivities where the generalized redundancy is inversely proportional to the elements' response sensitivity $\left(\text{Generalized redundancy } \alpha \dfrac{1}{\text{Response sensitivity}}\right)$. In their definition the redundancy of the structure is no longer a fixed quantity, but rather a function of the strength and the response of the structure at a given stage.

7. Hartley, D. and S. Ressler, "After-Fracture Redundancy of Steel Bridges: A Review of Published Research," ATLSS Report No. 89-13, Lehigh University, Bethlehem, Pa., 1989.

Hartley and Ressler (*7*) conducted a review of 16 articles and technical reports on the after-fracture redundancy of steel bridges. They concluded that no actual consistent definition of the word redundancy had been determined. As they mentioned, this could be because redundancy is characterized by a certain degree of inherent variability, which is very hard to quantify.

8. Csagoly, P.F. and L.G. Jaeger, "Multi-Load Path Structures for Highway Bridges," *Transportation Research Record 711,* Transportation Research Board, National Research Council, Washington, D.C., 1979, pp. 34–39.

Csagoly and Jaeger (*8*) provided a framework of reference for discussion of multiload path structures with the intention that such discussion will result in the elimination in future design of single-load path structures. Csagoly and Jaeger provided various examples of bridges that behaved "unintentionally" as multiload path structures as a result of having backup systems that prevented collapse when critical components of the structures failed. It is worth mentioning that the authors characterized single-cell steel box girders as single-load path structures, suggesting that their future design should be eliminated.

9. Frangopol, D.M. and J.P. Curley, "Effects of Damage and Redundancy on Structure Reliability," *Journal of Structural Engineering,* Vol. 113, No. 7, 1987, pp. 1533–1549.

Frangopol and Curley (*9*) used an example of redundant and nonredundant trusses to illustrate that the degree of redundancy is not an adequate measure of the system's overall strength. Rather, the strength measure is constituted by the behavior of the members (i.e., ductile versus brittle).

Frangopol and Curley recognized the need for the development of a better understanding and definition of redundancy in various types of bridges and used illustrative examples to define new ideas regarding redundancy in bridges. They defined the term R (the redundant factor for a bridge) as

$$R = \frac{L_{\text{intact}}}{L_{\text{intact}} - L_{\text{damaged}}}$$

where L_{intact} defines the overall collapse load of the bridge without damage, whereas L_{damaged} defines the overall collapse load of the damaged bridge. Therefore, R is the reserve strength between the component(s) damage and system collapse. They also stated that "important members could be identified in which their failure or severe damage would have a greater influence on the effectiveness of the redundancy of the bridge, and that more inspection and quality assurance should be given to such members."

10. The 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," *Journal of Structural Engineering,* Vol. 111, No. 12, Dec. 1985.

The Task Committee on Redundancy of Flexural Systems (*10*) recognized that "loads are carried by the system along a variety of simultaneous paths" and that "redundant load paths exist in most structures." That being said, they examined methods to determine the redundant path in flexural systems and presented the experiences of approximately 100 bridges. The following highlights what they discussed.

- Various analytical techniques could be used to investigate the linear elastic response of damaged flexural systems to loads that include reverse design using AASHTO; more sophisticated methods than AASHTO; and finite-difference, finite-strip, or finite-element methods. Nonlinear analysis for all critical stresses and deformations must be accurately determined.
- Welded members can suffer a failure as a result of excess stress that is the result of an imperfection in the weld.
- If several load paths are available in a structure, then weakening in the most critical location would be followed by a redistribution of the load in which the redistribution is a function of the changing stiffness of the structure.
- Steel stringer bridges are highly redundant for the load for which they were designed.
- Failure of one girder in a two-girder system may lead to collapse. However, large deflections could be the result of the redistribution of loads.
- Secondary members are often not included in the analysis for the transverse behavior. They are also not considered in the analysis for live load. However, these members could participate in the load redistribution resulting in reduction of the stresses in the main members.
- Fatigue cracks do not cause a change in the stiffness of the structure. However, if the crack length becomes critical, a sudden failure of the girder could take place. If the girder fails, then a drastic redistribution of the load to the adjacent girder would occur through the slab and the bracing systems.
- Fatigue is the most common cause of reported damage in bridges.
- Failure in bolted structures is unlikely to occur because if a crack originates it cannot propagate past the holes used for bolting.
- Behavior of the bridge superstructure is not greatly affected if the skew angle is more than 60 degrees.
- For loads up to the elastic limit, a fully composite interaction between the deck and the main girder could be assumed if the AASHTO code is used for the design even if the bridge is noncomposite.
- Based on previous studies, in composite bridges the reinforced concrete deck is likely to be the first component to exhibit nonlinearity.
- Bottom lateral wind bracing does not contribute greatly to live-load redistribution in straight bridges. The contribution of the bracing system to the load redistribution is high for curved girders.
- Data on steel bridges that suffered damage indicate that if redundancy is present then few steel bridges would collapse. Furthermore, those bridges that collapsed were mostly nonredundant truss bridges.

11. Frangopol, D.M. and J.P. Curley, "Effects of Redundancy Deterioration on the Reliability of Truss Systems and Bridges," *Conference Proceeding,* In *Effects of Deterioration on Safety and Reliability of Structures,* S. Marshall Ma, Ed., 1986.

The definition of structural redundancy including system reliability and damage assessment concept was utilized to analytically investigate the effect of redundancy deterioration on the reliability of truss systems and bridges. Using Figure D1, Frangopol and Curley (*11*) illustrated that the degree of redundancy of a system is not an adequate measure of system strength. The example showed that the redundant truss has an increase in ultimate load capacity compared with the nonredundant truss of 78%, 100%, and 131% for brittle, ductile and hardening behavior of members, respectively.

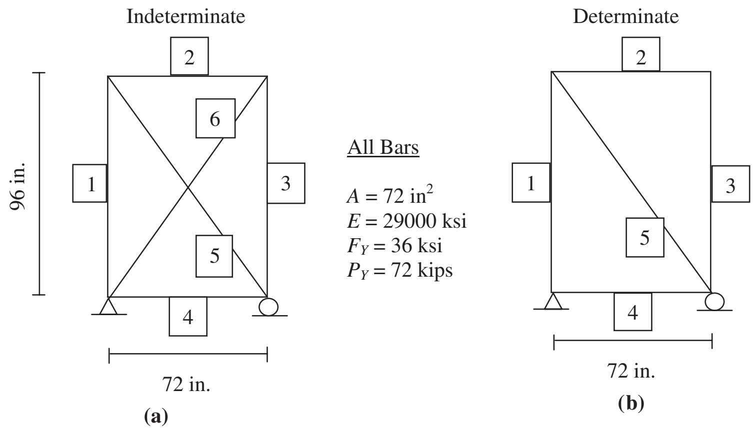

FIGURE D1 Two elementary trusses (ref. *11* in Appendix D).

Because the degree of redundancy of a system is not an adequate measure of system strength, Frangopol and Curley (*11*) introduced the redundancy factor R for bridges, which is the reserve strength between component(s) damage and system collapse. The redundancy factor R is equal to 1 when the damaged bridge has no reserve strength

where

$R = L_{intact}/(L_{intact} - L_{damage}) = \lambda/(\lambda - \lambda^*)$,
L_{intact} = the overall collapse load of the bridge without damage = λL,
L_{damage} = the overall collapse load of the damaged bridge = $\lambda^* L$, and
L = applied load on the bridge.

Using this equation with various damage factors, which represent the percent reduction in the load carrying capacity of a given member, the redundant factor R could then be calculated. The results are helpful in that it makes it possible to identify which member or group of members are critical in the structure.

12. Frangopol, D.M. and R. Nakib, "Redundancy Evaluation of Steel Girder Bridges," 5th International Conference on Structural Safety and Reliability, San Francisco, Calif., Aug. 7–11, 1989.

Considering corrosion damage scenarios, Frangopol and Nakib (*12*) reviewed several definitions of redundancy factors of existing steel girder bridges. Furthermore, an analytical model was developed to study the degree of redundancy of E-15-AF Colorado State Bridge in the presence of corrosion damage using the definition of the redundancy factor R provided in (*11*).

13. Frangopol, D.M. and K. Yoshida, "Loading and Material Behavior Effects on Redundancy," *Proceedings of Structures Congress,* Chicago, Ill., Apr. 15–18, 1996.

Frangopol and Yoshida (*13*) used a numerical example of a three-bar system to show that system redundancy could be greatly affected by variation in the applied load and/or material behavior. The angle in which the load was applied varied between 0° and 360°, whereas the material behavior was altered by choosing four different values, 0.25, 0.50, 0.75, and 1.00, to represent the yield stress ratio "a," where "a" is defined as the tension yield stress to the compression yield stress $\left(\sigma_y^C/\sigma_y^T\right)$. Failure loci were then generated with respect to first yielding and collapsing the system. Collapse and first yield loads (P_C and P_Y) were represented by the distance between the origin and the collapse loci and the origin and the first yield loci, respectively. The redundancy factor R_1 is then defined as:

$$R_1 = (P_C - P_Y)/P_C$$

14. Kudsi, T.N. and C.C. Fu, "Redundancy Analysis of Existing Truss Bridges: A System Reliability Approach," First International Conference on Bridge Management, Barcelona, Spain, July 14–17, 2002.

Kudsi and Fu (*14*) developed a new approach to the redundancy of structural systems in general bridges and truss bridges in particular. The approach consists of building a block diagram, which accounts for the degree of redundancy of the system and the possible amount of redundant members' combinations (in parallel configuration) to be laid in series with the nonredundant members in the system. Multiple failure modes are then defined and equations are implemented to obtain the system's pre- and post-failure reliability index and probability of failure, where the post-failure phase is defined as the phase when a member fails without causing collapse. The system is then defined as redundant or nonredundant by comparing the reliability index of the system after the failure of a particular member with the target reliability index.

15. Kritzler, R.W. and J. Mohammadi, "Probabilistic Evaluation of Redundancy of Bridge Structures," *Proceedings of the Sixth Specialty Conference,* sponsored by the Engineering Mechanics, Structural, and Geotechnical Divisions of the American Society of Civil Engineers, Denver, Colo., July 8–10, 1992.

Knowing that redundancy of structures could result from the reserve capacity of its members when stressed beyond their yield or buckling strength, Kritzler and Mohammadi (*15*) defined redundancy as "the degree of reverse strength available for preventing failure of an entire structural system." They evaluated the degree of redundancy of bridges based on the difference between the safety index of the redundant structure, considering all of its failure paths and the safety index of the exact same structure with no alternative load path where redundancy measure is computed by subtracting the damage safety index (β_s) from the controlling component safety index (β_f) (the controlling component safety index is calculated based on the probability of failure of one component of the system.)

16. Heins, C. and C.K. Hou, "Bridge Redundancy: Effect of Bracing," *Journal of the Structural Division,* Vol. 106, No. 6, June 1980, pp. 1364–1367.

Like many others, Heins and Hou (*16*) realized the need for examining the effect of bracing members in bridge structures on the load distribution of two-girder and multigirder systems after the development of a crack in one of the girders. This was done using an analytical space frame model in which the flanges are supported by vertical and diagonal web plates. The girders were stiffened by transverse diaphragms and lateral wind bracing. The cracks were introduced in the flanges of the girders by assigning a negligible stiffness to small beam element. A two-system girder and a three-girder system with different spans were used in the study. The systems were examined by first assuming a cracked bottom flange, and then cracked bottom and top flanges. The results revealed that when one crack develops and no bracing is used, the deformation increases by 40% for the two-girder system and 10% for the three-girder system. However, if bracings are considered, the deformation increases by 10% for the two-girder system, although almost no increase in the deformation is noticed in the three-girder system.

17. Heins, C.P. and H. Kato, "Load Redistribution of Cracked Girders," *Journal of the Structural Division,* Vol. 108, No. 8, Aug. 1982, pp. 1909–1915.

An analytical model was employed by Heins and Kato (*17*) to study the effect of bracing on load distribution in a two-girder bridge system when one of the girders is damaged. The two-girder system consisted of two longitudinal girders with cross floor beams and longitudinal stringers. The flanges, the webs, and the deck slab were idealized as a series of intersecting beam elements. A crack was introduced near the center span of the girder with the assumption that the web elements along the crack had zero stiffness. Such an assumption is conservative, resulting in deformations and stresses higher than those that would develop in an actual damaged bridge. The result of the analysis showed that for a 120-ft girder the deformation of the cracked girder was reduced by 39% when bottom bracings were incorporated in the analysis, whereas the uncracked girder had a 19% increase in its deformation when the bottom bracings were used. For a 180-ft girder, the deformation of the cracked girder was reduced by 54% when bottom bracings were incorporated in the analysis, whereas the uncracked girder had a 20% increase in its deformation when the bottom bracings were used.

18. Tang, J.P. and J.T.P. Yao, "Evaluation of Structural Damage and Redundancy," *Conference Proceedings,* In *Effects of Damage and Redundancy on Structural Performance,* D.M. Frangopol, Ed., American Society of Civil Engineers, New York, N.Y., 1987.

Realizing that the material properties and the damage state of a structure is random in nature, Tang and Yao (*18*) interrelated the system strength, redundancy, structural damage, and member damage such that the random nature of strength and damage could be considered.

19. Furuta, H., M. Shinozuka, and J.T.P. Yao, "Probabilistic and Fuzzy Representation of Redundancy in Structural Systems," Presented at the First International Fuzzy Systems Associated Congress, Palma de Mallorca, Spain, July 1985.

As stated by Furuta et al. (*19*), the ultimate strength of a damaged structure is better measured by the residual resistance factor (*RIF*) and the redundant factor (*RF*) where

$$RIF = \frac{R_u(X_i, Y_j, A_k)}{R_u(X_i, A_k)}$$

$$RF = \frac{R_u(X_i, A_k)}{R_u(X_i, A_k) - R_u(X_i, Y_j, A_k)}$$

$R_u = f(X_i, Y_j, A_k), \quad i, j, k = 1, 2, \ldots m$ (ultimate strength of intact structure),

X_i = member properties of the ith member,
Y_j = reduction in geometry properties of the jth member (zero for undamaged structure), and
A_k = constant representing original geometrical properties of the kth member.

These equations assume that material properties, original geometry properties, and the damage level of a member are known. However, because such information is difficult to obtain, Tang and Yao (*18*) considered a probabilistic approach and treat some of the variables as random variables and define *RIF* and *RF* in the average sense

where

$$\overline{RF} = \frac{E[R_u(X_i, Y_j, A_k)]}{E[R_u(X_i, A_k)]}$$

$$\overline{RF} = \frac{E[R_u(X_i, A_k)]}{E[R_u(X_i, A_k)] - E[R_u(X_i, Y_j, A_k)]}$$

where E [.] denotes expected value.

20. Moses, F., "Evaluation of Bridge Safety and Remaining Life," *Conference Proceeding,* In *Structural Design, Analysis, and Testing,* Alfred H.S. Ang, Ed., American Society of Civil Engineers, New York, N.Y., 1989, pp. 717–726.

Based on reliability principles and available data, Moses (*20*) considered the overstress limit state and the fatigue limit state for the evaluation of bridges. The overstress limit state refers to the extreme event in which the maximum load effect exceeds the bridge strength capacity. The fatigue limit state is determined by repetitive loading and is more sensitive in steel bridges. Provided that the bridge is redundant in which a multiload path exists in the bridge, a target reliability index of 2.3 was chosen by the author. The reliability index *b* was calculated using the formula $\beta = \overline{g}/\sigma_g$, where $\overline{g}$ = mean safety margin, and σ_g = standard deviation of g.

The value of g is calculated based on the safety index model:

$$g = R - D - L$$

where

R = member strength,
D = dead load effect, and
L = live load effect.

The statistical values of the load and load effects were assembled from weight-in-motion programs, field tests, and other load meter information.

21. Ghosn, M. and F. Moses, *NCHRP Report 406: Redundancy in Highway Bridge Superstructures,* Transportation Research Board, National Research Council, Washington, D.C., 1998, 50 pp.

In this report, Ghosn and Moses defined bridge redundancy as "the capability of bridge superstructure to continue to carry loads after the damage or the failure of one of its members." They recognized that redundancy is related to system behavior rather than individual component behavior. The current bridge specifications, however, generally ignore the interaction between members and structural components in a bridge. To overcome the lack of interaction between members and components in current bridge design codes, Ghosn and Moses introduced system factors, related to system safety and redundancy, to be multiplied to nominal resistance of members. The multiplier factors could be used when evaluating the degree of redundancy of an existing bridge or when designing a new bridge.

The load factors identified as LF_1, LF_u, LF_f, and LF_d could be calculated for any bridge configuration using a finite-element analysis. LF_1 is expressed as the maximum number of AASHTO HS-20 trucks that the structure can carry before first member failure. LF_1 could be calculated using linear elastic structural models and incrementing the load until the failure of the first member. LF_u is the ultimate limit state and could be calculated by performing a nonlinear analysis of the structure under the effect of the dead load and two side-by-side AASHTO HS-20 trucks. LF_u is then obtained by incrementing the truck loads until the system collapses. LF_f is the structure capacity to resist large displacements and is expressed as the number of AASHTO HS-20 trucks that will cause a violation of the functionality limit state, which is chosen to be span length/100. Finally, the damage condition, LF_d, is calculated by conducting a nonlinear analysis on the damaged structure using two side-by-side AASHTO HS-20 trucks. LF_d is then obtained by incrementing the truck loads until the system collapses.

To provide a reliability-based level of redundancy, relative reliability needs to be calculated.

$$\Delta\beta_u = \beta_{\text{ult}} - \beta_{\text{member}}$$

$$\Delta\beta_f = \beta_{\text{funct}} - \beta_{\text{member}}$$

$$\Delta\beta_d = \beta_{\text{damaged}} - \beta_{\text{member}}$$

The values of β_{ult}, β_{member}, β_{funct}, β_{damaged} could be obtained using Equations 2, 3, 4, and 5 in *NCHRP Report 406.*

It is important to note that an adequate level of redundancy is satisfied if obtained values of $\Delta\beta_u$, $\Delta\beta_f$, and $\Delta\beta_d$ are greater than or equal to 0.85, 0.25, and −2.70, respectively.

For direct system redundancy approach, adequate load factor ratios (system reserve ratios) are required to satisfy a minimum level of redundancy.

where

$$R_u = LF_u / LF_1$$

$$R_f = LF_f / LF_1$$

$$R_d = LF_d / LF_1$$

The redundancy of the bridge system is then considered adequate if R_u, R_f, and R_d are greater than or equal to 1.30, 1.10, and 0.50, respectively.

It is important to note that *NCHRP Report 406* indicates that bridges that do not meet the required load factor ratios could still provide a high level of system safety. This could be checked by investigating if the capacity of the member meets or exceeds the capacity required by the specifications.

22. Lenox, T.A. and C.N. Kostem, "The Overloading Behavior of Damaged Steel Multigirder Bridges," *Fritz Engineering Laboratory Report No. 432.11,* Lehigh University, Bethlehem, Pa., Mar. 1988.

Lenox and Kostem (*22*) conducted a parametric analytical study of three multigirder bridge models. Every model had six parallel girders and a span length different from the other models. The analysis revealed that after major damage has occurred in the exterior girder at the midspan, the load is redistributed mainly among the structural components in the vicinity of the damage. It was also noted that a significant amount of internal redundancy exists in simple-span steel highway bridges, resulting in a large increase in deformations and stresses at the vicinity of the exterior girder after it develops a sever damage. Bridge deck and cross bracing members played a major role in the redistribution of the load after the exterior girder is damaged. An insignificant response of the multigirder bridge was observed when the damage was only in the lower flange of the girder. However, a significant response was observed when half web crack was introduced in the exterior girder.

23. Daniels, J.H., W. Kim, and J.L. Wilson, *NCHRP Report 319: Recommended Guidelines for Redundancy Design and Rating of Two-Girder Steel Bridges,* Transportation Research Board, National Research Council, Washington, D.C., 1989, 148 pp.

Daniels et al. (*23*) provided guidelines for the design and rating of a redundant bracing system on new or existing two-girder steel bridges. They also investigated the after-fracture redundancy of simple span and continuous, composite and noncomposite, steel two-girder highway bridges. The investigation was done using an analytical model in which a near full-depth fracture was assumed to occur at any position along the length of one of the two girders. The fracture was introduced in the tension flange and in the full depth of the web (no fracture of compression flange). The bridge system used in the model consisted of top and bottom bracings (laterals) and diaphragms.

The analysis concluded that significant redundancy could be achieved if redundant bracing systems (top and bottom laterals and diaphragms) are properly designed. Furthermore, if the bracing system is not originally designed for redundancy, the bridge system could still exhibit significant after-fracture redundancy if the bracing system is properly configured. Finally, there is a need for a new redundancy rating, which could be calculated by the allowable stress or load factor methods.

24. Chen, S.S., J.H. Daniels, and J.L. Wilson, "Computer Study of Redundancy of Single Span Welded Steel Two-Girder Bridge," Report FHWA-PA-85-047+84-20, Federal Highway Administration, Washington, D.C., Mar. 1986, 72 pp.

Chen et al. (*24*) investigated the response of a simple-span welded right two-girder bridge to a midspan fracture of one of the two main girders. The investigation included a three-dimensional model of the bridge to allow for the interaction between the structural components.

The finite-element analyses confirmed the findings of many researchers, which is the critical role played by components and details in carrying the load after the fracture of one of the girders. For the bridge used in the study, under the dead load, the girder crack was followed by cracking and warping in the deck, failing of a fixed bearing, and buckling of several cross-frame horizontal members. Overall, a significant insight was gained on the structural behavior and the redistribution mechanism of the load after the through-depth fracture of one of the two girders at midspan.

25. "Final Report of the Bridge Safety Assurance Task Force to the New York State Commissioner of Transportation," New York State Department of Transportation, John Kozak, Chairmen of the Bridge Safety Assurance Task Force, June 1995.

The Bridge Safety Assurance Task Force was formed primarily in response to the fatal collapses of Connecticut's Mianus River Bridge in 1983 and the New York State Thruway Schoharie Creek bridge collapse in 1987 as a result of scour. The task force included experts in various fields related to bridge engineering in the areas of hydraulics, river mechanics, concrete, and steel. The individuals contributing in the area of steel bridges were Dr. John W. Fisher and Dr. J. Hartley Daniels. The report evaluates conditions significantly affecting structural safety of New York bridges and recommends action to enhance identification of vulnerable bridges and prioritization of corrective actions.

Vulnerability of steel bridges was examined and a procedure to assess vulnerability, based on average daily truck traffic, member type, susceptibility to fatigue, redundancy, and other factors were incorporated into the assessment procedure. The result of the assessment was a ranking that could be used to compare the vulnerability of a given bridge to other bridges and to permit more effective use of funds when considering which bridges to retrofit, repair, or replace.

Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
NASA	National Aeronautics and Space Administration
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation