NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM REPORT 319

RECOMMENDED GUIDELINES FOR REDUNDANCY DESIGN AND RATING OF TWO-GIRDER STEEL BRIDGES

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FOREWORD

By Staff
Transportation Research Board

This report presents the results of an investigation into the after-fracture redundancy of steel, two-girder highway bridges. Procedures, equations, and worked examples are provided for designing bracing systems to create redundancy in new or existing bridges and for computing redundancy ratings for bracing systems in terms of AASHTO truck loadings. Both simplified procedures and three-dimensional finite element computer analyses were used in the investigation and are recommended in the guidelines for determination of redundant load paths. Engineers involved in bridge design, rating, and rehabilitation will find the report of interest. Recommendations are also made for changes in AASHTO specifications and manuals for bridge design and evaluation.

Redundancy in a bridge has been generally defined as the absence of critical components whose failure would cause collapse of the structure. To minimize the risk of collapse, fracture-critical members (FCMs) in existing bridges generally require more frequent and thorough inspections than other members, and FCMs in new bridges require special design, fabrication, and materials. Considerable differences of opinion exist, however, about which types of steel bridges can be safely classified as redundant.

Current AASHTO specifications define an FCM as a nonredundant tension member or other component whose failure would be expected to cause collapse of the bridge because a suitable alternative load path is not present. Specific criteria, nevertheless, are not available to adequately define redundancy. Experience suggests that many bridge types have viable alternative load paths that are not easily identified. For example, longitudinal continuity, bracing, floor systems, and certain other structural components might have significant effects. Therefore, engineers need a better understanding of alternative load paths and specific criteria for redundancy.

Under NCHRP Project 12-28(10), "Guidelines for Determining Redundancy in Steel Bridges," Lehigh University, Bethlehem, Pennsylvania, investigated the development or creation of redundant load paths in steel girder highway bridges. The original intent of the project was to include all steel girder bridges, but early in the project, a mutual decision was made by the researchers and the NCHRP project panel to focus on simple span and continuous, composite and noncomposite two-girder steel bridges only. This decision was based on the complexity of the problem, the most critically perceived bridge configuration, and the financial resources available to the project.

The research produced recommendations to AASHTO for changes in the defi-

nition for redundancy and introduced a redundancy rating factor. Guidelines are provided for designing and evaluating redundant bracing systems and for retrofitting existing bracing systems. Simplified procedures and equations, as well as three-dimensional finite element computer analyses, were used in the investigation and are included in the recommended guidelines for evaluating redundancy in steel two-girder bridges.

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The research reported herein was performed under NCHRP Project 12-28(10) by the Department of Civil Engineering, Fritz Engineering Laboratory, Lehigh University. J. Hartley Daniels, Professor of Civil Engineering, Lehigh University, was the principal investigator. John L. Wilson, Professor of Civil Engineering, also with Lehigh University, was the co-principal investigator mainly responsible for computer-related studies. Mr. Wonki Kim was the Research Assistant on this project and is a Ph.D. candidate in the Department of Civil Engineering.

Special thanks are extended to the NCHRP project panel who provided valuable comments and suggestions. The panel also provided design drawings of actual two-girder steel bridges in California, New York, and Texas which were used extensively in the development of these guidelines.

The several discussions with Roger G. Slutter, Professor of Civil Engineering, Lehigh University were very helpful. His contributions to Chapter Four, particularly with respect to the use of tension cables or rods, are valuable and gratefully acknowledged. The Computer Aided Engineering (CAE) facilities of the Fritz Engineering Laboratory, John L. Wilson, Director, are acknowledged for assistance in the conduct of the research.

RECOMMENDED GUIDELINES FOR REDUNDANCY DESIGN AND RATING OF TWO-GIRDER STEEL BRIDGES

SUMMARY

This report is the result of an extensive investigation, conducted under NCHRP Project 12-28(10), into the redundancy of two-girder steel highway bridges. The term redundancy is used to describe the ability of a two-girder bridge to survive following the near full depth fracture of one of the two main girders. If the bridge does not collapse and, what is more important, remains serviceable under normal traffic conditions for a short time (a month, or so) after the fracture, the bridge is redundant; otherwise, it is nonredundant.

AASHTO classifies all two-girder steel highway bridges as nonredundant whether they are simple span or continuous. For this reason most states have avoided the design of new two-girder bridges for many years. More frequent inspections are being made of existing two-girder bridges in the belief that the traveling public must be protected against the uncertainties of potential localized or complete failure.

On the other hand, experience shows that two-girder highway bridges typically do not collapse following fracture of a girder. In fact, not only do they remain serviceable in some cases, but damage sometimes is not even suspected until the fracture is discovered accidentally or during an inspection. This experience does not suggest that fracture of a two-girder bridge is of no concern. It does suggest, however, that much needs to be learned about how the fractured bridge supports not only its own dead weight but also vehicles on the bridge. The members and components providing redundancy need to be identified. The arrangement of these members and components in as-built bridges which exhibit redundancy needs to be examined. Analytical models need to be developed for use in redundancy evaluation and design which consider the three-dimensional behavior of the as-built structure, not the behavior of the oversimplified planar model normally used in midspan. Design provisions to ensure redundancy need to be developed for application to simple-span and continuous-span deck-type two-girder bridges and to through-girder bridges.

This report is written to be a practical user's manual dealing with after-fracture redundancy evaluation and design of two-girder bridges. The behavior of fractured two-girder bridges is developed in considerable detail. Guidelines are provided for redundancy evaluation and design, either by means of procedures and equations developed from simple three-dimensional analytical models or by finite element modeling and computer analysis of the as-built three-dimensional structure containing a properly configured and located bracing system. Several worked examples are provided which illustrate the application of the guidelines.

A significant finding of this investigation is that the bracing system, consisting of lateral (wind) bracing and diaphragms, which is normally present in two-girder deck-type bridges, provides considerable after-fracture redundancy if properly configured and located. The bracing system can be designed specifically to provide not only after-fracture strength against collapse but also after-fracture serviceability of the bridge.

A new redundancy rating level, similar to the AASHTO inventory and operating levels, is developed for evaluating the after-fracture redundancy of a two-girder bridge.

The redundancy rating is computed either by the allowable stress or load factor method and provides an after-fracture H or HS rating which can be compared with the usual AASHTO inventory and operating ratings.

Guidelines are also provided for two-girder bridges that do not contain a suitable redundant bracing system. These include guidelines for retrofitting the existing bracing system so that it qualifies as a redundant bracing system. Also included are guidelines for providing after-fracture redundancy using tension cables or rods in lieu of a redundant bracing system and guidelines for providing redundancy for through-girder steel highway bridges.

Suggestions for further research conclude this report. Additional experimental research on two-girder bridges is needed to complete and verify the guidelines. Redundancy research is also needed for all other bridge types. That research should emphasize deck serviceability as well as collapse.

CHAPTER ONE

INTRODUCTION AND RESEARCH APPROACH

PROBLEM STATEMENT

The design of steel bridges in the United States requires the bridge engineer to design against fatigue resulting from repetitive live loads(*1*). The allowable stress ranges used in design depend on whether the bridge is considered to be a redundant or nonredundant load path structure. Article 10.3.1 of the AASHTO *Standard Specifications for Highway Bridges* (*1*) defines redundant load path structures as "structure types with multi-load paths where a single fracture in a member cannot lead to the collapse." Nonredundant load path structures are defined as structure types "where failure of a single element could cause collapse." The "element" referred to is defined as a "main load carrying component subject to tensile stress."

The allowable fatigue stress ranges for redundant load path structures provided in Table 10.3.1A of Art. 10.3.1 result primarily from research at Lehigh University over the past 25 years, much of it sponsored by the NCHRP (*2, 3, 4, 5, 6, 7, 8, 9*). The allowable fatigue stress ranges for nonredundant load path structures, also provided in Table 10.3.1A, are empirical and not based on research results. These reduced stress ranges are determined simply by shifting the values for redundant load path structures one column to the left and introducing additional values for over 2,000,000 cycles.

Design against fatigue by the use of the allowable stress ranges in Table 10.3.1A for either redundant or nonredundant load path structures does not guarantee that fracture of a steel bridge component or member cannot occur. Fracture is one possible outcome of undetected fatigue crack growth in any riveted, bolted, or welded steel structure.

AASHTO assumes, however, that the consequences of fracture of a redundant load path structure are not so severe in that total collapse is not likely to occur. Whether or not the fractured bridge presents little or no risk to the traveling public or to heavy vehicles traveling at normal highway speeds (the bridge remains serviceable) is not considered, however, and has not so far been addressed by AASHTO.

The consequences of fracture of nonredundant load path structures are assumed to be severe. It is, in fact, assumed by AASHTO that collapse of the superstructure will occur. Therefore, the reduced allowable stress ranges provided in Table 10.3.1A are intended to decrease the probability (reduced risk) of a nonredundant load path structure developing undetected fatigue crack growth which could lead to fracture and potential collapse.

As a guide to bridge engineers, AASHTO classifies structures that are to be considered redundant or nonredundant, in Art. 10.3.1, including a footnote to Table 10.3.1A in Ref. (*1*). As an example, AASHTO classifies multi-girder bridges as redundant and two-girder bridges as nonredundant. Such classifications are based on the simplified concepts widely held by bridge engineers on the behavior of as-built bridges under dead and live loads. These concepts, in turn, are based on the oversimplified AASHTO assumptions used in the design of steel girder bridges.

In the design of a straight two-girder steel bridge, the two girders alone (or the two composite Tee girders in composite construction) are considered to be the only design load paths available for transmitting all dead, live, and impact loads to the substructure. The deck, stringers and floor beams are considered only to transmit the vertical loads to each girder. The three-dimensional as-built structure is therefore reduced to a single girder for use in analysis and design. Because no longitudinal distribution of wheel loads is permitted by AASHTO, the resulting live load distribution to the girder is highly approximate. The bottom lateral bracing, top lateral bracing, if any, and diaphragms (cross bracing, cross frames, or cross trusses) are assumed to play no part in sharing the vertical loads with the two girders. For noncomposite construction the flexural and torsional strength of the deck is not considered. For composite construction the torsional strength of the composite deck/girder system is not considered. The actual wheel load distribution on the deck and the resulting influence on the girders is not considered. Stresses in the lateral bracing and diaphragms and their connections that are due to bending elongation and shortening of girder flanges under vertical loads, and those that are due to differential girder deflection under unsymmetrical live loads, are not even computed. Strain measurements on in-service bridges consistently show that these stresses may greatly exceed allowable stresses, depending on the as-built configuration of the structure (skew, offset diaphragms) (*9, 10, 11, 12*). In short, the three-dimensional behavior of all the components of the superstructure acting together to share the vertical loads, especially when unsymmetrical vertical loads exist, is not considered in design.

Although this elementary design model of the two-girder superstructure greatly simplifies the design of two-girder structures subjected to *static* loads and can be shown to be safe for *static* loads (the model may be and often is unsafe for dynamic load and design against fatigue), it fosters the erroneous idea that if one of the two girders of a simple-span bridge suffers a nearly full-depth fracture, say at midspan, all resistance to vertical loads vanishes, and the superstructure becomes geometrically unstable and collapse follows. The considerable amount of research into the stress history of in-service bridges as well as full-scale laboratory tests of three-dimensional bridge superstructure components indicate that the real behavior of steel bridges is significantly different from that assumed in analysis and design (Refs. 9 to 27). This report demonstrates that simple-span two-girder steel bridges do not necessarily collapse when one of the two girders fractures.

Clearly, the need exists for a better understanding of the real three-dimensional behavior of the as-built bridge structure under dead and live loads, especially for two-girder bridges, which are considered nonredundant by AASHTO. The alternate load paths that exist or that can be designed to provide redundancy in the event of fracture of one of the two girders need to be investigated. Simplified models of the after-fracture three-dimensional structure which retain the fundamental three-dimensional behavior of the bridge need to be developed for redundancy design and rating. Guidelines need to be prepared to assist in redundancy design and rating of two-girder steel bridges, and for establishing bridge inspection and replacement priorities. These needs for two-girder steel highway bridges are addressed in the investigation reported herein.

RESEARCH OBJECTIVES

The objectives of this investigation are (1) to develop a better understanding and definition of after-fracture redundancy, (2) to establish specific criteria for after-fracture redundancy, and (3) to develop guidelines for establishing after-fracture redundancy in two-girder bridges. Although the original research objectives, as defined in the NCHRP Project Statement, included various types of steel bridges, the NCHRP project panel agreed with the research investigators, early in the study, that the objectives should be redefined to concentrate the study effort to the after-fracture redundancy of two-girder bridges. The tasks, consistent with this modification, are defined as follows:

Task 1—Review relevant current domestic and foreign practice, performance data, and research findings. Assemble this information from both the technical literature and the unpublished experiences of bridge engineers and owners of steel bridges, placing emphasis on the performance of steel bridges in which fatigue cracking and/or fracture of one of the two girders is observed.

Task 2—Analyze and evaluate the information generated in Task 1 and establish a general definition of after-fracture redundancy with consideration of load levels. Consider new and innovative ideas as well as established practice.

Task 3—Develop a methodology for applying specific criteria for after-fracture redundancy to two-girder steel bridges.

Task 4—Prepare an interim report covering the results of Tasks 1, 2, and 3, and propose a detailed framework for the guidelines to be developed in the remaining Tasks, including examples illustrating the application of the methodology developed in Task 3.

Task 5—Verify the methodology developed in Task 3 for application to simple- and continuous-span two-girder bridges, including deck and through-girder bridges.

Task 6—Develop guidelines for establishing after-fracture redundancy in two-girder steel highway bridges. The guidelines are to be useful in the design of safe and economical new bridges as well as in establishing bridge inspection and replacement priorities for existing bridges. The guidelines are to be in a format suitable for consideration by the AASHTO Subcommittee on Bridges and Structures. The guidelines are to be accompanied by a detailed commentary and examples of specific applications intended to facilitate the understanding and use of the methodology.

SCOPE OF THE INVESTIGATION

The results of recent theoretical research into the behavior of two-girder steel highway bridges assuming a near full depth midspan fracture of one of the two girders indicates that the typical elementary model used by bridge engineers to analyze and design two-girder steel bridges cannot be used to predict the after-fracture behavior and redundancy of the bridge (*28*). The research reported in Ref. 28 clearly shows that following fracture of one of the two girders, the entire three-dimensional as-built structure is mobilized to resist the vertical dead and live loads. The bracing system consisting of lateral bracing and diaphragms is shown to be a major contributor to this resistance.

The investigation reported herein considerably extends the research reported in Ref. 28. Major emphasis is placed on utilizing the strength and stiffness of the bracing system to provide the after-fracture redundant alternate load path. Computer-based and noncomputer-based methodologies are developed for determining the after-fracture redundancy rating of an existing two-girder steel highway bridge, which contains a redundant bracing system, and for designing a redundant bracing system for new or existing two-girder bridges. Worked examples are included to illustrate the application of the noncomputer-based methodology. Guidelines are presented for the design and rating of two-girder steel highway bridges with redundant bracing systems. Of major significance is the direct provision for a bridge engineer specified serviceability limit to provide safe crossing of the bridge following a girder fracture.

Guidelines are also provided for two-girder bridges that do not contain a suitable redundant bracing system. These include guidelines for retrofitting the existing bracing system so that it qualifies as a redundant bracing system. Also included are guidelines for providing after-fracture redundancy using tension cables or rods in lieu of a redundant bracing system and guidelines for providing redundancy for through-girder steel highway bridges.

To minimize misinterpretation of the meanings of the words and phrases used in this report the following definitions are provided:

Alternate Load Path—In the event of fracture of one of the two girders, an alternate load path signifies the presence of a structurally stable system of members, components and connections in the superstructure, which is capable of transmitting vertical loads to the substructure.

Redundant Alternate Load Path—If the alternate load path is also capable of safely resisting the specified after-fracture dead and live loads and is further capable of maintaining after-fracture serviceability of the deck, it is called a redundant alternate load path.

After-Fracture Serviceability—After-fracture deflection-to-span length criteria established by the bridge engineer, which enable the possibility of fracture detection under dead load plus safety for heavy vehicles crossing the fractured bridge at normal highway speeds.

Inventory Rating—The maximum load level which may safely traverse an *unfractured* bridge for an indefinite period of time (*29*).

Operating Rating—The absolute maximum permissible load level to which an *unfractured* bridge may be subjected (*29*).

Redundancy Rating—The absolute maximum permissible load level to which a *fractured* bridge may be subjected for a short period of time. This is a new rating level introduced in this report and used for the after-fracture rating of a two-girder steel bridge using either the allowable stress method or the load factor method as defined in Ref. 29.

Redundant Bracing System—A bracing system, consisting of top and bottom lateral bracing and diaphragms, which conforms to the requirements specified in Chapter Three of this report.

In recent years, the NCHRP has sponsored several studies on bridge repair, rehabilitation, retrofitting, and strengthening which complement the guidelines presented herein and are excellent references. The NCHRP reports related to this investigation include the following:

NCHRP Report 102, "Effect of Weldments on the Fatigue Strength of Steel Beams," 1970 (*2*).

NCHRP Report 147, "Fatigue Strength of Steel Beams with Welded Stiffeners and Attachments," 1974 (*3*).

NCHRP Report 206, "Detection and Repair of Fatigue Damage in Welded Highway Bridges," 1979 (*4*).

NCHRP Report 227, "Fatigue Behavior of Full-Scale Welded Bridge Attachments," 1980 (*5*).

NCHRP Report 271, "Guidelines for Evaluation and Repair of Damaged Steel Bridge Members," 1984 (*30*).

NCHRP Report 293, "Methods of Strengthening Existing Highway Bridges," 1987 (*31*).

NCHRP Report 299, "Fatigue Evaluation Procedures for Steel Bridges," 1987 (*32*).

RESEARCH APPROACH

Task 1

This task was carried out in two parts: case studies of fatigue damaged and/or fractured two-girder steel highway bridges, and a literature review. These two parts are briefly described in the following sections.

Case Studies. Information was obtained and reviewed for 165 fatigue-damaged or fractured steel highway bridges in the United States, Canada, and Japan. Of these, 12 were two-girder steel bridges. This information was obtained primarily from files maintained by the Center for Advanced Technology for Large Structural Systems (ATLSS) at Lehigh University. ATLSS is an NSF-sponsored Engineering Research Center. The Center maintains a comprehensive, current file on all fatigue-damaged or fractured steel bridges that are reported throughout the world.

The Center does not have any report of fatigue-fracture related damage or collapsed steel two-girder highway bridges in Europe. During August 1988, the Principal Investigators of this project visited Switzerland and France to collect information, if any, from Europe. Discussions were held at the Institute Construction Metallique (ICOM) of Ecole Polytechnique Federale de Lausanne (EPFL) with experts on European highway bridges. They indicated that they do not know of any fatigue-fracture related damage or collapses of two-girder steel highway bridges in Europe (*33, 34*). They were concerned, however, that their current two-girder bridge designs were highly nonredundant and therefore are very vulnerable to future fatigue-induced fractures.

A description of the 12 two-girder steel highway bridges studied in Task 1 and a description of the fatigue-fracture damage of these bridges are presented in Appendix A.

Literature Review. Computerized literature searches were made using DIALOG Information Service Inc., through the Lehigh University Libraries which have access to the Highway Research Information Service (HRIS), the Computerized Engineering Index (EI), and other data bases. Research into redundancy as defined by Art. 10.3.1 of the AASHTO bridge design specification (*1*) can date only from the late 1970s with the introduction of the 1978 AASHTO *Guide Specification for Fracture Critical Nonredundant Steel Bridge Members* (*35*). As expected, the earliest relevant significant publications dated from 1979. A chronological review of these publications is presented in Appendix B.

Task 2

The main work of Task 2 was to establish a general definition of after-fracture redundancy in steel highway bridges. The term redundant and the associated terms nonredundant and redundancy have at least three different meanings in bridge engineering, only one of which was the subject of this investigation. These meanings are briefly defined as follows:

Statically Indeterminate Structure. This is often referred to as a redundant structure. It means that the internal stress resultants or reactions cannot be determined by the equations of equilibrium alone. Removal of the redundant members or supports, for example, will result in a statically determinate structure. This definition of redundancy is not the subject of this report.

Overdesigned Structure. Two-girder steel highway bridges are inherently overdesigned for static loads because of the simplified elementary planar model used in their analysis. The as-built structure has excess capacity compared with the design capacity. It is possible for an as-built two-girder steel bridge with a near full depth midspan fracture of one of the two girders to also have excess capacity compared with the original design capacity (*28*). Redundancy in terms of excess capacity is also not the subject of this report.

After-Fracture Redundancy Defined by Art. 10.3.1, Ref. 1. The term redundancy and related terms used in this report applies to after-fracture redundant and nonredundant load path structures as referred to in Art. 10.3.1 of the AASHTO bridge design specifications, including the footnote to Table 10.3.1A.

Much of the work of establishing a general definition of after-fracture redundancy in steel highway bridges has to do with clarifying the meanings of the various terms used by AASHTO, such as collapse. It requires the redefining of these terms considering the after-fracture behavior, strength, and serviceability of the real three-dimensional structure rather than the assumed behavior of the oversimplified elementary analysis and design model.

A general definition of after-fracture redundancy together with the definitions of terms used is presented in Chapter Two.

Task 3

Research since 1979 suggests that the bracing system, consisting of top and bottom lateral bracing plus diaphragms such as cross bracing, cross frames, and cross trusses, is a logical and practical source of redundancy. Research conducted at Lehigh University and reported in Ref. 28 developed a basic understanding of the bracing system as an alternate load path and broke new ground in developing design procedures to provide after-fracture redundancy. The research reported herein considerably extends the work of Ref. 28, and develops design and rating procedures for redundancy in new and existing two-girder bridges. This investigation also develops guidelines for redundancy of two-girder bridges without relying on the bracing system and for through-girder bridges, where a redundant bracing system cannot be provided.

Task 4

Task 4 required the preparation of an Interim report at the 12-month stage, presenting the results of Tasks 1, 2, and 3. It was clear during the first year of the investigation that if meaningful guidelines were to be prepared, the objectives and scope should be redefined to consider only two-girder steel highway bridges, an important bridge type, and not various types of bridges as originally intended. With the concurrence of the NCHRP project review panel at a meeting in Washington, D.C. on May 21, 1987, the investigators agreed to continue work on redundancy with respect to both design and rating of new and existing, simple-span and continuous-span, two-girder bridges, including deck and through-girder bridges.

Task 5

The decision in Task 4 to confine the research effort to two-girder bridges was critical to the progress of the investigation. Although a few months time was previously diverted to studying other types of bridges, it meant that a more efficient use of project resources could now be focused on the two-girder bridge. The preparation of an additional technical progress report for panel review in early 1988 helped consolidate ideas for the design and rating procedures and guidelines for two-girder bridges with redundant bracing systems. The development of redundant bracing system requirements together with worked design and rating examples are provided in Appendix C. The development of requirements for redundant tension cables and rods together with worked examples are included in Appendix D.

Task 6

Guidelines for two-girder bridges with redundant bracing systems were developed as a natural extension to the work of Task 5, and are presented in Chapters Three and Five. Because some two-girder bridge types cannot rely on redundant bracing systems, such as through-girder bridges, guidelines for alternate load paths independent of bracing systems were developed and are presented in Chapter Four. Chapter Four also provides guidelines for redundant bracing system retrofit and for continuous two-girder bridges.

Task 7

The purpose of Task 7 was to prepare a final report, documenting all the research undertaken in this investigation. Chapters Three, Four, and Five of this report present guidelines for providing after-fracture redundancy in simple-span and continuous-span, two-girder deck and through-girder steel highway bridges. Appendixes C and D provide technical details, the development of design and rating equations, and design and rating examples.

CHAPTER TWO

FINDINGS

STEEL GIRDER BRIDGE DAMAGE—CASE STUDIES

Information was obtained and reviewed for 165 fatigue or fracture damaged steel bridges of various types in the United States, Canada, and Japan. Of these, 12 were two-girder steel highway bridges, having fatigue and/or fracture damage of the girders or, in one case, of the floor beam connection to the girder.

Appendix A of this report provides a description of each of the 12 bridges together with a description of the fatigue or fracture damage. All 12 bridges are located in the northern United States. No fatigue- or fracture-damaged two-girder steel highway bridges were reported outside of the United States.

All 12 bridges are continuous, having 3 to 6 spans. The length of time the bridges were in service prior to developing substantial fatigue cracking or fracture varies from 1 to 25 years. Although significant fracture of a girder occurred in three of the bridges no collapse of a span is reported.

Table 1 summarizes the 12 case studies reviewed in Appendix A. The bridge number in the first column of the table corresponds to the number in Appendix A. The first column also shows schematic views of the girder profile together with the relative positions of fatigue cracks or fractures, the crack direction (vertical or horizontal) and the relative lengths of the cracks or fractures. The second column describes the type of detail involved. The third column provides a description of the extent of the observed cracking or fracture.

PREVIOUS RESEARCH

In 1978, the AASHTO *Guide Specification for Fracture Critical Nonredundant Steel Bridge Members* was introduced (*35*). Allowable stress ranges for nonredundant load path structures and examples of redundant and nonredundant load path structures were introduced into the 12th Edition of the AASHTO *Standard Specifications for Highway Bridges* with the 1979 Interim Specification. Neither the allowable stress ranges for nonredundant load path structures nor the examples for redundant and nonredundant load path structures were determined by rational research. Thus, previous research into redundancy as presently defined by the 13th Edition of the AASHTO *Standard Specifications for Highway Bridges* can only date from the late 1970s.

Deterministic-Based Research

The following is an overview of deterministic-based research into after-fracture redundancy. Brief abstracts of research significant to two-girder steel highway bridges are provided in Appendix B.

One of the first discussions of after-fracture redundancy in riveted and welded steel girder bridges is provided by Sweeney (*36*). He points out that fatigue and fracture are much more critical in welded girders than in riveted girders. This is because of the multitude of rivet holes and individual built-up plates, which have inherent redundancy (crack stoppers) and lower rigidity (lower stress ranges).

Another early investigation is by Haaijer, et al. (*37*), who introduce four design procedures to deal with redundancy and fatigue in a direct way. These procedures are based on the service load, overload, maximum load, and fail-safe load. Each design is based on a load level and limit state at that level. The fail-safe load is considered only in design for fatigue and assumes one separated or fractured component.

A third early investigation by Csagoly and Jaeger (*38*) considered the possibility of excluding single-load-path structures from future design. This investigation considered six examples of bridge collapses or severe damage that had occurred in recent years, many in the 1970s. The investigation led to the first design specification, in the 1979 Ontario Highway Bridge Design (OHBD) code, which attempted to deal with design for redundancy. It concludes that a mandatory backup system should be made part of the bridge design process.

In the early 1980s, Heins and Hou (*39*) and Heins and Kato (*40*) made the first attempts to investigate load redistribution following cracking or major fracture of a girder. The two studies show that the bottom lateral bracing plus cross-bracing diaphragms effectively create redundancy in two-girder steel bridges. A significant finding is that use of the bracing system results in a two-girder bridge behaving similarly to a three-girder bridge.

Sangare and Daniels (*41*) followed up on the work of Heins, et al., by investigating the after-fracture redundancy of a steel deck-truss bridge with the tension chord of one of the two parallel trusses completely fractured at midspan. The bracing systems were found to be very effective in providing redundancy. Under full service dead load plus four lanes of HS-20 lane loading and impact, all members of both main trusses remained elastic even though the bracing system contained some yielded and buckled components.

Reference 42 reviews the state of the art on redundant bridge systems as of 1985 (*42*). It is concluded that (1) little work has been done to quantify the degree of redundancy needed in bridges, (2) further research into redundancy is encouraged, and (3) quantifiable results are increasingly possible in view of developments in computer speed and available software. Unfortunately, this important paper does not state its specific definition of redundancy, and does not clarify the meaning of redundancy in the context of Art. 10.3.1 of the AASHTO bridge design specifications (*1*). Some of the authors appear to be thinking of redundancy in this context, while other authors are

Table 1. Summary of two-girder steel bridge damage—case studies.

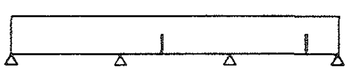
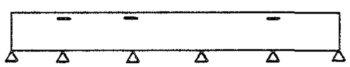
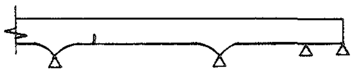
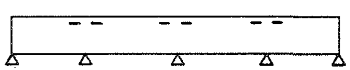
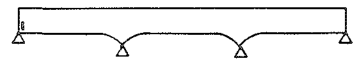
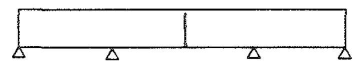
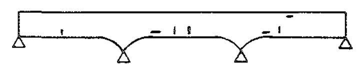
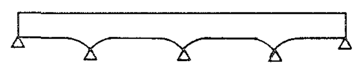
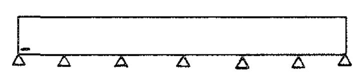
Bridge Number and Girder Profile	Type of Detail	Extent of Cracking and/or Fracture
1 	Floor beam-girder web gap Gusset plate cope to girder web	Horizontal crack, top of web Vertical crack, bottom of web
2 	Gusset plate welds to bottom flange	Substantial fracture of main girder
3 	Floor beam-girder web gap	Horizontal crack, top of web
4 	Floor beam-girder web gap	Vertical crack, top of web
5 	Lateral bracing connection plate	15 % of web
6 	Floor beam-girder web gap	2.5" to 10.5" horizontal crack, top of web

Table 1. Continued.

Bridge Number and Girder Profile	Type of Detail	Extent of Cracking and/or Fracture
7 	Gusset plate cope to girder web	4" vertical crack, bottom of web
8 	Bottom flange butt weld - electroslag weld	Fracture: full web, full bottom flange
9 	Gusset plate cope to girder web Connection plate to web gap	Vertical crack, bottom of web Horizontal crack, top of web
10 	Floor beam bracket (outrigger)	14" horizontal crack, floor beam web top 7" vertical crack, connection plate
11 	Lateral gusset plate	Fracture: 95 % of web full bottom flange
12 	Floor beam-girder web gap Longitudinal stiffener groove weld	19" horizontal cracks bottom of web Full depth web

apparently addressing redundancy as the excess capacity for static loads in a normally designed and undamaged structure.

Daniels, et al. (*28*), recently turned their attention to the redundancy of welded two-girder deck type steel highway bridges. This is the investigation that preceded this NCHRP Project 12-28(10). Daniels, et al., established that relatively simple guidelines could be proposed for the redundancy design and rating of new and existing two-girder steel bridges.

While research into after-fracture redundancy of steel girder bridges was underway, the bridge design profession was being called upon to provide redundancy in actual bridges. Parmelee and Sandberg (*43*) presented a paper to the New Orleans AISC National Engineering Conference, describing the provision for redundancy to a three-girder bridge by using the cross bracing to support a girder in the event of a near full depth girder fracture. This paper contains the following two very important conclusions. The first is that the redundant system should provide a clear signal that fracture has occurred and that repairs are needed. The second is that criteria need to be established for redundant live load levels, permissible allowable stresses, load factors, deflection limits (after-fracture serviceability), and critical fracture scenarios.

At the same AISC National Engineering Conference in New Orleans, Seim (*44*) presented a paper investigating economical ways to provide redundancy in steel bridges. The redundancy of a two-girder steel bridge is studied. A significant conclusion is that the cost of adding bracing to provide redundancy is far less than the cost of adding another steel girder.

Probabilistic/Reliability Based Research

The following is an overview of probabilistic/reliability based research into after-fracture redundancy. Brief abstracts of research significant to two-girder steel highway bridges are provided in Appendix B.

Galambos (*45*) examined the use of a simple first-order probabilistic method to assess the reliability of the 1977 AASHTO specifications for the design of steel bridges. It is demonstrated that the AASHTO load factor design (LFD) method provides a consistent reliability index but that the AASHTO allowable stress design (ASD) method does not. The study also investigated load- and resistance-factor design methods. These methods use multiple load factors and multiple resistance factors. It is concluded that load- and resistance-factor methods are shown to be the most reliable and economical. Uniform reliability can be achieved through the judicious choice of the load and resistance factors. The study concludes that there is sufficient statistical information on steel structures available to allow a probability-based design method to be developed.

Gorman (*46*) investigates the interaction between structural redundancy and system reliability. Structural redundancy is defined as the degree of static indeterminacy. The study concludes that, for truss examples, increasing structural redundancy increases system reliability up to a point. It is shown, however, that for highly redundant structures system reliability is only slightly improved, or even slightly reduced.

Moses and Verma (*47*) in a recent NCHRP project have implemented a reliability-based strategy for evaluating bridge components. The application is not intended to predict the probability of structural failure, but rather attempts to evaluate and adjust the safety factors in an evaluation code. The load and resistance factor design (LRFD) format was adopted for flexibility in dealing with different bridge components. The reliability of the partial safety factors is transparent to the code user and the designer would apply the LRFD check in a deterministic fashion. Strength rather than serviceability limit states is discussed. Safety is expressed in terms of a measure of the probability that the capacity will exceed the extreme load (legal or illegal) that may occur during the inspection interval. Data for the loading model have been assembled using Moses' weigh-in-motion (WIM) results (*48*). Load and resistance factors are recommended which lead to reliability levels. Numerous comparisons illustrate the effects on rating for different factors and options contained in the proposed rating guidelines. According to Moses these guidelines are suitable for inclusion in the AASHTO *Manual for Maintenance Inspection of Bridges* (*29*).

There is much useful literature and, of course, considerable differences of opinion about redundancy. In particular, there are differences of opinion on which types of steel bridges can be defined as redundant and which bridges are more redundant than others (redundancy classification). Various tools for safety evaluation have been proposed that presently are at different stages of development. Research topics include risk analysis, failure scenarios, progressive collapse, Bayesian uncertainty propagation models, strategies for ratings, inspections, and maintenance and knowledge-based expert systems, with fuzzy logic. Although many interesting results are available, the after-fracture behavior and reliability aspects of the bridge structural systems, which are the central focus of this research project, remain to be studied further (*49, 50*).

Though the further development of tools using probabilistic/reliability techniques for failure analysis, risk analysis and evaluation, and decision analysis is highly desirable, much more study is warranted. For example, more data need to be collected, compiled, and evaluated for model verification. In addition, the expert systems approach to damage assessment and decision support, such as SPERIL-1 (*51*), although extremely useful in earthquake situations, is not yet appropriate for after-fracture redundancy investigations, such as the one reported herein. The basic rationale behind expert systems, however, strongly suggests the potential for additional research and use in design and rating of bridges for after-fracture redundancy.

AASHTO DEFINITION OF REDUNDANCY

The 13th Edition of the AASHTO bridge design specifications contains the following definitions in Art. 10.3.1 (*1*):

Redundant Load Path Structures—Structure types with multi-load paths where a single fracture in a member cannot lead to the collapse. For example, a simply supported single span multi-beam bridge or a multi-element eye bar truss member has redundant load paths.

Nonredundant Load Path Structures—Main load carrying components subjected to tensile stresses that may be considered nonredundant load path members—that is, where failure of a single element could cause collapse—shall be designed for the allowable stress ranges in Table 10.3.1A for Nonredundant Load Path Structures. Examples of nonredundant load path members are flange and web plates in one or two girder bridges, main one-element truss members, hanger plates, and caps at single or two-column bents.

Both definitions hinge on the word "collapse". But collapse is not defined. The definitions also suggest that if multi-load paths are present collapse cannot occur. If the term collapse is used in the usual engineering sense to mean that the structure or, at least, a significant portion of it has dropped to the ground or into the river, such as happened with the Silver Bridge (*8*) and the Schohaire Bridge (*52, 59*), this definition is not necessarily true. Multi-load paths must not only exist but also be capable of resisting a certain level of dead and live loads following the fracture. Otherwise, even with the existence of multi-load paths, the structure may still be nonredundant.

Lacking from the AASHTO definitions is any reference to after-fracture serviceability of the bridge. The bridge or a significant part of it does not have to collapse to render it totally unusable or at least highly dangerous to cross. Vehicles traveling at normal highway speeds and at night may not be aware that a fracture has occurred until they attempt to cross the bridge. After-fracture deflections, twisting of the deck and local deck failures may be such that the vehicles and occupants cannot safely cross. The resulting effect may be as tragic as though the bridge or a span had collapsed.

After-fracture redundancy is, therefore, primarily concerned with serviceability of the bridge deck and safety of the traveling public and not so much with the word collapse. Deflection and twisting deformations of the deck consist of two parts. The first is the deformation under dead load alone. The second is the additional deformation under the live and impact loads. The ratio of the two is a function of the dead load to total load ratio for the particular bridge.

It is important that indications of a bridge fracture be reported as quickly as possible. Therefore, dead load deformations should be large enough that it is obvious, especially when viewed during daylight hours, that the bridge has suffered damage. However, the total dead, live, and impact deformations should be small enough that the bridge is still serviceable to heavy vehicles traveling at normal highway speeds. This is especially important during the night time hours when deformations of the bridge may not be visible within the range of the vehicle's headlights until the vehicle is about to enter or is crossing the bridge.

The AASHTO definitions provide examples of redundant and nonredundant load path structures. For example, two-girder bridges are considered to be nonredundant. It is assumed by AASHTO that if one girder fractures, collapse follows because AASHTO assumes that no alternate load path exists. As mentioned in Chapter One of this report this assumption is based on the traditional oversimplified AASHTO model of a two-girder bridge used in design and rating. In that model it is assumed that the two girders alone (or composite Tee girders) are the only load paths available to transmit all dead, live, and impact loads to the substructure. The deck, stringers, and floor-beams are considered only to transmit the loads to the girders but not to interact with them. The effect of the deck system in distributing the live loads longitudinally to the girders is ignored. The deck is assumed to behave like a series of narrow planks laid across the girders. The interaction of the bracing system with the girders is completely ignored. If this model were correct, upon fracture of one of the two girders, no resistance to vertical loads could be developed and the bridge or a major part of it would indeed collapse.

This model, however, is not correct and does not approximate the way real bridges carry loads in many cases (*28*). Although it is a safe model to use for static loads and with ductile materials, it is not necessarily safe for dynamic, fatigue producing loads, such as bridge live loads, and is not realistic when considering after-fracture redundancy of two-girder bridges where one girder is fractured.

Finally, the AASHTO definition of redundancy does not address the magnitude of level of loading which a fractured structure is expected to support. The AASHTO manual for maintenance inspection (*29*) states, "The factors of safety used in designing new bridges may provide for an increase in traffic volume, a variable amount of deterioration, and extreme conditions of long continued loading."

Although traffic volume may be larger than when the bridge was new, and some deterioration is likely, the fractured bridge is certainly not going to be subjected to "extreme conditions of long continued loading." Thus, the probability of the fractured bridge experiencing extreme overloads or even the design live loading is reduced. Consequently, it should be reasonable to consider reduced levels of loading and reduced factors of safety when providing for redundancy.

The AASHTO manual for maintenance inspection (*29*) also states, "The factors of safety used in rating existing structures must provide for unbalanced loads; reasonably possible overloads and illegal and careless handling of vehicles. For both design and rating factors of safety must provide for lack of knowledge as to the distribution of stress. . . ."

The probability of the fractured bridge seeing overloads, and illegal and careless handling of vehicles, is reduced if the fracture is quickly detected. If a more sophisticated analytical model is used in redundancy design and rating, the lack of knowledge as to the distribution of stress is reduced. These are further arguments in favor of reduced levels of loading and reduced factors of safety when providing for redundancy. Again, the fractured bridge is not expected to remain in service for an extended period of time. It is also reasonable to consider reduced levels of loading and factors of safety when evaluating the redundancy of a bridge.

AASHTO already considers reduced load levels and reduced factors of safety in its rating provisions (*29*). Load levels and separate factors of safety are provided for inventory and operating rating of bridges. Reduced factors of safety are used for operating ratings. An extension of these concepts would suggest separate loading and factor of safety provisions for the redundancy design and rating of new or existing bridges.

It should be mentioned here that Ref. 29 is being revised under an NCHRP-directed effort.

ALTERNATE DEFINITION OF REDUNDANCY

The following alternate definition of after-fracture redundancy was formulated for use during this investigation to address the issues discussed above and to provide a fundamental basis on which to develop guidelines for the redundancy design and rating of two-girder steel bridges (this definition should also be applicable with little or no modification to other steel bridge types as well): ***Redundant Load Path Structure:*** **New, existing or rehabilitated steel highway bridges where at least one alternate load path exists and is capable of safely supporting the specified dead and live loads and maintaining serviceability of the deck following fracture of a main load carrying member.**

A nonredundant load path structure, of course, is one which does not qualify as redundant. It is inappropriate to provide general examples of redundant and nonredundant load path structures in this report because much more research is needed. Each bridge type must be investigated separately. Appropriate realistic models for after-fracture redundancy design and rating must be developed and used to investigate redundancy in each bridge type. Although this investigation is concerned with the redundancy of two-girder bridges, the basic concepts and approaches developed and reported herein can be applied to other bridge types as well. Examples of redundant and nonredundant bridge types could be formulated after each bridge type is investigated. It is important, however, that the examples also indicate under what conditions a particular bridge type may be both redundant and nonredundant. For instance, it is shown in this report that two-girder bridges having certain combinations of span length, number of lanes, number of interior diaphragms, girder depths, and AASHTO bracing systems may be redundant, otherwise they are nonredundant. As a result of this investigation, it is shown herein that two-girder steel highway bridges can be considered redundant load path structures provided they are designed or retrofitted to meet the guidelines presented in this report. Otherwise, they may be classified as nonredundant.

AASHTO Operating and Inventory Ratings

As part of a nationwide bridge safety program, existing steel bridges are inspected at regular intervals not to exceed 2 years (*29*). A steel bridge is rated whenever it is obvious from the inspection that the conditions upon which the bridge was originally designed have changed significantly (*29*). These changes can include the following: (1) deterioration of the structure due to corrosion, overload, fatigue cracking, impact damage, and so forth; and (2) an increase in the vehicular loading intensity and frequency.

A bridge rating analysis is performed as part of a short or long term repair, retrofit, rehabilitation, or replacement plan. The outcome of a rating analysis may be to close the bridge, to post the bridge for maximum vehicle loading, and/or to schedule the bridge for repair, retrofit, rehabilitation, or replacement. In the rating analysis a rating factor, RF, is calculated which, when multiplied by the gross vehicle weight, GVW, of the rating vehicle, gives a rating, usually expressed in tons. A rating is performed for each member or component of the steel bridge superstructure and for more than one rating vehicle. Reference 29 provides three vehicles. States often add or substitute other vehicles, especially when the state's legal vehicles are more severe than the AASHTO vehicles. For each rating vehicle the bridge rating is determined as the minimum rating achieved among all the members and components considered.

If a bridge continues in service after a rating has been performed, the bridge is assumed to be able to function in accordance with the outcome of the rating, without considering the possibility of an impending disaster, until such time that a further rating is scheduled or considered necessary.

The next section discusses the need for a new AASHTO redundancy rating which provides an after-fracture redundancy rating of a two-girder bridge in the same way that the AASHTO inventory and operating ratings are applied to unfractured bridges. The new redundancy rating also uses the allowable stress and load factor method. Prior to introducing the new redundancy rating level and redundancy rating factor, RRF, the current AASHTO inventory and operating rating levels and methods are briefly reviewed in the following.

AASHTO Rating Levels

The girders are rated at two levels (*29*):

1. *Operating Rating Level,* which is the absolute maximum permissible load level to which the structure may be subjected.
2. *Inventory Rating Level,* which is a load level that can safely utilize an existing structure for an indefinite period of time.

AASHTO Rating Methods

The girders are rated using one or both of two methods (*29*).

Allowable Stress Method. In the allowable stress method, the girder is analyzed under service dead, live, and impact load combinations (*1*) using linear elastic theory. The rating factor, RF, is determined such that the maximum girder stress does not exceed the allowable stress. For noncomposite girders the *RF* for both the operating and inventory rating levels is given by

$$\mathrm{RF} = \frac{f_{\mathrm{all}} - f_D}{f_L} \tag{1}$$

where f_{all} = allowable stress for the operating or inventory rating level, f_D = dead load stress, and f_L = live load plus impact stress produced by the rating vehicle.

For unshored construction, the RF for composite girders for both the operating and inventory rating levels is given by

$$\mathrm{RF} = \frac{f_{\mathrm{all}} - f_{D1} - f_{D2}}{f_L} \tag{2}$$

where f_{D1} = dead load stress prior to hardening of the concrete, and f_{D2} = additional dead load stress due to loads (wearing surface, parapets, for example) applied to the composite girder.

Load Factor Method. In the load factor method, the girder is analyzed under factored dead, live, and impact load combinations using linear elastic theory. The rating factor, RF, is determined such that the load effect (bending moment, for example) does not exceed the strength of the girder determined using a strength reduction factor. For noncomposite girders, the RF for the operating rating level is given by

$$\mathrm{RF} = \frac{\phi S_u - \gamma_D D}{\gamma_L (L + I)} \tag{3}$$

where ϕ = strength reduction factor, S_u = member strength, D = dead load effect, $L + I$ = live plus impact load effect, and γ_D = dead load factor, and γ_L = live load factor.

For unshored construction, the RF for composite girders for the operating rating level, in terms of the tension stress in the girder, for example, is given by

$$\mathrm{RF} = \frac{\phi F_y - \gamma_D f_{D1} - \gamma_D f_{D2}}{\gamma_L f_L} \tag{4}$$

where F_y = yield stress.

The RF for the Inventory rating level is 0.6 times the corresponding operating ratings (*29*).

NEED FOR A REDUNDANCY RATING LEVEL

AASHTO operating and inventory ratings are performed for bridges in which the oversimplified AASHTO model used (but updated for current design practices) in the design is still applicable for rating. That is, except for corrosion damage, limited fatigue cracking, limited impact damage, missing rivets, bent flanges, changes in traffic lanes, unique approach conditions influencing impact values, the connectivity of the structural members is essentially the same as that assumed in the design. For this reason, the AASHTO assumptions on the distribution of loads to the girders are virtually identical in both design and rating even though bridge deterioration and significant changes in traffic conditions may have occurred.

A vastly different situation arises as a result of fracture of one of the girders of a simple-span two-girder bridge, for example. In this case the dead and live loads are redistributed in such a way that the three-dimensional behavior of the entire superstructure is involved (*28*). It is possible, in many cases, to find suitable alternate load paths that bypass the fractured girder, but this suggests a much different analytical model from that used in the traditional AASHTO design and rating analyses.

Also different is the expectation that, after fracture occurs, the bridge should continue to function, under normal traffic conditions, until the next inspection cycle. Although the fractured bridge should be expected to remain serviceable until the fracture is discovered, the time interval between fracture and detection of the fracture is probably quite short (day, week, month) in comparison to the usual remaining life expectancy of the bridge (many years). Recent experience suggests that the fracture would likely be detected within a relatively short period of time as a result of excessive deflections, other visible signs of distress, or during bridge maintenance or inspection (*7, 53*).

There is clearly the need for an additional rating level that addresses after-fracture bridge redundancy with respect to specific fracture scenarios. For two-girder steel bridges, this report proposes and uses the term redundancy rating level. The proposed redundancy rating would be performed, along with the AASHTO operating and inventory ratings of an existing two-girder steel highway bridge. The redundancy rating can be based on either a worst case fracture scenario or on one or more plausible fracture scenarios as revealed by design conditions or inspections for fatigue cracking. Whereas the operating and inventory ratings are carried out for every member and component of a bridge superstructure, the redundancy rating is confined to the members and components of the alternate load path.

Assuming that there is a low probability that the maximum design loading will occur in the time interval between fracture and fracture detection, the proposed redundancy rating could be based on elevated allowable stresses and reduced load factors as is currently done for the AASHTO operating rating. The same rating vehicles could be used. However, the number of traffic lanes might be less than that presently required for design and rating. Suitable allowable stresses and load factors need to be recommended. Because the after-fracture deck deflection is expected to be larger than the AASHTO design deflection, the impact used in a redundancy rating is expected to be larger.

Providing recommendations to the several points discussed above, based on extensive analytical or experimental research, is not within the scope of this investigation. However, recognizing that many of the AASHTO design and rating provisions are based on engineering judgment and experience, it is possible to provide guidelines suggesting extensions of these provisions to redundancy rating.

Chapters Three, Four, and Five, together with Appendixes C and D, propose such guidelines and demonstrate their application to composite and noncomposite, simple-span and continuous-span, steel two-girder highway bridges.

DESIGN FOR REDUNDANCY

Redundancy rating, as proposed in this report, is used to determine the after-fracture redundancy rating of an as-built or existing two-girder bridge in terms of a specified rating vehicle. If the resulting rating is less than the rating required to provide safety and after-fracture serviceability of the bridge, it is necessary to retrofit the existing alternate load path or design a new alternate load path to provide the required rating. If a redundant alternate load path is to be provided as part of the design process for new two-girder bridges, guidelines and procedures are required for design for redundancy. Chapters Three, Four, and Five, together with Appendixes C and D, also propose guidelines for design for redundancy and demonstrate their application to two-girder steel highway bridges.

CHAPTER THREE

GUIDELINES FOR REDUNDANT BRACING SYSTEM DESIGN AND RATING

APPLICATIONS

These guidelines are intended for application to new or existing simple-span, steel, two-girder deck-type highway bridges. The girders may be riveted or welded. The bridges may be composite or noncomposite. These guidelines are applicable to each of the following design and rating situations: (1) the design of an alternate load path to provide redundancy in a new bridge; (2) the design of a new alternate load path to provide redundancy in an existing or rehabilitated bridge; (3) the retrofit design of an as-built or existing alternate load path to provide redundancy in an existing or rehabilitated bridge; and (4) the analysis of an as-built or existing alternate load path to determine the after-fracture redundancy rating of an existing bridge.

Representative cross sections of two-girder bridges to which these guidelines are applicable are shown in Figures 1 to 7 (*54,*

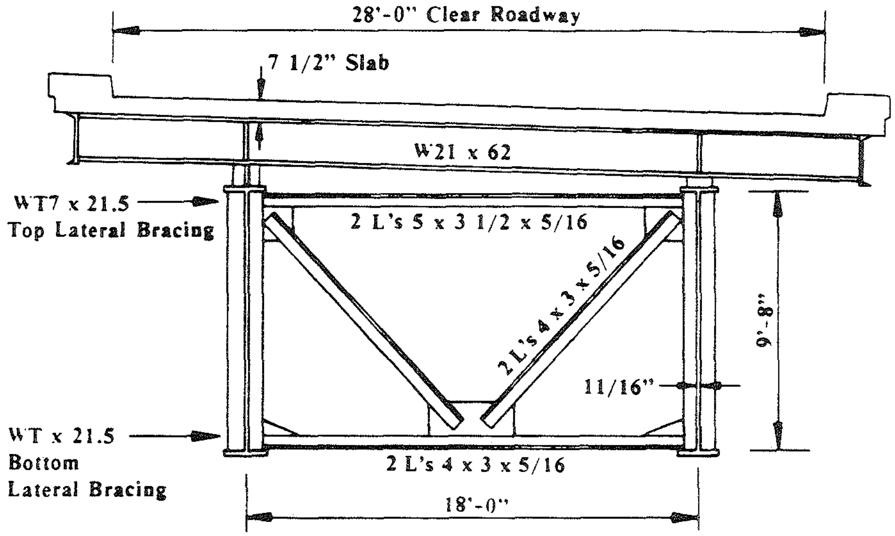


Figure 1. Cross section of 138-ft simple span, noncomposite two-girder bridge (Ref. 54).

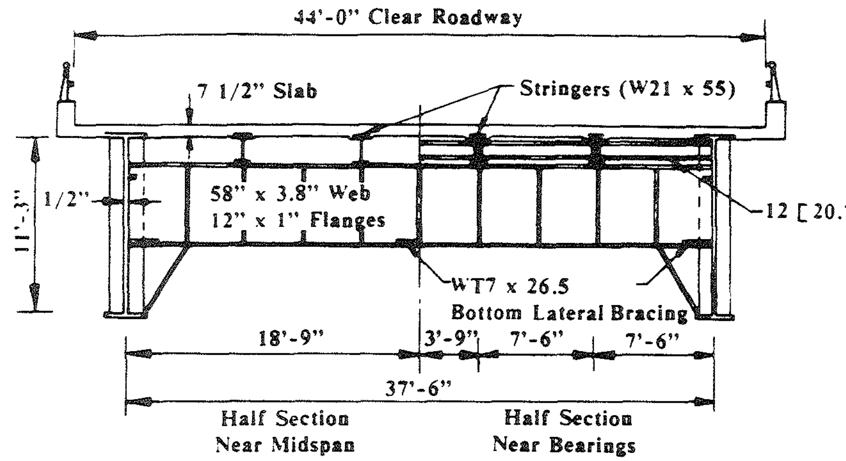


Figure 3. Cross section of 180-ft simple span, noncomposite three-lane, two-girder bridge (Ref. 55).

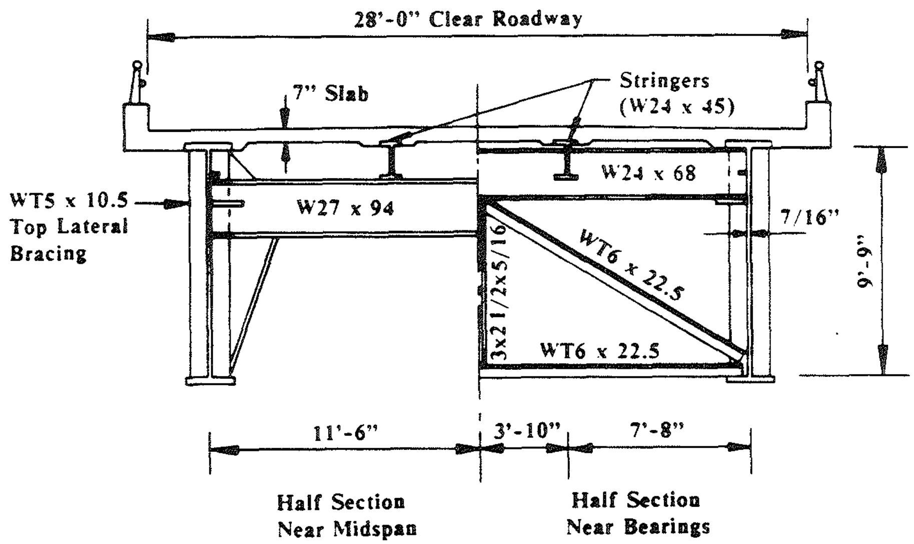


Figure 2. Cross section of 180-ft simple span, noncomposite two-girder bridge (Ref. 55).

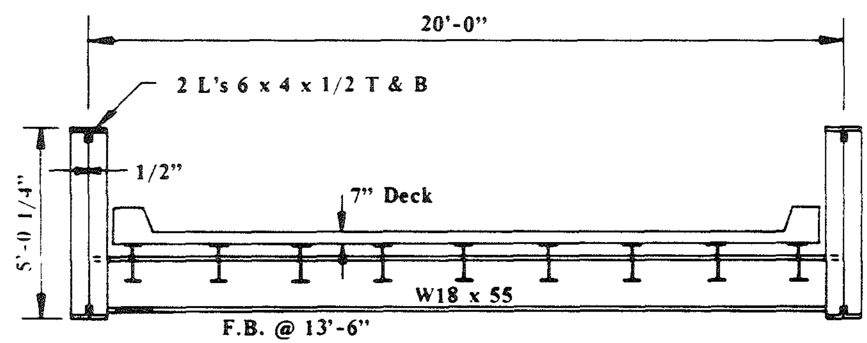


Figure 4. Cross section of 55-ft simple span, through-girder bridge (Ref. 56).

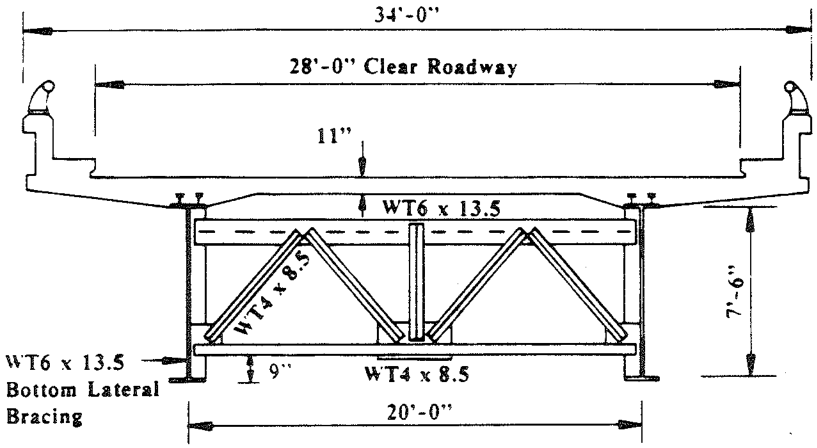


Figure 5. Cross section of 128-ft simple span, composite two-girder bridge (Ref. 57).

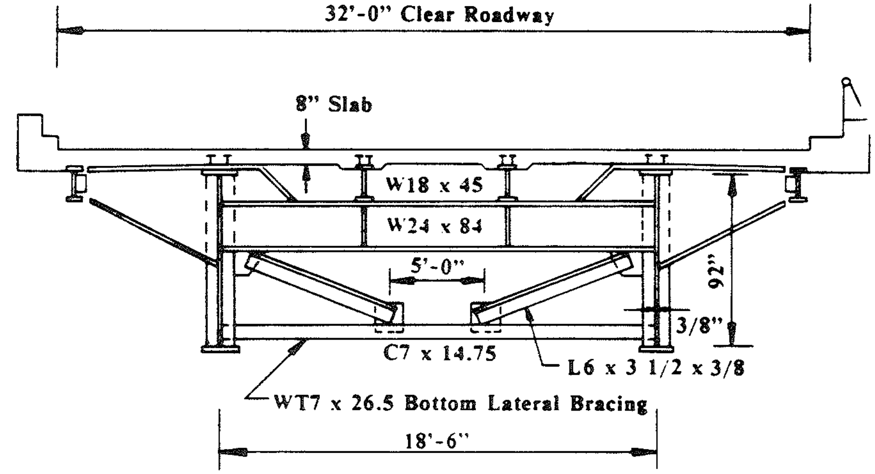


Figure 6. Cross section of 180-ft 2-span continuous, composite two-girder bridge (Ref. 58).

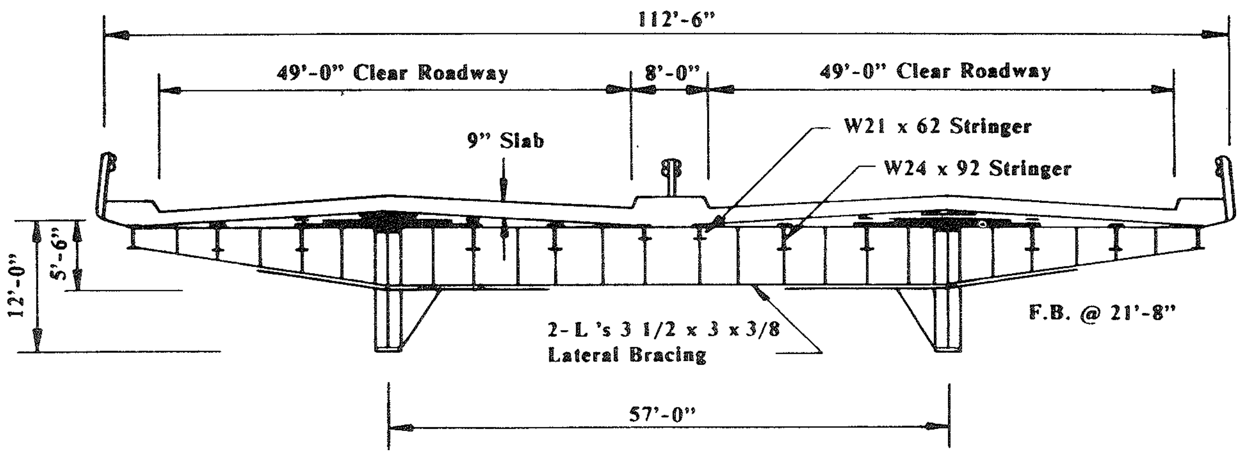


Figure 7. Cross section of 120-ft simple span, noncomposite two-girder bridge (Ref. 59).

55, 56, 57, 58, 59). The guidelines are particularly applicable to those bridges in which the as-built or existing lateral and cross-bracing systems and connections can be economically and practically retrofitted, if necessary, to provide the redundant alternate load path.

Figure 1 is an example of such a bridge. The remaining figures, except Figure 4, are examples of new or existing bridges with bracing systems that require more extensive redesign or retrofit to provide the redundant alternate load path. The guidelines in this chapter are applicable whether or not the bracing system is to be designed or retrofitted to an alternate load path.

The through-girder bridge in Figure 4 is an example of a bridge with no bracing system or perhaps only a bottom lateral system. In this case an alternate load path must be provided independently of the bracing system. Chapter Four presents guidelines for such bridges.

The retrofit design of an as-built or existing bracing system to provide a redundant alternate load path likely will involve the evaluation and repair of damaged members in the existing bridge. It may also be necessary in many cases to strengthen as-built or existing members and connections. The NCHRP reports listed in Chapter One will provide considerable help in these situations and should be used together with these guidelines.

DESCRIPTION OF FRACTURE

The guidelines in this chapter apply to new or existing steel two-girder bridges in which a near full depth fracture is assumed to occur in one of the two main girders. Although the exact reason for the fracture is not important in applying the guidelines and procedures developed in this report, fracture most likely results from unstable crack growth associated with fatigue cracking. The guidelines and procedures have been developed assuming that a fracture can occur anywhere along the length of a girder. The most likely locations resulting from fatigue cracking are the following (*4, 8, 30, 31*): (1) at details with the lowest fatigue strength, (2) in zones of highest tension stress range, (3) at details exhibiting displacement induced fatigue, and (4) at defects such as section loss due to corrosion or flaws.

The fracture is assumed to penetrate the tension flange. Although the fracture will likely penetrate only partially through the depth of the web, it is assumed to extend more or less vertically through the full height of the web. The compression flange is assumed to remain intact. For noncomposite bridges and near midspan fracture (which is the most probable location for simple span bridges), the compression flange is assumed to resist the relatively low shear and high compression forces at the fracture location. For composite bridges the deck is assumed to participate in resisting the shear and compression forces at the fracture location. For other fracture locations the remaining portion of the web above the fracture is assumed to resist the higher shear forces at these locations.

EXPECTED FRACTURE LOCATIONS

Although these guidelines and procedures are independent of the fracture location along the girder, the decision to provide an alternate load path in an existing or rehabilitated bridge could depend on the location and extent of fatigue cracking along the girder as revealed by inspection. The following is a guide to locations where fatigue cracks are most likely to occur (*4, 8, 60, 61, 62*).

1. *Groove-welds:*
   a. Flange groove welds—relatively older structures with groove welds made prior to adequate nondestructive inspection techniques.
   b. Web groove welds—relatively older structures with groove welds made prior to adequate nondestructive inspection techniques.
   c. Groove welds in longitudinal stiffeners—longitudinal stiffeners on girder webs are structural components and the welds should be treated as structural welds. Older bridges seldom had these components inspected.
   d. Groove welds between longitudinal stiffeners and intersecting members—often lack of fusion exists in the transverse weld connection, particularly when no cope exists at the web.
2. *Ends of welded cover plates on tension flanges:*
   a. At toe weld or in throat of weld at midwidth of flange on cover plates with end welds.
   b. End of longitudinal welds on cover plates without end welds.
3. *Ends of various reinforcement or attachment plates welded on girder flange or web:*
   a. Welded splices between adjacent parts—lateral gusset plates (equivalent to cover plates).
   b. Repairs of flanges or webs using doubler plates (equivalent to cover plates if more than 8 in. long).
   c. Repairs of webs using fish plates (equivalent to cover plates if more than 8 in. long).
   d. Attachments for signs, railings, pipe supports, etc., when the attachment plate is parallel to the girders (equivalent to cover plates if more than 8 in. long).
   e. Welded attachment plates perpendicular to the girder (higher fatigue strength than in a, c, and d).
4. *Diaphragm connections:*
   a. Ends of welded diaphragm connection plates on girder webs where the connection plate is not welded to the flange (displacement induced fatigue cracking)—cracks may occur at the gap (cope) either horizontally along the web-to flange weld, or at the top of the web-to-connection plate weld. Cracks can occur at the top or bottom of the connection plate when no positive attachment is made to the flange.
   b. Ends of riveted diaphragm connection angles on girder webs where the angles are not connected to the flange (displacement induced fatigue cracking)—cracks may occur in the girder web either horizontally along the flange, or vertically along the angles, and also in the first (highest or lowest) line of rivets. Web cracks are most likely when connection angles do not overlap the flange angles. Also rivet or bolt heads may crack from prying.
5. *End connections of floor beams or diaphragms:*
   a. Copes and blocked flanges at ends of floor beams: cracks may occur at the reentrant angle of the cope or the blocked flange, particularly when the reentrant angle is flame cut, with reentrant notches.
   b. Connection plates and angles may have cracks similar to those described above for diaphragm connections.

6. *Floor beam brackets (outrigger bracket):*
   a. Bracket connections to girder webs—similar to cracks in diaphragm connections.
   b. Tie plates connected between top flanges of outrigger brackets and the floor beams (displacement induced fatigue cracking)—cracks may develop from edge of rivet holes of these plates if connected to top flange of longitudinal girder. Relative movement may also result in web cracks in the floor beams and webs of brackets.
7. *Top and bottom lateral (wind) bracing connections to girders:*
   a. Gusset plates welded to girder web or flange—when the gusset plate is attached to the web but not to the diaphragm connection plate, cracks may occur in the web gap, at the toe of the weld. These gusset plates are welded attachments and also force-transmitting connection plates. Forces arise due to the displacement induced live load forces in the lateral bracing produced by elongation and shortening of the girder tension and compression flanges.
   b. Gusset plate to diaphragm connection plate welds—special attention should be given to these types of details. Displacement induced live load forces are produced in the diaphragms because of differential girder deflection. If the welds joining the gusset plates to the web and the welds joining the gussets to the diaphragm connection plate intersect, high restraining forces develop in this region. The probability of defects in this region increases the probability of fatigue crack growth with cracks entering the girder.
8. *Transverse stiffeners:*
   a. Intermediate transverse web stiffeners are not connection plates for diaphragms or floor beams. These are transverse attachments usually having adequate fatigue strength.
   b. At ends of cut-short intermediate stiffeners because of handling, transportation or web plate vibration. At fitted stiffeners cracks can be revealed by cracking of the paint film.
9. *Tack welds:*
   a. At tack welds used for attaching bridge components during construction and erection—these tack welds are often sources of fatigue cracking.
   b. Tack welds often occur between gussets and main members, between bearing plates and beam flanges, between floor beam top flanges and outrigger bracket tie plates, between riveted and bolted connection angles and webs.
10. *Plug welds:*
   a. At any plug weld—a plug weld may have been made in fabrication to correct a misplaced drilled hole, or in the field during repairs or retrofit. Plug welds are often hidden below a paint film. Fatigue cracking is revealed by cracking of the paint.

BEHAVIOR BEFORE FRACTURE

The usual application of the AASHTO provisions for the design and rating of two-girder steel bridges assumes that only the two-girders support all vertical loads (*1, 29*). Thus, the two girders are considered in the oversimplified AASHTO model of the bridge to be the only load paths available for transmitting all vertical dead, live, and impact loads through the deck, stringers, and floor beams to the bearings. Secondary members, such as lateral (wind) bracing and diaphragms, are not assumed to participate in transmitting vertical loads. Although secondary bracing members do, in fact, share part of the vertical loads by developing displacement induced forces, which are not necessarily small, they are only designed to resist wind loads and to maintain some rigidity of the structure particularly during construction.

The assumptions of the simplified model are reasonably good for a right (no skew) bridge having a symmetrical cross section and loaded symmetrically about the longitudinal axis. In this case, the two girders are equally loaded and deflect equally. Stresses induced in the diaphragm members are minimal. Stresses are developed in the lateral bracing by elongation and shortening of the tension and compression flanges. These are not large. However, for skewed bridges, unsymmetrical cross sections and particularly for unsymmetrical loading, the girders are not loaded equally and do not deflect equally. The bracing members are now stressed mainly because of the differential deflection of the girders. Additional stresses arise from torsion or rotation about the bridge longitudinal axis. Large stresses are developed at the connections of offset diaphragms to the girders.

The AASHTO model greatly simplifies the design and rating of two-girder bridges and provides a conservative solution, but only for static loading conditions. It can be shown that, for ductile structures subjected to static loads, a safe (sometimes overly safe) design or rating, in terms of load capacity, is obtained by ignoring the bracing members.

This is not the case, however, for dynamic or cyclic loading even if static loads are adjusted by impact factors. The AASHTO model may not provide a safe design or rating in terms of serviceability. Fatigue cracking, for example, is a function of the real live load stress range at a detail, not by the artificial stress range calculated by the AASHTO model. For main members, such as the two girders, the real stress range may or may not be greater than the calculated stress range. To make matters worse, stress ranges are not even calculated for details associated with bracing members simply because these members are not included in the AASHTO model. It is not unusual to measure strains in bracing members and connections of actual bridges which indicate displacement induced stresses near yield stress levels. At such details, fatigue cracking occurs very early in the life of the bridge. Such cracks, if undetected, can quickly propagate into the main girders, setting up the strong potential for a near full depth fracture of the girder.

BEHAVIOR AFTER FRACTURE

After-fracture redundancy of welded steel two-girder bridges was studied extensively and is reported in Ref. 28. The investigation was sponsored by the Pennsylvania Department of Transportation (PADOT) and the Federal Highway Administration (FHWA). The investigation employed elastic-plastic finite element computer analyses of the complete three-dimensional bridge including girders, deck, stringers, floor beams, diaphragms, and bottom lateral bracing. The analysis is coupled with upper-bound and lower-bound plasticity concepts and computer graphics to predict the after-fracture behavior and maximum strength of the simple and continuous span bridges. Descriptions of the spans studied and the results obtained are briefly discussed in this section. Further details of the study are reported in Ref. 28.

The spans were selected from the Betzwood Bridge, which is shown in Figure 8 (*58*). Continuous spans 2 and 3 supporting the southbound lanes were selected for the two-span continuous bridge study. Span 8 was selected for the simple-span bridge study. The cross section shown in the figure is the same for both the simple and continuous spans. The cross section of the southbound bridge at the same locations is also shown in Figure 6. The Betzwood Bridge is designed to the 1961, 8th Edition of the AASHTO *Standard Specifications for Highway Bridges,* for HS-20 loading.

In the study, all load carrying welded and bolted connections are assumed to develop the member strengths. The deck is assumed to be composite only over the positive moment regions as defined by the girder bending moments throughout the elastic-plastic range of behavior. The shear connectors are assumed to develop the deck or girder strength. The bottom lateral X-type bracing is at the level of, and connected to, the girder bottom flanges. The K-type cross-bracing diaphragms are spaced at 17 ft 10 in. in the simple span and at about 17 ft 5 in. in the continuous span. Note in Figure 6 that the K-type bracing diagonals do not come together midway between girders, but are separated by about 5 ft.

The simple span and continuous girders are shown in Figure 9. For the continuous girder a bolted field splice occurs in only one span and is located 23 ft 11 in. from the interior bearing. The splice is designed for 75 percent of the strength of the girder at the splice location.

The after-fracture analysis of the three-dimensional simple-span bridge assumes a near full depth fracture at midspan of one of the girders. Only the top flange is assumed to remain intact. For the two-span continuous bridge a near full depth fracture is also assumed at midspan of one of the girders, but is located in the span containing the splice.

In addition to the dead load, two lanes of AASHTO HS truck loading with 30 percent impact are used. The trucks are offset laterally towards the fractured girder and located in the same longitudinal positions that would be used in normal design to compute the maximum bending moment at midspan of the girder.

Because nonlinear elastic-plastic behavior is expected to occur, an incremental loading procedure is used. The dead load is gradually applied until the full service dead load is obtained. Then, the HS loading including 30 percent impact is gradually applied until either the maximum capacity of the bridge or very large deflection is attained.

Figure 10 shows the resulting load-deflection behavior of the two bridges. Midspan deflection of each bridge refers to the vertical deflection of the fractured girder at the fracture location.

At full application of service dead load, *DL,* on the simple span, the deflection, Δ, is about $2\frac{1}{2}$ in., which is three times the deflection of the unfractured bridge, yet still relatively small. Damage is confined primarily to yielding and buckling of the

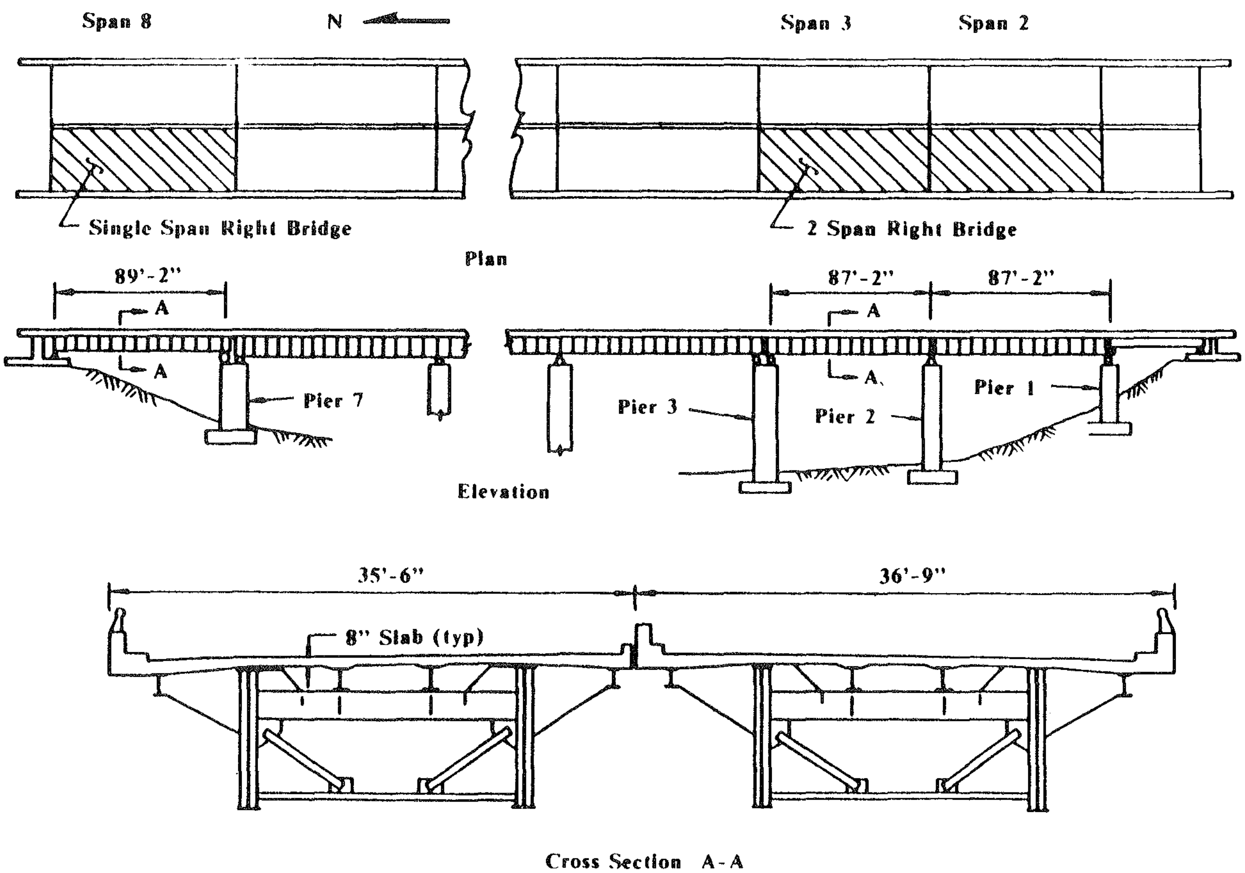
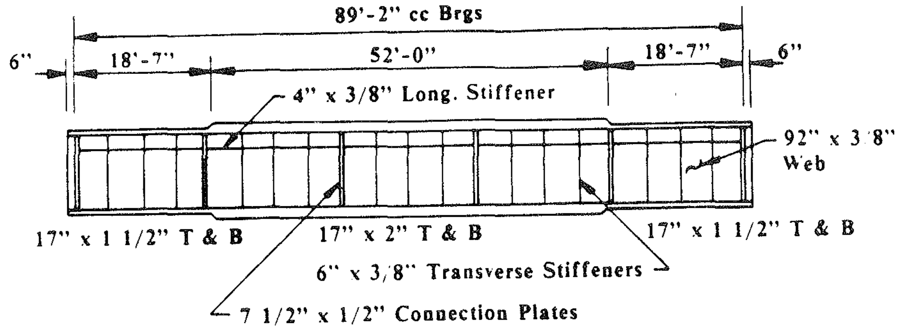
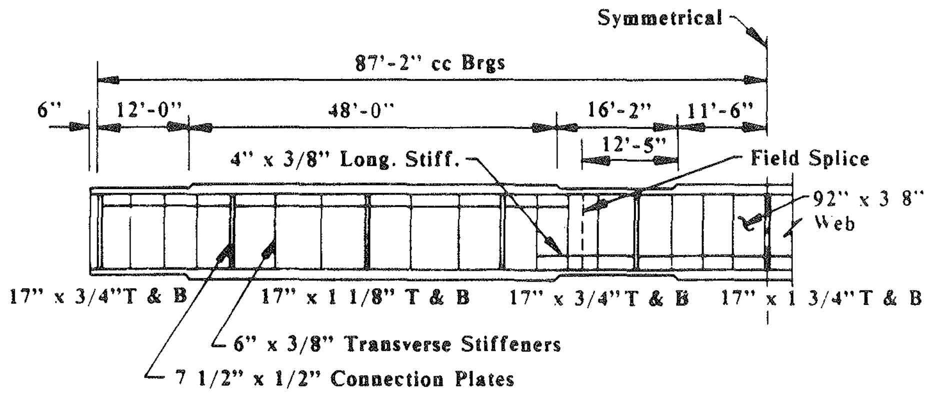


Figure 8. Plan, elevation, and cross section of the Betzwood Bridge (Fig. 6 and Refs. 58 and 28).



(a) Span 8 - Simple Span



(b) Spans 2 & 3 - Continuous

Figure 9. Simple and continuous span bridge girders.

C7 × 14.75 bottom horizontals of some of the K-type bracing (Fig. 6). At service dead load plus two lanes of HS-20 truck loading, including 30 percent impact, additional damage has occurred and the deflection is now approximately 6 in. For an 89-ft span this is still a fairly small deflection. Although a 6-in. deflection would likely be noticeable (noticeable deflection is desired so that the fracture is quickly detected), trucks should have no difficulty crossing the bridge at normal highway speeds. Damage at this point has spread from the K-type bracing to yielding of the diaphragm connection plates and transverse stiffeners on the fractured girder. Buckling of two bottom lateral diagonal members has also occurred. However, considerable additional live load capacity still exists as Figure 10 indicates. The study shows that the maximum bridge capacity is in excess of HS-60. The deflection, however, is more than 30 in., which is probably too large for safe crossing by trucks at normal highway speeds. From a serviceability point of view the maximum capacity is probably closer to HS-30, which corresponds to a deflection of about 10 in. or a deflection-to-span ratio of about 1/100.

At full application of service dead load, *DL*, on the two-span bridge the deflection, Δ, is about 1.5 in., or about twice the deflection of the unfractured bridge. Damage is very slight and confined to buckling of one cross-bracing horizontal near the fracture. At service dead load plus two lanes of HS-20 loading, including 30 percent impact, the additional damage is still small and the deflection is about 2.5 in. At HS-40 loading major damage occurs, as Figure 10 indicates. This damage involves yielding and lateral buckling of the bottom compression flange of the fractured girder near the interior support. The midspan fracture requires the fractured girder to cantilever from the interior support out to midspan. Therefore, the negative bending moments in the fractured girder are much larger than anticipated in the original design. The increased compression stress in the bottom flange of the fractured girder near the interior support eventually leads to lateral flange buckling. The girder, in effect, transforms itself into a fractured simple span by buckling of the compression flange at the interior support.

The girder splice, originally located in the inflection region is now also subjected to considerable negative bending moment. Because the splice is designed for 75 percent of the girder strength it carries the increased moment. A smaller splice may not.

For the two-span study bridge, the deflection at HS-40 suddenly increases about 16 in., as shown in Figure 10, because of compression flange buckling. Analysis was terminated at a deflection of about 27 in. The maximum capacity of the two-span bridge is in excess of HS-50.

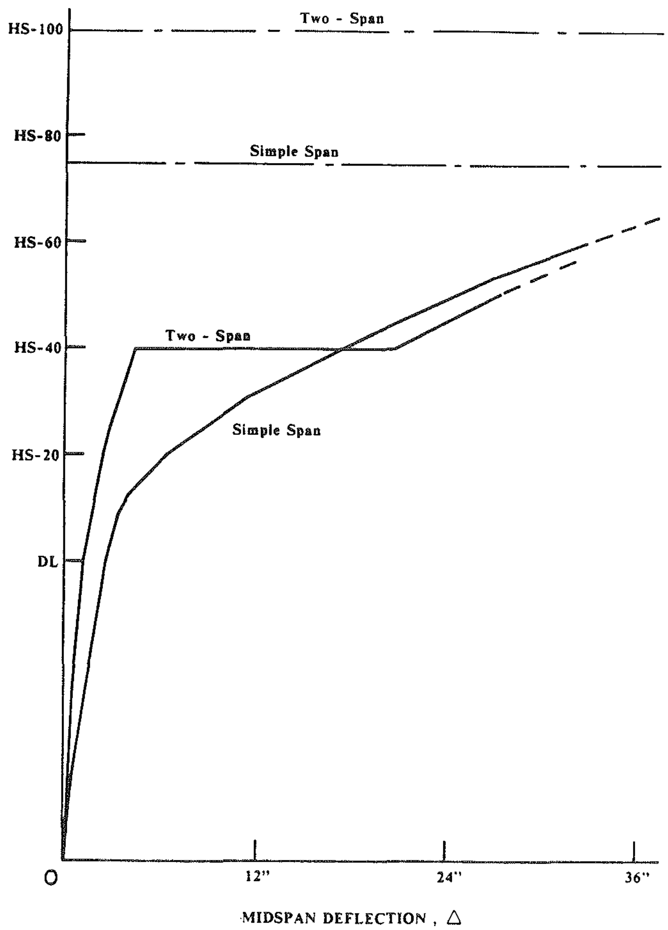


Figure 10. Comparison of load vs midspan deflection for simple span and two-span continuous bridges.

Comparing the behavior of the two bridges, it is evident that for equal deflections, after compression flange buckling, the two-span bridge has less live load capacity, therefore less after-fracture redundancy than the simple span bridge. Although this seems to contradict the usual impression that continuous spans should automatically be more redundant than simple spans, this is easily explained. A review of Figure 9 indicates that the average cross section of the continuous girder is smaller than that of the simple span girder. Thus, even though the continuous girder has slightly smaller span lengths, after compression flange buckling the fractured girder is converted to a fractured simple span with less load carrying capacity than the simple span girder.

There is usually agreement among bridge engineers that continuous spans offer more redundancy than simple spans. The results of this study, although for only one two-girder bridge, contradicted this concept. This is not unexpected, because bridge engineers naturally extrapolate their design and field experience with *unfractured* bridges to the *fractured* case. It should be obvious that such an extrapolation is risky.

Bridge engineers need to begin developing redundancy design and rating experience with fractured bridges in order to more rationally access and classify the degree of redundancy not only in simple span and continuous two-girder bridges but in other bridge types as well. Relative redundancy between bridge types cannot be based on the simplified design concepts commonly used for normal unfractured bridges.

Figures 11 and 12 show the computer graphics generated, finite element models of the two fractured study bridges in their deflected states. Each bridge is carrying full service dead load plus two lanes of HS-20 trucks including 30 percent impact. For the simple span and continuous study bridges the deflections at the fracture locations are 6 in. and 2.5 in. respectively. Vertical deflections in the two figures are plotted to a scale of 10 and 30 times actual.

POTENTIAL ALTERNATE LOAD PATHS

The redundancy of the simple span study bridge in Ref. 28 depends mainly on the elastic-plastic strength of its bracing system. Although the bracing system is not designed or intended to provide after-fracture redundancy, the study shows that it did function as an alternate load path and considerable redundancy was achieved even though considerable inelastic behavior is needed to achieve this redundancy.

This suggests that it may be possible to define an optimum configuration and location of the bracing system members and components which would provide redundancy within the elastic range of behavior. The majority of the remainder of this report is devoted to guidelines and procedures applicable to the design and/or rating analysis of bracing systems that are specifically configured and located to provide after-fracture redundancy in new or existing bridges within the elastic range. These bracing systems are, hereafter, referred to as redundant bracing systems.

Guidelines applicable to redundant bracing systems are provided in the remainder of this chapter. Appendix C develops the theoretical basis for a simplified conservative approach to be used either for designing a new or retrofit redundant bracing system for new or existing two-girder bridges, or for computing a redundancy rating for a two-girder bridge with a properly configured and located redundant bracing system. Appendix C also summarizes these design and rating equations and provides a number of worked examples showing the application of redundancy design and rating using the allowable stress and load factor and methods. Worked examples in Appendix C also show the application of serviceability criteria in redundancy design and rating.

Chapter Four provides guidelines for retrofitting existing bridges to the requirements of a redundant bracing system. That chapter also provides guidelines for designing alternates to a redundant bracing system and guidelines for continuous two-girder and through-girder bridges.

Chapter Five provides guidelines for redundancy design and rating of simple-span two-girder bridges by finite element modeling and analysis of the entire three-dimensional composite or noncomposite bridge, independently of Appendix C. Use of the guidelines in Chapter Five assumes a properly configured and located bracing system.

ALTERNATE LOAD PATH SURVIVABILITY—FAIL SAFE

The alternate load path is relied on to provide redundancy of a two-girder steel highway bridge in the event of fracture of one of the girders sometime during the life of the bridge. The redundant alternate load path therefore performs a "fail-safe" function. If the alternate load path is to perform this fail-safe function, it must survive the girder fracture. This requires fre-

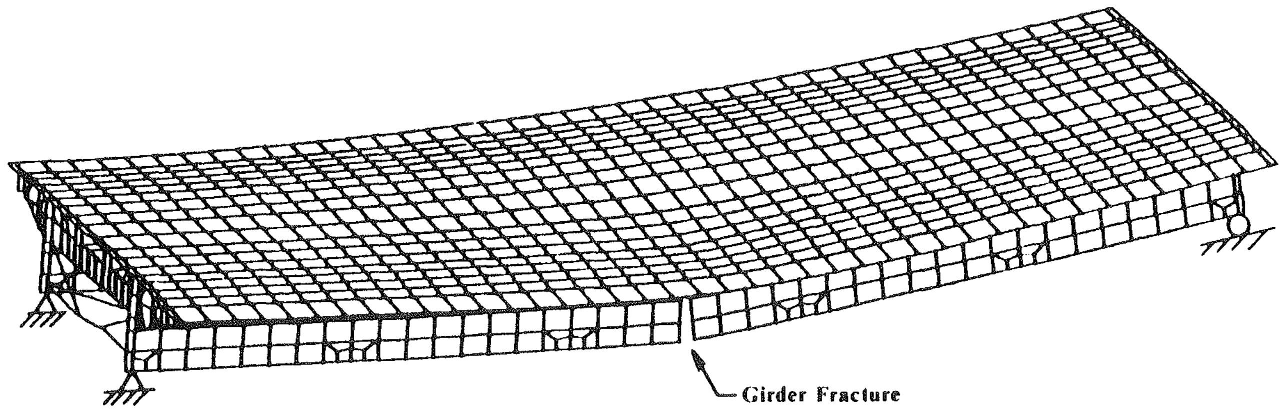


Figure 11. Computer graphics display of simple span right bridge under full service dead load plus HS-20 truck loading (vertical deflections are amplified 10 times actual).

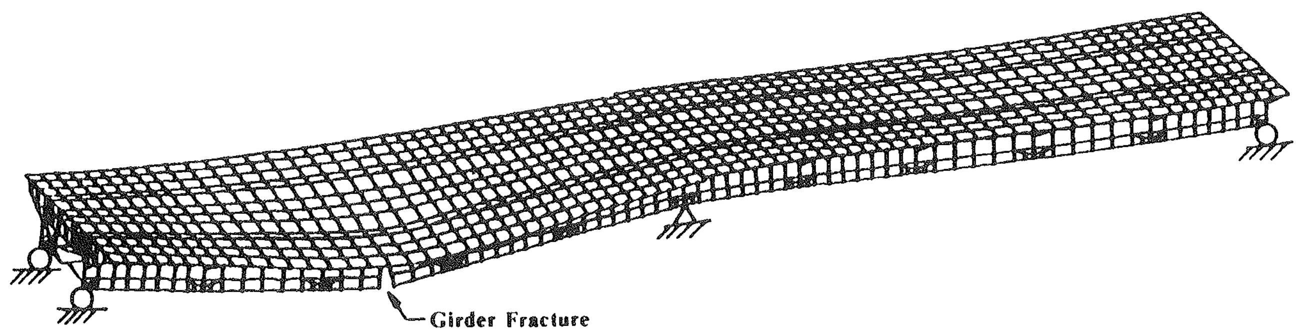


Figure 12. Computer graphics display of two-span continuous bridge under full service deadload plus HS-20 truck loading (vertical deflections are amplified 30 times actual).

quent inspections to reveal corrosion damage and fatigue cracking of the alternate load path itself. To maintain the fail-safe function repairs must be made quickly when needed.

Survivability of the alternate load path depends to a large extent on whether it is part of the primary or secondary structural system prior to fracture. Members and connections of the primary system are stressed by the dead, live, and impact loads. The decks, stringers, floor beams, girders and all their load carrying connections comprise the primary system. The secondary system consists of members and connections that are stressed because of deflection of the primary system. The bracing system, consisting of the top and bottom laterals and the diaphragms, comprises the secondary system under the applied vertical loads.

If the alternate load path is part of the primary structural system prior to girder fracture, it is also susceptible to fatigue damage resulting from the cyclic live and impact loads. For example, an alternate load path employing steel tension cables, rods, or shapes attached to the girder flanges may be part of the primary system before fracture. Whether or not the design assumes that the redundant system participates with the girder in supporting the live and impact loads it may be subjected to stress ranges due to these loads. Such an alternate load path may not be fail safe. A worst case scenario arises if the cables, rods, or shapes and their connections corrode or fracture prior to girder fracture. In this case the bridge is nonredundant even though a redundant alternate load path is provided. As another example, an alternate load path can be provided by bolting or welding another tension flange or girder to the existing girder. Under live loads this flange or girder is part of the primary system and is subjected to stress range. Every detail connection is therefore fatigue critical.

Alternatively, a redundant bracing system consisting of top and bottom lateral (wind) bracing and diaphragms is part of the secondary structural system prior to fracture. These members and connections are stressed primarily by relative deflection of the girders and longitudinal extension and shortening of the girder tension and compression flanges. The redundant bracing system may also be susceptible to fatigue, mainly due to displacement induced stresses. If connections and details are designed to eliminate displacement induced stresses, if vibration of long bracing members is minimized, and if corrosion damage is repaired on a timely basis, the bracing system is, to a high degree, fail safe. After a girder fractures, the bracing members and connections become part of the primary structural system and provide the alternate load path for carrying the after-fracture dead and live loads.

This report concentrates on design and rating guidelines and procedures for the use of a redundant bracing system to provide a reasonable fail safe alternate load path. To provide redundancy to deck-type two-girder bridges, the redundant bracing system must be specifically configured and located so that it will provide the necessary strength and stiffness to support the dead and live loads following fracture of a girder. A bracing system conforming to these requirements is termed a redundant bracing system in this report. The remainder of this chapter discusses the AASHTO bracing system, then develops the specific requirements of a redundant bracing system. The chapter concludes by providing guidelines for the design and rating of a redundant bracing system.

AASHTO BRACING SYSTEM

The bracing system normally provided for deck-type two-girder bridges usually consists of at least one and sometimes all of the following three components and their connections:

1. *Bottom lateral bracing*—a horizontal truss located at or near the bottom flanges of the girders (Figs. 1, 5, and 6) or near the bottom of the floor beams (Figs. 3 and 7).
2. *Top lateral bracing*—A horizontal truss located at or near the top flanges of the girders (Fig. 1) or at the floor beam level (Fig. 2). In bridges composite with the two girders the deck may also serve as the top lateral bracing.
3. *Diaphragms*—partial and full depth interior and end diaphragms such as cross bracing (Figs. 1 and 6), cross frames (Figs. 3 and 7) and cross trusses (Fig. 5).

Top and bottom laterals are designed for wind and meet AASHTO requirements for minimum member size and stiffness. Diaphragms stiffen the cross section and transmit wind to the opposite girder. The three components of a typical AASHTO bracing system are shown schematically in Figure 13 for a simple span bridge.

REDUNDANT BRACING SYSTEM REQUIREMENTS

For the redundant bracing system to be relied on to provide redundancy and to function as a fail safe alternate load path after fracture of one of the girders, all of the above three bracing system components and their connections must be present and conform to the following requirements

1. *Top and bottom lateral bracing:*
   a. X-type top and bottom lateral bracing must be present over the full span length and be located at or immediately above or below the girder flanges.
   b. The top and bottom lateral bracing is to be configured as horizontal trusses whose diagonal members resist the applied loads primarily by axial tension and/or compression.
   c. To accommodate fracture at any section along the girder, identical diagonal members are to be used for the full length of the bottom lateral bracing. Similarly, identical diagonal members are used for the full length of the top lateral bracing.
   d. The diagonal members are to be designed for the max-

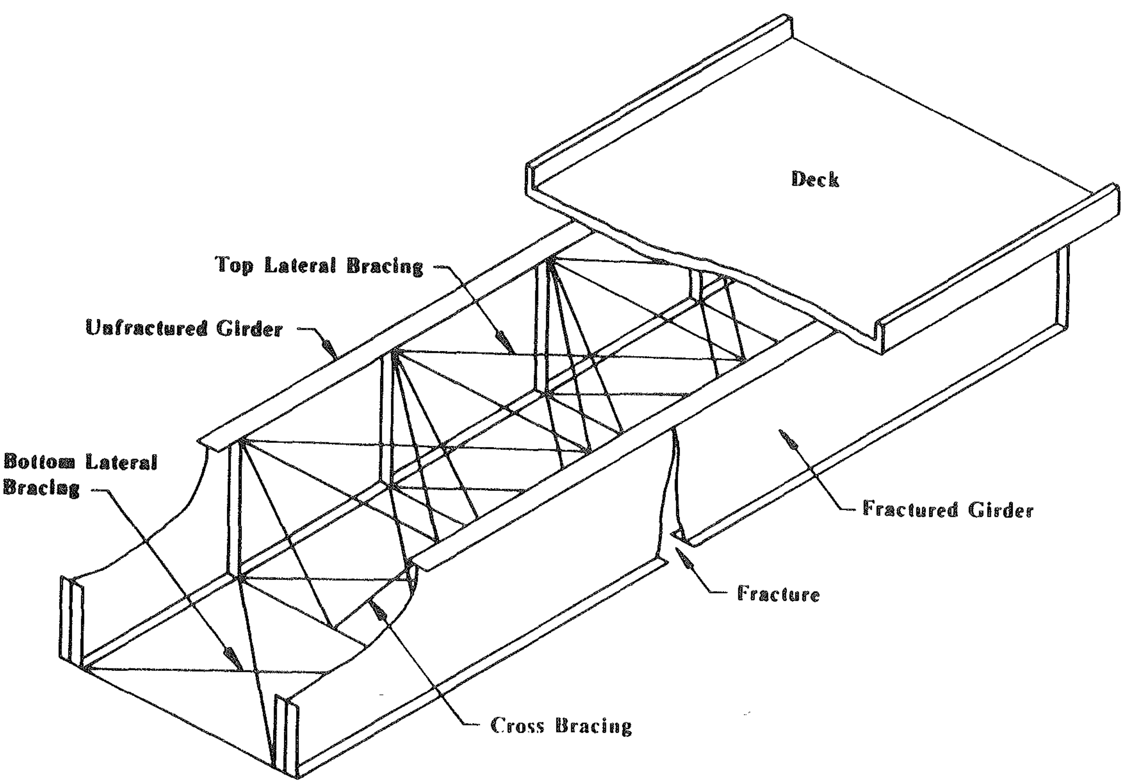


Figure 13. Schematic showing typical components of a redundant bracing system for a simple span bridge.

on major highways or roads with high average daily truck traffic (ADTT) and where a significant length of time may lapse between fracture and fracture detection (more than a month, say) it is recommended that the operating rating provisions of Ref. 29 be used for both redundant design and rating.

Allowable Stresses (Redundancy Rating). For two-girder bridges on highways or roads with low ADTT or where a relatively short time lapse between fracture and fracture detection is likely (less than a month, say) because of pronounced after-fracture deck deflection under dead loads, it is recommended that the following redundancy rating provisions be used for both redundant design and rating. These provisions are based on the AASHTO operating rating provisions (*29*) but with increased allowable stresses and decreased factors of safety.

Referring to Table 5.4.2B of Ref. 29, the following allowable stresses for redundancy design and rating are proposed:

- Axial tension in members without holes: 0.85 F_y or 0.65 F_u.
- Axial tension in members with holes and tension in extreme fibers in members subjected to bending: 0.85 F_y on the gross section, or 0.74 F_u on the net section, whichever is smaller.
- Axial compression in splice material: 0.85 F_u on the gross section.
- Compression in extreme fibers of bending members where the compression flange is:
  (A) Supported laterally its full length: 0.85 F_y on the gross section.
  (B) Partially supported or unsupported: change 1.37 to 1.55.
- Compression in concentrically loaded columns: F.S. = 1.40.
- Shear in girder webs 0.50 F_y on the gross section.

The above provisions assume the following conditions: (1) the reduced probability that the bridge is subjected to the AASHTO design live loads during the comparatively short time interval between fracture and fracture detection; and (2) the excess load capacity beyond first yield or the stability limit inherent in the conservative procedures developed in Appendixes C and D or in the guidelines developed in Chapter Five for computer modeling and analysis. This is a result of requiring all similar members or components of the redundant bracing system to be identical throughout the span, requiring their design and rating to be based on a worst case fracture, neglecting the strength of other components such as the deck, and establishing first yield as the limit state in the allowable stress method. In the load factor method most members on the redundant alternate load path are at or below yield under the redundant dead and live loads.

The basis used to determine the increased allowable stresses is that the allowable stress for tension members should not exceed 0.85 F_y.

Load Factor Model

It is recommended that redundancy design and rating be performed using the AASHTO load factor (strength) method defined in Ref. 29. It is recommended that load factors corresponding to either the operating or redundancy rating level be used for redundancy design and rating by the conservative methods of Appendixes C and D or by the finite element modeling and analysis described in Chapter Five and for the reasons discussed above for allowable stresses.

Load Factors (Operating Rating). For two-girder bridges on major highways or roads with high ADTT and where a significant length of time may lapse between fracture and fracture detection (more than a month, say), it is recommended that the operating rating provisions of Ref. 29 be used for both redundant design and rating.

Load Factors (Redundancy Rating). For two-girder bridges on highways or roads with low ADTT or where a relatively short time lapses between fracture and fracture detection (less than a month, say), it is recommended that the following redundancy rating provisions be used for both redundant design and rating. These provisions are based on the AASHTO operating provisions (*29*) but with a decreased load factor.

Referring to Ref. 29, the following load factor for redundancy design and rating is proposed: Maximum strength $\geq$ 1.18 (D + RF (L + I), where RF = redundancy rating factor.

The selection of the 1.18 load factor is based on calibrating the allowable stress and load factor methods for tension members having a factor of safety of 1/0.85 = 1.18, and assuming a unit dead to live load ratio.

Serviceability

Serviceability refers to the ability of vehicles to safely cross the deflected bridge at normal highway speeds following a girder fracture. Deflection criteria are necessary because a vehicle approaching the bridge at night, for example, may not observe any unusual deflection of the bridge deck.

Some adverse deflection of the deck is expected after girder fracture. It should not be so large that a heavy vehicle attempting to cross the bridge would lose control. On the other hand, it should not be so small that, even in daylight, evidence of fracture would not be provided.

After-fracture deflection of the deck occurs in two stages: (1) The first is the deflection under dead load only. This deflection should be large enough to be detected by a vehicle approaching the bridge during daylight hours. (2) The second is the additional deflection produced by the rating vehicle crossing the bridge. The total deflection under dead, live, and impact loads should not exceed the value which would enable an unsuspecting vehicle to safely cross the bridge.

The after-fracture deflection of the fractured girder is proportional to the level of stress permitted in the bottom lateral diagonal members at the fracture location. For example, for midspan fracture, the deflection is proportional to the level of tension stress in the midspan diagonals as shown in Chapter C2 in Appendix C. An increase in permitted stress results in an increase in deflection. The dead load and total load after-fracture deflections are therefore controlled by the stress in the bottom lateral diagonals. Appendix C also provides worked examples showing the application of serviceability criteria.

The redundant bracing system is very effective in controlling deflections. For the low strength structural steels permitted by AASHTO (Table 10.2A, Ref. 1) the midspan deflection-to-span length ratio, Δ/L, would likely be low enough to permit a heavy vehicle to safely cross the bridge, but may not be large enough for fracture detection under dead load alone.

Selection of an appropriate value of Δ/L for the total load condition is based on the following two serviceability criteria: (1) the maximum longitudinal deck slope, $\theta_{\text{long.}}$, for a worst case fracture location; and (2) the maximum transverse deck slope, $\theta_{tran.}$, which results from a midspan fracture. Both of these situations are discussed in Chapter C5 of Appendix C.

The selection of the appropriate total load Δ/L also depends on the particular bridge geometry and on the judgment of the bridge engineer as to what constitutes safe versus unsafe deck slopes.

For an example, consider a simple span bridge with 100-ft span, 20-ft girder spacing, $n = 7$ and a $\Delta/L = 1/150$. Using the equations in Appendix C, the midspan deflection for midspan fracture is 8 in. The maximum transverse deck slope is $\theta_{tran.} = 0.67/20 = 0.03$ rad $= 1.9$ deg. The longitudinal deck slope over the fractured girder is $0.67/50 = 0.01$ rad $= 0.8$ deg. An estimate of the maximum longitudinal deck slope over the fractured girder for worst case fracture location (Eq. C5.12, Appendix C) is $\theta_{long.} = 0.02$ rad $= 1.3$ deg.

On the other hand, for a 100-ft span, $n = 3$, $\Delta/L = 1/150$, but with a 50-ft girder spacing (similar to the Schoharie Creek Bridge, Ref. 60) $\theta_{tran.} = 0.01$ rad $= 0.8$ deg and $\theta_{long.} = 0.02$ rad $= 1.2$ deg.

The bridge engineer, in the foregoing examples, has to decide whether the calculated longitudinal and transverse slopes are acceptable or excessive, depending on factors such as vehicle speed, effect of a change in longitudinal slope near the fracture, effect of transverse slope on the controllability of the vehicle, and other factors. If $\Delta/L = 1/150$ is acceptable, the required stress, f_B, in the midspan diagonals to produce this deflection can be computed from the equations presented in Appendix C, as follows.

In the first example above, assuming $\alpha = 1.6$, $d = 96$ in., and $E = 30{,}000$ ksi, then $f_B = 87.5$ ksi (Eq. C2.25, Appendix C). For the second example, using the same values, $f_B = 37.5$ ksi. These stresses should not exceed the allowable stresses as proposed in this chapter.

The next question to be addressed is the deflection under dead load conditions alone. For example, if the dead load is two-thirds of the total load in the above example, the dead load deflection-to-span length ratio $(\Delta/L)_D = 1/225$. This corresponds to a midspan deflection of 5.3 in. for midspan fracture. Again, the bridge engineer must judge if this is sufficient deflection to provide warning of fracture.

Examination of Eq. C2.25 indicates that for a given bridge with a given Δ/L ratio, f_B is primarily dependent on the number of panels n, which also determines α. The smallest number of panels, n, will produce the least stress f_B, to achieve a given Δ/L ratio.

Corrosion, Deterioration, and Collision Damage

Redundancy design and rating in all situations involving an existing two-girder bridge should take note of the present state of the bridge. A thorough inspection is needed to reveal the extent of corrosion of members and connections, missing members rivets and bolts, cracked welds, and damage to members from collision impact. A redundancy rating must account for the corrosion and deterioration of the members and connections, especially of the redundant bracing system. Alternatively, the redundancy rating can be made assuming that the members and connections are repaired to their original specifications.

Connections

As previously discussed for redundant bracing system requirements, all load carrying connections must develop the full strength of the connected members. A redundancy rating of an existing bracing system following the procedures of Appendix C and Chapters Three, Four, and Five assumes that all connections either meet this requirement or are repaired and/or retrofitted to meet this requirement.

It is also assumed that new or retrofitted redundant bracing systems for new or existing two-girder bridges are provided with connections that develop the full strength of the connected members.

Care must be exercised in repairing or retrofitting existing connections and in the design of new or retrofit connections not to provide a detail that will deteriorate the fatigue strength either of the redundant bracing system and connections or of the girders. Fatigue cracks in existing details should be repaired in accordance with current practice (*4, 8, 30*).

Fracture at a Connection

Because the redundant bracing system is not part of the primary vertical load carrying system and providing that displacement induced fatigue details are avoided, the probability of fracture of the redundant bracing system and connections prior to girder fracture is low. Girder fracture can occur, however, at the location of a connection of the redundant bracing system members to the girder. It is unlikely that girder fracture will occur simultaneously on both sides of a connection resulting in fracture in two adjacent panels. It is assumed that girder fracture involves only one panel, and is associated with a welded, bolted, or riveted detail in the panel either adjacent to a connection or between two connections.

Shear Capacity at a Fracture

The guidelines, procedures, and equations developed in this report have assumed that the after-fracture shear can be resisted by the girder at the fracture location. For midspan fracture of a simple span bridge the shear force is relatively small and this assumption is reasonably good. For fracture towards the end of a simple span the shear force is relatively large. In these situations the question arises whether or not the girder can resist this shear force at the fracture location.

This question has not been positively addressed in this research and is a question that should be explored in further research into redundant bracing systems. For purposes of this report, it is assumed that the girder can resist the shear at the fracture location for the following reasons.

For a redundant bracing system designed or retrofitted in accordance with these guidelines, a large elastic restraint is provided at the girder fracture. This elastic restraint limits the crack opening width. The fracture is restrained against running to the full depth of the web as assumed in this report. Until further research establishes the length of the fracture as a function of the elastic restraint, it is assumed that sufficient unfractured web remains to resist the shear force at the fracture location.

After-Fracture Girder Stresses

These guidelines provide that the fractured bridge carry less than the AASHTO design live load. The larger allowable stresses imum effects of the dead, live, and impact loads considering the worst case location of the fracture along the girder.

e. The chords of the top and bottom horizontal trusses (lateral bracing) are provided by the girder flanges.
f. All load carrying connections are to develop the full axial tension or compression strengths of the connected members.
g. All connections are to be designed to eliminate displacement induced fatigue stresses.
h. All diagonal members are to have sufficient flexural stiffness to minimize vibration induced stresses.

2. *Diaphragms:*
   a. Full depth interior and end diaphragms must be present. Diaphragms consisting of cross bracing, cross frames, or cross trusses must be used.
   b. The diaphragms are to be connected to the four flanges of the girders at the location of every connection of the top and bottom lateral bracing to the flanges.
   c. Cross bracing and cross-truss members are to resist the applied loads primarily in axial tension and/or compression.
   d. Cross frames are to resist the applied loads primarily by bending and are to be sufficiently rigid to minimize cross-section distortion.
   e. To accommodate fracture at any section along the girder and to provide maximum cross-section stiffness, all diaphragms in the span are to be identical.
   f. All diaphragms are to be designed for maximum effect of the dead, live, and impact loads considering the worst case location of the fracture along the girder.
   g. All load carrying connections are to develop the full strengths of the connected members.
   h. All connections are to be designed to eliminate displacement induced fatigue stresses.
   i. All diaphragm members are to have sufficient stiffness to minimize vibration induced stresses.

PSEUDO SPACE TRUSS CONCEPT

These requirements essentially describe a three-dimensional, load carrying, pseudo space truss. The two girders are the vertical side "trusses". The lateral bracing provides the top and bottom horizontal trusses. The diaphragms connect to the four chords to distribute torsional forces and to maintain rigidity of the cross section.

Such a structure is fully capable of supporting vertical loads, even with a near full depth girder fracture anywhere in the span. The following example will illustrate.

Figure 14 shows a load carrying prismatic space truss with a missing side panel. Side ADHE represents the fractured girder. Panel BCGF is the missing panel and represents the location of the fracture. For this illustration the fracture is modeled by removing the bottom chord GF and the diagonals BG and CF.

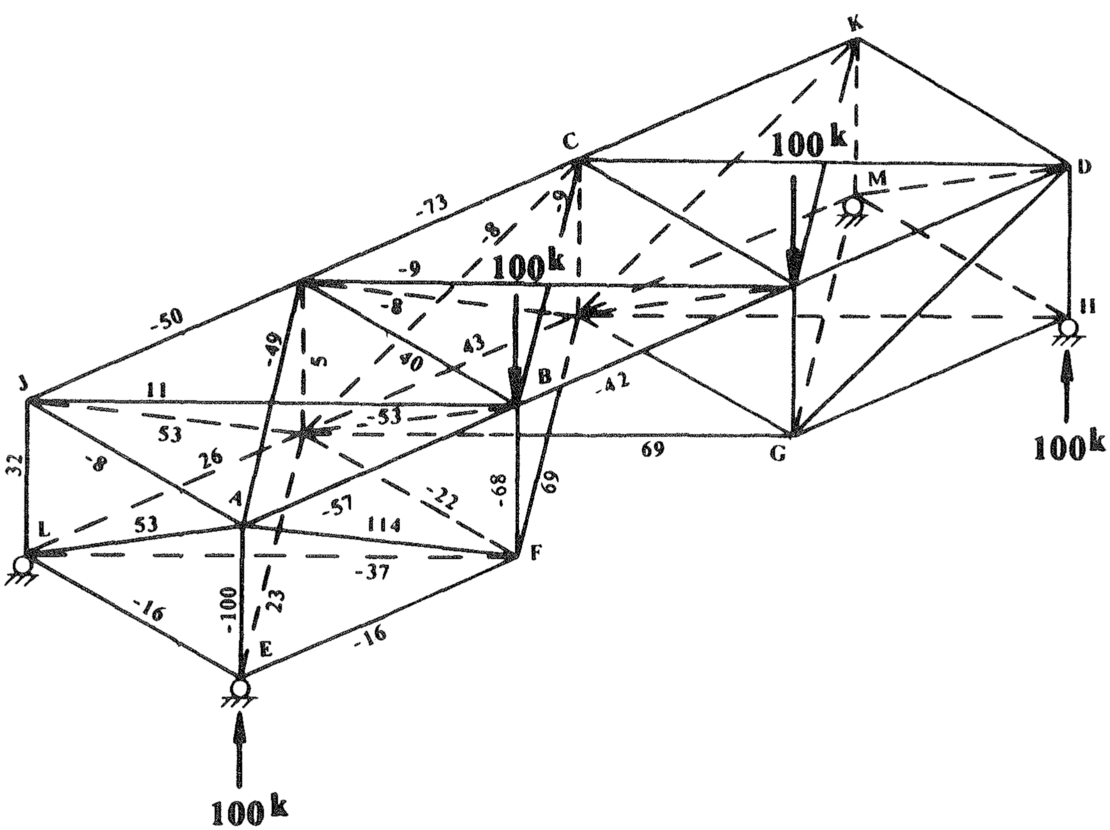


Figure 14. Load carrying space truss with missing side panel.

Side JKML represents the unfractured girder. The top and bottom lateral bracing are represented by JKDA and LMHE, respectively. Two end and two interior cross-bracing diaphragms are provided.

The space truss carries vertical loads of 100 (lb, kips, etc.) at B and C, which represent the dead, live, and impact loads. In the analysis, all members are pin ended and of equal cross-section area. Members EF and EL are each 4 units long. JL is 3 units high. The structure is stable and highly indeterminate. The axial forces are shown only in two bays due to symmetry. Axial tension is positive. All member forces are in the same units as the loads.

Note that corresponding diagonal and lateral members of the top and bottom lateral bracing trusses have opposite signs. Note also that because of symmetry, cross-bracing diagonals carry equal forces. Also because of symmetry the reactions at E and H are equal to the applied loads. Because the loads are applied in the plane of truss ADHE, there are no reactions at M and L.

GUIDELINES FOR REDUNDANT BRACING SYSTEMS

Design and Rating Situations

In the remainder of this chapter these guidelines are applicable to the design and rating of a two-girder bridge containing a redundant bracing system that meets all of the requirements stated above for redundant bracing systems. The application of these provisions will depend on whether the bridge is new or existing and whether a redundancy design or a redundancy rating is required.

The five situations typically encountered in redundant bracing system design and rating are shown in Table 2 together with the applicable chapter and appendix references. A description of the lettered items is as follows:

a. A new redundant bracing system is designed to provide redundancy for a new bridge using redundant design loads.
b. A new redundant bracing system is designed to provide redundancy for an existing bridge that does not contain a bracing system meeting redundant bracing system requirements. Redundant design loads are used. This case normally applies when the existing AASHTO bracing system cannot be practically or economically retrofitted to meet redundant bracing system requirements.
c. The existing AASHTO bracing system in an existing bridge does not meet redundant bracing system requirements, but may be practically and economically retrofitted to those requirements. Redundant design loads are used.
d. The existing AASHTO bracing system meets all the redundant bracing system requirements which enables a redundancy rating of the bridge to be determined using redundancy rating loads and methods.
e. The existing AASHTO bracing system does not meet all the redundant bracing system requirements, but may be practically and economically retrofitted so that a redundancy rating of the bridge can be determined using redundancy rating loads and methods. This case normally applies when, for example, the three required bracing system components are present but the connections do not develop the full strength of the connected members. Also when one component, such as the top or bottom lateral bracing is not present but can be practically and economically installed.

The remainder of this chapter provides guidelines applicable to the design and rating of deck-type simple-span two-girder bridges with redundant bracing systems. Guidelines are provided for selecting the redundant design and rating loads, and for selecting the allowable stresses and load factors for use in the Allowable Stress and Load Factor methods. Guidelines are also provided for selecting serviceability criteria.

These guidelines may also be used, where applicable, for the redundancy design of two-girder bridges using tension cables and rods as provided in Chapter Four and Appendix D.

Table 2. Design and rating situations.

Bridge	Design	Rating	Reference
New	a) New		Chaps. 3, 5 App. C
Existing	b) New	d) Existing	Chaps. 3, 5 App. C
	c) Retrofit	e) Retrofit	Chap. 4

Redundant Design and Rating Loads

Dead Loads. Dead loads for redundancy design or ratings are to be computed in accordance with AASHTO design provisions (Refs. 1 and 29). Dead loads for redundancy rating are to be computed in accordance with the conditions existing at the time of the analysis. Units weights are to be in accordance with the AASHTO provisions (Ref. 1).

Live Loads. Live loads for redundancy design and rating are to be computed in accordance with the AASHTO rating provisions (Ref. 1), except as follows: (1) Because the fractured bridge is expected to remain in service for only a short time before the fracture is discovered, and the bridge closed for repair, the probability of the bridge being subjected to full live load in the number of traffic lanes specified by AASHTO (Refs. 1 and 29) is significantly reduced. For this reason, it is recommended that no more than one traffic lane be loaded for two-lane bridges and no more than two traffic lanes be loaded for bridges with three or more traffic lanes. (2) Also noted as an exception is when the center of the outside wheel of any vehicle is applied at a distance not less than 2.0 ft from the curb.

Impact. Because fractured bridges are expected to experience deflections somewhat larger than the AASHTO design deflections (*1*), and the AASHTO anticipated rating deflections (*29*), it is recommended that a constant 30 percent impact factor be used regardless of span length.

Allowable Stress Method

It is recommended that redundancy design and rating be performed using the AASHTO allowable stress (service load) method defined in Ref. 29. It is recommended that allowable stresses corresponding to either the operating or redundancy rating level be used for redundancy design and rating by the conservative methods of Appendixes C and D or by the finite element modeling and analysis guidelines described in Chapter Five.

Allowable Stresses (Operating Rating). For two-girder bridges and smaller load factors used for design and rating of the redundant bracing system are also applicable to the girders. Thus, for the vertical dead and live load condition the girders have sufficient capacity. However, additional stresses are introduced by the three-dimensional after-fracture behavior of the structure consisting of the girders and the redundant bracing system.

This situation was studied in the research reported in Ref. 28. It was found that although local stresses may exceed allowables (as is the case in any bridge design to present AASHTO specifications as discussed earlier), the overall stress condition in the fractured and unfractured girders is acceptable.

In application of the procedures and equations developed in Appendix C, it is assumed that girder stresses are acceptable and no provisions for checking these stresses are established. In application of the computer modeling and analyses guidelines of Chapter Five, girder stresses can and should be checked.

Bearings

Figure C-17(a) of Appendix C shows the displacement of the fractured girder corresponding to midspan fracture. For simple spans, fixed-expansion bearings are normally provided. The fixed bearing is not normally designed for large longitudinal forces. The expansion bearing is usually designed only for the normal temperature change movements.

The research reported in Ref. 28 indicates that the longitudinal displacements at the ends of the girder, as shown in Figure C-17(a), will likely occur even at the fixed bearing. It is not advisable to redesign the fixed bearing to restrain this displacement because the forces are considerable. It is more practical to allow the displacement to occur even if the anchor bolts at the fixed bearings are sheared off. As part of design or retrofit for redundancy, it is suggested that both fixed and expansion bearings be evaluated or retrofitted to allow for a displacement equal to h, as shown in Figure C-17(a), even though the restraining forces applied to the girder flange by the bottom lateral diagonals will result in end displacements a little smaller than h.

For example, referring to Figure C-17(a) for $\Delta/L = 150$ and $d = 150$, $h =$ 2-in. For the fixed bearing the restraint against longitudinal displacement should be large enough to prevent displacement under normal conditions prior to fracture. However, this restraint should be small enough to be easily overcome to allow the 2-in. displacement to occur after fracture. The expansion bearing should also allow for a 2-in. displacement.

Allowable Fatigue Stresses

A two-girder bridge that has been designed or retrofitted to provide a redundant bracing system is, by definition, a redundant alternate load path structure. The applicable allowable fatigue stresses for design or rating are those specified in AASHTO Table 10.3.1A, Art. 10.3.1 (*1*).

Skewed Two-Girder Bridges

Although skewed two-girder bridges were not studied in the NCHRP-sponsored research leading to the guidelines presented in this report, a skewed two-girder bridge was studied as part of the PADOT-sponsored research (*28*).

The following guidelines are conservative and may be useful in evaluating, or providing for, after-fracture redundancy of skewed two-girder bridges: (1) It is assumed that the diaphragms are perpendicular to the girders. (2) The number, n, of bays of bracing to use in the equations presented in Appendix C is to be the largest number of rectangular bays each containing equal length diagonal members. (3) The span length, L, to use in the equations presented in Appendix C is the total span length of a girder.

CHAPTER FOUR

GUIDELINES FOR BRACING SYSTEM RETROFIT OR PROVISION OF AN ALTERNATE REDUNDANT LOAD PATH

APPLICATIONS

These guidelines are intended for application to new or existing steel two-girder simple span or continuous deck or through-girder highway bridges. The girders may be riveted or welded. The bridges may be composite or noncomposite. These guidelines are applicable to the following situations: (1) the retrofit of an existing bracing system to meet all of the requirements of a redundant bracing system, as defined in Chapter Three; (2) the design of an alternate redundant load path where it is uneconomical or impractical to provide a redundant bracing system; (3) special conditions for continuous deck-type bridges; and (4) design or retrofit of new or existing through-girder bridges to provide an alternate redundant load path.

BRACING SYSTEM RETROFIT—SIMPLE SPANS

The following guidelines apply to an existing bridge with an AASHTO bracing system that does not meet all of the requirements of a redundant bracing system as provided in Chapter Three, but that may be practically and economically retrofitted to those requirements. Refer also to the retrofit design and rating situations discussed in Chapter Three, Table 2, items (c) and (d).

Bottom Lateral Bracing Retrofit

Bracing Not Provided. If bottom lateral bracing is not present in the existing bridge, X-type bracing should be provided in accordance with the guidelines for redundant bracing systems discussed in Chapter Three. The cross-section areas, A_B, of all the diagonal members is computed, as shown in Appendix C, or by referring to the appropriate summary equations and examples in Appendix C. All connections between the diagonal members and a girder flange are to develop the full strength of the connected members.

Retrofit X-Type Bracing. If bottom lateral bracing is at the level of the girder flange, but is not an X-type bracing, it should be retrofitted to conform with the design and rating procedures, equations, and examples for X-type bracing presented in Appendix C.

Figure 15 shows several examples of bottom lateral bracing configurations that can be easily retrofitted to the X-type. Figure 15(a) shows X-type bracing in all five panels. No retrofit is needed. Use $n = 5$ in Appendix C.

In Figure 15(b), the connections between the diagonals and the girder flanges do not occur at the diaphragm locations as required in Chapter Three for redundant bracing systems. A suggested retrofit is to add new diaphragms, as shown in the figure, to provide four full panels of X-type bracing. The half-panel at each end can be left as shown. For the equations in Appendix C use $n = 5$.

Figure 15(c) shows five panels with only one diagonal in each panel. The addition of one more diagonal in each panel will produce the required X-type bracing. Use $n = 5$ in Appendix C.

Figure 15(d) shows K-type bracing in six panels. Extending the diagonals in the two panels shown in the figure will provide three full panels of X-type bracing. Use $n = 3$ in Appendix C.

Figure 15(e) is another example of K-type bracing in seven panels. However, the bracing overlaps in the middle panel. Extending the diagonals shown in figure will provide four full panels of X-type bracing, where the two interior panels overlap. Use $n = 4$ in Appendix C.

Bracing Not at Girder Flange Level. An example of a deck-type bridge where the bottom lateral bracing is above and not directly connected to the bottom flanges of the two girders is shown in Figure 5. In this case the bracing is 9-in. above the flanges. The retrofit used to provide the required connection to the flange is one based on practical considerations and relative costs. The following guidelines should assist in designing an economical retrofit.

If the bracing is attached to the girder within 2-in. or so of the bottom flange, it may be possible simply to make up the small gap with filler plates. The bottom lateral bracing can be bolted through the filler plates to a lateral gusset plate that is bolted to the underside of the girder bottom flange.

If the gap is larger, such as the 9-in. gap shown in Figure 5, a WF, ST, or some other shape or built up member can be placed in the gap and run parallel to the girder. The length of the member should be sufficient to develop the connection forces in the direction of the girder. Lateral connection forces are developed by the diaphragms and are not transmitted to the girder flange.

An alternate solution to the situation shown in Figure 5 is to lower the bottom lateral bracing to the level of the girder bottom flange and deepen the cross trusses. This may be the most economical solution in the event that the cross-section areas of the bottom lateral diagonals or the cross trusses have to be increased anyway to achieve redundancy (see examples in Appendix C).

If the bottom lateral bracing is considerably above the girder bottom flange, such as shown in Figures 3 and 7, the economical solution is probably to install additional X-type bracing at the level of the bottom flanges.

Study of the procedures, equations, and examples in Appendix C will indicate that the lower the bottom lateral bracing is placed the smaller are the required areas, A_B, of the diagonal members.

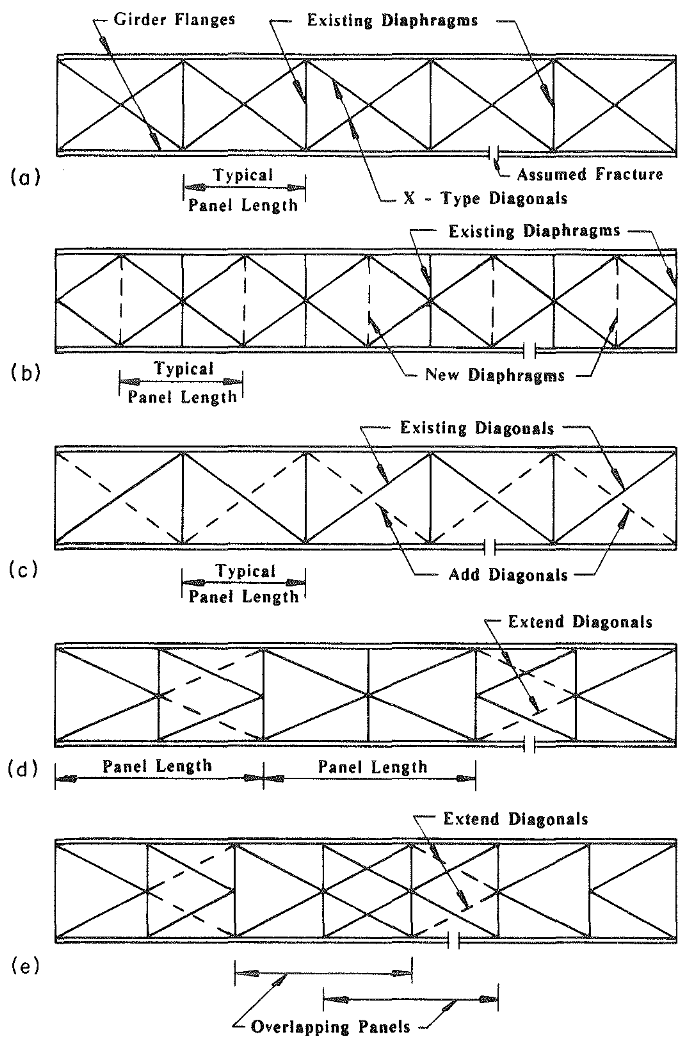


Figure 15. Examples of bottom lateral bracing configurations which can be retrofitted to X-type.

Diaphragm Retrofit

Acceptable diaphragms consist of cross bracing (X-type or K-type), cross frames, or cross trusses as defined in Appendix C. Existing diaphragms need to be installed or retrofitted to one of these types.

For example, the diaphragm shown in Figure 6 is not acceptable K-type cross bracing because the diagonals do not intersect with the bottom horizontal at one point midway between the girders as required to form structurally stable triangles. A possible retrofit is to remove the existing diagonals and replace them with longer members and a new connection midway between the girders. Although two new members can be added, forming an inverted V, extending from the two connections on the bottom horizontal up to a single connection on the bottom of the floor beam midway between the two girders, Appendix C does not provide explicit design equations for this case. Forces in the diagonal members can, however, be computed by extending the concepts used in Appendix C.

Also in Figure 6, the top horizontal member of the cross bracing consists of a substantial floor beam plus haunches that connect to top flanges of the girders. This arrangement is satisfactory, providing that all forces in the members and connections can be computed along the lines shown in Appendix C (see Fig. C-24, for example).

Alternatively, the two diagonals shown in Figure 6 can be removed and additional haunches installed between the floor beam and the bottom flanges of the girders. In this case the diaphragm is converted to a cross frame and designed or rated as shown in Appendix C (see Fig. C-24, for example).

Figure 16 shows an example of a cross-truss retrofit. Cross trusses are often used when the two girders are spaced far apart. Stringers can then be supported by the cross trusses. The top horizontal member of a cross truss may not attach directly to the top flanges of the girder but some distance below, as shown in the figure. A possible retrofit is the installation of new struts as shown in Figure 16. Referring to the development in Appendix C (Fig. C-24, for example), one notices that the two diagonal struts are subjected to the net applied force of $U - F$. Each strut should be designed for a controlling compression force of $(U - F)\ k_d/2$.

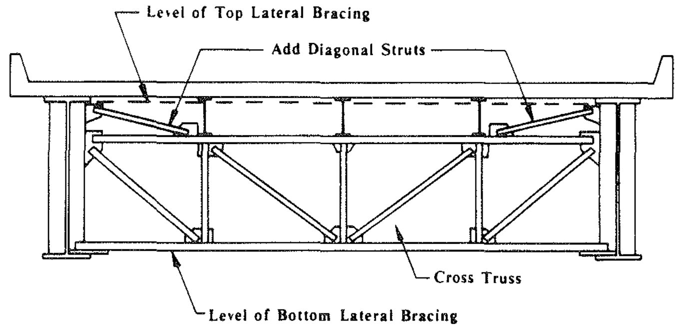


Figure 16. Example of a cross-truss retrofit.

Top Lateral Bracing Retrofit

Top lateral bracing is a very important component of the redundant bracing system. As discussed in Chapter Three and Appendix C, the forces released by the fractured girder are transmitted first to the bottom lateral diagonals and then to the top lateral diagonals through the diaphragms. The strength and stiffness of the fractured two-girder bridge are ultimately dependent on the transverse strength and stiffness of the top lateral bracing.

Some two-girder bridges do not contain top lateral bracing. This is illustrated in Figures 3, 5, 6, and 7. The composite decks shown in Figures 5 and 6 may have been relied upon in the original design to provide lateral strength and stiffness under wind loads in lieu of top lateral bracing. But they likely need to be retrofitted to provide redundancy.

Other two-girder bridges have top lateral bracing that may or may not have been designed for wind loads. If not, the bracing was probably installed to provide stability to the tops of the girders during construction. The existing top lateral bracing also may not conform to the requirements of a redundant bracing system. An example of this is shown in Figure 2 where top lateral bracing is provided but not at the level of the top flanges of the two girders.

Bracing Not Provided. If top lateral bracing is not present in the existing bridge, X-type bracing should be provided in accordance with the guidelines for redundant bracing systems discussed in Chapter Three. The cross-section areas, A_T, of all the diagonal members is computed as shown in Appendix C or by referring to the appropriate summary equations and examples in Appendix C. All connections between the diagonal members and a girder flange are to develop the full strength of the connected members.

Installation of top lateral bracing with the deck in place is likely to be difficult and expensive. Considering fatigue, welding rather than bolting is acceptable because the girder top flanges are in compression for simple spans. However, overhead welding in a confined space is not too practical. This option should be given consideration, however, if it is planned to replace the existing deck as part of a rehabilitation program.

Alternatively, it may be possible to develop the required transverse strength and stiffness using the reinforced concrete deck in lieu of the top lateral bracing. Guidelines are presented below under the heading "Use of Deck as Top Lateral Bracing."

Retrofit to X-Type Bracing. If top lateral bracing exists and is at the level of the girder flange, but is not X-type bracing, it should be retrofitted to conform with the design and rating procedures, equations, and examples for the X-type bracing presented in Appendix C. The retrofit guidelines discussed above for bottom lateral bracing and the examples shown in Figure 15 are equally applicable to top lateral bracing. As discussed above, this retrofit option may be practical and economical only if the existing deck is to be replaced as part of a rehabilitation program.

Alternatively, in lieu of retrofitting the top lateral bracing, it may be possible to use the reinforced concrete deck as the required top lateral bracing. Guidelines are presented below under the heading "Use of Deck as Top Lateral Bracing."

Bracing Not at Girder Flange Level. An example of a deck-type bridge where the top lateral bracing is below and not directly connected to the top flanges of the two girders is shown in Figure 2. In this case, the bracing is over 24 in. below the flanges. As previously discussed for the bottom lateral bracing, the retrofit used to provide the required connection to the flanges is one of practicality and relative costs. Because of the confined working space under the deck and the presence of stringers, retrofit will be difficult and expensive unless the deck is to be replaced as previously mentioned. The retrofit guidelines, discussed above, for bottom lateral bracing are equally applicable, with appropriate modifications, to top lateral bracing retrofit.

Alternatively, in lieu of retrofitting the top lateral bracing, it may be possible to use the reinforced concrete deck as the required top lateral bracing, as discussed next.

COMPOSITE DECK AS TOP LATERAL BRACING

For simple-span two-girder bridges, reinforced concrete deck

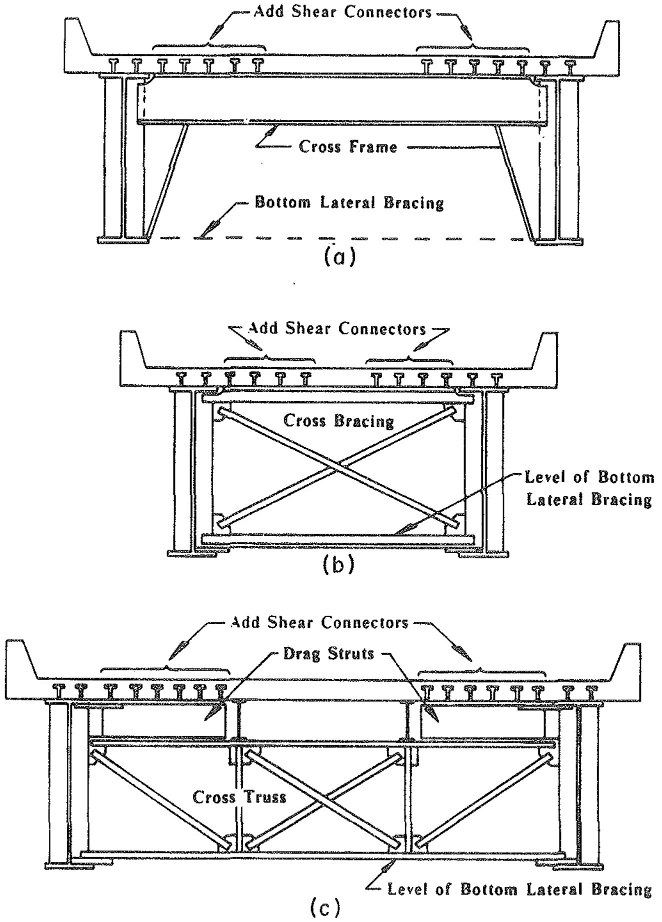


Figure 17. Alternative ways of providing composite connection between the deck and diaphragms.

This transverse force can be transmitted to the deck, however, by making the deck composite with the diaphragm. Figure 17 shows alternative ways of providing this composite connection. In Figure 17(a) the deck is made composite with the floor beams that are part of a cross-frame diaphragm. If the deck is not already composite the floor beams, it can be made composite by coring the deck so that stud connectors can be welded to the floor beams. Alternatively, high strength ASTM A325 bolts can be used which require drilling the floor beam flange. After providing the required number of shear connectors, the holes in the deck are grouted. Further details of providing such a shear connection are contained in *NCHRP Report 293* (*31*).

Figure 17(b) shows a similar composite connection to the top horizontal member of a cross-bracing diaphragm. It may be necessary to replace the existing horizontal member of the diaphragm with a larger member in order to transmit the required transverse force without exceeding allowable stresses.

Figure 17(c) illustrates the use of drag struts to transmit the transverse force from a cross-truss diaphragm to the deck. The drag strut might be the same section as the stringers. The deck is made composite with the top flange of each drag strut. The bottom flanges of each drag strut are bolted to the cross truss horizontal member. The drag strut is also connected to the girder flanges. A lateral gusset plate attachment is used to connect the girder flange to the drag strut, as shown in Figure 17. The drag struts serve two purposes, (1) to provide the required transverse shear connection between the deck and the diaphragm and (2) to provide the required connection between the cross truss and the top flanges of the two girders.

The total transverse force which is to be transmitted to the deck at every diaphragm location by the shear connection is $U - F$, as shown in Figures C-18 and C-24. This force might be developed over a relatively short distance and not the whole width of the deck, as shown in Figure 17, where the transverse shear connection is developed only between the girders and the adjacent stringers.

is available to replace the top lateral bracing, but only if the deck is fully composite with the two girders over the entire span length. In effect, top lateral X-type bracing, which acts like a horizontal truss, is replaced with a horizontal plate girder. The flanges of the horizontal girder, like the flanges of the horizontal truss, are the top flanges of the two bridge girders. The diagonals of the horizontal truss are replaced by a web consisting of the reinforced concrete deck. The shear connectors are needed so that the girder flanges are continuously attached to the deck as flanges are attached to the web of a plate girder.

As demonstrated in Appendix C, and discussed in Chapter Three (Fig. 14), following the fracture of one of the two bridge girders, concentrated transverse forces are developed at the intersections of the top lateral diagonals, the diaphragm members and girder flanges. If the deck replaces the top lateral bracing, this concentrated transverse force must be transmitted from the diaphragm members through the shear connectors between the bridge girder and the deck. The shear connectors are therefore subjected to forces perpendicular to the girder flanges. Because the stiffness of the girder flange in the transverse direction is relatively small, the concentrated transverse force will be distributed to relatively few connectors in the vicinity of the diaphragms. In view of the magnitude of the concentrated transverse force to be transmitted to the deck, these few shear connectors will likely be substantially overstressed.

REDUNDANT TENSION CABLES, RODS, OR SHAPES

From the point of view of ease of installation and perhaps economy, the use of tension cables, rods, or shapes may be a practical means of providing after-fracture redundancy to some two-girder deck-type bridges in-lieu of providing a redundant bracing system, as described in Chapter Three and Appendix C. The requirements of unpoststressed tension cables, rods, or shapes are developed in Appendix D together with design examples.

Although the use of tension cables, rods, or shapes may appear at first to be a relatively attractive solution, there are several potential concerns that should be carefully addressed before making a decision. Some of these are briefly discussed in the following:

1. If the tension cables, rods, or shapes are tightened (snug, not pretensioned) the redundant system, including the anchorages, is subjected to live load stress ranges under normal traffic conditions. The redundant system is therefore not fail safe, as defined in Chapter Three, because the possibility exists that it may develop fatigue cracks and fracture prior to girder fracture. Care, therefore, is needed in the design, fabrication, and installation of the redundant system, especially the intermediate and end connections

and the attachment of the anchorages to the girder (refer to Fig. D-1, Appendix D).

2. If the tension cables, rods or shapes are left sufficiently slack, the redundant system is not subjected to live load stress range during service. However, sudden fracture of a girder may subject the redundant system to large impact loads. These impact loads should be considered in the design of the system, including anchorages. Materials with sufficient toughness are required to prevent brittle fracture in the presence of normal fabrication and installation flaws.
3. Figure D-1 of Appendix D shows the installation of intermediate supports for the tension cables, rods, or shapes between the two anchorages. Prior to girder fracture these intermediate supports are spaced mainly to prevent excessive sag and to eliminate fatigue stresses due to aolean (wind-induced) vibration. However, when fracture occurs the girder may deflect significantly. After fracture, the supports are now required to maintain the alignment of the cables or rods and keep them below the girder. These supports are placed in regions that are subjected to large live load stress range prior to fracture. Extreme care must be exercised in the design and installation of the supports to ensure that fatigue cracking and fracture of the girder does not originate at one of the supports. A bolted connection provides a high fatigue strength detail and is preferred for both riveted and welded girders. For ductile connections, use multiple rows of bolts perpendicular to the girder. A single row of bolts perpendicular to the direction of stress always provides a brittle connection.
4. The redundant design example in Chapter D2 of Appendix D illustrates the use of tension rods. Tension cables or shapes could also be used. The steel used for the redundant system should conform to AASHTO provisions (*1*) or be suitable for exposed, unprestressed application. The steel should also be suitable for threading if nuts are used at the anchorages.

   The material for tension rods should conform to Table 10.2A of AASHTO, Ref. 1. Material shown in Table 10.2B of Ref. 1 is not permissible because this material is specified only for pins, rollers, and rockers. Similarly, material for wire and strands intended for prestressed concrete application and specified in Art. 9.3.1 of Ref. 1 is not permissible. High strength steel bars intended for pretensioned or post-tensioned concrete structures, such as ASTM A 722, cannot be used for the redundant system. A 722 rods are limited to $1\frac{3}{8}$ in. diameter, have low ductility, very low toughness, and very low corrosion resistance (*63, 64*). Such materials are intended for application only under full pretension or post-tension and protected from corrosion, such as in uncracked concrete.

   The material used for tension cables (wire ropes and strands) should conform to Table 10.2A of Ref. 1 or to AASHTO M 277 (material for moveable bridges), ASTM A 603 (rope) or ASTM A 586 (strand).
5. If relatively small diameter tension rods of A 517 material are used, a protective coating is required. Although galvanizing provides excellent protection, the hot dipped process is risky due to hydrogen embrittlement. This is usually only a problem for tensile strengths above 150 ksi, but can also be a problem with A 517 steel.
6. Recent experience with coated strands and cables for cable-stayed bridges indicates that the kinds of coatings and protection used in recent years is not providing the corrosion protection needed (*65, 66*). Use of redundant tension cables, rods, or shapes to provide a redundant system is not unlike cable-stayed application.

ADDING GIRDERS TO AN EXISTING TWO-GIRDER BRIDGE

Whether or not it is cost effective, and regardless of the alternate methods presented in this report, there may be good reasons to provide after-fracture redundancy to an existing two-girder bridge by adding new girders.

There are several alternative ways to add girders. For example, one girder may be installed midway between the existing girders to create a three-girder bridge. Or additional girders could be added between the existing girders to create a multiple-girder bridge. Or the new girders could be installed only adjacent to the existing girders to retain the "two-girder" concept.

The following guidelines may be found useful when retrofitting the existing two-girder bridge to one of these alternatives.

Retrofit to a Three-Girder Bridge

Unless the bridge deck is removed prior to installation of the new girder, or unless the new girder is preloaded by jacking against the floor beams, the new girder is subjected only to live and impact loading.

The proportion of live and impact loading carried by the new girder is a function of the relative moments of inertia of the new and existing girders. The usual assumptions used in new three-girder bridge design on the distribution of live and impact loading to the three girders can be used in the retrofit design providing that the moments of inertia of the new and existing girders are the same.

Retrofit to a Multiple-Girder Bridge

As above, unless the bridge deck is removed, or unless the new girders are preloaded by jacking against the floor beams, the new girders are subjected only to live and impact loading. If the AASHTO live load distribution provision is used for the new girders, the moments of inertia of the new and existing girders should be the same.

Retrofit to Retain the Two-Girder Concept

The new girders are installed adjacent to the existing girders, but spaced sufficiently far apart to enable inspection for cracking and corrosion and to enable painting.

Unless the deck is removed or the existing girder shored, the new girder(s) will be subjected only to live and impact loading. If the new and existing girders are to share the live and impact loading equally, each new girder must have the same moment of inertia as the existing adjacent girder.

The new girder(s) must also be designed to carry the full dead and live plus impact loading in order to provide sufficient strength in the event that the existing adjacent girder fractures.

THROUGH-GIRDER BRIDGES

Figure 4 shows the cross section of a typical through-girder steel bridge. Floor beams are connected to the bottoms of the two girders as shown in the figure. There may or may not be a bottom lateral system under the floor beams. The stringers may be located, as shown in the figure, or they may be placed on top of the floor beams. Diaphragms and top lateral bracing obviously cannot be used in a through-girder bridge. Therefore, redundancy cannot be provided with a redundant bracing system as discussed in Chapter Three and Appendix C.

Redundancy can be provided using tension cables, rods, or shapes as discussed in the previous section and in Appendix D.

Alternatively, if space exists at the abutments, another girder could be fabricated and installed outside of the existing girders. The redundant girders can be connected to the existing girders at the floor beam locations. The redundant girders are designed using the usual design procedures for through-girder bridges but using the redundant loads, and allowable stresses or load factors suggested in Chapter Three.

CONTINUOUS TWO-GIRDER BRIDGES

Redundant Bracing System

The concept of providing after-fracture redundancy with a redundant bracing system, as discussed in Chapter Three and Appendix C, can easily be extended to continuous two-girder steel bridges, as provided by the following guidelines.

1. *End and Interior Spans:* A worst case scenario would include the possibility that a fracture first occurs in the negative moment region over an interior support, originating in the top (tension) flange, followed by a fracture in an end or interior span because of the sudden impact loading resulting from the first fracture. These fractures essentially reduce the continuous structure to a simple span structure in a local region. It is therefore suggested that redundancy design and rating of each end and interior span be performed as described in Appendix C, considering each span to be simply supported.
2. *Negative Moment Regions:* The top flange is the tension flange in the negative moment regions. Sufficient top lateral bracing should be provided to develop the force released by the girder following a near full depth fracture over an interior support. This force could be calculated in a manner similar to that shown in Chapter C2 of Appendix C (see Fig. C-9 and Eq. C2.1), but the simplified model and resulting equations are much more complex. Fortunately, a sufficiently accurate alternative is available. The ratio of forces released after a fracture in the negative or positive moment regions is approximately equal to the ratio of the maximum areas of the tension flanges in the negative and positive moment regions. This suggests the following guidelines for proportioning the top lateral diagonal members in the negative moment region over a particular interior support.

a. Compute the ratio of the maximum tension flange area over an interior support to the average of the maximum tension flange areas in the positive moment regions on either side of that support.
b. Compute the average areas of the bottom lateral diagonals in the two spans adjacent to that support.
c. Assuming that all top and bottom lateral diagonals are of steel with the same yield stress, provide top lateral bracing diagonal members over that support with areas equal to the area computed in (b) times the ratio computed in (a) but not less than the maximum area for the top lateral bracing which is computed assuming each adjacent span to be simply supported as described in item 1 above.
d. The top lateral bracing computed in (c) should extend at least a quarter of the span on each side of that support.

3. *Inflection Regions:* In the usual design of continuous girders, flange areas are often reduced in the inflection regions where the dead and live load bending moments are reduced. Splices may also occur in these regions. If the AASHTO bracing system described in Chapter Three is insufficient to provide redundancy after fracture of a girder in an end or interior span, the resulting redistribution of bending moment towards the negative moment region, as the girder cantilever from the support, may be sufficient to fail the girder or splice in the inflection region (see Ref. 28 for a detailed description of this behavior). If a redundant bracing system is provided, as described in Chapter Three and Appendix C, and extended to continuous girders, as suggested above, there should not be a significant redistribution of bending moment and the inflection regions or splices should not be subjected to bending moments significantly larger from those assumed in design. In fact, because the redundant live loads suggested in Chapter Three are less than the design live loads, this should offset the above effects.
4. *Compression Flange Buckling:* The comments in item 3 apply equally to the possibility of compression flange buckling in the negative moment regions. Buckling should not be expected providing that a redundant bracing system, as described in Chapter Three and Appendix C and extended to continuous girders, is provided.

Tension Cables, Rods, or Shapes

Tension cables, rods, or shapes, as described in this chapter and Appendix D, can also be used to provide after-fracture redundancy to continuous girders. The following guidelines may be useful:

1. Provide tension cables, rods, or shapes for each span as described in Appendix D, treating each span as simply supported.
2. Provide tension cables, rods, or shapes for each interior support extending at least a quarter of the span on each side of the support.
3. The required area of tension cables, rods, or shapes over an interior support can be computed using the flange area ratio similar to that described above for top lateral bracing.
4. If the deck is above the top flange of the girder (stringers are on top of the floor beams), it may be relatively easy to install the tension cables, rods, or shapes in the negative moment region.
5. If the top flange of the girder is cast into the deck, installation

of tension cables, rods, or shapes will be much more difficult. The anchorages could be attached to the underside of the top flange, but in this case the tension cables or rods will pass through web and bearing stiffeners requiring special details. Extreme care is needed when retrofitting the stiffeners to accommodate the cables, rods, or shapes because fatigue cracking and fracture could originate at one of these retrofitted details.

CHAPTER FIVE

GUIDELINES FOR COMPUTER MODELING AND ANALYSIS

APPLICATIONS

These guidelines are intended for bridge engineers who wish to provide for or evaluate redundancy in new or existing simple-span two-girder bridges by a finite element analysis of the three-dimensional structure rather than by the procedures developed in Appendix C. As elsewhere in this report the bridges may be riveted or welded, and composite or noncomposite, but are assumed to contain a properly configured and located redundant bracing system as defined in Chapter Three. These guidelines are applicable in each of the following design and rating situations: (1) the design of an alternate load path to provide redundancy in a new bridge; (2) the design of a new alternate load path to provide redundancy in an existing or rehabilitated bridge; (3) the retrofit design of an as-built or existing alternate load path to provide redundancy in an existing or rehabilitated bridge; and (4) the analysis of an as-built or existing alternate load path to determine the redundancy rating of an existing bridge.

In each of the above situations if the alternate load path consists of top and bottom lateral bracing and diaphragms configured and located as defined in Chapter Three, the preliminary cross-section areas of the bracing system members that are needed in the analysis can be found using the equations presented in Chapter C9 of Appendix C.

In a finite element analysis of the structure in accordance with the guidelines presented in this chapter, the bridge engineer is not limited to X-type top and bottom lateral bracing as defined in Chapter Three and developed in Appendix C. Other configurations such as those shown in Figure C-4 may be used. Diaphragm configurations other than those considered in Appendix C are also possible. However, the locations of the top and bottom lateral bracing and the diaphragms must conform to the guidelines presented in Chapter Three. If alternate configurations are employed, preliminary cross-section areas of the members used in the finite element analysis must be found by trial and error or other means, as is common to all indeterminate structural analysis.

For a composite bridge with continuous shear connection between the girders and the deck, if no top lateral bracing is provided, or suitable top lateral bracing is not available, the required composite connection can be provided using drag struts between the deck and diaphragms or floor beams as suggested in Chapter Four. In this case the three-dimensional model should include elements representing the in-plane stiffness of the deck instead of top lateral bracing.

These guidelines should be used in conjunction with the guidelines presented in Chapters Three and Four, where applicable. A linear finite element analysis program can be used to provide a redundancy design or rating based on the allowable stress, load factor, and serviceability guidelines contained in Chapter Three. Load factor design or rating is based on the least strength of a member on the alternate load path, not on partial yielding and redistribution of forces as is assumed in Appendix C.

MEMBERS AND COMPONENTS TO BE INCLUDED

Finite element modeling and analysis for redundancy of composite and noncomposite simple-span two-girder steel bridges can be accurately performed by including the major members and components of the alternate load path, as follows:

Noncomposite Structure

The members and components to be included in a finite element analysis consist of the unfractured and fractured girders, floor beams, floor beam connection plates, bottom lateral bracing, diaphragms, top lateral bracing, and bearings.

Composite Structure

The members and components to be included in a composite structure are the same as those for a noncomposite structure. In addition, the composite deck is included. However, only the in-plane stiffness, not bending stiffness, of the deck is used in order to adhere to the assumption used in the remainder of this report that the bending strength and stiffness of the deck are not considered. If composite action is provided between the deck and the diaphragms or floor beams as suggested in Chapter Four, the top lateral bracing (if any) need not be included in the three-dimensional model.

FINITE ELEMENT MODELING

Recently, a number of finite element types have been developed to efficiently and accurately represent a real structure. Not all of these are needed in a finite element analysis to determine the redundancy of two-girder steel bridges. The primary elements needed for the usual two-girder steel bridges are the truss,

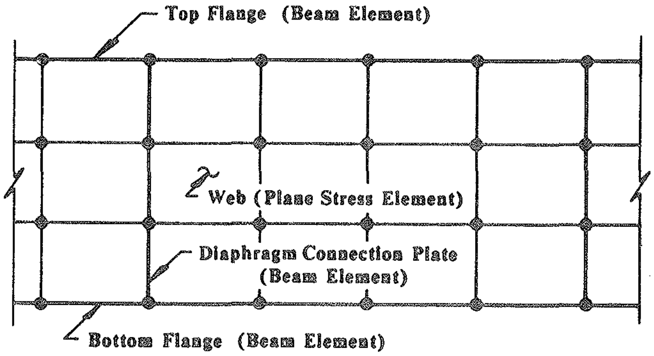


Figure 18. Typical planar model for the two main girders.

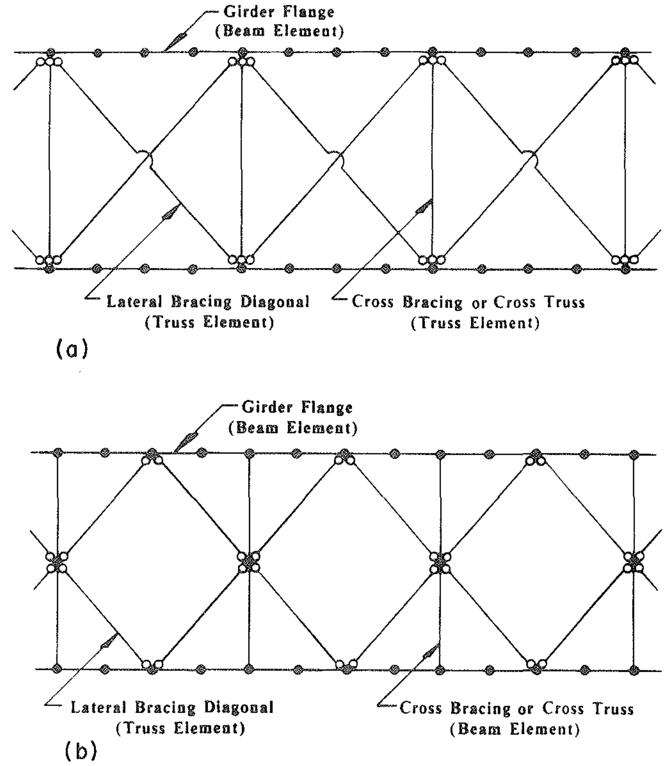


Figure 19. Examples of modeling of top and bottom lateral bracing.

beam, plane stress, and plate bending elements. Guidelines on the use of these elements in modeling two-girder composite and noncomposite steel bridges containing a near full depth fracture of one girder are as follows.

Main Girders

Each of the two main girders are modeled as planar structures consisting of the top flange, web, bottom flange, and transverse diaphragm and/or floor beam connection plates. Figure 18 shows a typical planar model for the two main girders. Beam elements are used to model the top and bottom flanges and the transverse connection plates. Plane stress elements are used for the web.

The beam elements representing the girder flanges are to have axial stiffness only. The flexural and torsional bending moments of inertia are to be suppressed. The beam element therefore functions like a truss element. The main reason for using the beam element is that in three-dimensional models the use of truss elements frequently causes numerical instability problems in the computer analysis. Another reason for using beam elements is so that shear forces can be transmitted at the cross section containing the girder fracture.

The girder web is modeled by plane stress elements. The small bending strength of the web is ignored. However, this is conservative and enables in-plane web stresses to be more easily determined. Depending on the computer program being used, out-of-plane instability may or may not occur with the plane stress elements. It may be necessary to suppress out-of-plane degrees of freedom. Alternatively, plate bending elements can be used for the girder web.

The floor beam connection plates are also modeled as beam elements. The flexural and torsional stiffnesses of this beam element are suppressed because it is assumed that the top and bottom horizontals of cross bracing or cross trusses or the flange of a cross frame are connected to the main girder flanges.

The web element sizes and corresponding beam element lengths depend mainly on the web element aspect ratio. The aspect ratio of a web element should be less than two. Also, a minimum three but preferably four or more rows of web elements through the girder depth are recommended.

Top And Bottom Lateral Bracing

Figure 19 shows examples of modeling of the top and bottom lateral bracing. A typical X-type bracing is shown in Figure 19(a). As previously discussed, main girder top and bottom flanges are modeled as beam elements. Lateral bracing diagonals and cross bracing or cross-truss transverse members, however, are modeled as truss elements. The main reason for using truss elements is that because bending moments in bracing members are displacement induced and relatively small,. even after girder fracture, they need not be considered. Ignoring these small moments is conservative.

In Figure 19(a) each diagonal member connecting the girder flanges is modeled as one truss element. Even if the diagonals are actually connected together in the structure where they cross, it is conservative to assume that they are not connected in the finite element model. Note that four truss elements connected to a central node cannot be used in the plane of the lateral bracing. Because they cannot resist out-of-plane displacement at the central node, a numerical instability will occur in the computer analysis. The transverse members are also modeled as truss elements connecting the girder flanges.

Figure 19(b) shows an example of another configuration of bottom lateral bracing and its modeling for finite element analysis. All the diagonal truss elements are shown connected to the cross bracing or cross-truss members which are modeled as beam elements. The beam elements require suppression of the flexural stiffness and use of only the axial stiffness of the element.

If cross frames are used, the transverse members in Figure 19 are modeled as discussed below for cross frames. Other configurations of lateral bracing can also be used as long as they

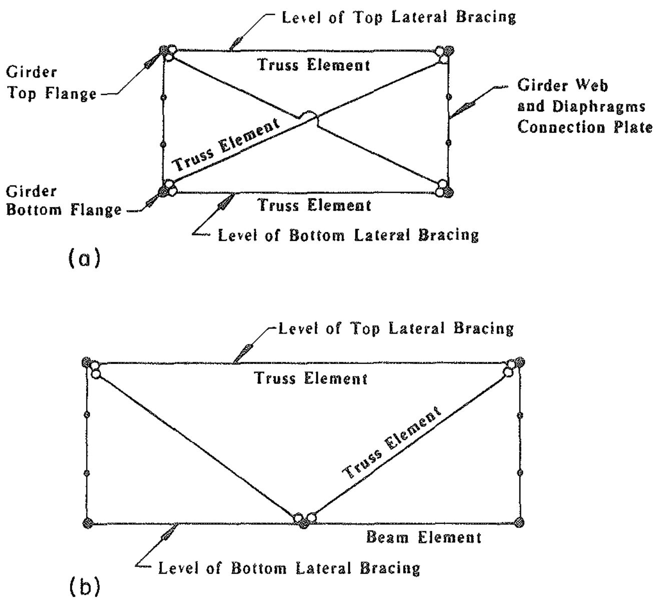


Figure 20. Examples of cross-bracing modeling.

are triangulated to form a stable structure. They are modeled similar to the guidelines suggested above.

Cross Bracing

Figure 20 shows examples of cross-bracing modeling. It is assumed in the figure that the structures are noncomposite and contain top and bottom lateral bracing.

Guidelines for modeling of X-type cross bracing, as shown in Figure 20(a), are identical to those for X-type lateral bracing as discussed above. Figure 20(b) shows modeling of K-type cross bracing. The cross-bracing diagonals and top horizontal are modeled as truss elements, while the bottom horizontal is modeled as a beam element. The beam elements require suppression of the flexural stiffness.

Cross Truss

Figure 21 shows examples of cross truss modeling. Figure 21(a) shows the cross truss top and bottom chords at the level of the girder flanges. Figure 21(b) shows the use of diagonal struts to transfer lateral force directly between cross truss and girder top flanges (Ch. Four).

The cross truss top and bottom chords are modeled as beam elements in which the flexural and torsional stiffnesses are suppressed. All the cross-truss diagonals and verticals are modeled as truss elements. Also the diagonal struts are modeled as truss elements as shown in Figure 21(b). Other configurations of cross truss can be modeled in a similar way providing that they consist of triangulated stable bracing members.

Cross Frame

Figure 22 shows examples of cross-frame modeling. In order to transfer the lateral forces directly between the girder flanges and the cross frame, four knee struts are used as shown in Figure 22(a). If the cross frame is directly connected to the girder top flanges as shown in Figure 22(b), only two knee struts to the bottom flanges are needed.

The cross frame requires a planar modeling whose element types are identical to the main girders as shown in Figure 18. The knee struts are also included in the planar model. If the knee struts have a web and flange, the flange is modeled as a beam element containing axial stiffness only. The web is modeled

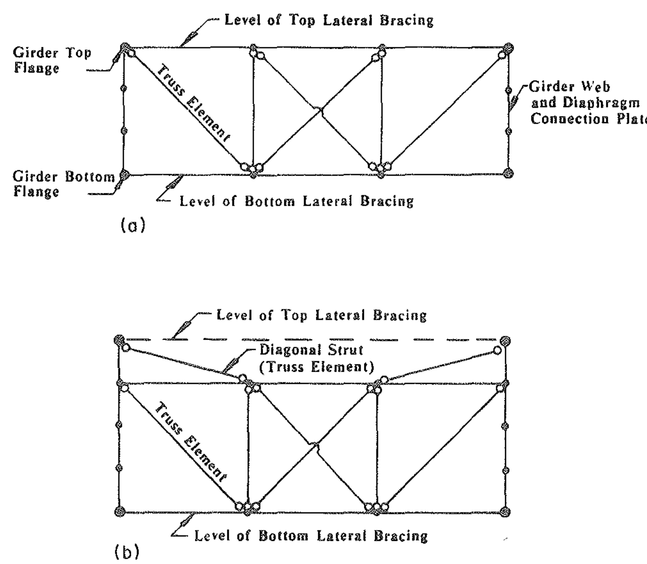


Figure 21. Examples of cross-truss modeling.

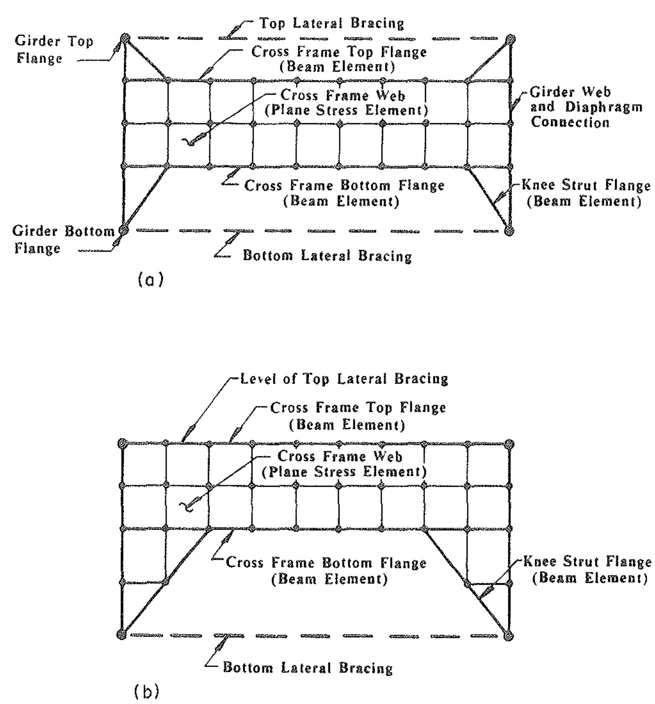


Figure 22. Examples of cross-frame modeling.

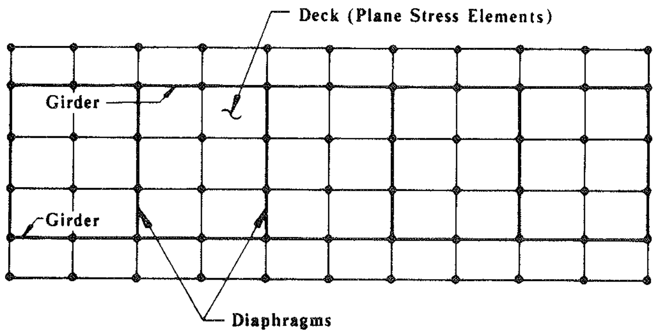


Figure 23. Example of deck modeling for composite structures.

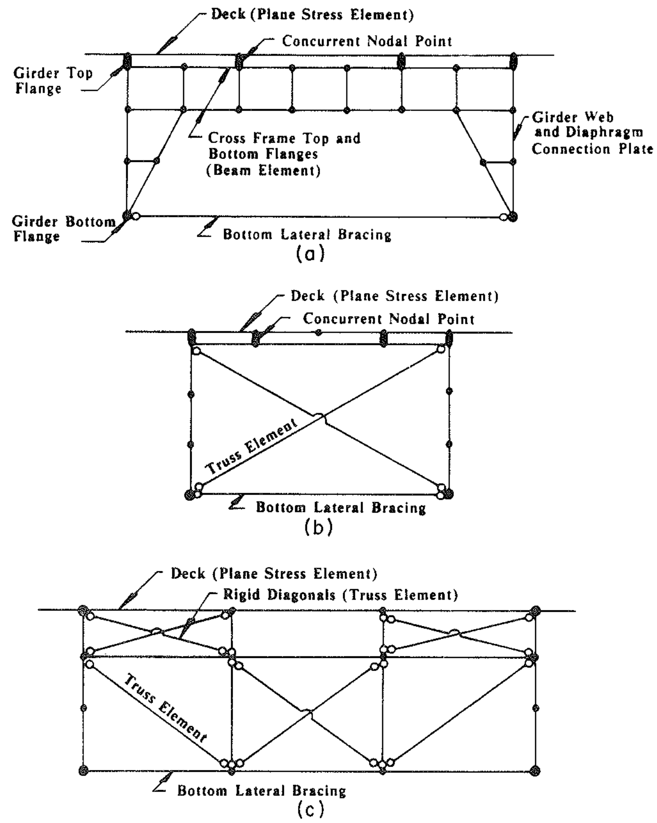


Figure 24. Example of diaphragm modeling where the composite deck replaces the top lateral bracing composite (Ch. Four).

with plane stress elements as shown in the figure. Refer to the earlier discussion regarding potential out-of-plane instability when using plane stress elements. Suitable discretization of the cross frame requires a minimum of two, but preferably three or more, rows of web elements in the depth of the cross frame.

Deck

Figure 23 shows the deck modeling for a composite structure. The locations of the two girders and the diaphragms are also shown. The deck is modeled using plane stress elements which develop membrane stiffness only. The elastic modulus of the deck elements can be taken as the actual concrete modulus neglecting reinforcing bars and the effects of cracking. The discretization of the deck depends on the desired deck element aspect ratios. Aspect ratios less than two are recommended. Depending on the computer program being used, it may be necessary to suppress the out-of-plane degrees of freedom to prevent out-of-plane instability with plane stress elements. However, as explained previously, plate bending elements should not be used to model the deck for composite bridges.

Figure 24 shows examples of diaphragm modeling where composite connection to the deck is provided using drag struts (Ch. Four). The three example structures shown in the figure are the same as those shown in Figure 17 of Chapter Four. None of these structures contain top lateral bracing. The in-plane stiffness of the deck elements replaces the top lateral bracing as discussed in Chapter Four.

Figure 24(a) shows the modeling required to provide shear connection between cross frames and the deck. Because it is assumed that the shear connection is strong enough to safely carry the required transverse force, the cross frame top flange can be directly connected to the deck using concurrent nodal points in the three-dimensional model, as shown in the figure.

Figure 24(b) shows the modeling required to provide shear connection between cross bracing and the deck. Concurrent nodal points are used to directly connect the cross bracing top horizontal to the composite deck as for cross frames.

Figure 24(c) shows the modeling required to provide shear connection between cross trusses and the deck using drag struts and shear connectors as shown in Figure 17(c). The drag struts are simple rigid members connecting the cross truss top chord and the deck. For simplicity the drag struts are modeled as rigid X-type bracing as shown in the figure. The diagonals and verticals of the rigid X-type bracing are truss elements with very large elastic modulus.

Fracture

For steel girder bridges there are many possible fracture scenarios. The following presents guidelines for modeling a typical girder fracture which may be placed at any position along the span. The fracture is assumed to pass through the bottom flange and the full depth of the web, but not through the top flange. Only one fracture is assumed to occur in a girder, and only one girder is assumed fractured.

Figure 25 shows the modeling of a typical main girder fracture. Figure 25(a) shows the model configuration recommended for GTSTRUDL program users (*67*). The web elements and the bottom flange elements are separated by using adjacent edge nodes at the fracture. For the purpose of accommodating fractures at any location, inactive nodes are provided at other potential fracture locations as shown in the figure. The model configuration recommended for SAP program users is illustrated in Figure 25(b) (*68*). All possible fracture locations are modeled by using coupled nodes. Except for the fracture location of interest, the other node couples are rigidly linked by introducing the master/slave option.

Bearings

An important issue is the modeling of the bridge bearings. Most bridges contain expansion and fixed bearings. Typical fixed bearings have anchor bolts to resist longitudinal and lateral displacement. Also, expansion bearings may have special provisions, such as keeper plates, to resist lateral movement. From a theoretical point of view, these restraints make the bridge highly indeterminate externally. It has been demonstrated that, when a girder fractures, the longitudinal and lateral displacements at all four bearing locations, but especially at the ends of the fractured girder, are large enough to shear the anchor bolts and remove any other restraints (*28*). This motion occurs as though no restraints existed. This is because typical bearings are not designed for large restraint forces. It is important, therefore, in modeling the bridge bearings to provide only sufficient restraint in the horizontal plane to prevent rigid body motion of the entire structure in this plane and not restrain motion of the other bearings. In effect the bearings are modeled as though they consisted of spheres, providing only vertical support. Additional lateral restraints are then provided to prevent rigid body motion of the bridge in a horizontal plane.

Figure 26 shows the minimum horizontal support boundary conditions needed for simple span and continuous two-girder bridges. Only three horizontal restraints are needed for each bridge. One longitudinal restraint can be provided at a fixed bearing location, say, on one of the girders. Two additional lateral restraints are provided as shown in the figure. With the horizontal constraints shown, no horizontal reactions will result. Also stress or force output for all members in the three-dimensional model will be identical no matter where the horizontal restraints are located. Only the horizontal deflections will be different. But these deflections are of little interest.

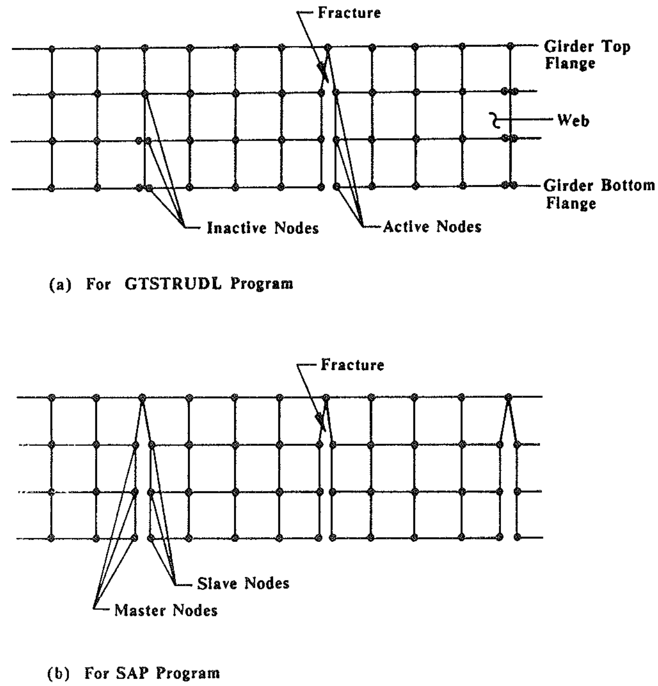


Figure 25. Modeling of main girder fracture.

REDUNDANCY EVALUATION

Methods and Factors

The methods for redundancy evaluation using finite element modeling and analysis are basically the same as the methods discussed in Chapter Three and Appendix C. The only exception is that while the load factor method in Appendix C assumes yielding and buckling of bottom lateral bracing tension and compression diagonals, respectively, the load factor method using computer modeling and analysis requires the model to be linear elastic. Consequently, the guidelines for finite element computer modeling and analysis are applicable not only to the allowable stress but also to the load factor method. For redundancy evaluation both methods require various inputs such as dead and live loads load factors, impact factors, allowable stresses, deflection-to-span length ratio limits, longitudinal girder slope limits, and transverse deck slope limits. All of these can be obtained from the guidelines presented in Chapter Three.

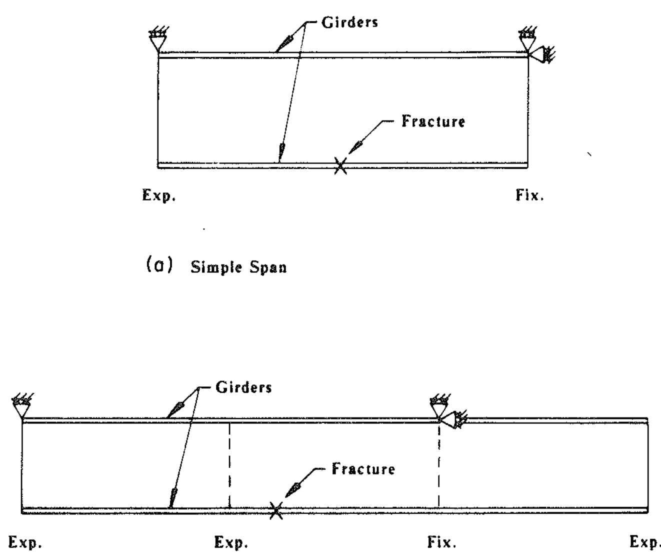


Figure 26. Recommended horizontal support boundary conditions.

Loading

Loads needed for redundancy evaluation are the redundant dead and live plus impact loads. The dead load refers to the as-built or the as-designed superstructure weight including a future wearing surface. The dead load can be easily computed by the analyst. The live load refers to the rating vehicle used in the after-fracture redundancy design or rating of the bridge. The vehicle and number of lanes loaded can be determined as suggested in Chapter Three.

Dead load may be automatically calculated and applied to the three-dimensional computer model when using software requiring member areas and unit weights. Because the concrete deck is not included in the computer model for a noncomposite bridge, the dead load of the deck should be applied separately to the girder top flanges as joint loads. For composite bridges the deck is represented by plane stress elements as explained previously. In this case the dead load of the deck should also be applied directly to the girder top flanges as joint loads. Any additional dead load of the steel superstructure can also be applied as joint loads. If the computer does not automatically apply the dead loads, all dead loads are applied as joint loads.

Live plus impact loads for the rating vehicle are applied on the girder top flanges as joint loads. Figure 27 shows an example of truck load application. An HS-20 truck configuration is shown in the figure for illustration.

Figure 27(a) shows a cross section at an axle location. The wheel loads are W. The resulting equivalent wheel loads on the unfractured and fractured girders are $(1-\beta)\ W$ and βW respectively, where β is given in the figure.

The locations of the equivalent wheel loads on the fractured and unfractured girders are shown in Figures 27(b) and 27(c). It is recommended that the center of gravity of the loads be placed at the cross section containing the fracture. The figures show the magnitudes of the wheel loads for an HS-20 truck including an impact factor, I. The impact factor to use in a redundancy design or rating is suggested in Chapter Three.

Members and Deflections

The following guidelines are provided for performing a redundancy evaluation based on the reactions, deflections, and stresses or stress resultants obtained from the finite element analysis. Deflections and stresses should be evaluated based on the guidelines in Chapter Three. Reactions are not important for two reasons. First, the distribution of the vertical reactions of a fractured simple span bridge is not significantly different from that of the unfractured bridge. Second, the rating loads are less than the design loads, resulting in smaller than design reactions.

The following guidelines are relevant to checking after-fracture deflections and deck slopes, and comparing with the respective limits established by the bridge engineer: (1) Maximum deflection of the fractured girder is used to determine the deflection-to-span length ratio. (2) Maximum slope of the fractured girder is used as a measure of the maximum longitudinal deck slope. (3) Maximum deflection of the fractured girder is used as a measure of the maximum transverse deck slope.

For redundancy evaluation all members and components should be checked and evaluated in accordance with the guidelines in Chapter Three. Specifically, redundant bracing systems are to be checked and evaluated as described in that chapter.

Axial stress in the elements representing the following members and components is to be checked: (1) unfractured girder top and bottom flanges, (2) fractured girder top flange above the fracture, (3) bottom lateral bracing tension diagonals in the panel containing the fracture, (4) bottom lateral bracing compression diagonals in the panels adjacent to the fracture, (5) top lateral bracing compression diagonals in the panels adjacent to the fracture, (6) cross bracing (or cross truss) top and bottom horizontals adjacent to the fracture, (7) cross bracing (or cross truss) compression diagonals adjacent to the fracture, (8) cross frame top and bottom flanges adjacent to the fracture, (9) cross frame knee or haunch flange adjacent to the fracture, and (10) composite deck above the fracture.

In addition, girder, cross frame, and knee or haunch webs are to be checked for shear and membrane stresses.

All connections are to be checked for their ability to safely resist the applied forces. Redundancy rating and evaluation may be affected by connection strengths if it is not planned to retrofit them.

The results of the finite element analysis and redundancy evaluation, as described above, are used for both redundancy design and redundancy rating as described in the following sections.

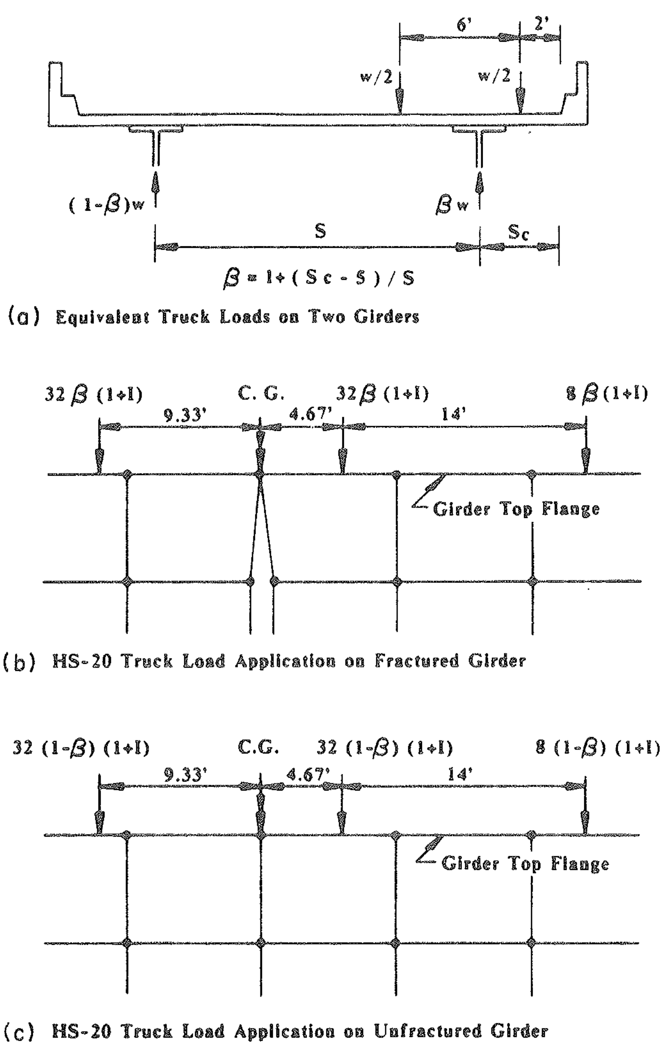


Figure 27. Example of truck load application.

REDUNDANCY DESIGN

The finite element results are used in redundancy design in the following three design situations: (1) the design of an alternate load path to provide redundancy in a new bridge; (2) the design of a new alternate load path to provide redundancy in an existing or rehabilitated bridge; and (3) the retrofit design of an as-built or existing alternate load path to provide redundancy in an existing or rehabilitated bridge.

In these situations the rating vehicle is specified and a new or retrofitted redundant bracing system, together with adequate connections, is provided to assure redundancy.

In redundancy design the following four steps are followed (refer to Chapter C8, Appendix C, for definitions and notation).

Step 1. Check the primary members and the members and connections of the redundant bracing system to assure redundancy according to the provisions of Chapter Three and the concepts in Appendix C. If the allowable stress method is used, all stresses should be less than the allowable stresses, i.e., $(f_D + f_L) \leq f_{\text{all}}$. If the load factor method is used, load effects should not be larger than the member strength reduced by the strength reduction factor, i.e., $\{\gamma_D D + \gamma_L(L+I)\} \leq \phi\, S_u$. Examples of maximum strength, S_u, are P_y, P_{cr}, M_u, and their combinations as used in the AASHTO specification.

Step 2. Check serviceability criteria such as maximum deflection-to-span length ratio (Δ/L), maximum longitudinal girder slope ($\theta_{\text{long.}}$), and maximum deck transverse slope ($\theta_{\text{tran.}}$). Serviceability is checked under service dead and live plus impact loads, not under factored loads. The serviceability limits for dead and total loads are to be determined by the bridge engineer for each bridge, as discussed in Chapter Three.

Step 3. If, after performing steps 1 and/or 2, the structure violates allowable stress, load factor, or serviceability criteria, the members and components including the connections are modified, another finite element analysis is performed, and steps 1 and 2 are repeated before continuing with Step 4. Refer to the retrofit guidelines of Chapter Four.

Step 4. In redundancy design all connections, particularly those of the redundant bracing system, are to be checked and redesigned, if necessary, to carry the redundant loads.

REDUNDANCY RATING

The finite element results are used in redundancy rating to analyze an as-built or existing alternate load path to determine the redundancy rating of an existing bridge. In this situation the structure is given and a redundancy rating factor, RRF, is calculated in terms of a specific rating vehicle.

In redundancy rating the following four steps are followed (refer to Chapter C8 of appendix C for definitions and notation).

Step 1. Rate all primary and alternate load path members using the following equations which are discussed in Appendix C.

For the allowable stress method:

$$\text{RRF} = \frac{f_{\text{all}} - f_D}{f_L} \tag{5}$$

For the load factor method:

$$\text{RRF} = \frac{\phi S_u - \gamma_D D}{\gamma_L(L + I)} \tag{6}$$

Examples of maximum strength, S_u, are P_y, P_{cr}, M_u, and their combinations as used in the AASHTO specifications.

Step 2. All connections, particularly those of the redundant bracing system are to be checked for their capacity to resist the applied forces. The RRF can reflect the capacity of the connections, or the connections can be retrofitted so that the RRF reflects the capacity of the members and components of the redundant bracing system.

Step 3. The RRF for the structure is determined as the minimum value of the RRF obtained in steps 1 and 2. For example, a RRF = 0.8 for HS-20 truck loading provides a live load rating of HS-16.

Step 4. Check serviceability criteria such as maximum deflection-to-span length ratio (Δ/L), maximum longitudinal girder slope ($\theta_{\text{long.}}$), and maximum deck transverse slope ($\theta_{\text{tran.}}$). Serviceability is checked under service dead and live plus impact loads, not factored loads. The total load deflection, Δ, is calculated as

$$\Delta = \Delta_D + \text{RRF}\ \Delta_L \tag{7}$$

where Δ_D and Δ_L are the dead and live plus impact load deflections determined in the finite element analysis.

CHAPTER SIX

CONCLUSIONS AND SUGGESTED RESEARCH

CONCLUSIONS

The following conclusions are a result of the investigation reported herein:

1. The most significant conclusion is that a new or existing deck-type steel two-girder highway bridge, which is provided with a properly designed redundant bracing system, consisting of top and bottom laterals plus diaphragms, exhibits considerable redundancy after the near full depth fracture of one of the two main girders.
2. Another significant conclusion is that an existing deck-type two-girder bridge may also exhibit after-fracture redundancy providing the as-built bracing system is properly configured and located, even though the bracing was not originally designed for redundancy.
3. A two-girder deck-type bridge with a fractured girder does not behave the way most bridge engineers think it does. The fractured bridge behaves instead like a "psuedo space truss" and not like the oversimplified behavioral model assumed in design.
4. A through-girder bridge which cannot contain or be provided with a redundant bracing system can still be made redundant using tension cables, rods, or shapes or by providing redundant girders alongside the existing girders.
5. A deck-type two-girder bridge may also be made redundant using tension rods, cables, or shapes, in lieu of a redundant bracing system.

6. Redundancy can be provided to simple span or continuous, composite, or noncomposite, new or existing steel highway bridges.
7. A significant conclusion is that a new redundancy rating is required, similar to the AASHTO inventory and operating ratings. A redundancy rating is calculated using either the allowable stress or load factor methods and provides an AASHTO H or HS rating of a fractured two-girder bridge.
8. Redundancy design and rating can be performed either by means of procedures and equations developed from simple three-dimensional analytical models or by finite element modeling and computer analysis of the as-built three-dimensional structure containing a properly configured and located bracing system.
9. The definition of redundancy as presently defined and used in Art. 10.3 of the AASHTO *Standard Specifications for Highway Bridges* should be changed to stress the after-fracture serviceability of the bridge rather than collapse.
10. The redundant system should be fail safe. That is, the members and components of the redundant alternate load path should not be part of the primary structural system, subjected to live load stress range prior to girder fracture so that fatigue cracking and fracture of the redundant system is not likely to occur prior to girder fracture.
11. A redundant bracing system is fail safe. Tension cables, rods, or shapes are not necessarily fail safe.
12. Guidelines can be formulated to assist in redundancy design and rating of two-girder steel bridges as part of the effort needed to establish bridge inspection, repair, retrofit, rehabilitation, and replacement priorities.

FURTHER RESEARCH—GENERAL

Fisher and Yen (*69*) in response to a request from the Pennsylvania Department of Transportation (PADOT) prepared a list showing the susceptibility of steel bridge superstructures to complete failure because of fatigue cracking and brittle fracture. This list was included with Ref. 62 and is shown in Appendix E. The list is based on fatigue and fracture of actual bridges, some of which experienced collapse (Mianus River and Point Pleasant ("Silver") Bridge). The list includes bridge types and configurations that should be of concern from the point of view of redundancy.

The investigation reported herein has resulted in guidelines for the redundancy design and rating of the two-girder bridges in items 6 and 7 of the list in Appendix E. This investigation confirms the statement made on redundancy for the bridge in item 8, of Appendix E.

The Bridge Upgrade Program Committee of the New York State Department of Transportation (NYSDOT) has prepared a similar, but expanded, list of steel bridge superstructures susceptible to complete failure resulting from fatigue cracking and brittle fracture (*70*). However, the NYSDOT list attempts to rank the bridges as to the degree of redundancy available. This list is shown in Appendix F. The ranking is approximate, as is stated in Appendix F.

In the NYSDOT list, two-girder bridges are considered to have no redundancy and are ranked 4-2 and 5. Although this ranking, based on the conditions stated, is conservative, the investigation reported herein suggests that not all two-girder bridges are nonredundant. In addition, this investigation suggests that redundancy can be practically and economically provided to all new and many existing two-girder bridges.

It is evident from Appendixes E and F that there are many types of steel bridge superstructures, each with its own degree of susceptibility to complete failure because of fatigue cracking and brittle fracture.

It is important to note that the degree of susceptibility and the approximate ranking in Appendixes E and F have been established on the concept of "complete failure," presumably "collapse." This investigation has pointed out that, although collapse is an important failure condition, it may not be the most important failure condition for most bridges. Most of the bridges listed in Appendix F, for example, are not likely to collapse, but may still fail because of the bridge deck not remaining serviceable following a fracture. Redundancy ranking or classification of steel bridge superstructures should therefore be based on the concept of after-fracture serviceability of the deck where collapse is simply the worst case of bridge deck serviceability.

It is suggested that further research be conducted to establish the relative redundancy of steel bridges based on the relative serviceability of the bridge deck for heavy vehicles traveling at normal highway speeds. As discussed in this report vehicles attempting to cross the bridge do not necessarily suspect that fracture has occurred. For each bridge type the particular fracture conditions which establish whether or not the deck remains serviceable should be identified. The following examples will illustrate this point.

Fracture of a Stringer. Because the fractured stringer is oriented parallel to the direction of traffic, the deck deflection is confined to the width between the adjacent nonfractured stringers. If the deck collapses over a single stringer, vehicles traveling within this width are endangered, others are not. In this case the bridge should likely be classified as nonredundant. If the deck deflects over the fractured stringer but the deflection is not too large, vehicles can cross safely. In this case, the bridge might be classified as redundant. Further research should establish the conditions which define the boundary between redundancy and nonredundancy in this situation.

Fracture of One Girder in a Multiple Girder Bridge. This condition is similar to the fracture of a stringer discussed above. However, in this case, because the span is longer, larger deflection of the deck over the fractured girder and between adjacent girders could be tolerated for the bridge to remain redundant. The research suggested above, under "Fracture of a Stringer," should also consider this case.

Fracture of a Floor Beam. A floor beam is transverse to the traffic and intercepts all traffic between the adjacent girders or trusses. Collapse of the deck over the fractured floor beam would result in a nonredundant bridge. However, if the deck deflection over the fractured girder allows heavy vehicles to cross safely, the bridge remains redundant. Research is needed to define the boundary between redundant and nonredundant in this situation.

Fracture of One Girder of a Two-Girder Bridge. As shown in this report, as-built bridges may or may not be redundant depending on the existence and configuration of the AASHTO bracing systems. All new bridges and retrofitted existing bridges can be designed to provide redundancy and serviceability of the deck. Further analytical and experimental research is needed, however, as outlined in the next section.

Suspended Spans. Failure of one of the two hanger plates (by corrosion pack-out) at one of the four pin and hanger supports does not automatically lead to collapse, as was demonstrated by the Mianus River bridge. It is believed that the suspended span continued in service for about a year after one of the two hanger plates failed. The pin at this support was capable of supporting the adjacent hanger plate. It was not until the adjacent plate failed that the suspended span collapsed even though it was still supported at three corners. Even though the Mianus River bridge showed that a suspended span is quite redundant the real problem was that the span did not give sufficient warning during the year after the hanger plate failed that collapse would eventually occur. In effect, the span remained too serviceable. Further research is needed on this aspect of redundancy.

Bearing Failure. Bearings can fail by tipping over resulting in deflection of the deck. This is not likely to be serious at interior supports of continuous bridges. However, at the abutments of continuous spans and for simple spans deflection of the deck relative to the adjacent span or approach span may be large enough to result in a serious accident. Research is needed to define the boundary between redundancy and nonredundancy in this situation.

Deck-Truss Type Steel Bridges. A deck-truss highway bridge usually contains two parallel trusses together with top and bottom laterals and cross bracing. Fracture of the tension chord (probably at a connection) of one truss results in after-fracture behavior similar to that for two-girder bridges. The results of this investigation can be extended with little effort to such bridges. Additional research is necessary to produce guidelines applicable to the fracture of a tension diagonal in one of the trusses. Following fracture of a tension chord or a tension diagonal member, the truss bridge carries dead and live loads as a space truss as described in Chapter Three. Further research is needed to define redundancy in terms of strength and serviceability.

The foregoing examples illustrate the role of deck serviceability in classifying the after-fracture redundancy of steel highway bridges. They also suggest the approach needed in further redundancy research. Although collapse is an important consideration in redundancy, serviceability is probably equally or more important, because the bridge must remain serviceable after fracture occurs in order to reduce the risk to the traveling public.

For each bridge type, suitable after-fracture three-dimensional analytical models need to be established for use in developing simplified redundancy design and rating procedures and equations or for computer modeling and analysis as was demonstrated in this investigation. For each bridge type the tolerable after-fracture deflection of the deck is to be established and procedures developed for computing total load and dead load deflection ratios as was done in this investigation. Although the guidelines presented in this report suggest live load levels, allowable stresses, and load factors for the redundancy design and rating of two-girder bridges, they also need to be studied and recommended for other bridge types as well.

FURTHER RESEARCH—TWO-GIRDER BRIDGES

To complete the investigation into the after-fracture redundancy of two-girder steel highway bridges and the preparation of additional guidelines, the following analytical and experimental research is suggested.

Analytical

1. Establish the shear capacity remaining at the fracture. Experience indicates that the fracture in an existing simple span girder usually occurs near midspan and stops short of the compression flange. With a redundant alternate load path provided to the bridge, the additional elastic restraint should result in a shorter fracture length because of compliance. It is necessary to establish a relationship between compliance and fracture length, especially for continuous girders where a fracture near an interior support is in a region of high shear.
2. The investigation reported herein assumed specific configurations of the redundant bracing system, such as X-type top and bottom laterals. These were selected because they were practical and easy to use in formulating redundancy design and rating procedures, equations, and guidelines. Other configurations are possible that may also be practical as well as optimal.
3. The investigation reported herein relied on a computer study of a specific simple-span bridge with variations to establish the parameters needed to calculate the variation in stress in the bottom lateral diagonals between the fracture and the support. This variation should be studied further for a wider selection of two-girder bridges.
4. Continuous two-girder bridges need additional study. The requirements of the redundant bracing system in the negative moment regions, although established in this report, need further verification.
5. Application of the results of the investigation reported herein to skewed two-girder bridges should be studied.
6. After-fracture redundancy of horizontally curved steel two-girder bridges should be investigated. The diaphragms in curved bridges are already proportioned for the resulting torsional stresses. Additional torsional stresses are introduced as a result of girder fracture. The approach used in this investigation could be extended to curved bridges. The popular V-Load method used in design of curved girder bridges might be modified to provide the required redundancy to curved two-girder bridges.

Experimental

1. Field testing of an existing unfractured two-girder steel bridge, which is closed and scheduled for replacement, could be performed to provide experimental verification of redundancy design and rating procedures. Depending on conditions below the bridge, one of the girders could be shored. An artificial fracture would be introduced, say, at midspan. The existing bracing system would be retrofitted to the guidelines for a redundant bracing system. The shoring would be lowered a few inches. Strains would be measured at strategic locations under dead load and specific live loads. The results would provide needed verification of the analytical research.
2. As-built two-girder bridges which experience near full depth fracture of a girder, but do not collapse, should be studied prior to making repairs or replacing the superstructure. Much can be learned by examining all the members and components of an actual bridge failure.
3. Laboratory tests can also be conducted on simulated fractured two-girder steel bridges. Testing facilities, such as the

new large facility just completed at Lehigh University for the Center for Advanced Technology for Large Structural Systems (ATLSS), are large enough to provide span lengths up to nearly 100 ft, and loads which simulate AASHTO HS truck loading. The large-scale test bridge can be designed for multiple tests where alternate redundant load paths would be examined for their ability to transfer after-fracture loads and for their effect on deflection. The girders can be designed in such a way that a simulated fracture can be introduced at any one of several strategic locations along the span.

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APPENDIX A

TWO-GIRDER STEEL BRIDGE DAMAGE - CASE STUDIES

1. CANOE CREEK BRIDGE (Ref's. A-1 and A-2)

Description of Structure

The Canoe Creek bridge is located on I-80 in Clarion County, Pennsylvania. The bridge consists of two separate structures, one supporting eastbound traffic and the other, westbound traffic. References A-1 and A-2 describe studies of the westbound bridge, which consists of a five-span continuous two-girder structure, as shown in Fig. A-1(a). The continuous girders are haunched over the piers. The girder depth at the piers is 14 ft in all spans and the web thickness is 0.5 in. The girder depth at midspan is 8 ft and the web thickness is 0.375 in.

The cross section at midspan, and at a pier is shown in Fig. A-1(b). The floor beams are built-up members framing into the girders. The two end spans of the structure have a floor beam spacing of 23.5 ft, whereas for the interior spans this spacing is either 23.33 ft. or 23 ft. The floor beam webs are bolted to connection plates, which are fillet welded to the girder webs. The connection plates are also fillet welded to the girder compression flanges and cut short of the girder tension flanges. Bottom lateral bracing diagonals (ST7x39) are connected to the floor beams, as well as to the girders using gusset plates at each connection plate. The level of the gusset plates varies depending on the girder depth, as shown in the figure. The gusset plates are bolted to gusset plate tabs. The gusset plate tabs are fillet welded to the girder web.

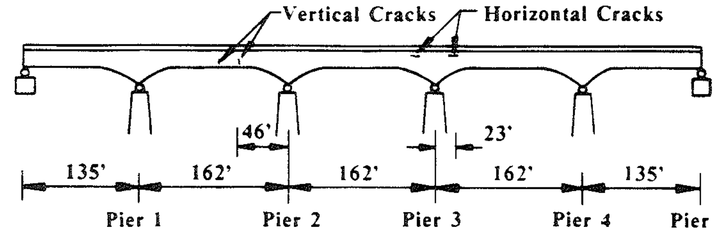
All steel is ASTM A36 mild carbon steel. The noncomposite reinforced concrete deck is supported by stringers (W21x55) and the two longitudinal girders. The top flanges of the girders and stringers are cast into the concrete deck.

Description of Fatigue Damage

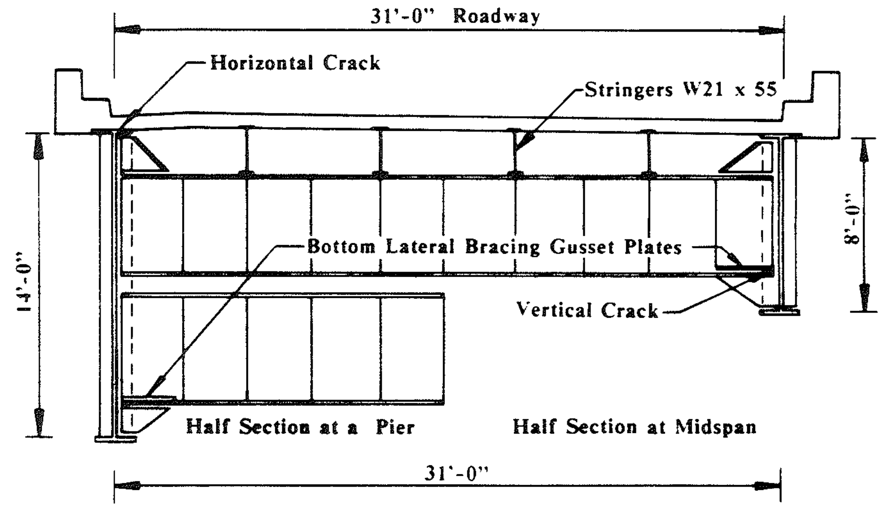
The Canoe Creek Bridge was built during the 1960's. In 1983, horizontal cracking was discovered in the gap between the web-to-top flange fillet weld and the cut short floor beam connection plates in the negative moment regions of the westbound structure as shown in the figure. Three-quarter inch holes were drilled at the crack tips. Subsequent site inspections revealed that the fatigue cracks had reinitiated from the drilled holes. Additional holes were drilled at the new crack tips in an attempt to arrest the fatigue crack growth. No cracks were found above the floor beam connection plates at the pier locations.

In October of 1984, inspection of the lateral bracing gusset plates revealed three types of fatigue crack indications. The first occurred at the end of the gusset plate tabs at the weld toe of the tab. The second occurred in the small horizontal gaps between the vertical floor beams connection plate and the lateral gusset plate tabs. The third type of crack indication was related to the horizontal web gap, but occurred on the outside surface of the girder web along every floor beam connection plate at the level of the lateral gusset plates.

A number of retrofit arrangements were applied to the floor beam connection plates and the gusset plate regions. Drilled holes alone at crack tips at the top end of floor beam connection plates were not effective arresting the cracks. The gusset plates connecting the bottom lateral diagonals to the girder were modified. The horizontal gaps between the gussets and the connection plates were increased from 1 to 2 inches. Some floor beam connection plates were attached directly to the girder top flange in the negative moment region.



(a) Elevation



(b) Cross Section

Figure A-1 Elevation and Cross Section – Canoe Creek Bridge

2. DEKORRA BRIDGE

Description of Structure

The Dekorra Bridge is located in the City of Madison, Wisconsin. The bridge consists of a three-span continuous welded two-girder structure as shown in Fig. A-2(a). Figure A-2(b) shows a cross section of the structure. The plate girders, 21 feet apart, are connected to W21x62 floor beams and cross trusses. Each floor beam supports three W18x50 stringers. All spans have conventional bottom lateral bracing at the level of the bottom flanges of the girders. The bottom lateral bracing (ST7x21.5) is connected to the girder bottom flanges every 20 ft. using gusset plates welded to the top of the flanges as shown in the figure.

Description of Fatigue Damage

The Dekorra Bridge was constructed arround 1965. Vertical fractures were discovered 10 years after construction, in the interior and one end span as shown in Fig. A-2(a). In the interior span cracks occurred first in the longitudinal weld under a gusset plate, then proceeded across the bottom flange along a transverse weld. In the end span the crack occurred at a vertical web butt weld.

In an attempt to stop further fatigue cracking of the flange in the interior span, hangers were installed midway between the girders to reduce vertical vibration of the bottom lateral bracing. However, a number of the hangers fractured in a relatively short time and fatigue cracking of the girder continued which eventually led to a substantial fracture.

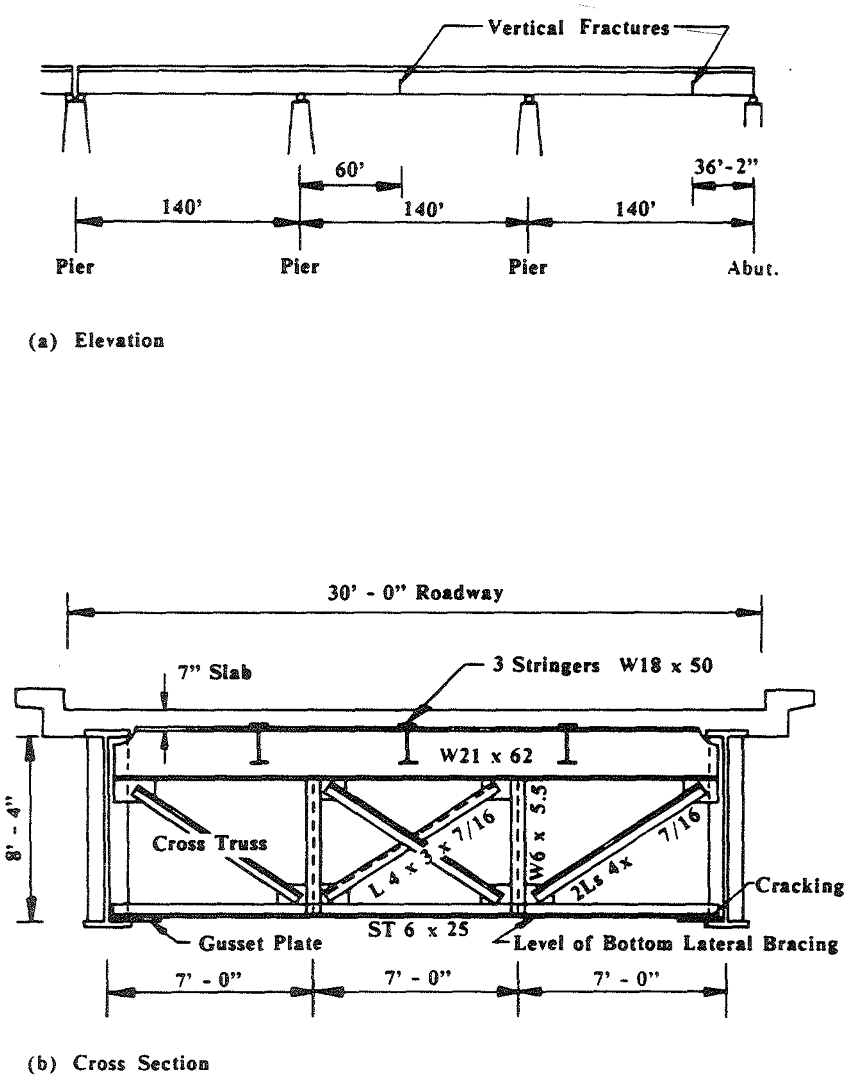


Figure A-2 Elevation and Cross Section – Dekorra Bridge

3. DES MOINES BRIDGE (Ref. A-3)

Description of Structue

The Des Moines (Polk County) Bridge carries east- and westbound traffic over the Chicago Rock Island and Pacific Railroad tracks and the East Four Mile Creek on Route 163. The bridge is located near Des Moines, Iowa. The east- and westbound bridges are two similar but separate structures. The bridge consists of a 37 degree skewed, five-span continuous two-girder structure as shown in Fig. A-3. The two girders, 26 ft. apart, are connected to plate girder floor beams. The floor beams above the piers frame into connection plates located nine inches from the bearing stiffeners. The connection plates are not welded to the top flange of the girders. Each floor beam supports two W18x45 stringers. The bridges were designed to carry two lanes of H20-S16 live loading according to the 1961 AASHO Specifications and 1960 Iowa-DOT Standard Specifications. All members were fabricated from A36 steel.

Description of Fatigue Damage

The Des Moines Bridge was constructed in 1962 and opened to traffic in 1963. Fatigue cracks were discovered in the girder web on September 17, 1979. At five locations, three in one girder and two the opposite girder, there are 3 to 6 in. long horizontal cracks in the girder web along the edge of the web to top flange weld. An additional 2 in. diagonal crack occurs at three locations. At all five locations there are several 1 to 1.5 in. long vertical cracks at the top of the connection plates.

The bridge was retrofitted in 1980. In the negative moment regions away from the piers the top portion of the connection plate was removed by flame cutting, and the remaining weld removed by grinding. At the ends of the web cracks 3/4 in. diameter holes were then drilled in order to isolate the cracks and ensure no further growth. In the negative moment regions at the piers the bearing stiffeners were bolted to the girder top flange, using a clip angle on each side of the bearing stiffener.

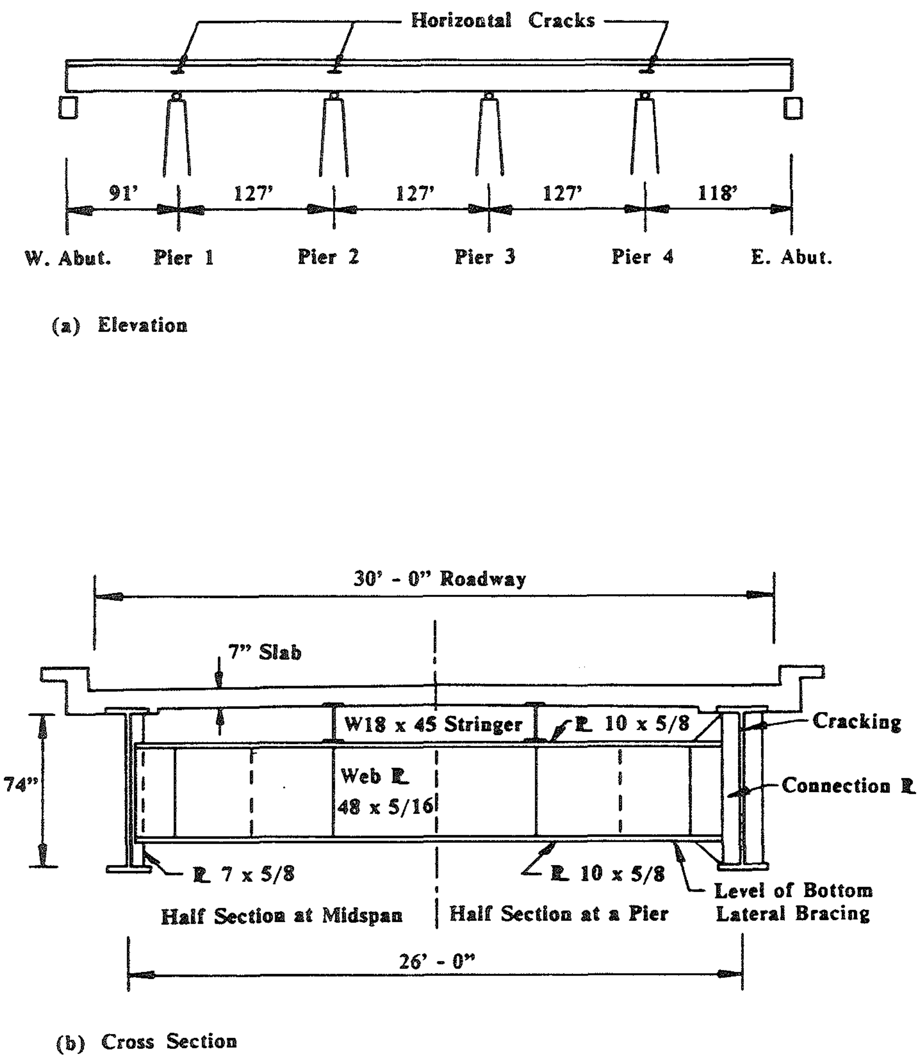


Figure A-3 Elevation and Cross Section – Des Moines bridge

4. DES PLAINES RIVER BRIDGE

Description of Structure

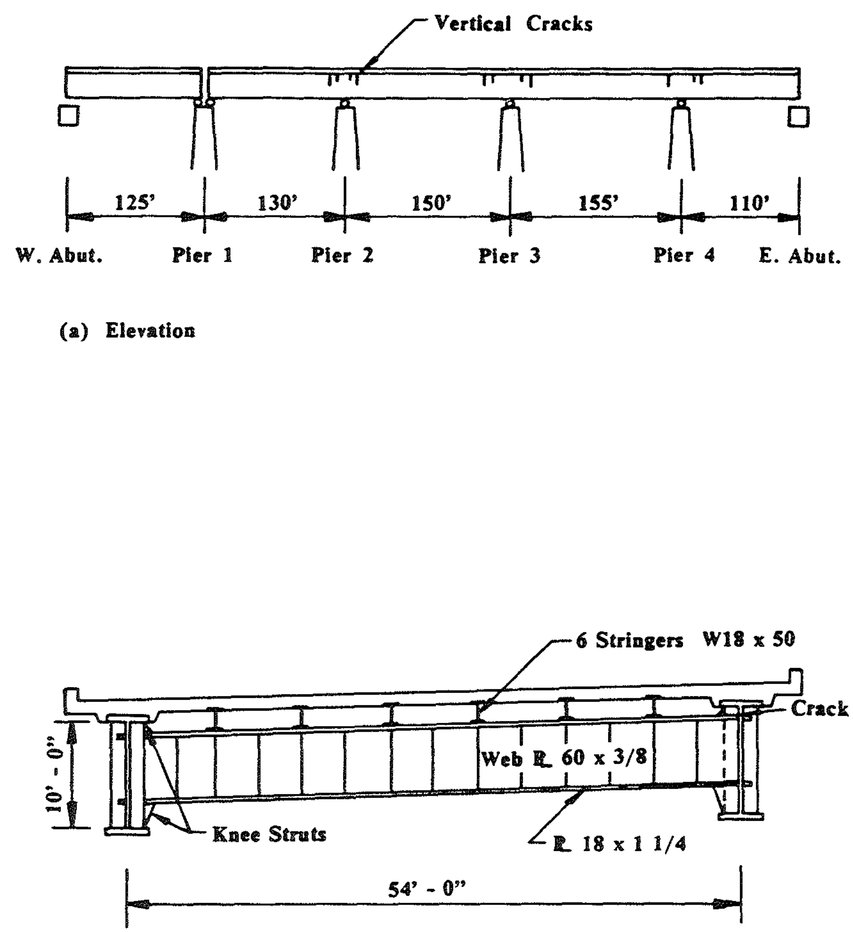
The Des Plaines River Bridge is located on I-55 over the Des Plaines River in Cook County, Illinois. The bridge is a four-span continuous two-girder structure as shown in Fig. A-4(a). The skew varies from 50 to 65 degrees. Figure A-4(b) shows a cross section of the structure. The plate girder floor beams are spaced at 20 ft, and support six W18x50 stringers. The girder webs are 120" x 7/16", and the flanges vary from 30" x 1-1/4" to 30" x 3-1/2". Floor beam webs are 60" x 3/8" with knee struts to the bottom and top of the girder connection plates as shown in the figure.

Description of Fatigue Damage

The Des Plaines River Bridge was opened in the Fall of 1964. Twenty vertical web cracks in the negative moment regions of the girders were reported in August, 1975. They were located at the upper ends of the floor beam connection plates. By Feburary 1977 some of the cracks had propagated downward along the web welds or into the web base metal away from the connection plate as much as 2–5/8 in.

It appears that differential deflection of the skewed bridge girders induces out-of-plane bending near the piers resulting in high stress ranges at the upper ends of the floor beam to web connection plates.

Temporary repairs were performed in 1975. In an attempt to stop cracking, 1/2 in. holes were drilled at the crack tips. Permanent repairs were performed in Feburary 1977. The permanent repairs included installation of reinforcement plates to resist twisting of the web to flange connections. No additional crack growth was observed during an inspection in May 1981.



(a) Elevation

(b) Cross Section

Figure A-4 Elevation and Cross Section – Des Plaines River Bridge

5. DRESBACH BRIDGE

Description of Structure

The Dresbach Bridge is located on I-90 over the Mississippi River in LaCrosse County, Campbell City, Wisconsin. The bridge consists of a four-span continuous two-girder structure as shown in Fig. A-5(a). Figure A-5(b) shows a cross section of the structure. The haunched welded plate girders are spaced 24 ft. apart. The girders are connected to floor beams and K-type cross bracing as shown in the figure.

Description of Fatigue Damage

The Dresbach Bridge was opened in the 1960's. One of the girders experienced a vertical web crack about 17 in. long that originated in a gusset plate connecting the cross bracing to the floor beam connection plate. The gusset plate is located about 12 in. above the bottom flange. The crack grew both up and down the web from the gusset plate. The downward crack appeared to terminate in the web to bottom flange weld.

In an attempt to stop further cracking holes were drilled at the corners of the gusset plate to connection plate welds. Also two reinforcement plates were bolted on the girder bottom flange below the crack.

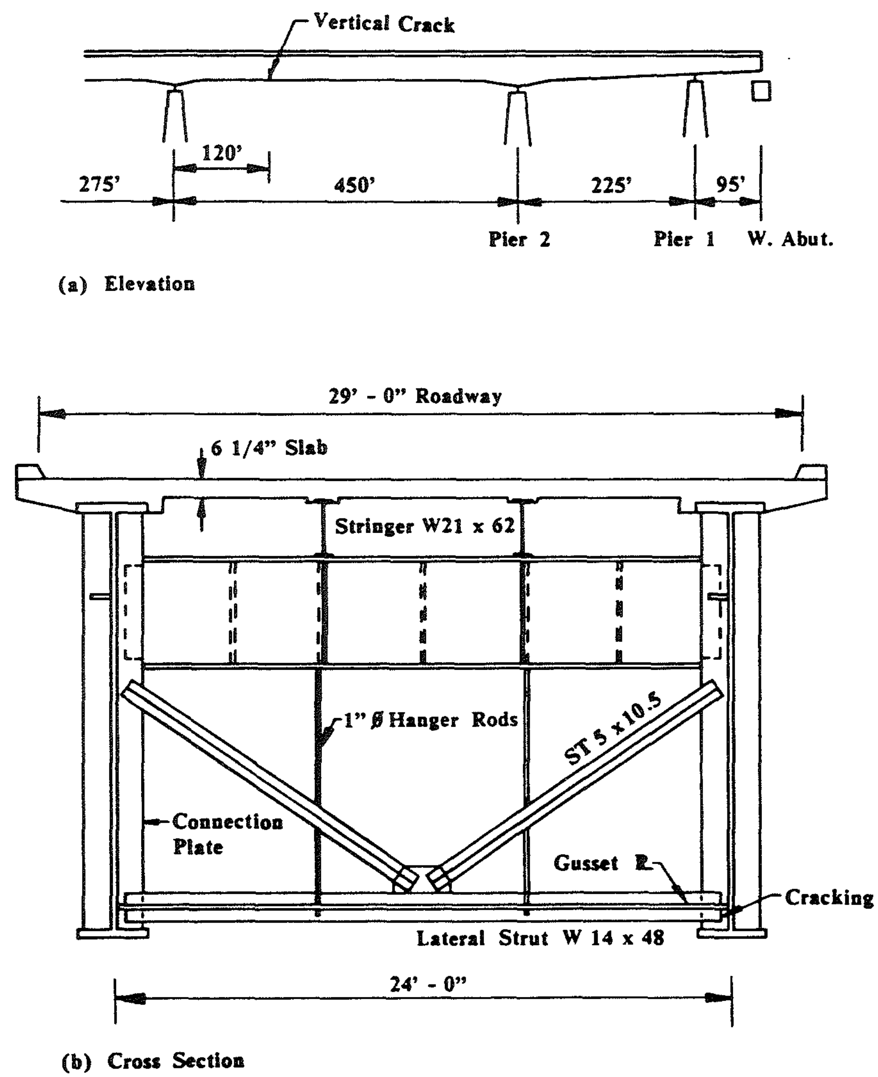


Figure A-5 Elevation and Cross Section – Dresbach Bridge

6. IOWA CITY BRIDGE

Description of Structure

The Iowa City bridge is located on I-80 over US 6 in Iowa City, Johnson County, Iowa. The bridge consists of a four-span continuous welded two-girder structure as shown in Fig. A-6(a). Figure A-6(b) shows a cross section of the structure. The floor beams consist of welded plate girders and support two W18x45 stringers. Bottom lateral bracing extends only from the piers to the first floor beam on each side of the pier, and only from the abutments to the first interior floor beam. In the positive moment regions the floor beam connection plates are welded to the top flange. In the negative moment regions they are welded to the bottom flange. The bottom lateral bracing gusset plate connections are welded to the floor beam connection plates and to the girder webs. The bridge was designed to carry H20-S16-44 live loading in accordance with the 1957 AASHO Specification.

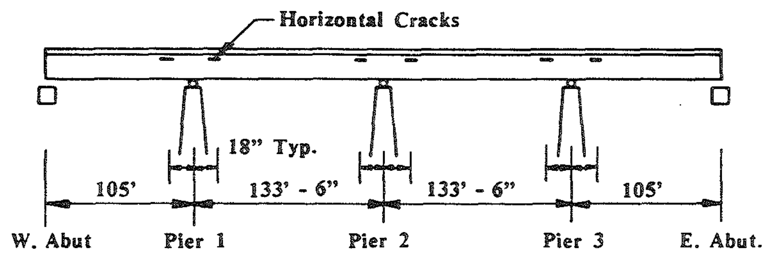
Description of Fatigue Damage

The Iowa City Bridge was opened in the 1960's. Cracks were discovered at 24 locations in August 1979. All cracks occurred in the negative moment regions at the first floor beam each side of a pier and near the top flange of the girder where the floor beam connection plate is welded to the girder web but not to the top flange.

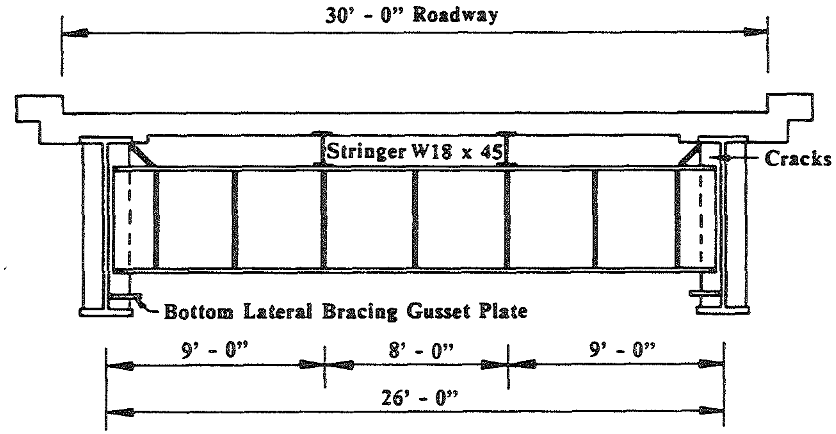
At each of these 24 locations there is a horizontal crack, from 2-1/2 to 10-3/4 in. long, in the girder web along the girder flange to web weld. At about six of these locations there are one or more horizontal cracks in the web located 2 to 3 inches below the girder top flange. At most of these 24 locations there are also vertical cracks in the girder web at the top of the connection plate. Some of these cracks angle outward away from the connection plate.

It appears that these cracks are caused by out of plane deformation of the girder web in the small gaps above the cope in the floor beam connection plate. At these locations the connection plate is not welded to the top flange. Any deformation at this location has to be resisted by out of plane deformation of the web.

In order to prevent these cracks from propagating, a 3/4 in. diameter hole was drilled about 1/2 in. farther along from the observed end of each crack. Permanent repairs at these locations consisted of either removing the upper part of the floor beam connection plate and part of the floor beam or cutting large oblong holes in the web of the girder at these locations.



(a) Elevation



(b) Cross Section

Figure A-6 Elevation and Cross Section – Iowa City Bridge

7. I-70 OVER PATAPSCO RIVER BRIDGE

Description of Structure

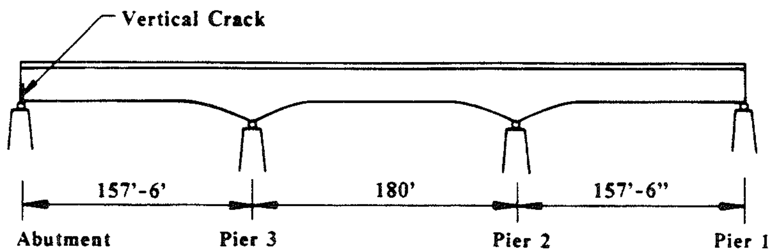
The Patapsco River Bridge carries I-70 northbound over the Patapsco river in Maryland. The bridge consists of a three-span continuous two-girder structure as shown in Fig A-7(a). Figure A-7(b) shows a cross section of the structure. The plate girders are connected by floor beams and K-type cross bracing. The floor beams support eight W21x55 stringers. Interior cross bracing is attached to vertical connection plates while end cross bracing is attached to the bearing stiffeners. The bridge contains X-type bottom lateral bracings with panel lengths of 22'-6". Gusset plates connecting the bottom lateral bracing to the girders are located 6 in. above the girder bottom flange level. The gusset plates contain a 1/2 in. cope at the connections of girder web to bearing stiffener and other floor beam connection plates. The bridge was designed in 1964.

Description of Fatigue Damage

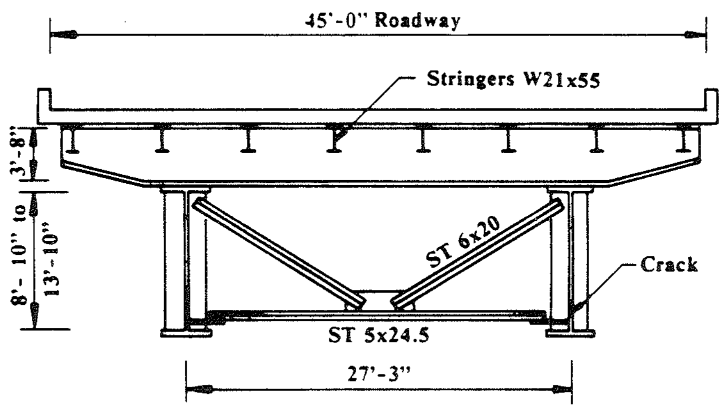
Inspections of the bridge in 1985 revealed fatigue cracking of the girder webs at the bottom lateral bracing gusset plate connections to the girders. Two 4 inch long vertical cracks were discovered at both sides of a bearing stiffener over the abutment as shown in Fig. A-7(a).

A study of the fatigue cracking indicates that the cracks originated from the root of the partial penetration weld connecting the gusset plate to the girder web due to a lack of fusion initial flaw. The web at the 1/2 in. cope is stressed by displacement induced deformations. As the fatigue cracking progresses the crack in the girder web extends outward away from the bearing stiffener.

Repair involved removing the lateral gussets from the web and bolting them to the flange. Also 2 in. diameter holes were drilled at each observed crack tip.



(a) Elevation



(b) Cross Section

Figure A-7 Elevation and Cross Section – I-70 Over Patapsco River Bridge

8. I-79 BACK CHANNEL BRIDGE (Ref. A-4)

Description of Structure

The Back Channel Bridge carries I-79 northbound over the Ohio River backchannel between Moon Township and Neville Island, which is approximately eight miles downstream from Pittsburgh, Pennsylvania. The bridge consists of two parallel three-span continuous haunched two-girder structures carrying the northbound and southbound highways. An elevation of the bridge is shown in Fig. A-8(a). The pairs of main girders under each roadway are framed together at 25 ft. intervals with cross trusses. The cross trusses support longitudinal stringers spaced at 8 ft. centers. The adjacent interior main girders of the two structures are joined together at 50 ft. intervals with cross bracings designed to transfer one half of the maximum live load and impact from one parallel structure to the other.

Description of Fatigue Damage

The I-79 Back Channel Bridge was opened approximately September 1976. A fracture was discovered on January 28, 1977 which extended through the bottom flange and the full depth of the web plate. The bottom flange is 3–1/2 in. x 30 in., and the web is 1/2 in. x 132 in. The fracture ended at the underside of the top flange. The fracture initiated at an electro-slag welded shop splice in the bottom flange at approximately the mid-point of the center span. The fracture was of a brittle nature with little or no apparent plastic deformation of the steel.

Field surveys showed that the crack in the bottom flange opened approximately 1–3/4 in. The concrete bridge deck slab deflected 5 in. below the theoretical elevation over a localized area above the fracture. Although the top flange was cast into the deck it separated from the underside of the concrete deck over a length of approximately 50 ft. and deflected an additional 5/8 in. vertically at failure location. The total after-fracture deflection is approximately 1/750 of the interior span. The undamaged top flange of the fractured girder deflected horizontally outward approximately 3/8 in. with respect to the concrete deck and sheared approximately 30 ft. off the concrete haunch containing the top flange. No cracks were discovered in the concrete deck, parapets and barriers.

The bridge was closed to all traffic for about two months. Repaires were performed from a barge mounted platform moored in the Ohio river. Splice plates were placed across the fractured web and bottom flange, and field drilled and bolted after large jacks were used to pull the fractured bottom flange back to its original position.

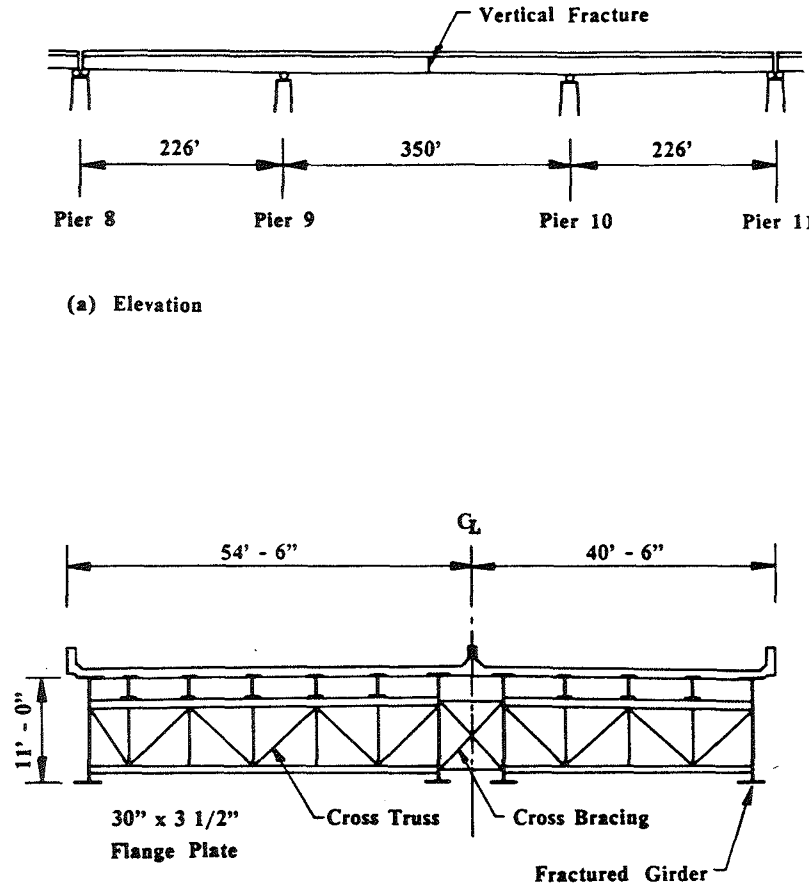


Figure A-8 Elevation and Cross Section – I-79 Back Channel Bridge

9. I-79 BRIDGE 2682 (Ref's. A-5 and A-6)

Description of Structure

Bridge No. 2682 carries I-79 over Big Sandy Creek approximately 34 miles north of Charleston, West Virginia. The bridge consists of two separate structures, one supporting northbound traffic and the other, southbound traffic. The bridge consists of three-span continuous two-girder structure as shown in Fig. A-9(a). The continuous girders are haunched over the piers. The cross section at midspan is shown in Fig. A-9(b). The floor beams are built-up members framing into the girders. The two end spans have a floor beam spacing of 20 ft. The spacing for interior span is 21.66 ft. The floor beams are bolted to connection plates which are fillet welded to the girder. The connection plates are also fillet welded to the girder compression flanges and cut short of the tension flanges. Bottom lateral bracing diagonals (ST6x13.5) are connected to the connection plate as well as to the girder webs by gusset plates. The gusset plates are 6 in. above the girder bottom flange.

Description of fatigue Damage

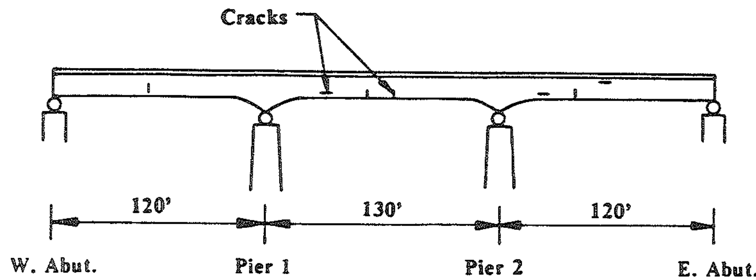
Bridge No. 2682 was opened to traffic in 1972. Inspectors from the West Virginia Department of Highways discovered cracks in the bridge girder in the Spring of 1984. A total of 73 cracks were found. They are almost equally distributed between the northbound and southbound structures. Also the cracks are uniformly distributed along the continuous girders. Some of the larger cracks are shown in Fig. A-9(a). The cracks are located either in the girder webs or in the floor beams. The floor beam cracks are all similar in nature, while the girder web cracks are divided into 3 groups.

The cracks in the floor beam webs occur at the point where the top flange terminates short of the bolted connection to the girder web. Twenty-three horizontal cracks of this type were found. These cracks propagated along the toe of the top flange-to-web welds.

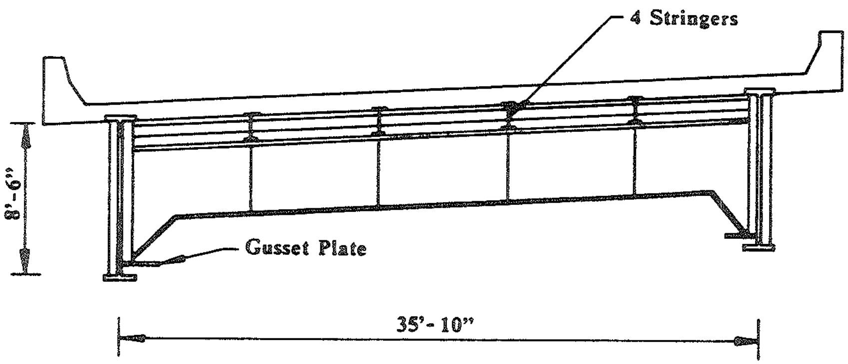
The first group of girder web cracks is comprised of 31 cracks which occur at the lateral gusset plate connection to the girder web. These cracks are oriented vertically in the web gap between the floor beam connection plate and the bottom lateral gusset plate which is coped around the connection plate. The largest crack is 4 inch long.

The second group of girder web cracks includes 16 cracks which typically occur in the gap between a cut-short stiffener and either the top or bottom flange of the girder. The cracks were found at the top flange in the haunched sections and at the bottom flange in the constant depth sections. The largest crack is 3 inch long.

The third group of girder web cracks is comprised of horizontal cracks typically located at the floor beam connection to the web. The cracks occur just above the termination of the connection plate-to-web weld near the floor beam top flange and just below the termination of the connection plate-to-web weld near the bottom flange. The largest crack is 3 inch long.



(a) Elevation



(b) Cross Section

Figure A-9 Elevation and Cross Section — I-79 Bridge 2682

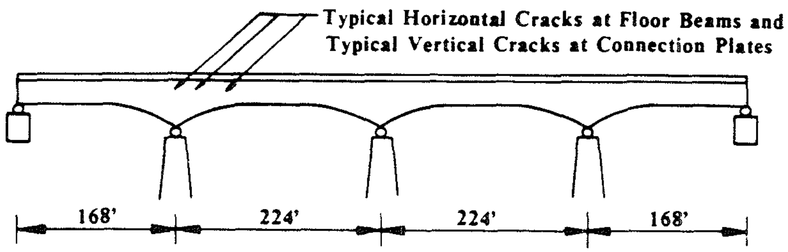
10. I-84 BRIDGE OVER HOUSATONIC RIVER

Description of Structure

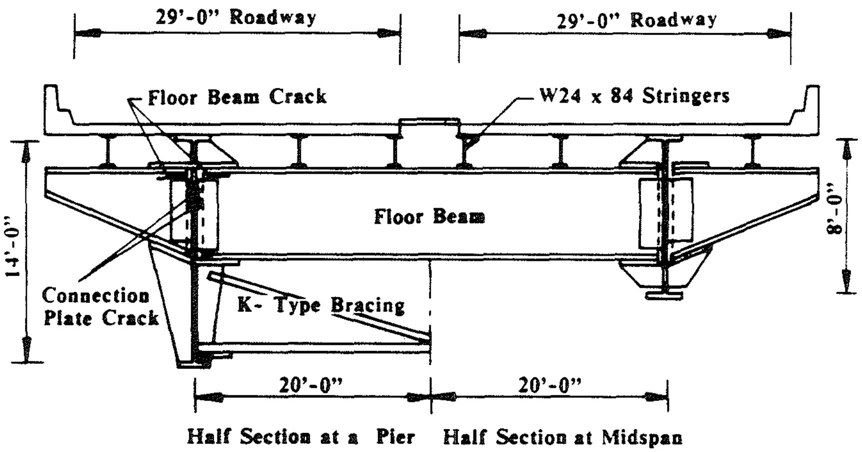
The Housatonic River Bridge carries I-84 westbound over the Housatonic river in Connecticut. The bridge consists of a four-span continuous two-girder structure as shown in Fig. A-10(a). The continuous girders are haunched over the piers and vary in depth. The cross section at midspan and at a pier is shown in Fig. A-10(b). The girder depth at a pier is 14 ft, and 8 ft. at midspan. The plate girders, 40 ft apart, are connected to floor beams above K-type cross bracing at the piers. At midspan there is no cross bracing. The floor beams are built-up members spaced at 28 ft. typically. Floor beam brackets (outriggers) are connected to the girders as shown in Fig. A-10(b). The top flanges of the floor beams and outriggers are connected with tie plates. The web of the floor beams and outriggers connected to the girder web with connection plates. The structure contains X-type bottom lateral bracings with panel lengths of 56 ft. The bottom lateral diagonals vary in size between ST6x18 and ST6x49.5. The bridge is designed to carry H20-S16 live loading.

Description of Fatigue Damage

The Housatonic River Bridge was designed in 1973-1974 in accordance with the 1973 AASHTO Specification and was opened to traffic in 1978. This bridge was inspected in 1984. Numerous vertical cracks were discovered at the toe of the connection plate welds both sides of the girder web. These cracks are up to 7 in. long. Horizontal 14 in. cracks were found at the top of the webs in both the floor beams and outriggers as shown in Fig. A-10(b).



(a) Elevation



(b) Cross Section

Figure A-10 Elevation and Cross Section — I-84 Bridge Over Housatonic River

11. LAFAYETTE STREET BRIDGE (Ref. A-3)

Description of Structure

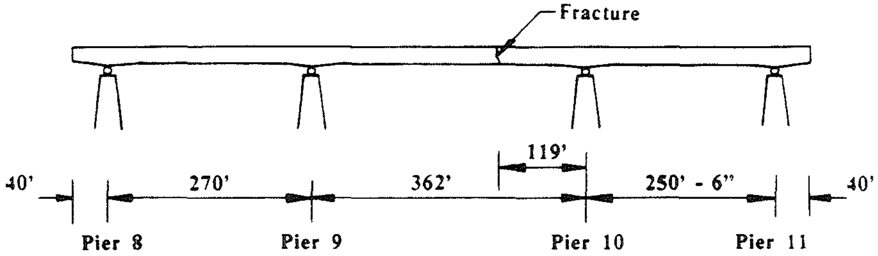
The Lafayette Street Bridge spans the Mississippi River at St. Paul, Minnesota. The main channel crossing consists of two parallel structures composed of two girders extending over three spans as shown in Fig. A-11(a). The transverse cross section consists of two main plate girders connected by transverse floor beams and K-type cross bracing as shown in Fig. A-14(b). The transverse floor beams support two W21x62 stringers. The web and flanges of the main girders were fabricated from ASTM A441 steel.

Description of Fatigue Damage

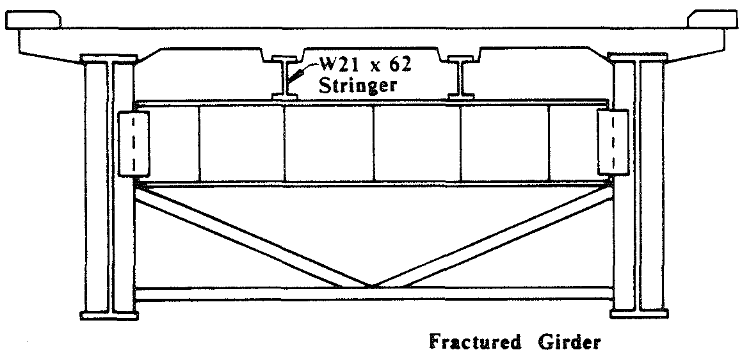
The Lafayette Street Bridge was opened to traffic in November 1968. The southbound lanes were closed between May 20 and October 25, 1974, for repairs to the deck and placement of deck overlays.

On May 7, 1975, a fracture was discovered in the east girder of the southbound structure as shown in Fig. A-11(a). It appears that fatigue crack growth originated in the weld between the gusset plate and the transverse stiffener as a consequence of a large lack of fusion discontinuity in this location. A brittle or cleavage fracture occurred after the fatigue crack propagated into the web through the gusset plate-stiffener weld. The fracture continued upwards and also extended down into the bottom flange fracturing the bottom flange. The fracture in the web was arrested within 7.5 in. of the top flange.

All gusset plates located in the regions of cyclic stress range and tensile stress were retrofitted to prevent other fatigue crack growth into the girder webs. The original fracture was bolt spliced after the cracked girder was jacked up from the adjacent bridge.



(a) Elevation



(b) Cross Section

Figure A-11 Elevation and Cross Section – Lafayette Street Bridge

12. POPLAR STREET BRIDGE (Ref. A-3)

Description of Structure

The Poplar Street Approach Eridges are located on the bank of the Mississippi River in East St. Louis, Illinois. The complex is one of the largest and busiest interchanges in the state of Illinois. The complex consists of several ramps and viaducts with multispan continuous two girder bridges. The majority of the bridges in the complex are on horizontal curves with approximately 1800 ft. radii of curvature. The fatigue damaged bridge is a six-span continuous structure as shown in Fig. A-12(a). Figure A-12(b) shows a cross section of the structure. The two girders are connected by W36x170 floor beams. The floor beams support four W18x14 stringers. The girder webs are generally 0.5 in. thick. The gaps between the girder web to flange fillet welds and the connection plate welds varies from 0.5 in. to 1 in.

Description of Fatigue Damage

The Poplar Street Bridges were designed in 1964, and built between 1967 and 1971. In late 1973 the complex was subjected to the first in-depth inspection. During this inspection several types of cracks as well as web buckling were discovered and reported. The cracks were located at the ends of the continuous main girders as shown in Fig. A-12(a). Additional inspections and field measurements in 1975 showed that cracks also existed in the negative moment regions of the main girders.

The fatigue cracks were located near the abutment and supports or adjacent to the interior piers of the bridges in the complex. These cracks can be grouped into three general types.

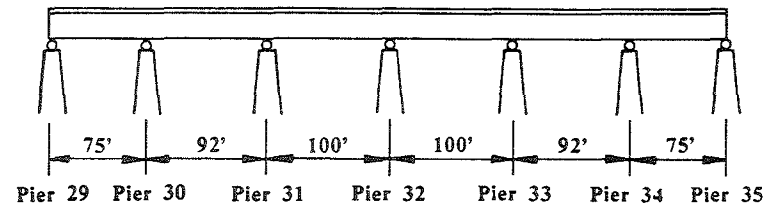
The first type comprises fatigue cracks in the girder web near the support in the gap between the lower end of the floor beam to main girder connection plate and the bottom flange of the main girder.

The second type comprises fatigue cracks in the web in the region between the top end of the floor beam to girder connection plate and the top flange of the main girder.

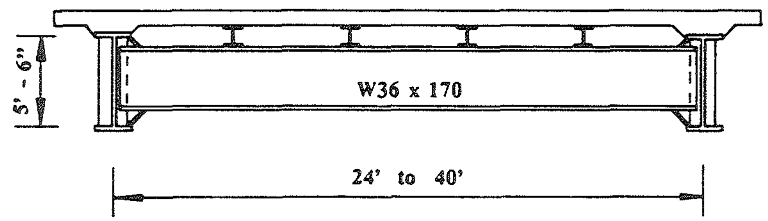
The third type includes fatigue cracks at the ends of bearing stiffeners which were also used as floor beam to girder connection plates.

For repair and corrective action of the girder end cracks, the floorbeam connection plates were welded to the top and bottom flanges in order to prevent relative displacement between the ends of the connection plates and the girder flanges. One-half in. holes were drilled through the web at the ends of the existing web-to-flange connections. Drilled holes were also placed at the ends of the web cracks at the ends of connection plates or stiffener connection plates. The cracks were gouged out and welded with a full-penetration groove weld up to the hole.

For the cracks in the negative moment regions, holes were drilled at each end of the cracks. Holes were drilled near the ends of the crack along the web-flange weld and on each side of the stiffener. This procedure permitted the crack to develop between the holes and thus softened the connection to accommodate the out-of-plane displacements.



(a) Elevation



(b) Cross Section

Figure A-12 Elevation and Cross Section – Poplar Street Bridge

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A-4. Schwendeman, L. P., and Hedgren, A. W., "Bolted Repair of Fractured I-79 Girder" ASCE Spring Convention and Exhibit, Pittsburgh, Pennsylvania, Preprint 3170 (April 24-28, 1978) 11 pp.

A-5. Modjeski and Masters, "Inspection of Bridges No. 2680 and No. 2682 for West Virginia Department of Highways", Modjeski and Masters Consulting Engineers, Harrisburg, Pennsylvania (November 1984) 33 pp.

A-6. Fisher, J. W., Kardara, A., Kaufmann, E. J., Lee, J. J., Pense, A. W., and Yen, B. T., "Final Report on Cracking of I-79 Bridges 2680 and 2682", Fritz Engineering Laboratory Report No. 501-1(85), Lehigh University (May 1985) 105 pp.

APPENDIX B

PREVIOUS RESEARCH

DETERMINISTIC BASED RESEARCH

1. Sweeney, R.A.P., "Importance of Redundancy in Bridge-Fracture Control", Transportation Research Board, Transportation Research Record 711, (1979) p. 23-29.

Sweeney investigates the importance of redundancy in riveted and welded steel girder bridges (36). It is shown that fatigue and fracture are much more critical problems in welded structures than in riveted structures. This is because riveted structures have an inherent component redundancy and lower rigidity. Therefore, riveted structures tend to be fail-safe while welded structures are generally not component fail safe.

The study concludes that designers, fabricators, and inspectors must ensure that welded structures will not develop large cracks because they don't have the inherent crack stoppers which riveted structures have. This is absolutely critical for nonredundant load-path welded structures. The importance of steel bridge repairs are also discussed. If welded repairs are to be used, it is shown that they must be of American Welding Society (AWS) quality. Otherwise, the weld may destroy the initial component redundancy of the structure. It is concluded that the safe-life approach is an absolute requirement for nonredundant load-path structures.

2. Haaijer, G., Schilling, C. G., and Carskaddan, P.S., "Bridge Design Procedures Based on Performance Requirements", Transportation Research Board, Transportation Research Record 711 (1979) p. 30-33

Haaijer, Schilling, and Carskaddan introduce four new design procedures which deal with redundancy and fatigue more directly (37). These procedures are based on the service load, overload, maximum load, and fail-safe load and close the gap between design and actual conditions. Each design procedure is based on a load level and a limit state for a primary structural performance requirement at that load level.

The study presents an investigation into fail-safe analysis. The structural performance requirement at the fail-safe load is to provide adequate load-carrying capacity when a bridge has one separated component. It is noted that a fail-safe load need only be considered when the design of a member is governed by fatigue. If the design of a bridge were governed by either overload or maximum load, a fail-safe check would not be necessary. However, if the design life of the structure were less than a certain value, say 100 years, a fail-safe analysis would be required. Questions such as: load level, elements to be considered, and acceptable level of damage are introduced. The study concludes that a great deal of research is needed before fail-safe analysis becomes a realistic design tool. As an example, the design life at which the probability of separation becomes significant should be established on the basis of statistical analyses. These new methods call for the use of redundant structures which are more rationally designed.

3. Csagoly, Paul F. and Jaegar, Leslie G., **"Multi-Load-Path Structures For Highway Bridges"**, Tranaportation Research Board, Transportation Research Record 711 (1979) p. 34-39

Csagoly and Jaegar established, by providing proper definitions, a framework of reference for further discussion into the merits of excluding single-load-path structures from future designs (38). Historical background with the following six case studies of bridge collapses or severe damage are presented.

1. Silver Bridge – combination of a three-span chain suspension system and stiffening trusses.
2. Lafayette Street and I-79 Bridges – consist of two welded plate girders 11 ft high and continuous over three spans.
3. Ontario-35 Bridge – consists of four steel girders continuous over three spans.
4. Ontario-33 Bridges – simple span arch truss bridge.
5. Truss Bridges – general discussions including, as an example, Hubby Bridge over the Des Moines river.
6. Excessive Movement of Pier – general discussions.

The six cases show that many existing bridges are unintentionally of the multi-load-path type. The main girders of two bridges, the Lafayette Street Bridge over the Mississippi River in Minneapolis-St. Paul(1974) and the I-79 bridge over the Ohio River(1977), failed due to a combination of brittle fracture and fatigue that originated from incomplete fusion of welds of a wind-bracing gusset plate and an electroslag flange joint, respectively. Both superstructures consist of two welded-plate girders 11 ft. high and continuous over three spans. The plate girders are interconnected by cross frames, wind bracings, and a composite concrete deck. Both failures occured in one girder in the central span, close to midspan. Although in both cases the tension flange and approximately 90 percent of the web fractured, neither bridge collapsed. The failure of each bridge was discovered in time by the respective authorities. They were closed and quickly reconstructed. In preparation for the reconstruction of the I-79 bridge, an extensive study was carried out regarding the distribution of loads following the failure. After several attempts that used traditional simplified methods of analysis, the structure was modeled as a space frame for a STRUDL-type analysis. This analysis clearly indicats that, because of torsional stiffness and longitudinal continuity of the superstructure, a significant redistribution take place that permits the bridge to carry all dead loads with some margin to spare after the failure of one main load-carrying component.

The study also introduces the following key definitions of collapse, component, failure, and multi-load-path structure which are included in the 1979 Ontario Highway Bridge Design (OHBD) Code.

1. Collapse - A major change in the geometry of the structure that renders it unserviceable.
2. Component - A structural element or combination of elements that requires individual design consideration.
3. Failure - A state in which the load-carrying capacity of a component or connection has been exceeded.
4. Multi-load-path structure - A structure in which the failure of a component or connection does not result in the collapse of the structure.

This is the first design specification attempting to deal with the issue of bridge design for redundancy.

The study concludes that a mandatory backup system should be made a part of the design process. The increase in cost for multi-load-path considerations should easily be covered by the cost of bridge replacement as a result of collapse. The cost of the extra design work is only a small fraction of the potential savings. It is claimed that the introduction of compulsory backup systems will reduce the probability of collapse to nearly zero. This is shown with a simple calculation of probability of collapse by comparing the failure of a primary member in single-load-path structure to the failure of both the primary and backup systems in a multi-load-path structure.

4. Heins, C.P. and Hou, C.K., **"Bridge Redundancy: Effects of Bracing"**, Journal of the Structural Division, ASCE, Vol. 106, No. ST6 (June 1980) p. 1364-1367.

Heins and Hou studied the effects of cross bracing (diaphragms) and bottom lateral bracing on bridge redundancy (39). The study focused on two and three-girder bridges where one or both flanges of one of the girders is assumed to be cracked. For the two-

girder bridge only the bottom flange is assumed cracked. The study apparently is conducted only in the elastic range.

It is shown that bracing can effectively reduce the deformations in the girders. The study indicates that the effect of flange cracking on the three-girder bridge is negligible but quite important for the two-girder system. It concludes that if bracing is utilized, the two-girder bridge behaves similar to the three-girder bridge.

5. Heins, C.P. and Kato, H., **"Load Redistribution of Cracked Girders"**, Journal of the Structural Division, ASCE, Vol. 108, No. ST8 (August 1982) p. 1909-1915.

Heins and Kato followed up on the study performed by Heins and Hou with an investigation of load redistribution in cracked girders (40). The study focuses on two-girder bridges where one girder is assumed to be fractured near midspan.

It is concluded that the influence of the bottom lateral bracing on load redistribution is significant. Further, the study concludes that utilization of the secondary members (cross bracing and bottom lateral bracing) effectively creates redundancy in two-girder bridges. Unfortunately, specific design procedures and guidelines for the design of the bracing members to ensure redundancy are absent. This study also appears to have been conducted only in the elastic range.

6. Sangare, M., **"Computer Study of Redundancy of a 3-D Steel Deck Truss Bridge"**, Report Presented in partial fulfillment of MS degree requirements, Lehigh University, Bethlehem, PA, supervised by Prof. J.H. Daniels (May 1983) 69 pp.

Sangare and Daniels conducted a computer study of the redundancy of a steel deck truss bridge (41). Post-elastic member behavior is considered. In the investigation, one of the 340 ft. suspended spans of the Newburgh-Beacon Bridge No.2 over the Hudson River at Newburgh, N.Y. was modeled for finite element analysis.

The bridge, designed by Modjeski and Masters Consulting Engineers, Harrisburg, PA., is a deck type cantilever truss bridge carrying four design traffic lanes supported by two steel trusses. Each truss is 48-ft deep. The trusses are spaced 33-ft apart. Each truss contains ten panels 34-ft in length. In the redundancy investigation the tension (bottom) chord of one truss is assumed to be completely fractured at midspan.

The analytical results (elastic and inelastic ranges) show that although the span would be considered nonredundant by most bridge engineers it carries at least full calculated dead load (load factor of 1.0) plus four lanes of HS20 lane loading (load factor of 1.0) plus AASHTO impact in all four lanes. Even in the fractured condition all members of both main trusses remain elastic. Considerable redundancy is provided by the cross bracing system and top and bottom lateral bracing systems even after many members of these bracing systems have yielded in tension or buckled in compression.

7. 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, ASCE, Vol. 111, No. 12 (December 1985) p. 2517-2531

This paper reviews the state-of-the-art on redundant bridge systems as of 1985. Of the 51 references listed only 8 are dated since 1980 (42). Among the conclusions in the review are the following statements:

1. Little work has been done on quantifying the degree of redundancy that is needed in bridges.
2. It is hoped that further research into structural redundancy in bridge systems will be conducted.
3. Computer speed and available software has made evaluation of redundancy more quantifiable than previously possible.

It is interesting to note in reading this paper, which is generated by several individuals, that their use of the term "redundant" does not appear to be consistent. The early and latter parts of the paper use the term mainly in the context of AASHTO Art. 10.3.1. The middle parts of the paper, those dealing with analysis of redundancy, types of analysis and modeling for analysis, appear to refer to "redundancy" as the excess capacity inherent in a normally designed and undamaged structure. For example, the use of the term 'overload' must refer to the latter definition of redundancy since one would not likely be investigating the overload capacity of a fractured structure if the term overload is used in its normal context to mean over the AASHTO design load. Rather, in a fractured bridge the designer should be content to design for a specified "underload"(ie: under the AASHTO design load) to ensure redundancy as defined in Art. 10.3.1 of AASHTO.

8. Daniels, J.H., Wilson, J.L., and Chen, S.S., **"Redundancy of Simple Span and Two-Span Welded Steel Two-Girder Bridges"**, Fritz Engineering Laboratory Report No. 503.2, Lehigh University (November 1986) 272 pp.

Daniels, Wilson and Chen conduct an investigation into the redundancy of simple span and two-span continuous welded steel two-girder bridges (28). The purpose is to study the behavior of three real two-girder bridge spans to determine if alternate load paths exist, and if so, to suggest preliminary design procedures and guidelines to ensure redundancy of the case study two-girder bridges.

The three bridge spans selected for the investigation are:

1. Simple-span right, 90-ft span, two lane, 32-ft clear roadway.
2. Simple-span skew, 90-ft span, 45 degree skew, two lane, 32-ft clear roadway.
3. Two-span continuous right, two 90-ft span, two lane, 32-ft clear roadway.

All three spans are taken from the Betzwood Bridge carrying LR 10461 over the Schuylkill River and Reading Railroad in Montgomery County, Pennsylvania, and were designed as noncomposite and to HS-20 truck loading. Since no top lateral bracing is provided the deck is connected to both girders so that cross bending of the deck would provide the required transverse stiffness of each span following fracture of one girder.

Upper bound elastic plastic analyses of all three bridges were conducted. Lower bound analyses of the simple span right and two-span bridges were also performed using finite element analyses. Excellent agreement is achieved between the upper and lower bound stability limit loads.

9. Parmelee, R.A. and Sandberg, H.R., **"Redundancy - A Design Objective"**, National Engineering Conference and conference of Operating Personnel Proceedings, New Orleans (May 1987) p. 39-1 to 39-12.

Parmelee and Sandberg presents the design of an actual three span continuous bridge for a given level of redundancy (43). It is decided to use a three girder design. For the study, girder fracture is defined as the placement of a hinge at any point in one of the girders. Redundancy is provided by designing the cross bracing to carry the necessary transverse loads. The typical cross bracing is designed to yield under the application of the redundant load, while functioning normally under service loads. Redundant, or stiffened, cross bracing is placed at the field splices. A computer model shows that redundancy is provided by the interaction of the stiffened cross bracings and the failed girder.

It is concluded that redundancy is more than a question of having three or more main longitudinal members. It is necessary to have "reliable" redundancy. Redundant paths must give visual signs of distress before they fail. This "warning system" is needed so that it is clear that the bridge is in need of repair after the fracture occurs. The study points out the need to be aware of the possibility of failure in members along the redundant path that were not designed to function as they actually did. The designer must investigate weak links along the redundant path that may prevent its use. It is emphasized that criteria need to be established for live load levels, permissible allowable stresses, load factors, deflection limits, and critical fracture scenarios.

10. Seim, Charles, **"Increasing Redundancy of Steel Bridges"**, National Engineering Conference & Conference of Operating Personnel, New Orleans (May 1987) p. 46/1-12

Seim investigates economical ways in which redundancy can be achieved in steel bridges (44). The study includes using parallel structural elements in the form of cables. The placement of cables across critical tension areas such as ties of arches or flanges of girders is suggested. If the steel develops a crack, the stress has the alternate load path of the cables available. The design of the Coushatta Bridge crossing the Red River in Louisiana is presented. This three-span continuous bridge consists of a 40-ft wide concrete deck and is supported by two girders. This structure is considered nonredundant by current AASHTO Specifications. Computer simulation of a tension flange fracture in five different locations was examined. It is shown that the structure would survive carrying one lane of HS20 truck loading for all five fracture scenarios although reinforced cross-frames and lateral bracing were not supplied. If torsionally effective cross frames and bottom lateral bracing system were assumed, safety factor would increased from 1.05 to 1.30 in one of the five cases.

The study concludes that the cost of adding the bracing is far less than the cost of adding a girder. It is emphasized that further research is needed to develop rules and proper factors of safety. Questions such as, "What role does a concrete deck play?" and "What is the most effective way to develop a torsion tube without adding a lot of costly bracing?" are introduced. Also it is suggested that the torsional performance could be improved by narrowing the spacing between girders to obtain more of a square cross-section shape.

1. Galambos, T.V., **"Probabilistic Approaches to the Design of Steel Bridges"**, Transportation Research Board, Transportation Research Record 711 (1979) p. 7-14.

Galambos introduces basic probability concepts with first-order probabilistic principles (45). A safety index equation is demonstrated by using means and coefficients of variation of a resistance factor, R, and a load effect Q. The use of a simple first-order probabilistic method is examined to assess the reliability of the 1977 AASHTO Specifications for the design of steel bridges.

It is demonstrated that the AASHTO LFD specification provides a consistent reliability index but that the AASHTO ASD specification does not. As an example, the safety index according to the calibration for compact steel multi stringer bridges designed by the 1977 AASHTO ASD specification varies from about 3.0 to 5.0. Reliability increases as the dead-to-live load ratio increases. However, Vincent's calibration, which proceeds from nonprobabilistic premises, is excellent and results in a nearly uniform safety index of approximately 3.2 for the entire dead-to-live load range.

The study also investigates load- and resistance-factor design (LRFD) provisions. These provisions use multiple load factors and multiple resistance factors. It is shown that LRFD provisions are the most reliable and economical. Uniform reliability can be achieved through the judicious choice of load and resistance factors. These factors are most easily obtained by calibration and by using one of several ways in which first-order probabilistic principles can be applied.

The study concludes that there is sufficient statistical information available on steel structures to allow a probability-based design method to be developed. It is possible to derive resistance factors for members and connectors used in steel bridges. Furthermore, the probability-based method need not be significantly different in format and design use from the present AASHTO LRFD specification.

2. Gorman, M.R., **"Structural Redundancy"**, 4'th ASCE Specialty Conference on Probabilistic Mechanics and Structural Reliability, Berkeley, Proc. (Jan. 84) p. 45-49.

Gorman investigates the interaction between structural redundancy and system reliability (46). Structural redundancy is the degree of static indeterminacy. Increasing structural redundancy tends to increase the number of members that must fail before the system fails. However, increasing structural redundancy also increases the number of failure modes. The effect of these two different influences on system reliability is examined for a series of optimal trusses with varying structural redundancy.

The study concludes that for the truss examples increasing structural redundancy increases system reliability. The greatest benefit is achieved in going from statically determinate to two or three times indeterminate. It is shown that for highly redundant structures system reliability is only slightly improved, or even slightly reduced.

3. Moses, Fred and Yao, James T. P., **"Safety Evaluation of Buildings and Bridges"**, Symposium on Structural Design, Inspection, and Redundancy, Williamsburg, VA, (November, 1983) 35 pp.

This paper reviews and summarizes the general practices of structural engineers in the design, inspection, and redundancy implementation of buildings and bridges (49). Some examples include steel bridge failures which led to code restrictions on nonredundant configurations including severe safety factor penalties especially for fatigue loading. The paper states that in building design, the trend is towards failure scenarios and progressive damage evaluations. However, the required controls are not often clear to designers and further research on system performance and risks are needed. This results because fail-safe reliability depends on both structure configuration and loading.

Various tools and their applications including failure analysis, risk analysis and evaluation and decision analysis are discussed. More over, the expert system approach and its application to damage assessment are described along with other possible methodologies.

4. Frangopol, Dan M., and Curley, James P., **"Effects of Damage and Redundancy on Structural Reliability"**, Journal of the Structural Division, ASCE, Vol. 113, No. 7 (July 1987) p. 1533-1549.

Frangopol and Curley investigate the effects of damage and redundancy on the reliability of structural systems (50). Their investigation is based on a definition of structural redundancy including both system reliability and damage assessment concepts.

This paper states that there are considerable differences of opinion about the definition of structural redundancy. And four different definitions of structural redundancy are introduced in conjuction with damage assessment using deterministic, probabilistic, and fuzzy interpretations of several measures. By incorporating the definitions of structural redundancy, redundancy in bridges is presented by means of illustrative example. Truss systems and bridges analyzed for different damage scenarios are used to develop the theoretical concepts and to illustrate their practical applications.

To assess the effect of structural damage on system performance, the deterministic safety concept and the probabilistic concept are applied to a redundancy factor. The study reaches the following conclusions: (1) in a deterministic context, the redundancy factor provides a realistic means to evaluate the overall system strength of a damaged structure; and (2) in a probabilistic context, system reliability methods should be used to examine structural behavior of damaged structures beyond single-element failure.

This paper indicates in the conclusions that additional studies of probabilistic measures of redundancy in structural systems are in progress. These studies include: (1)

investigation of load, strength, and damage correlation effects on system redundancy; and (2) application of system redundancy measures to quality assurance and inspection strategies.

5. Rashedi, Reza and Moses, Fred, **"Identification of Failure Modes in System Reliability"**, Journal of the Structural Division, ASCE, Vol. 114, No. 2 (February 1988) p. 292-313

This paper studies the reliability of framed structures assuming loads and component strengths are random variables. An incremental loading method is used for the identification of failure modes with multiple loadings by holding the component strengths and all loads at nominal values except for one of the loadings which will be incremented. In this method the failure of components is assumed variously such as ductile, brittle, and semibrittle. For the brittle component failure, an efficient technique is introduced.

The method is also extended to simultaneous multiple loads, and strategies are established to enumerate significant failure modes of a structure. Incomplete failure modes of structures are also studied, and it is found that incomplete failure modes are path-dependent even for rigid plastic behavior. The relation between the path-dependent failure modes involving the same ductile components are formulated.

This paper shows that the application of system reliability methods can also be extended to quality assurance and inspection strategies. It is shown that the Monte Carlo simulation can be utilized effectively and efficiently to obtain mean and variance of the system failure and, consequently, a system safety index.

APPENDIX C

DEVELOPMENT OF REDUNDANT BRACING SYSTEM REQUIREMENTS

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CHAPTER C1

INTRODUCTION AND RESEARCH APPROACH

OBJECTIVE AND SCOPE

In addition to the two main girders, typical deck type two-girder steel bridges also contain bottom lateral (wind) bracing and diaphragms spaced at regular intervals along the span. Top lateral bracing may or may not be present depending upon the design and whether the bridge is composite or noncomposite. It will be shown that after near full depth fracture of one of the two main girders, after-fracture redundancy can be provided if properly configured top and bottom lateral bracing and diaphragms are present. It will also be shown that the level of redundancy provided in terms of live load capacity is dependent upon the strength and stiffness of the redundant bracing system. Together with the two girders the redundant bracing system provides the after-fracture alternate load path for supporting dead and live loads.

The objective of this appendix is to explain the development of the strength and stiffness requirements of the redundant bracing system. These requirements are formulated in terms of allowable stress, load factor and serviceability (deck deflection and slope) criteria. The bridge design engineer can use these requirements to design a new or retrofit redundant bracing system to ensure redundancy. The bridge inspection engineer can use these requirements to assess the level of redundancy available in existing bridges and to establish inspection, repair, retrofit, rehabilitation and replacement priorities.

This appendix is limited in scope to simple span, noncomposite, deck-type two-girder steel bridges containing a redundant bracing system consisting of properly configured top and bottom lateral bracing and regularly spaced diaphragms of the cross bracing, cross frame or cross truss type. Redundancy is formulated for the case of near full depth fracture of one of the two girders. The probability of both girders fracturing simultaneously or one girder containing two or more simultaneous near full depth fractures is assumed to be extremely low.

Research reported in Ref. C-1 (Note: references follow Chapter C-10) and briefly discussed in Chapter 3 of the main report shows that two-girder continuous span bridges are not necessarily more redundant than simple span bridges. Redundancy design and rating should therefore be based on methods for simple span bridges.

COMPONENTS OF THE REDUNDANT BRACING SYSTEM

The redundant bracing system must develop sufficient torsional strength and stiffness to safely support the dead, live and impact loads and to prevent excessive deck deflections and slopes in order to maintain the after-fracture serviceability of the deck as is discussed in Chapter 3. The redundant bracing system must therefore contain the following three main components which are connected to the fractured and unfractured girders (See Chapter 3):

1. Top lateral bracing at or immediately adjacent to the top flanges of the two girders.
2. Bottom lateral bracing at or immediately adjacent to the bottom flanges of the two girders.
3. Full depth interior and end diaphragms consisting of cross bracing, cross frames or cross trusses.

The three components of the redudant bracing system are shown schematically in Fig. C-1 for a typical noncomposite simple span two-girder bridge. Figure C-2 shows a typical top lateral bracing configuration. It consists of n equal length panels where the

panel length is defined by adjacent diaphragms. The span length is L and the girder spacing is S. The top lateral bracing functions as a horizontal truss and, for the work developed in the Appendix, must contain two chords and web members consisting of transverse (diaphragms) and X-type diagonal members. The truss chords consist of the top flanges of the two girders. For this reason the top lateral bracing must be near enough to the top flanges to allow for efficient and economical connections to transfer forces from the diaphragms to the diagonals of the top lateral bracing.

Similarly, Fig. C-3 shows a typical bottom lateral bracing configuration. Except for the girder fracture, the geometric configurations of the top and bottom lateral bracing are similar. The bottom lateral bracing must also be near enough to the bottom flanges of the girders to allow efficient and economical connections to transfer forces from the girder flanges to the diagonals of the bottom lateral bracing.

Figure C-4 shows typical configurations of top and bottom lateral bracing. The derivations and equations developed in this Appendix apply only to the X-type bracing shown at the top of the figure. Typical configurations of cross bracing, cross frames and cross trusses are shown in Fig. C-5.

Deck-type two-girder bridges exhibit a variety of cross sections. Two examples are shown in Fig. C-6. In Fig. C-6(a) the floor beams rest on top of the girders allowing top lateral bracing to be placed near the top flanges of the girders. In Fig. C-6(b) cross trusses support the interior stringers. For this bridge to exhibit redundancy, as developed in this Appendix, top lateral bracing is required at the top of the girders as shown in the figure, and may have to be installed in an existing bridge.

METHODS OF EVALUATING AND ASSURING REDUNDANCY

Current AASHTO operating and inventory ratings are performed for existing two-girder bridges in which the simplified model used in the design is still applicable for rating (C-2). Except for corrosion damage, limited fatigue cracking, missing rivets, bent flanges, etc., the two girders are still assumed to support the vertical loads. Connectivity of the structural members is the same as that assumed in the design. Thus, assumptions on the distribution of the vertical loads to the two girders are the same for both design and rating, even though the bridge may exhibit corrosion and minor collision damage as well as significant changes in traffic conditions.

A vastly different situation exists as a result of near full depth fracture of one of the two main girders. In this case, the vertical loads are redistributed to the entire three-dimensional structure consisting of the fractured and unfractured girders plus the top and bottom lateral bracing and diaphragms as discussed in Chapter 3. Just as a simplified conservative model can be used to design and rate a two-girder bridge before fracture occurs, another conservative (although somewhat more complex) model can be used to design and rate the redundant bracing system following fracture of a girder (C-1).

In the conservative after-fracture model, the forces released by the fractured girder are assumed to be transmitted through connections of the girder bottom flanges to the bottom lateral bracing diagonals. In effect, the bottom lateral bracing acts as an alternate bottom flange to the fractured girder. The forces in the members of the bottom lateral bracing are then transmitted through the diaphragms and the girder top flanges to the top lateral bracing. The diaphragms and top lateral bracing, plus connections, must provide the required strength and stiffness to develop the forces in the bottom lateral bracing and to prevent excessive deflections and slopes of the bridge deck.

It is shown in the Appendix that the current AASHTO rating levels and methods used to rate unfractured two-girder bridges can be easily modified and extended to provide the after-fracture redundancy rating of existing two-girder bridges or the after-fracture redundancy design of new or existing two-girder bridges.

RESEARCH APPROACH

The research reported in this Appendix has identified the need for an additional redundancy rating level for use when assuming a near full depth fracture of one of the two main girders. For existing bridges a redundancy rating can be performed along with the familiar AASHTO operating and inventory ratings (C-2). The same or different rating vehicles can be used to compute a redundancy rating.

For new and existing bridges the redundancy rating concept can be used to compute the after-fracture strength and stiffness requirements of the top and bottom lateral bracing and diaphragm members in terms of the familiar AASHTO allowable stress and load factor methods (C-2), plus the consideration of after-fracture serviceability of the bridge. These requirements are found by setting the redundancy rating factor (RRF) equal to unity. Thus, the required strength and stiffness of the redundant bracing system is defined in terms of a specified after-fracture live load capacity.

Extensive computer modeling and finite element analyses were performed during both the development and verification phases of this research. Further details are presented in Ref's. C-3 and C-4. Actual two-girder bridges having spans from 100 to 200 ft. were used in this study. Details of these study bridges were obtained from the investigator's research files as well as from plans submitted by the NCHRP Project 12-28(10) Panel members, such as the Catskill Creek, NY, bridge in Ref. C-5.

The research reported in this Appendix considers only near full depth fracture of one of the two main girders. Redundancy rating factors, RRF, are derived for the as-built or existing members of the top and bottom lateral bracing and diaphragms in terms of a user specified rating vehicle. The cross-section area requirement of each member of the redundant bracing system is derived in terms of a user specified rating vehicle by setting the redundancy rating factor, RRF, equal to unity. All derivations are performed for a worst case fracture location along the girder. All derivations consider only an odd number of panels in the span (n=odd). Studies show that for practical span lengths and values of n the results for n=even are nearly the same. Ref's. C-3 and C-4 provide further details of the computer analyses used to verify the derivations presented in this appendix. An important assumption in all of the work presented in this appendix is that all connections are designed or retrofitted to develop the full strength of the connected members.

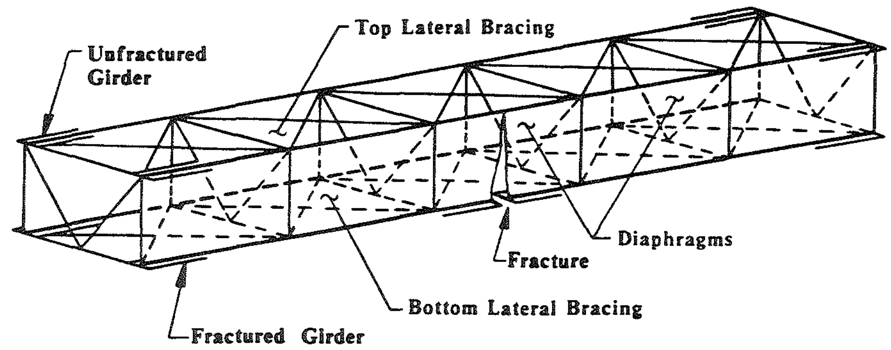


Figure C-1 Components of the Redundant Bracing System

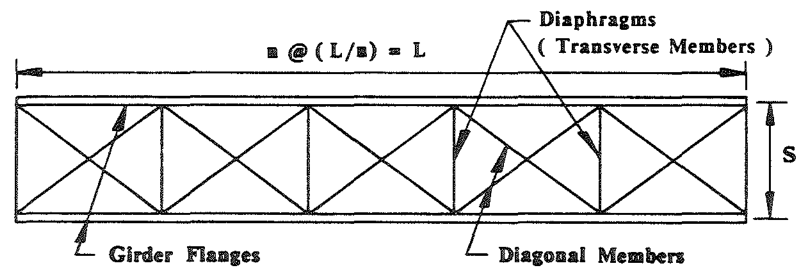


Figure C-2 Typical Top Lateral Bracing

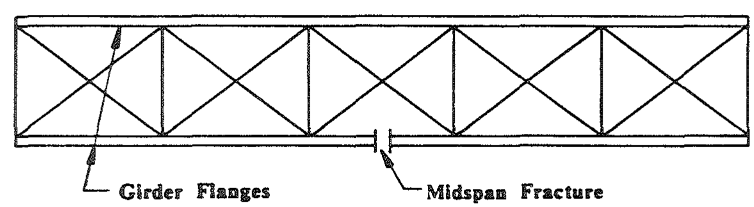


Figure C-3 Typical Bottom Lateral Bracing

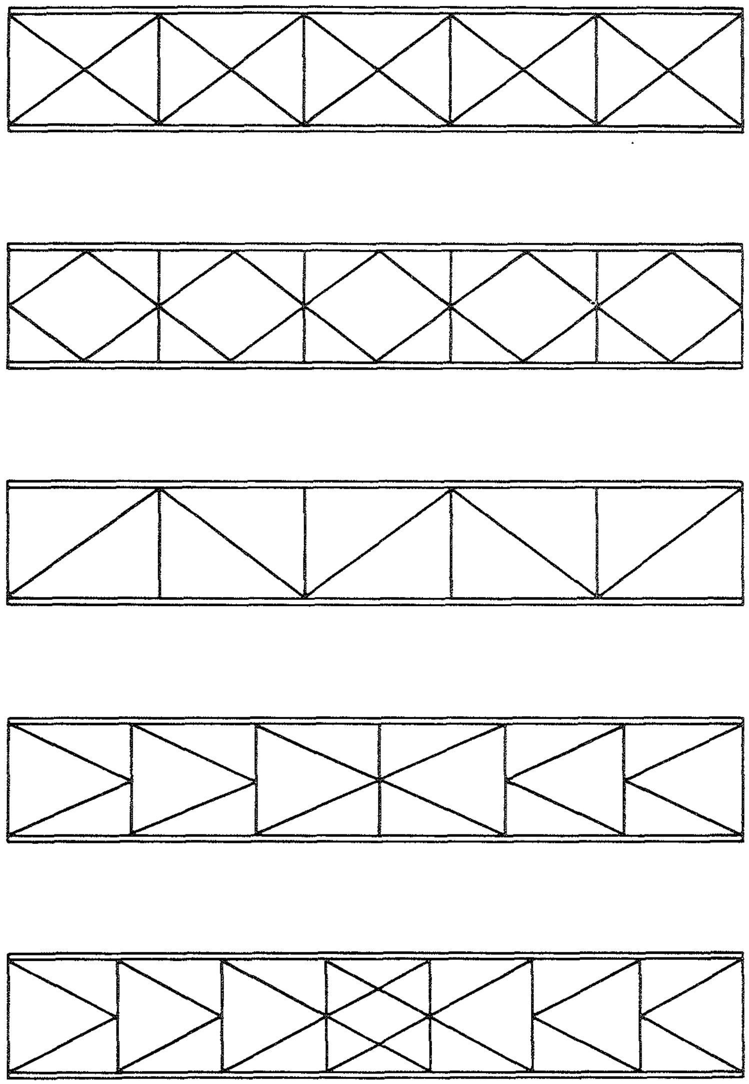


Figure C-4 Typical Configurations of Top and Bottom Lateral Bracing

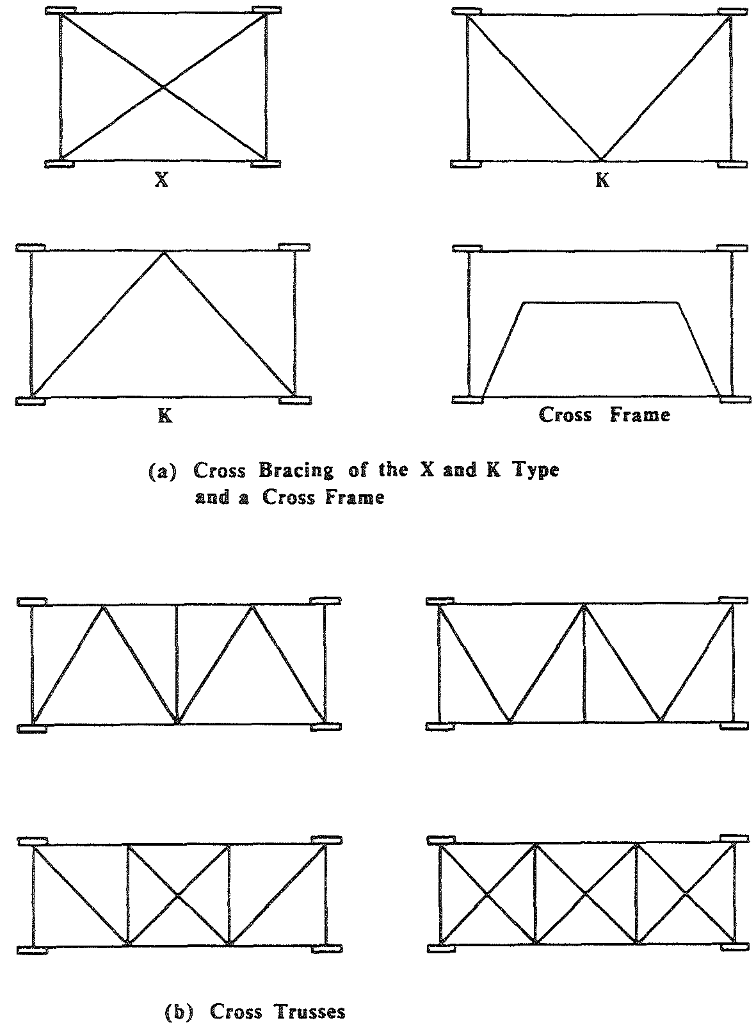
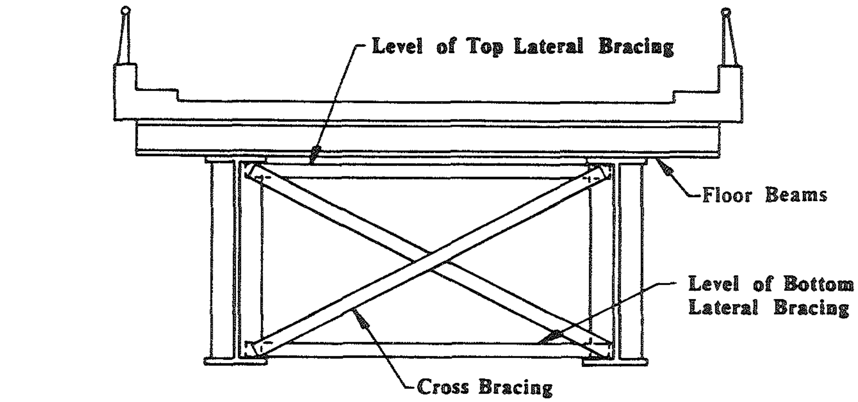
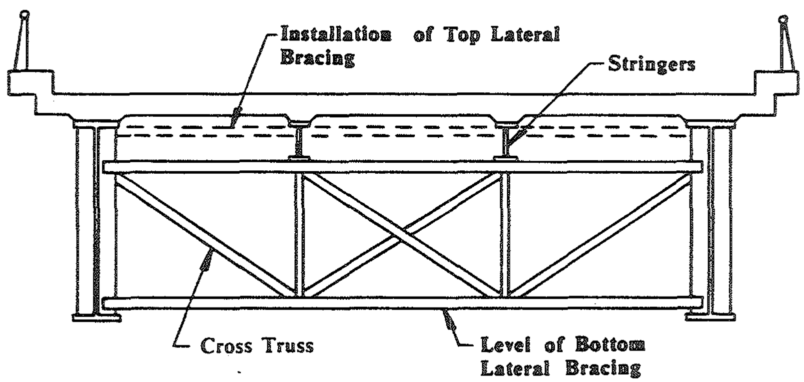


Figure C-5 Typical Configurations of Cross Bracing, Cross Frames and Cross Trusses



(a)



(b)

Figure C-6 Typical Deck Type Two-Girder Bridge Cross Sections

CHAPTER C2

REQUIREMENTS OF THE BOTTOM LATERAL BRACING TENSION DIAGONALS

DESCRIPTION OF THE CONSERVATIVE ANALYTICAL MODEL AND LOADING

Figure C-7 shows the loads and reactions acting on a schematic three-dimensional representation of a fractured two-girder simple span bridge. The fracture is assumed to occur at midspan and to penetrate the bottom (tension) flange and extend to the top of the web. The top (compression) flange is not fractured and is assumed to resist the resulting shear and axial (normal) stresses above the fracture. (Research into the shear strength actually available at the fracture location has not been done, but is needed.) Also the top flange is assumed to behave like a hinge above the fracture. The span consists of n panels of top and bottom lateral bracing of an X-type, where all diagonal members in the span are assumed to have equal cross-sectional areas. (Equations have not been developed for other than X-type configurations). The span length is L, the girder spacing is S and the depth of both girders is d.

In Fig. C-7 the distributed load, w, represents the amount of total dead load carried by each girder. When computing forces in the bottom lateral bracing members due to truck loads it is not necessary to consider the individual axle locations as is usual in design. Instead the fraction β of the total truck live plus impact loading (L+I) which is tributary to the fractured girder can be applied as a resultant concentrated load at midspan as shown in Fig. C-7. The corresponding truck loading on the unfractured girder is $(1-\beta)$(L+I). If the bridge is to be designed for redundancy using lane loading the distributed load w can be increased to account for both the dead load and the distributed lane loading which is tributary to each girder. In this case β(L+I) and

$(1-\beta)(L+I)$ would represent the porportion of the applicable concentrated lane load (for moment) which is tributary to each girder.

For midspan fracture and the symmetrical loading shown in Fig. C-7 the four reactions are statically determinate. The sum of the reactions at C and D are obtained by considering equilibrium about an axis through reactions at A and B. Also, by symmetry, the reactions at C and D are equal. The reactions at A and B are found in a similar way.

Figure C-8 shows a plan view of the bottom lateral bracing consisting of X-type diagonal members. Fracture has occurred at midspan of one of the girders.

Figure C-9 shows an elevation of the fractured girder considered as a free body isolated from the rest of the structure. Under the dead and live loads applied to this girder a concentrated rotation occurs in the top flange above the fracture as the girder deflects downward. The resulting movement of the bottom flange away from midspan generates forces F_1, F_2 and F_3 at the points where the bottom flange is connected to the bottom lateral bracing. Vertical forces applied to the girder by the cross bracing are conservatively neglected (just as they are in the usual design and rating procedures for two-girder bridges). The free body of the fractured girder shown in Fig. C-9 is the conservative analytical model used in this appendix to calculate the forces in the bottom lateral bracing diagonals (as well as the entire redundant bracing system).

Figure C-10 again shows a plan view of the bottom lateral bracing but this time the deflected positions of the members are shown. Notice that for X-type diagonals the member forces are alternately tension and compression. For diagonal tension and compression members of equal cross-sectional areas (a typical situation) research shows that the diagonal member forces are largest at midspan and decrease towards the end panels (C-1, C-3, C-4). For a given structure the difference between the maximum and minimum forces is a function of the stiffness (cross-sectional areas) of the diagonal

members. As the stiffness of the diagonal members increases the difference becomes larger, even though the average force in all the diagonal members remains constant, and equal to $F_1 + F_2 + F_3$ in Fig. C-9.

In the following section, the after-fracture tension force in the two midspan diagonal members is derived for the midspan fracture condition. This tension force is used to develop criteria for the redundancy rating factor, RRF, and required cross-section area of tension diagonals for use in the Allowable Stress Method and in determining after-fracture serviceability. Similar criteria are developed for use with the Load Factor Method.

For midspan fracture the largest compression force occurs for the compression diagonal in the panel adjacent to midspan. Corresponding criteria and procedures for computing the critical buckling load of compression diagonals is presented in Chapter C6.

Chapter C5 considers the influence of the worst case fracture which may be located other than at midspan.

AFTER-FRACTURE TENSION FORCE IN MIDSPAN DIAGONALS

Referring again to Fig. C-9, after midspan fracture occurs, the bottom lateral bracing exerts forces F_1, F_2 and F_3 on the bottom flange of the fractured girder as shown. Let the sum of these forces over the half span (or the corresponding sum of forces over a half span for a bridge with any number of panels) be F. Thus, considering the condition of zero bending moment at mispan of the fractured girder

$$F = \frac{1}{d}\left(\frac{wL^2}{8} + \frac{\beta\,(L+I)\,L}{4}\right) \qquad \text{(C2.1)}$$

If αF is the sum of the axial tension and compression forces in all the bottom lateral bracing diagonal members to one side of midspan, then

$$\alpha F = \frac{\alpha}{d}\left(\frac{wL^2}{8} + \frac{\beta(L+I)\,L}{4}\right) \qquad (C2.2)$$

where α is the ratio of the length of an X-type diagonal member to the panel length L/n.

For a span with n panels, where n is an odd integer, and with X-type bottom lateral diagonals, then the number of diagonal members which are connected to the bottom flange of the fractured girder to one side of midspan is n. Thus the average force in a bottom lateral diagonal member is $\alpha F/n$ and is alternately tension and compression as shown in Fig. C-10.

For given linear elastic bottom lateral bracing with X-type diagonals of equal cross-sectional areas, as the cross-sectional areas of the two girders increase the forces in all the diagonals approach the average force. Alternatively, for given girders, as the cross-sectional areas of the diagonals decrease the forces in the diagonals approach the average force. For practical ranges of girder and diagonal member areas, and for midspan fracture compatibility between the diagonal members and the girders requires that the forces in the two diagonals in the midspan panel be above average and that the corresponding forces in the end panels be below average.

Since all diagonal members in the span are assumed to be equal in cross-sectional area then for midspan fracture the largest tension force exists in the midspan diagonals. The largest compression force exists in the panel adjacent to midspan as shown in Fig. C-10. The ratio of the maximum tension force to the average force depends not only on the relative cross-sectional areas of the girders and diagonal members, as discussed above, but also on the relative dead and live load effects.

Let V_D be the ratio of the maximum tension force to the average force in the midspan diagonals due to dead load. Let V_L be the similar ratio due to live plus impact loading. Then from Eq. C2.2 maximum tension force, F_D, due to dead load, and the maximum tension force, F_L, due to live load plus impact, are given by

$$F_D = \frac{\alpha\, w\, L^2\, V_D}{8\, d\, n} \qquad (C2.3)$$

$$F_L = \frac{\alpha\;\beta(L+I)\, L\, V_L}{4\, d\, n} \qquad (C2.4)$$

Equations C2.3 and C2.4 can now be used to develop redundancy rating factors, RRF, in terms of the familiar AASHTO allowable stress and load factor methods (C-2). These rating factors provide the after-fracture redundancy rating of the bridge for midspan fracture of one girder in terms of the controlling midspan tension diagonals in the bottom lateral bracing. The influence of alternate fracture locations and the influence of the controlling compression diagonals in the adjacent panel on the redundancy rating is addressed in Chapters C5 and C6.

Equations C2.3 and C2.4 can also be used to compute the required cross-sectional areas of the bottom lateral tension diagonal members in order to design for after-fracture redundancy in terms of a given vehicle. These requirements are developed in terms of both allowable stress and load factor methods plus consideration of after-fracture serviceability of the deck.

ALLOWABLE STRESS METHOD

Redundancy Rating Factor - RRF

Following the concepts contained in Ref. C-2, the redundancy rating factor, RRF, for the diagonal tension members at midspan (Fig. C-10), is defined as follows:

$$V_{LB} = 0.8 + 0.18 \frac{L}{f_{all}} > 1.0 \qquad (C2.15)$$

where, L is in feet and f_{all} is the allowable stress for bottom lateral diagonals in ksi.

The values of allowable stress, f_{all}, used in developing Fig's. C-14 and C-15 were selected from Ref. C-2, and represent the usual maximum and minimum allowable stresses currently used to rate bridges at the operating level.

LOAD FACTOR METHOD

Redundancy Rating Factor - RRF

Following the concepts contained in Ref. C-2, the redundancy rating factor, RRF, is defined as follows:

$$RRF = \frac{\phi\, S_u - \gamma_D D}{\gamma_L\, (L+I)} \qquad (C2.16)$$

where, for the midsapn diagonal tension members of the bottom lateral bracing,

- ϕ = resistance factor, equal to 1.0 for the midspan tension diagonals
- S_u = maximum strength in tension
- D = dead load force in the midspan tension diagonals
- L+I = live load plus impact force in the midspan tension diagonals due to the given redundancy rating vehicle
- γ_D = dead load factor
- γ_L = live load factor

It is assumed that all tension diagonals have reached their yield stress. The midspan tension diagonals will yield first. All tension diagonals and connections are assumed to provide sufficient ductility so that the tension diagonals in the end panels

will just reach their yield stress level under the given factored loads. All compression diagonals are assumed to buckle and are ignored in the following development. Figure C-16 is a plan view of the bottom lateral bracing model and shows the alternating yielded tension diagonals and buckled compression diagonals.

As a result of the above assumptions the number of tension diagonals which are subjected to the total force αF (Eq. C2.2) is $(n+1)/2$ where n is an odd integer. Since all tension diagonals are of equal cross sectional area and are all at the yield stress level, then $V_D = V_L = 1$ when computing the RRF or the required cross section area.

Thus, the tension force due to dead load, F_D, and the tension force due to live load plus impact, F_L, in any tension diagonal are (Eq. C2.2)

$$F_D = \frac{\alpha\, w\, L^2}{4\, d\, (n+1)} \qquad (C2.17)$$

$$F_L = \frac{\alpha\, \beta\, (L+I)\, L}{2\, d\, (n+1)} \qquad (C2.18)$$

where all terms were previously defined.

Substituting Eq's. C2.17 and C2.18 into Eq. C2.16, then for $\phi = 1.0$,

$$RRF = \frac{F_y\, A_B - \dfrac{\gamma_D\, \alpha\, w\, L^2}{4\, d\, (n+1)}}{\dfrac{\gamma_L\, \alpha\, \beta (L+I)\, L}{2\, d\, (n+1)}} \qquad (C2.19)$$

Required A_B to Assure Redundancy

The required area, A_B, of all the bottom lateral tension and compression diagonal members to a design for redundancy for a given live loading is found by setting the RRF in Eq. C2.19 equal to unity and solving for A_B.

$$\text{Req'd. } A_B = \frac{\alpha\, L}{4\, d\, (n+1)\, F_y}\left(\gamma_D\, w\, L + 2\, \gamma_L\, \beta(L+I)\right) \qquad \text{(C2.20)}$$

This area is required for both the tension and the compression diagonals because a compression diagonal for fracture in one girder is a tension diagonal if the fracture occurs in the other girder.

SERVICEABILITY

It is discussed in Chapter 3 of this report that the deflection-to-span length ratio, Δ/L, is important for two reasons. First, under the dead load alone, a sufficiently large Δ/L is an important indicator that fracture has occurred. Secondly, Δ/L should be small enough under the total dead, live and impact loading that a heavy vehicle travelling at normal highway speeds can safely cross the bridge. In the following, equations are developed which enable the Δ/L ratios to be calculated under the dead and total loads for a simple span bridge.

Deflection - Stress Relationship

For midspan fracture the after-fracture dead load deflection of the fractured girder is a function of the axial stress in the tension diagonals at midspan. As the axial stress increases, deflection increases. It is also assumed, as before, that all tension and compression bottom lateral diagonals are identical and have equal cross section areas.

It is assumed that the unfractured girder does not deflect. It is also assumed that the deflected shape of the fractured girder consists of two straight lines extending from each support to midspan and that the deck deflection above the fracture equals the deflection of the fractured girder. These conditions are shown in Fig. C-17(a).

From similar triangles in Fig. C-17(a),

$$\frac{\Delta}{L} = \frac{h}{2d} \qquad \text{(C2.21)}$$

where h is one half of the crack opening width at the bottom of the fractured girder and Δ is the midspan deflection of the fractured girder.

Figure C-17(b), which is similar to Fig. C-10, shows the after-fracture displacements of the bottom lateral bracing. Axial stresses in both the tension and the compression diagonals are assumed to be at or below the allowable stresses under service dead and live loads as recommended in Chapter 3. It is assumed that the girders remain the same distance apart. Any increase in deflection resulting from shortening of the diaphragm (reduction in girder spacing) is assumed to be offset by reductions in deflection due to bending and torsional stiffness of the deck, for example, which is neglected in this analysis.

Figure C-17(c) shows the displacements of the midspan panel and the tension diagonals at midspan. The strain, ϵ_B, in a tension diagonal is

$$\epsilon_B = \frac{h/\alpha}{\alpha\, L/n} = \frac{n\, h}{\alpha^2\, L} \qquad \text{(C2.22)}$$

The resulting deflection-to-span length ratio, Δ/L, is found by substituting h from Eq. C2.21.

$$\frac{\Delta}{L} = \frac{\alpha^2\, L\, f_B}{2\, E\, d\, n} \qquad \text{(C2.23)}$$

where f_B is the axial stress in the tension diagonal and equal to $E\, \epsilon_B$.

If f_B is the total dead and live load plus impact stress in the diagonals at midspan, then from Eq. C2.23

$$f_B = \frac{2\,E\,d\,n}{\alpha^2\,L}\left(\frac{\Delta}{L}\right) \tag{C2.24}$$

where $f_B < f_{all}$ and Δ/L is the desired deflection-to-span length ratio selected by the bridge engineer for total load conditions.

Deflections for Allowable Stress Method

The dead load stress, f_D, and the live load plus impact stress, f_L, are given by Eq's. C2.6 and C2.7. Substituting Eq's. C2.6 and C2.7 into Eq. C2.23 the total load deflection ratio $\left(\frac{\Delta}{L}\right)$ is

$$\left(\frac{\Delta}{L}\right) = \frac{\alpha^3\,L^2}{16\,E\,d^2\,n^2\,A_B}\left(w\,L\,V_{DA} + 2\,\beta(L+I)\,V_{LA}\right) \tag{C2.25}$$

where A_B is the required area given by Eq. C2.9. Coefficients V_{DA} and V_{LA} are to be calculated by using the required area from Eq. C2.9.

Similarly, substituting only Eq. C2.6 into Eq. C2.23, the dead load deflection ratio $\left(\frac{\Delta}{L}\right)_D$ is

$$\left(\frac{\Delta}{L}\right)_D = \frac{\alpha^3\,L^2}{16\,E\,d^2\,n^2\,A_B}\left(w\,L\,V_{DA}\right) \tag{C2.26}$$

Also, from Eq's. C2.25 and C2.26 the dead load deflection ratio can be expressed as

$$\left(\frac{\Delta}{L}\right)_D = \frac{w\,L\,V_{DA}}{w\,L\,V_{DA} + 2\,\beta(L+I)\,V_{LA}}\left(\frac{\Delta}{L}\right) \tag{C2.27}$$

Deflections for Load Factor Method

Equations C2.25, C2.26 and C2.27 can be used to compute the service load deflections when using the Load Factor Method providing that

$$\frac{\text{Req'd. } A_B \text{ by LFM}}{\text{Req'd. } A_B \text{ by ASM}} \geq \frac{1}{\gamma_D} \text{ or } \frac{1}{\gamma_L} \tag{C2.28}$$

where it is assumed that γ_L and γ_D are equal as recommended in Chapter 3. Equation C2.28 can still be used, however, if the dead to live load ratio is high. Eq. C2.28 ensures that the tension stress in the highest stress diagonal does not exceed the yield stress under the service loads when using the Load Factor method.

If Eq. C2.28 is not satisfied, the real deflection will be higher than the deflections computed by Eq's. C2.25 and C2.26. Since plastic deformations are involved computation of the real deflection is quite complex and is not attempted in this investigation.

Table C-1 Details of Bridges Used for FE Analyses

Bridge Span	100 ft.		150 ft.		200 ft.	
Number of panels, n	5	7	7	9	9	13
Girder Depth, d	80"		120"		160"	
Flanges: Midspan	8" x 2.5"		22" x 2.75"		25" x 3.0"	
Quarterspan	18" x 1.875"		22" x 2.0"		25" x 2.25"	
Web	80" x 0.5"		120" x 0.75"		160" x 1.0"	
A_B (in^2)	9.36 18.72 37.44	8.00 15.99 31.98	9.74 19.47 38.94	8.54 17.07 34.14	9.95 19.89 39.78	8.54 17.07 34.14

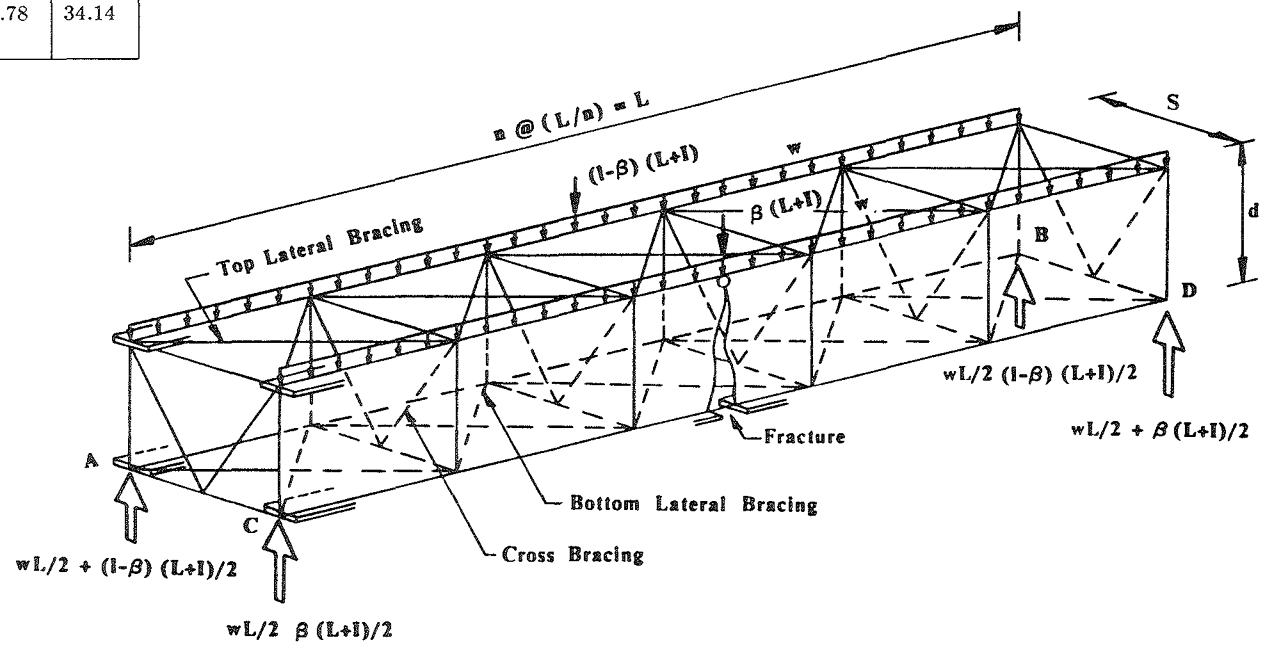


Figure C-7 Loads and Reactions Acting on a Schematic Three-Dimensional Representation of a Fractured Two-Girder Simple Span Bridge

$$RRF = \frac{f_{all} - f_D}{f_L} \tag{C2.5}$$

where,

f_{all} = allowable tension stress

f_D = dead load stress in the tension diagonals

f_L = live load plus impact stress in the tension diagonals due to the given redundancy rating vehicle.

The dead and live plus impact stress in the diagonal tension members are found from Eq's. C2.3 and C2.4 as follows.

$$f_D = \frac{\alpha\ w\ L^2\ V_{DA}}{8\ d\ n\ A_B} \tag{C2.6}$$

$$f_L = \frac{\alpha\ \beta(L{+}I)\ L\ V_{LA}}{4\ d\ n\ A_B} \tag{C2.7}$$

where: A_B = Cross-section area of a diagonal tension or compression member

V_{DA} = Value of V_D for allowable stress conditions

V_{LA} = Value of V_L for allowable stress conditions

Substituting Eq's. C2.6 and C2.7 into Eq. C2.5, then the RRF for tension diagonals is

$$RRF = \frac{f_{all} - \dfrac{\alpha\ w\ L^2\ V_{DA}}{8\ d\ n\ A_B}}{\dfrac{\alpha\ \beta(L{+}I)\ L\ V_{LA}}{4\ d\ n\ A_B}} \tag{C2.8}$$

Required A_B to Assure Redundancy

The required cross-section area, A_B, of each bottom lateral diagonal member in design for redundancy for a given live loading is found by setting the RRF in Eq. C2.8 equal to unity and solving for A_B. In this case, however, when RRF = 1, the factors V_{DA} and V_{LA} will change to V_{DB} and V_{LB}. Solving for A_B from Eq. C2.8 with RRF = 1 gives

$$\text{Req'd}\quad A_B = \frac{\alpha\ L}{8\ d\ n\ f_{all}}\left(w\ L\ V_{DB} + 2\ \beta(L{+}I)\ V_{LB}\right) \tag{C2.9}$$

where: V_{DB} = value of V_D for allowable stress conditions when RRF = 1

V_{LB} = value of V_L for allowable stress conditions when RRF = 1

The corresponding maximum tension force, F_B, in the midspan diagonals, for midspan fracture, is found by multiplying Eq. C2.9 by f_{all}:

$$F_B = \frac{\alpha\ L}{8\ d\ n}\left(w\ L\ V_{DB} + 2\ \beta(L{+}I)\ V_{LB}\right) \le A_B\ F_y \tag{C2.10}$$

where F_y = yield stress in tension.

Coefficient V_D and V_L

The coefficient V_D and V_L are amplification factors which determine the increase in tension force in the midspan tension diagonals over the average force. The degree of amplification depends not only on the relative areas of the diagonals and girders but also on whether the loading is distributed or concentrated at midspan.

In addition, V_D and V_L are each different depending on whether they are used to calculate the RRF as in Eq. C2.8 or used to calculate the required area A_B of all the bottom lateral diagonals for a unit RRF as in Eq. C2.9.

Practical values of V_D and V_L for use in Eq's. C2.8, C2.9 and C2.10 were obtained from extensive finite element (FE) analyses of 18 variations in design of 3 noncomposite two-girder bridges having X-type top and bottom lateral bracing and X-type cross bracing. The three bridges used in the FE analyses were based on the bridge described in Ref. C-5. The computer models were formulated considering the three-dimensional geometry of the bridges consisting of the fractured and unfractured girders, plus the floor beams, bearings and bracing systems. The FE analyses were performed using GTSTRUDL (C-6) and the Computer Aided Engineering Laboratory (CAE) facility, Dr. John L. Wilson, Director, located at Fritz Engineering Laboratory, in the Department of Civil Engineering, Lehigh University. Further details of the computer modeling and analyses are contained in Ref's. C-3 and C-4.

Figure C-11 shows the details of the 150 ft. simple span two-girder bridge described in Ref. C-5 which was designed for two lanes of AASHTO HS-20 loading. Prior to developing the computer models, the bridge shown in Fig. C-11 was redesigned to span lengths of 100 and 200 feet, maintaining AASHTO HS-20 truck and lane loading and A36 steel. The span-to-depth ratio was kept constant at 15 to maintain similar deflection control. The flange thickness transition was maintained at the quarter point. The girder spacing was kept constant at 18 feet.

The computer models developed for the FE analyses were based on the given 150 ft. bridge plus the two bridges redesigned to 100 and 200 feet. Table C-1 shows the details of the bridges used for FE analyses. The number, n, of panels of equal top and bottom lateral bracing was varied from 5 to 13, providing a range of values for the ratio α (Eq. C2.2). The cross-sectional areas, A_B, of the bottom lateral diagonals were varied from 8.0 to 39.78 in.^{2}, as shown at the bottom of the table providing a range of diagonal-to-girder axial stiffness ratios. For each span length and number of panels the areas of the diagonals were varied three times, providing 18 different analyses. For each analysis the maximum force due to dead load, F_D, and the maximum force due to either HS-20 truck or lane loading, F_L, was obtained for the diagonals in the midspan panel. Values of V_D and V_L were then computed from Eq's. C2.3 and C2.4 and plotted, as the solid lines in Figs. C-12 and C-13 as functions of the diagonal-to-girder axial stiffness

ratio, R, and the number of panels, n, where,

$$R = \frac{A_B}{\alpha^3 \bar{A}_f} \tag{C2.11}$$

and, $\bar{A}_f = A_f + 1/6 \, A_w$ (Ref. C-7)

where A_f = average area of girder bottom flange

A_w = average area of girder web

$\bar{A}_f$ is obtained by replacing the actual girder having top and bottom flanges, each of average area A_f, and web of average area A_w, with an equivalent girder having top and bottom flanges, each of area $\bar{A}_f$, and zero web area, but maintaining the same flexural stress in the flanges of the actual and equivalent girders. The average flange area, A_f, is computed as the total volume of the flange divided by the span length. The average web area is computed in a similar way.

The dashed curves in Fig's. C-12 and C-13 were analytically fitted to the computed results (solid lines) and used to determine V_{DA} and V_{LA} for use in Eq. C2.8, as follows:

$$V_{DA} = \frac{n^2 R + 2.4}{n \, R + 2.4} \tag{C2.12}$$

$$V_{LA} = \frac{n^2 R + 7.0}{n \, R + 7.0} \tag{C2.13}$$

Figures C-12 and C-13 were used together with Eq. C2.9 to construct Fig's. C-14 and C-15. Further details are contained in Ref's. C-3 and C-4. Each solid line is a best conservative fit to the computed data points. The equations of these solid lines were used to determine V_{DB} and V_{LB} for use in Eq. C2.9 as follows:

$$V_{DB} = 0.8 + 0.36 \frac{L}{f_{all}} > 1.0 \tag{C2.14}$$

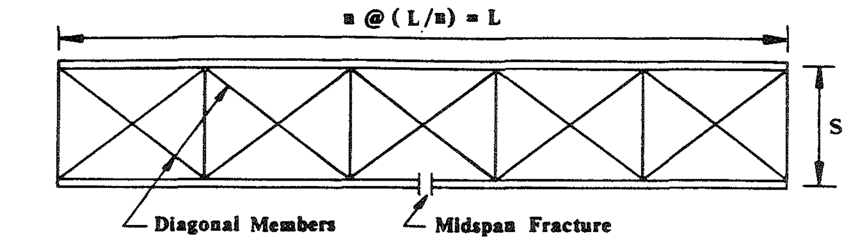


Figure C-8 Plan View of X-Type Bottom Lateral Bracing

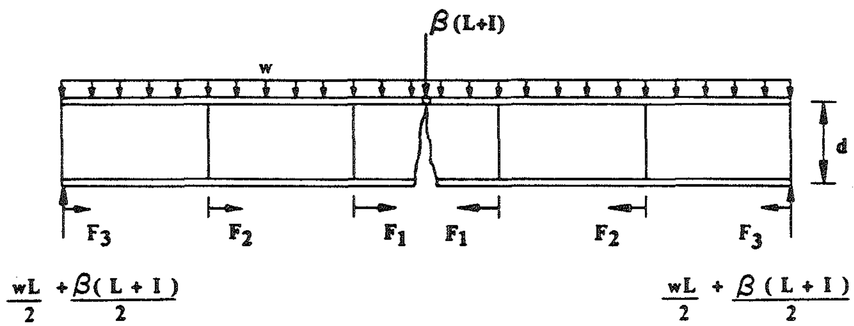


Figure C-9 Elevation View of Fractured Girder as a Free Body

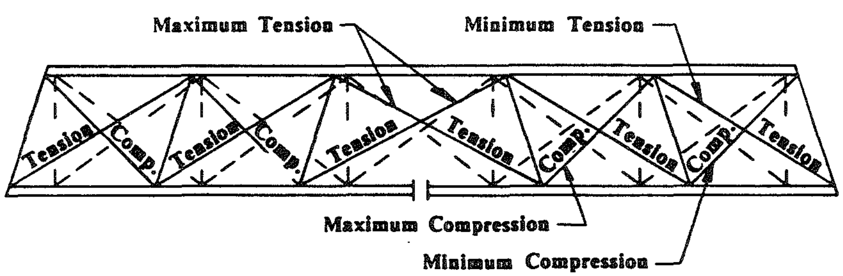


Figure C-10 Plan View of Bottom Lateral Bracing in the After-Fracture Deflected Position

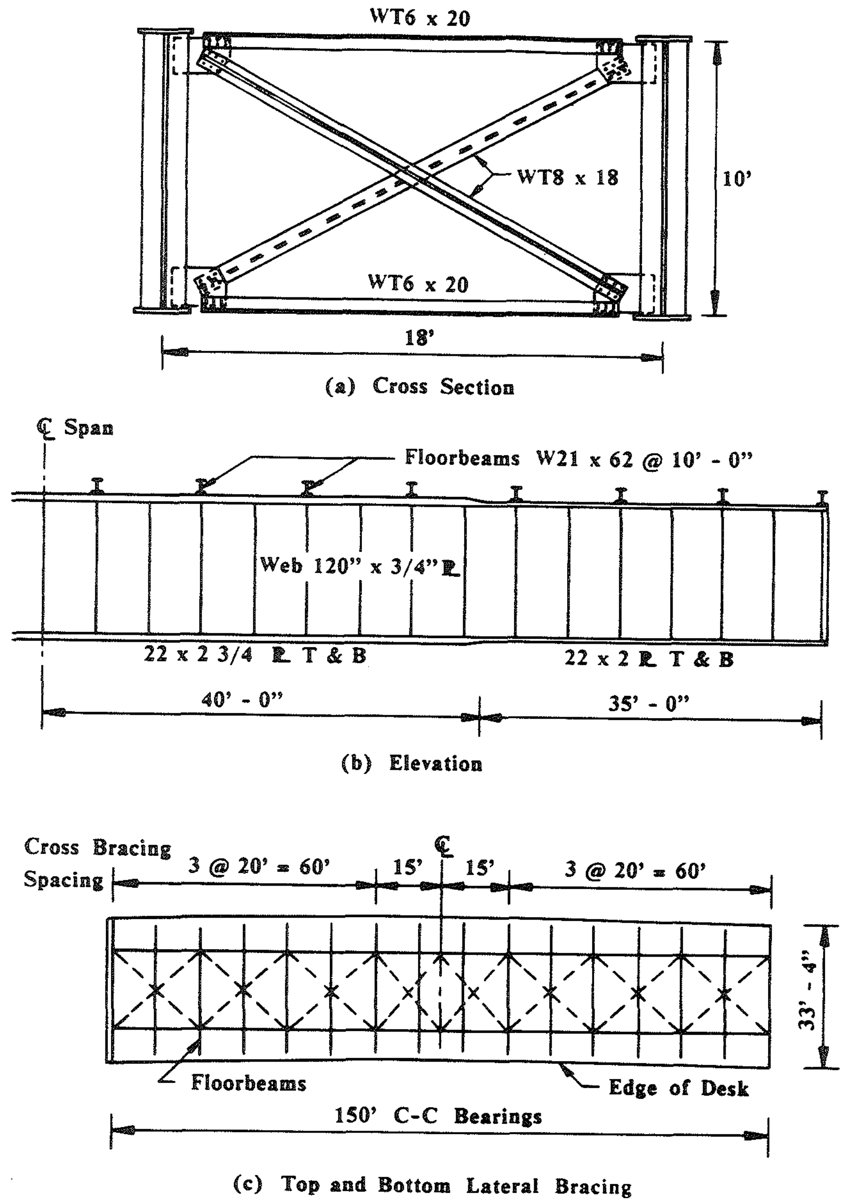


Figure C-11 Details of the Bridge in Ref. C-5

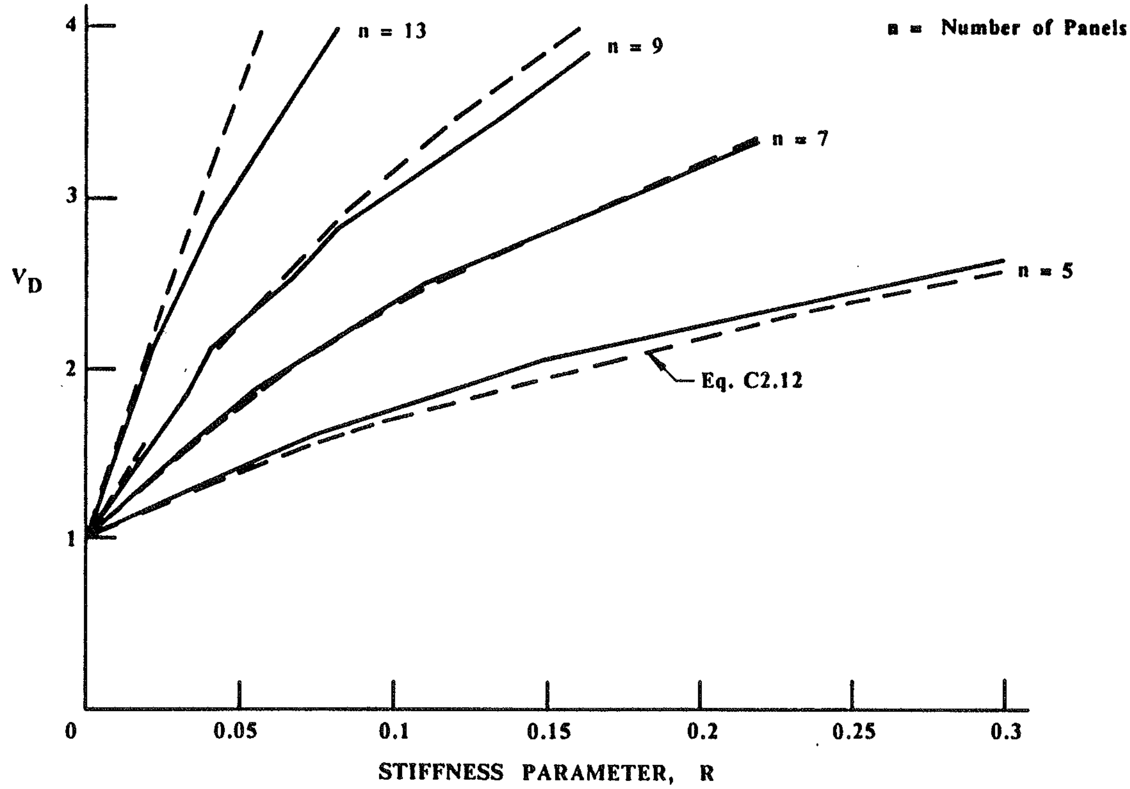


Figure C-12 V_D versus R as a Function of Number of Panels, n C2-20

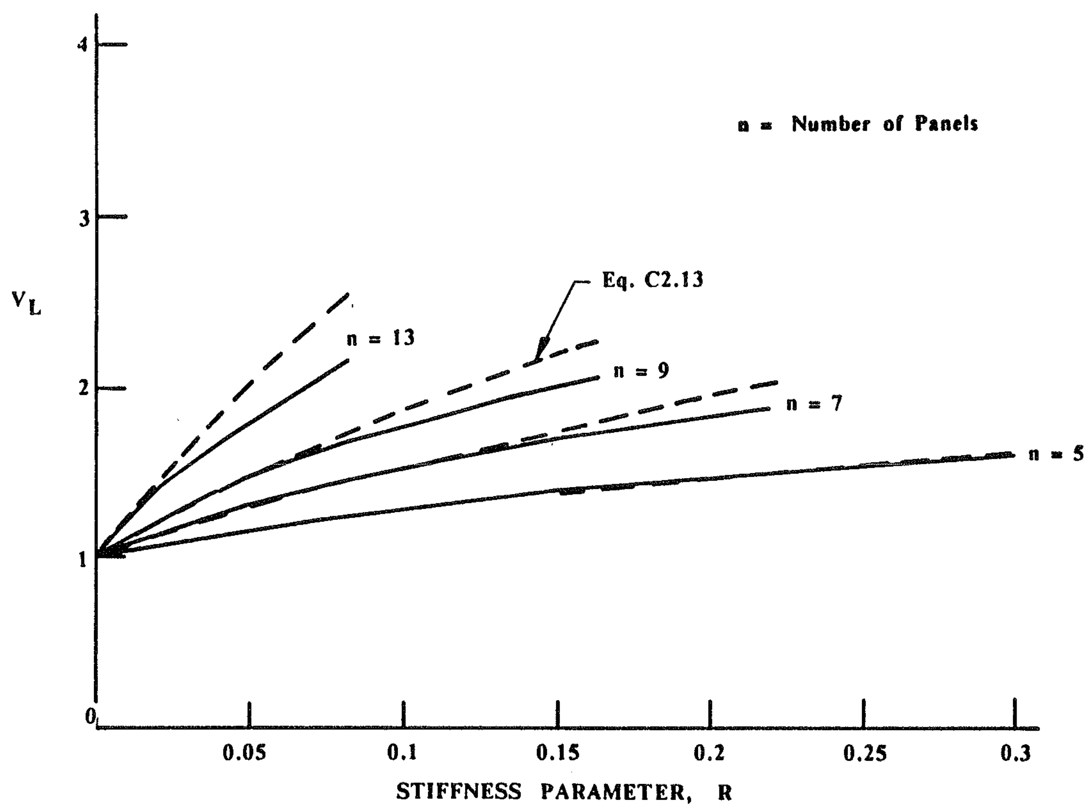


Figure C-13 V_L versus R as a Function of Number of Panels, n C2-21

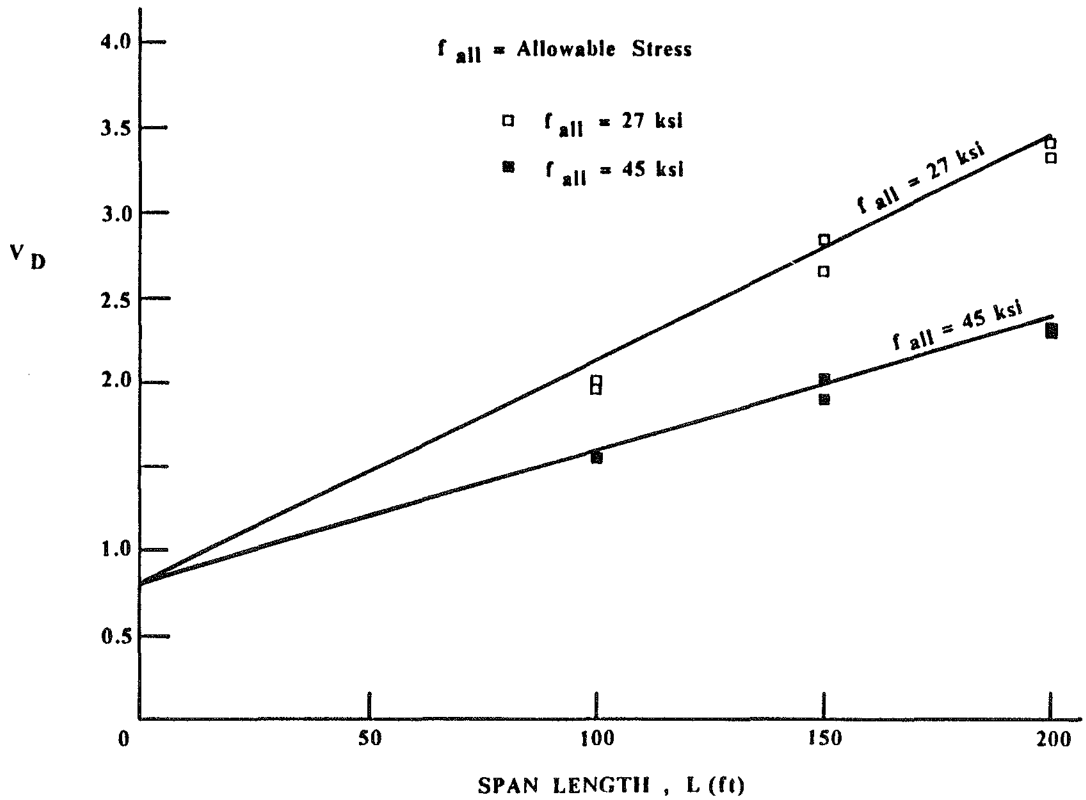


Figure C-14 V_D versus L as a Function of Allowable Stresses C2-22

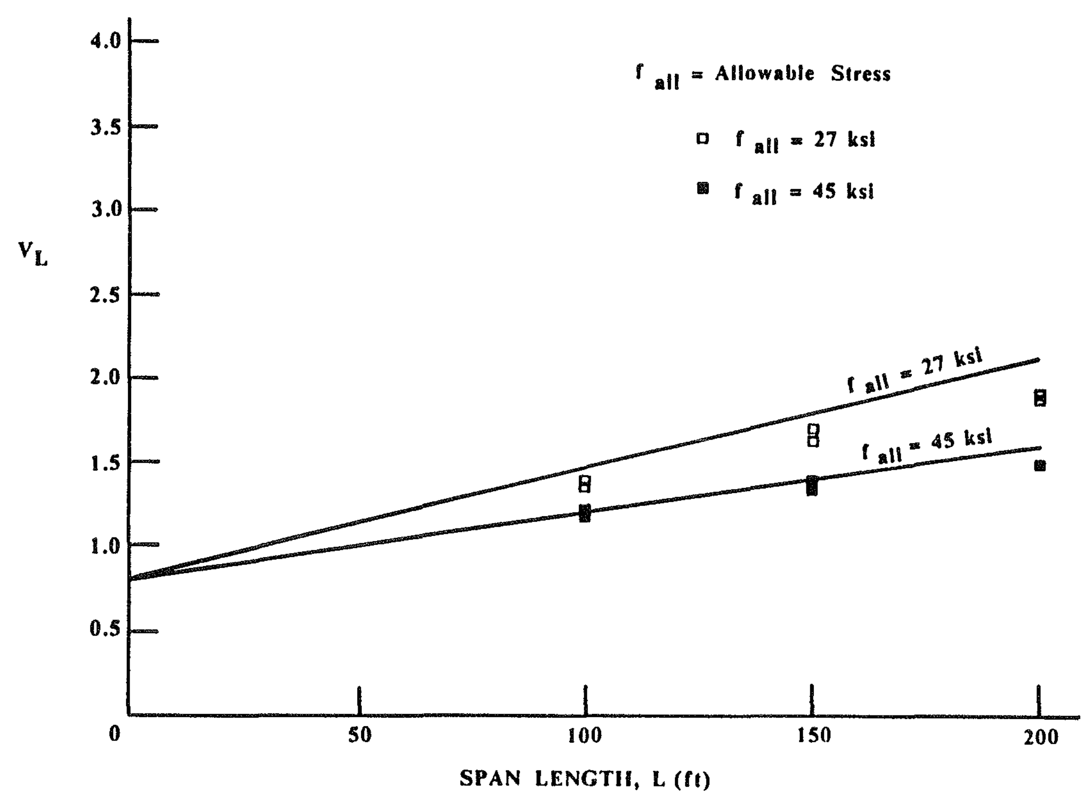


Figure C-15 V_L versus L as a Function of Allowable Stresses C2-23

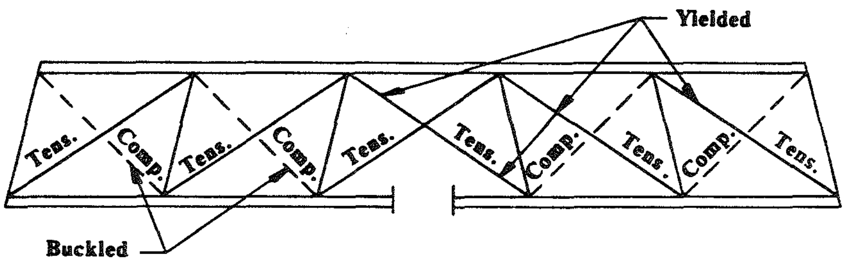


Figure C-16 Plan View of the Bottom Lateral Bracing Model for the Load Factor Method

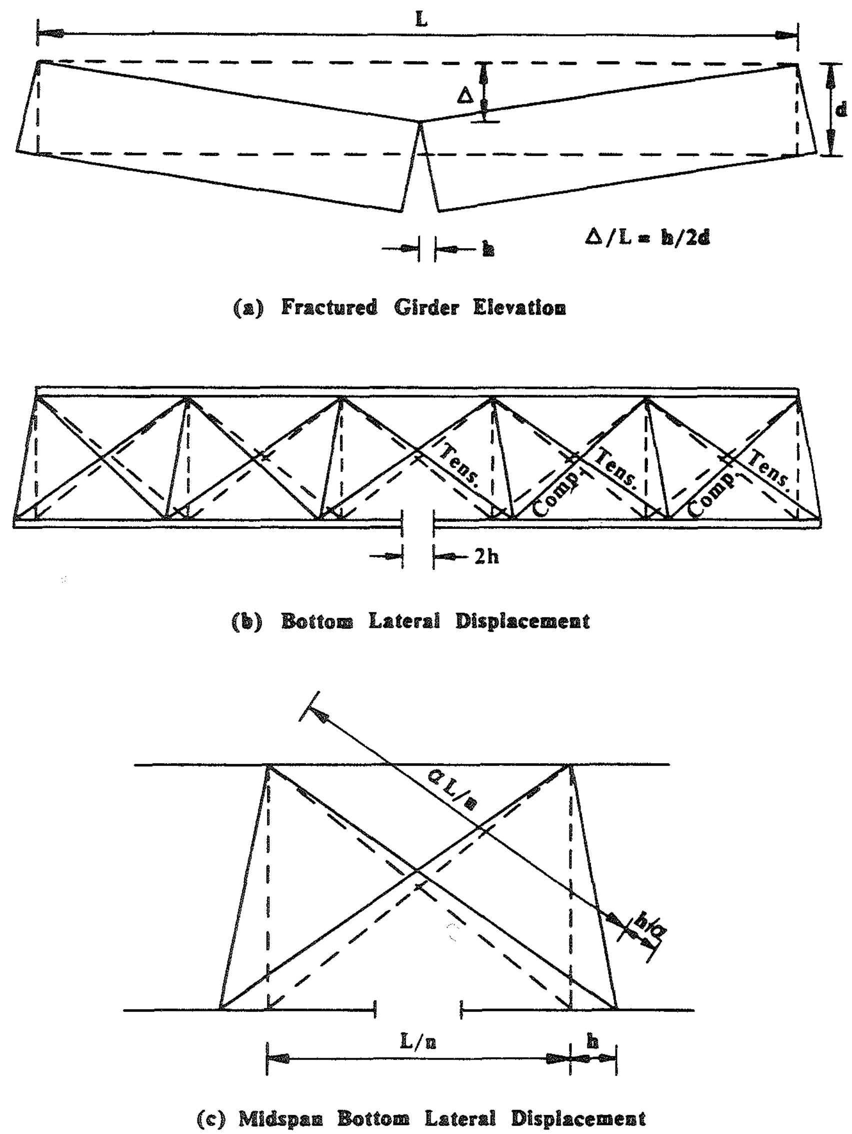


Figure C-17 Displacement Relationships Used to Establish Serviceability Criteria for Midspan Fracture

CHAPTER C3

REQUIREMENTS OF THE CROSS BRACING

FORCES INDUCED IN THE CROSS BRACING

In order for the after-fracture forces in the bottom lateral tension and compression diagonals to develop, the bottom lateral diagonals must be anchored to the top lateral bracing by the cross bracing diaphragms. All end and interior cross bracing diaphragms are assumed to be identical (a practical situation) in the following.

The resultant transverse components of the forces in the bottom lateral diagonals which are applied to bottom flanges of the unfractured and fractured girders are U and F, respectively, as shown in Fig. C-18(a). These forces are resisted by the cross bracing members. It is assumed that for the fracture in the right girder the forces in the two diagonal cross bracing members are equal, one in tension, F_{CD}, the other in compression $-F_{CD}$ as shown in Fig. C-18(a). Also shown in the figure is the force, $-F_{CH}$, in the bottom horizontal member which is also a transverse member of the bottom lateral bracing and is always in compression.

The three configurations of cross bracing diagonals considered in this appendix are shown in Fig's. C-18(b), (c), and (d). The forces in the diagonals and bottom horizontal member for each cross bracing configuration are also shown in the figures, where the right girder is assumed fractured, and where k_d is the ratio of the length of a cross bracing diagonal member to its horizontal projection.

CROSS-BRACING FORCES

The forces in the bottom lateral diagonals of the midspan and the two adjacent panels of a simple span bridge with a midspan fracture are shown in Fig. C-19. The after-fracture tension force in the midspan diagonals is F_B where F_B is given by Eq.

C2.10. As previously discussed, the tension and compression forces in the diagonals away from midspan are progressively less than F_B. Therefore, the largest cross bracing forces will occur in the two diaphragms adjacent to midspan as shown in the figure.

Let ϕ_c F_B and ϕ_t F_B be the compression and tension forces, respectively, in the bottom lateral diaognals in the panels adjacent to midspan as shown in Fig. C-19, where ϕ_c and ϕ_t are force reduction factors. Therefore, the forces U and F, discussed above, which are applied to the two diaphragms adjacent to midspan are

$$U = (1+\phi_t)\, F_B\, k_H \tag{C3.1}$$

$$F = (1-\phi_c)\, F_B\, k_H \tag{C3.2}$$

where k_H is the ratio of the girder spacing to the length of a bottom lateral diagonal member. Therefore, referring to Fig. C-18, forces in the diagonal members and the bottom horizontal members for each of the cross bracing configurations are as follows:

X-Bracing and K-Bracing (Type 1)

$$F_{CH} = \left(1 + 0.5\, (\phi_t - \phi_c)\right)\, F_B\, k_H \tag{C3.3}$$

$$F_{CD} = 0.5\, (\phi_t + \phi_c)\, F_B\, k_D \tag{C3.4}$$

K-Bracing (type 2)

$$F_{CHF} = (1 - \phi_c)\, F_B\, k_H \tag{C3.5}$$

$$F_{CHU} = (1 + \phi_t)\, F_B\, k_H \tag{C3.6}$$

$$F_{CD} = 0.5\, (\phi_t + \phi_c)\, F_B\, k_D \tag{C3.7}$$

where $k_D = k_d\, k_H$

Note that since the horizontal member for Type 2 has a constant cross section area either Eq. C3.5 or Eq. C3.6 will produce the largest force.

ALLOWABLE STRESS METHOD

Redundancy Rating Factor - RRF

The results of the finite element analyses of 18 configurations of 3 simple span bridges previously discussed in Chapter C2 were used to evaluate the force reduction factors ϕ_c and ϕ_t. Further details are reported in Ref's. C-3 and C-4. The results of this study are shown in Table C-2 where the range of redundancy rating factors, RRF, shown in Table C-2 were computed for the range of areas A_B, used for the bottom lateral diagonals as shown in Table C-1.

Recognizing the greater influence of dead load on the cross bracing forces, the following conservative values were selected from Table C-2 for use either in determining the RRF for as-built cross bracing members or in determining the required design areas, A_{CH} and A_{CD}, of the cross bracing horizontals and diagonals, respectively, corresponding to a RRF of unity.

$$\phi_t = 0.85 \tag{C3.8}$$

$$\phi_t - \phi_c = 0.70 \tag{C3.9}$$

$$\phi_t + \phi_c = 1.50 \tag{C3.10}$$

Note that ϕ_c is not shown separately here or in Table C-2 since Eq. C3.6 will produce the maximum force in the horizontal member for Type 2.

The resulting forces in the diagonals and the bottom horizontal member for each cross bracing configuration shown in Fig. C-18 are as follows:

X-bracing and K-Bracing (Type 1)

$$F_{CH} = 1.35\ F_B\ k_H \tag{C3.11}$$

$$F_{CD} = 0.75\ F_B\ k_D \tag{C3.12}$$

K-bracing (Type 2)

$$F_{CH} = 1.85\ F_B\ k_H \tag{C3.13}$$

$$F_{CD} = 0.75\ F_B\ k_D \tag{C3.14}$$

Note that since $F_{CHU} > F_{CHF}$ (Eq's. C3.5 and C3.6), F_{CHU} is replaced by F_{CH}.

Following the concepts contained in Ref. C-3 and the procedures developed in Chapter C2, the redundancy rating factor, RRF, for the cross bracing members is defined as follows:

$$RRF = \frac{f_{all} - f_D}{f_L} \tag{C3.15}$$

where,

f_{all} = allowable tension or compression stress

f_D = dead load stress in a cross bracing horizontal or diagonal member

f_L = live load plus impact stress in a cross bracing horizontal or diagonal member due to the given redundancy rating vehicle

X-Bracing and K-bracing (Type 1)

Horizontal Member

Combining Eq's. C2.6, C2.7 and C3.11, where A_{CH} is the cross sectional area of the horizontal member, the dead and live load stresses f_D and f_L are:

$$f_D = 1.35\ \frac{\alpha\ w\ L^2\ V_{DA}}{8\ d\ n\ A_{CH}}\ k_H \tag{C3.16}$$

$$f_L = 1.35\ \frac{\alpha\ \beta(L+I)\ L\ V_{LA}}{4\ d\ n\ A_{CH}}\ k_H \tag{C3.17}$$

Substituting Eq's. C3.16 and C3.17 into Eq. C3.15, then

$$\mathrm{RRF} = \frac{f_{all} - \dfrac{\alpha\ w\ L^2\ V_{DA}\ k_H}{5.93\ d\ n\ A_{CH}}}{\dfrac{\alpha\ \beta(L+I)\ L\ V_{LA}\ k_H}{2.96\ d\ n\ A_{CH}}} \qquad (C3.18)$$

where k_H was previously defined for use in Eq's. C3.1 and C3.2.

Diagonal Members

In a similar manner using Eq's. C2.6, C2.7, C3.12 and C3.15 where A_{CD} is the cross-sectional area of a diagonal members (assuming equal diagonals)

$$\mathrm{RRF} = \frac{f_{all} - \dfrac{\alpha\ w\ L^2\ V_{DA}\ k_D}{10.67\ d\ n\ A_{CD}}}{\dfrac{\alpha\ \beta(L+I)\ L\ V_{LA}\ k_D}{5.33\ d\ n\ A_{CD}}} \qquad (C3.19)$$

where $k_D = k_d k_H$, as before.

K-Bracing (Type 2)

Horizontal Member

Similarly, combining Eq's. C2.6, C2.7 and C3.13, and substituting into Eq. C3.15,

$$\mathrm{RRF} = \frac{f_{all} - \dfrac{\alpha\ w\ L^2\ V_{DA}\ k_H}{4.32\ d\ n\ A_{CH}}}{\dfrac{\alpha\ \beta(L+I)\ L\ V_{LA}\ k_H}{2.16\ d\ n\ A_{CH}}} \qquad (C3.20)$$

Diagonal Members

Similarly, combining Eq's. C2.6, C2.7 and C3.14, and substituting into Eq. C3.15,

$$\mathrm{RRF} = \frac{f_{all} - \dfrac{\alpha\ w\ L^2\ V_{DA}\ k_D}{10.67\ d\ n\ A_{CD}}}{\dfrac{\alpha\ \beta(L+I)\ L\ V_{LA}\ k_D}{5.33\ d\ n\ A_{CD}}} \qquad (C3.21)$$

In Eq's. C3.18 to C3.21, f_{all} is controlled by compression. Thus $f_{all} = f_{cr}/F.S.$ for all horizontal and diagonal members, where f_{cr} is the buckling stress (see Chapter C6) and F.S. is the factor of safety (see Chapter 3 of this report).

Required A_{CH} and A_{CD} to Assure Redundancy

The required cross-section areas A_{CH} and A_{CD} of the cross bracing bottom horizontal member and diagonal members to design for redudancy for a given vehicle is found by setting the redundancy rating factors given by Eq's. C3.18 to C3.21 equal to unity, replacing V_{DA} and V_{LA} with V_{DB} and V_{LB} as discussed in Chapter C2 and solving for the required A_{CH} and A_{CB}, as follows:

X-Bracing and K-Bracing (Type 1)

Horizontal Member

$$\text{Req'd. } A_{CH} = \frac{\alpha\ L\ k_H}{5.93\ d\ n\ f_{all}}\left(w\ L\ V_{DB} + 2\ \beta(L+I)\ V_{LB}\right) \qquad (C3.22)$$

Diagonal Members

$$\text{Req'd. } A_{CD} = \frac{\alpha\ L\ k_D}{10.67\ d\ n\ f_{all}}\left(w\ L\ V_{DB} + 2\ \beta(L+I)\ V_{LB}\right) \qquad (C3.23)$$

K-Bracing (Type 2)

Horizontal Members

$$\text{Req'd. } A_{CH} = \frac{\alpha\ L\ k_H}{4.32\ d\ n\ f_{all}}\left(w\ L\ V_{DB} + 2\ \beta(L+I)\ V_{LB}\right) \qquad (C3.24)$$

Diagonal Members

$$\text{Req'd. } A_{CD} = \frac{\alpha\, L\, k_D}{10.67\, d\, n\, f_{all}}\left(w\, L\, V_{DB} + 2\,\beta(L+I)\; V_{LB}\right) \qquad (C3.25)$$

In Eq's. C3.22 to C3.25, f_{all} is controlled by compression. Thus $f_{all} = f_{cr}/F.S.$ for all horizontal and diagonal members.

LOAD FACTOR METHOD

Redundancy Rating Factor - RRF

Following the concepts contained in Ref. C-2, and the procedures developed in Chapter C2, the redundancy rating factor, RRF, for the cross bracing members is defined as follows:

$$RRF = \frac{\phi S_u - \gamma_D D}{\gamma_L\,(L+I)} \qquad (C3.26)$$

where,

- ϕ = resistance factor, equal to 1.0 for the cross bracing
- S_u = maximum strength in tension or compression
- D = dead load tension or compression force in a cross bracing horizontal or diagonal member
- L+I = live load plus impact tension or compression force in a cross bracing horizontal or diagonal member
- γ_D = load factor for dead load
- γ_L = load factor for live load

The tension forces, F_B, in the bottom lateral tension diagonals are shown in Fig. C-20 for a simple span bridge. As discussed in Chapter C2 all tension diagonals are assumed to yield and all compression diagonals area assumed to buckle. The maximum

cross bracing forces are again in the two diaphragms adjacent to midspan as shown in the figure. The forces U and F shown in Fig. C-18(a) are given by Eq's. C3.1 and C3.2 where $\phi_t = 1$ and $\phi_c = 0$.

$$U = 2\, F_B\, k_H \qquad (C3.27)$$

$$F = F_B\, k_H \qquad (C3.28)$$

where $F_B = A_B\, F_y$ (Eq. C2.10)

Therefore, from Fig. C-18 the forces in the diagonal members and the bottom horizontal member for each cross bracing configuration are as follows:

X-Bracing and K-Bracing (Type 1)

$$F_{CH} = 1.5\, F_B\, k_H \qquad (C3.29)$$

$$F_{CD} = 0.5\, F_B\, k_D \qquad (C3.30)$$

K-Bracing (Type 2)

$$F_{CH} = 2.0\, F_B\, k_H \qquad (C3.31)$$

$$F_{CD} = 0.5\, F_B\, k_D \qquad (C3.32)$$

Equations C3.29 to C3.32 correspond to the allowable stress Eq's. C3.11 to C3.14.

For the bottom lateral bracing diagonals the tension force due to dead load, F_D, and the tension force due to live load plus impact, F_L, in any diagonal are given by Eq's. C2.17 and C2.18. Substituting these equations into Eq's. C3.29 to C3.32 and combining with Eq. C3.26 gives the following RRF equations.

X-Bracing and K-Bracing (Type 1)

Horitzontal Member

$$\mathrm{RRF} = \frac{S_u - \dfrac{\gamma_D \alpha w L^2 k_H}{2.67 d (n+1)}}{\dfrac{\gamma_L \alpha \beta (L+I) L k_H}{1.33 d (n+1)}} \qquad (C3.33)$$

Diagonoal Members

$$\mathrm{RRF} = \frac{S_u - \dfrac{\gamma_D \alpha w L^2 k_D}{8 d (n+1)}}{\dfrac{\gamma_L \alpha \beta (L+I) L k_D}{4 d (n+1)}} \qquad (C3.34)$$

K-Bracing (Type 2)

Horizontal Member

$$\mathrm{RRF} = \frac{S_u - \dfrac{\gamma_D \alpha w L^2 k_H}{2 d (n+1)}}{\dfrac{\gamma_L \alpha \beta (L+I) L k_H}{d (n+1)}} \qquad (C3.35)$$

Diagonal Members

$$\mathrm{RRF} = \frac{S_u - \dfrac{\gamma_D \alpha w L^2 k_D}{8 d (n+1)}}{\dfrac{\gamma_L \alpha \beta (L+I) L k_D}{4 d (n+1)}} \qquad (C3.36)$$

In Eq's. C3.33 to C3.36, S_u is controlled by compression. Thus $S_u = 0.85\, f_{cr}\, A_{CH}$ for the horizontal member, and $S_u = 0.85\, f_{cr}\, A_{CD}$ for the diagonal members. The use of the 0.85 factor is consistent with the AASHTO Load Factor equations for column sections in Ref. C-2.

Required A_{CH} and A_{CD} to Assure Redundancy

The required cross-section areas A_{CH} and A_{CD} of the cross bracing bottom

horizontal member and the diagonal members to design for redundancy for a given vehicle is found by setting the redundancy rating factors given by Eq's. C3.33 to C3.36 equal to unity and solving for the required A_{CH} and A_{CD} as follows:

X-Bracing and K-Bracing (Type 1)

Horizontal Member

$$\text{Req'd. } A_{CH} = \frac{\alpha L k_H}{2.67 d (n+1)\, 0.85 f_{cr}} \big(\gamma_D w L + 2\gamma_L \beta (L+I)\big) \qquad (C3.37)$$

Diagonal Members

$$\text{Req'd. } A_{CD} = \frac{\alpha L k_H}{8 d (n+1)\, 0.85 f_{cr}} \big(\gamma_D w L + 2\gamma_L \beta (L+I)\big) \qquad (C3.38)$$

K-Bracing (Type 2)

Horizontal Members

$$\text{Req'd. } A_{CH} = \frac{\alpha L k_H}{2 d (n+1)\, 0.85 f_{cr}} \big(\gamma_D w L + 2\gamma_L \beta (L+I)\big) \qquad (C3.39)$$

Diagonal Members

$$\text{Req'd. } A_{CD} = \frac{\alpha L k_D}{8 d (n+1)\, 0.85 f_{cr}} \big(\gamma_D w L + 2\gamma_L \beta (L+I)\big) \qquad (C3.40)$$

Table C-2 Variation of ϕ_t, $(\phi_t - \phi_c)$ and $(\phi_t + \phi_c)$

Bridge Span			100 ft.		150 ft.		200 ft.	
Number of panels, n			5	7	7	9	9	13
ϕ_t	Dead	RRF > 1.0	0.70	0.72	0.72	0.73	0.73	0.75
	Dead	RRF = 1.0	0.73	0.76	0.73	0.75	0.73	0.76
	Dead	RRF < 1.0		0.77	0.74	0.76	0.75	0.78
	Live	RRF > 1.0	0.82	0.83	0.82	0.83	0.82	0.82
	Live	RRF = 1.0	0.85	0.86	0.84	0.84	0.83	0.84
	Live	RRF < 1.0	0.86	0.87	0.84	0.86	0.85	0.87
$\phi_t - \phi_c$	Dead	RRF > 1.0	0.75	0.59	0.74	0.62	0.70	0.52
	Dead	RRF = 1.0	0.55	0.42	0.61	0.51	0.64	0.48
	Dead	RRF < 1.0			0.54	0.45	0.51	0.38
	Live	RRF > 1.0	0.43	0.38	0.50	0.45	0.53	0.40
	Live	RRF = 1.0	0.28	0.23	0.39	0.33	0.46	0.36
	Live	RRF < 1.0			0.31	0.28	0.34	0.27
$\phi_t + \phi_c$	Dead	RRF > 1.0	0.65	0.85	0.70	0.84	0.76	0.98
	Dead	RRF = 1.0	0.91	1.10	0.85	0.99	0.82	1.04
	Dead	RRF < 1.0		1.20	0.94	1.07	0.99	1.18
	Live	RRF > 1.0	1.31	1.28	1.14	1.21	1.15	1.24
	Live	RRF = 1.0	1.42	1.49	1.29	1.35	1.20	1.32
	Live	RRF < 1.0		1.60	1.37	1.44	1.36	1.49

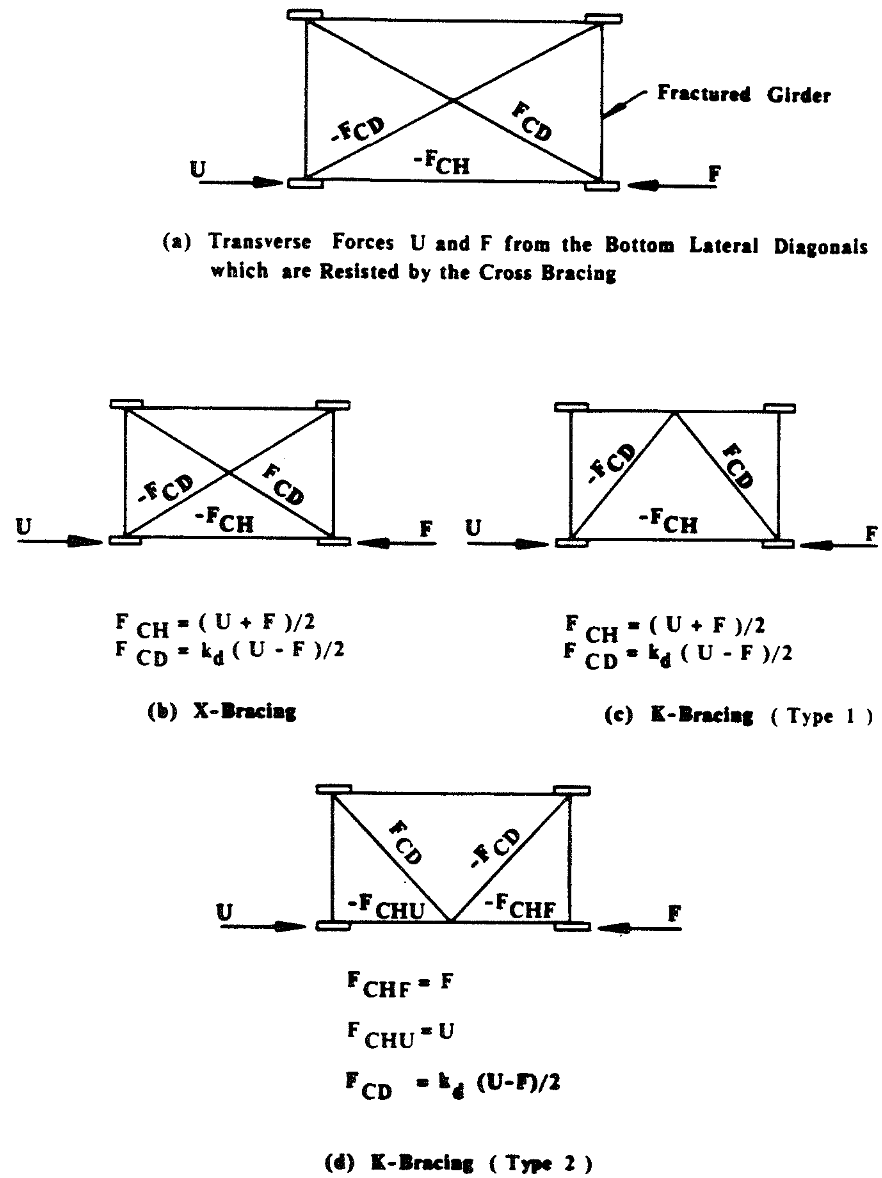


Figure C-18 Forces Induced in the Cross Bracing Diaphragms

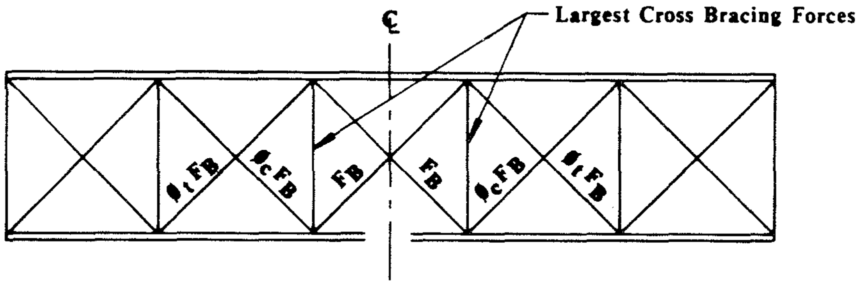


Figure C-19 Forces in the Bottom Lateral Diagonals of the Midspan and Two Adjacent Panels

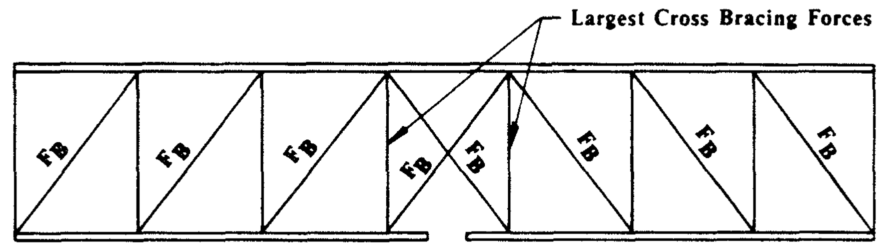


Figure C-20 Forces in the Bottom Lateral Tension Diagonals - Load Factor Method

CHAPTER C4

REQUIREMENTS OF THE TOP LATERAL BRACING DIAGONALS

FORCES INDUCED IN THE DIAGONAL MEMBERS

The forces from the cross bracing diagonal members shown in Fig. C-18 are resisted by the top lateral bracing which is also assumed to be of the X-type. It is also assumed that the cross-section areas of all top lateral diagonal members are equal. It is further assumed that the transverse shear force (the shear force in the plane of the top lateral bracing perpendicular to the girders) in any given panel is shared equally by the two diagonal members, one in tension, the other in compression.

In the absence of applied horizontal loads, equilibrium requires that the transverse shear in a given panel of the top lateral bracing be equal and opposite to the transverse shear in the same panel of the bottom lateral system. Thus for symmetrical loading there is zero shear in the midspan panel of top lateral bracing and zero force in the corresponding diagonals. The maximum shear force occurs in the two panels adjacent to midspan (for midspan fracture). Thus the maximum tension or compression force, F_T, in a top lateral bracing diagonal member is

$$F_T = 0.5\,(\phi_t + \phi_c)\,F_B \qquad \text{(C4.1)}$$

where F_B is the tension force in the midspan diagonal members of the bottom lateral bracing due to dead, live and impact loads (Eq. C2.10) and ϕ_t and ϕ_c are force reduction factors as defined in Chapter C3.

ALLOWABLE STRESS METHOD

Redundancy Rating Factor - RRF

Multiplying Eq's. C2.6 and C2.7 by A_B and adding

$$F_B = \frac{\alpha\ w\ L^2\ V_{DA}}{8\ d\ n} + \frac{\alpha\ \beta(L+I)\ L\ V_{LA}}{4\ d\ n} \qquad \text{(C4.2)}$$

With $0.5\ (\phi_t + \phi_c) = 0.75$, then from Eq. C4.1,

$$F_T = \frac{\alpha\ w\ L^2\ V_{DA}}{10.67\ d\ n} + \frac{\alpha\ \beta(L+I)\ L\ V_{LA}}{5.33\ d\ n} \qquad \text{(C4.3)}$$

If $F_T = A_T\ f_T$ where A_T is the cross-section area and f_T is the tension or compression stress in a top lateral bracing diagonal member, then

$$f_T = \frac{\alpha\ w\ L^2\ V_{DA}}{10.67\ d\ n\ A_T} + \frac{\alpha\ \beta(L+I)\ L\ V_{LA}}{5.33\ d\ n\ A_T} \qquad \text{(C4.4)}$$

where the dead load stress, f_D, and the live load plus impact stress, f_L, are

$$f_D = \frac{\alpha\ w\ L^2\ V_{DA}}{10.67\ d\ n\ A_T} \qquad \text{(C4.5)}$$

$$f_L = \frac{\alpha\ \beta(L+I)\ L\ V_{LA}}{5.33\ d\ n\ A_T} \qquad \text{(C4.6)}$$

(Eq's. C4.5 and C4.6 can be compared with Eq's. C2.6 and C2.7).

Substituting Eq's. C4.5 and C4.6 into Eq. C2.5.

$$\mathrm{RRF} = \frac{f_{all} - \dfrac{\alpha\, w\, L^2\, V_{DA}}{10.67\, d\, n\, A_T}}{\dfrac{\alpha\, \beta(L+I)\, L\, V_{LA}}{5.33\, d\, n\, A_T}} \qquad \text{(C4.7)}$$

where V_{DA} and V_{LA} are given by Eq's. C2.12 and C2.13, and f_{all} is the allowable tension or compression stress for a top lateral bracing diagonal member. (Eq. C4.7 can be compared with Eq. C2.8.)

Required A_T to Assure Redundancy

The required area A_T to design for redundancy is found by setting the RRF (Eq. C4.7) equal to unity and changing V_{DA} and V_{LA} to V_{DB} and V_{LB} as was discussed in Chapter C2.

$$\text{Req'd. } A_T = \frac{\alpha\, L}{10.67\, d\, n\, f_{all}}(w\, L\, V_{DB} + 2\, \beta(L+I)\, V_{LB}) \qquad \text{(C4.8)}$$

where V_{DA} and V_{LA} are given by Eq's. C2.12 and C2.13. (Eq. C4.8 can be compared with Eq. C2.9).

LOAD FACTOR METHOD

Redundancy Rating Factor - RRF

As before, the redundancy rating factor, RRF, for the top lateral bracing members is given by

$$\mathrm{RRF} = \frac{\phi S_u - \gamma_D D}{\gamma_L\, (L+I)} \qquad \text{(C4.9)}$$

where: ϕ = resisstance factor, equal to 1.0 for the top lateral bracing diagonal member

S_u = maximum strength in tension or compression

D = dead load tension or compression force in the top lateral bracing diagonal member

L+I = live load plus impact tension or compression force in the top lateral bracing diagonal member

γ_D = load factor and dead load

γ_L = load factor for live load

Since, for the bottom lateral bracing diagonal members, all tension diagonals have yielded and all compression diagonals have buckled, then the maximum tension or compression force, F_T, in a top lateral bracing diagonal member is

$$F_T = 0.5\, F_B \qquad \text{(C4.10)}$$

The corresponding forces due to dead and live load plus impact are therefore one-half the values given by Eq's. C2.17 and C2.18, or

$$F_{TD} = \frac{\alpha\, w\, L^2}{8\, d\, (n+1)} \qquad \text{(C4.11)}$$

$$F_{TL} = \frac{\alpha\, \beta(L+I)\, L}{4\, d\, (n+1)} \qquad \text{(C4.12)}$$

Substituting Eq's. C4.11 and C4.12 into Eq. C4.10,

$$\mathrm{RRF} = \frac{S_u - \dfrac{\gamma_D\, \alpha\, w\, L^2}{8\, d\, (n+1)}}{\dfrac{\gamma_L\, \alpha\, \beta(L+I)\, L}{4\, d\, (n+1)}} \qquad \text{(C4.13)}$$

where $S_u = f_y\, A_T$ for tension diagonals, and $S_u = 0.85\, f_{cr}\, A_T$ for compression diagonals.

Required A_T to Assure Redundancy

As before, setting Eq. C4.13 equal to unity, the required area A_T to design for redundancy is

$$\text{Req'd. } A_T = \frac{\alpha\, L}{8\, d\, (n+1)\, 0.85\, f_{cr}} \left(\gamma_L\, w\, L + 2\, \gamma_L\, \beta(L+I)\right) \qquad \text{(C4.14)}$$

CHAPTER C5

INFLUENCE OF ALTERNATE FRACTURE LOCATION

The redundancy rating factors, RRF, and required cross-section areas of the various members of the alternate load path system consisting of top and bottom lateral bracing and cross bracing were determined in Chapters C2, C3 and C4 considering midspan fracture of one of the two main steel girders. The studies reported in Ref's. C-3 and C-4 investigate the influence of fracture at locations other than at midspan. For fracture locations which produce lower RRF's or larger required cross-section areas, reduction or amplification factors, respectively, can be used to adjust the previous results. These factors are briefly discussed and evaluated in the following sections. Further details are reported in Ref's. C-3 and C-4.

BOTTOM LATERAL BRACING

Allowable Stress Method

The finite element study described in Chapter C2 was also used to investigate the influence of alternate fracture locations. Near full-depth girder fractures were introduced into the computer models in each consecutive panel from midspan to the end of the girder.

The results of this study are shown in Table C-3. Two span lengths and three values of n (number of panels) were used in the study. For each bridge the cross-section area, A_B, of each bottom lateral diagonal member was calculated by Eq. C2.9 and kept constant, as the fracture was moved to each panel. The maximum stress always occurred in the two tension diagonals at midspan, for midspan fracture, or in one of the two diagonals in the adjacent panel, when fracture occurred in that panel.

The influence of variations in A_B was also studied. These results are shown in Table C-4. The highest and lowest values of A_B represent variations of 25% of the Req'd. A_B given by Eq. C2.9.

The maximum stress again always occurs in one or both of the tension diagonals in the panel containing the fracture, either at midspan or adjacent to midspan. The stresses shown in Table C-4 are for the 150 ft. span. Similar results were obtained for the 100 and 200 ft. spans.

Redundancy Rating Factor - RRF The stress increases shown in Table C-4 are applicable to the situation where the RRF is being calculated for an existing bridge. The largest increases occur for a RRF < 1.0. The study reported in Ref's. C-3 and C-4 recommends that when computing the RRF a 10% increase in tension stress in the bottom lateral diagonals be assumed. Thus, multiplying Eq's. C2.6 and C2.7 by 1.1, Eq. C2.8 becomes

$$RRF = \frac{f_{all} - \dfrac{\alpha\, w\, L^2\, V_{DA}}{7.3\, d\, A_B\, n}}{\dfrac{\alpha\, \beta(L+I)\, L\, V_{LA}}{3.6\, d\, A_B\, n}} \qquad (C5.1)$$

Required A_B to Assure Redundancy The stress increases shown in Table C-3 are applicable when computing the required design area A_B for the condition RRF = 1.0. The largest increase is less than 5%. It is suggested, however, that a 10% increase be conservatively assumed so that the required A_B can be determined by setting Eq. C5.1 equal unity and replacing V_{DA} and V_{LA} with V_{DB} and V_{LB} as before.

$$\text{Req'd. } A_B = \frac{\alpha\, L}{7.3\, d\, n\, f_{all}}\left(w\, L\, V_{DB} + 2\, \beta(L+I)\, V_{LB}\right) \qquad (C5.2)$$

Load Factor Method

Figure C-21 shows the mathematical model used to compute the tension diagonal forces as a function of fracture location. As before it is assumed that all compression diagonals are buckled and all tension diagonals are yielding on the short side of the fracture as shown by the dashed and heavy lines in the figure. The number of tension diagonals which resist the total force $F = F_3 + F_4 + F_5$ is $\left((n+1)/\, 2 - i\right)$, where n is the number of panels in the span length $\left(n=9, \text{ for example, in Fig. C-21(a)}\right)$, and i is the number of full panels from midspan to the panel containing the fracture $\left(i=2, \text{ for example, in Fig. C-21 (a)}\right)$.

The forces acting on the fractured girder, again conservatively ignoring the vertical forces from the cross bracing, are shown in Fig. C-21(b). As before, assuming zero moment at the top of the fracture, F is given by

$$F = \left(1 - \left(\tfrac{2\,i}{n}\right)^2\right)\left(\frac{w\, L^2}{8\, d} + \frac{\beta(L+I)\, L}{4\, d}\right) \qquad (C5.3)$$

Since the number of tension diagonals is ((n+1)/ 2 - i) then the force F_B carried by each tension diagonal is

$$F_B = \frac{\alpha\left(1 - \left(\tfrac{2\,i}{n}\right)^2\right)}{\frac{n+1}{2} - i}\left(\frac{w\, L^2}{8\, d} + \frac{\beta(L+I)\, L}{4\, d}\right) \qquad (C5.4)$$

where α is the ratio of the diagonal length to the panel length, as before.

The amplification factor, AF, is therefore found by dividing Eq. C5.4 by the sum of Eq's. C2.17 and C2.18.

$$AF = \frac{\left(1 - \left(\tfrac{2\,i}{n}\right)^2\right)\frac{n+1}{2}}{\frac{n+1}{2} - i} \qquad (C5.5)$$

Table C-5 shows the value of the amplification factor as a function of fracture location for several combinations of i and n. The maximum, or near maximum AF occurs when $i = (n-3)/2$ which is the second panel from the end of a span.

Substituting this value of i into Eq. C4.5 gives the near maximum value of AF as follows:

$$AF = 0.75\left(1 + \frac{1}{n}\right)\left(2 - \frac{3}{n}\right) \qquad (C5.6)$$

Redundancy Rating Factor - RRF The tension force due to dead load, F_D, and the tension force due to live load plus impact, F_L, are now determined by multiplying Eq's. C2.17 and C2.18 by Eq. C5.6.

$$F_D = AF \frac{\alpha\, w\, L^2}{4\, d\, (n+1)} \qquad (C5.7)$$

$$F_L = AF \frac{\alpha\, \beta(L+I)\, L}{2\, d\, (n+1)} \qquad (C5.8)$$

Thus redundancy rating factor for the bottom lateral tension diagonals is

$$RRF = \frac{F_y\, A_B - AF \dfrac{\gamma_D\, \alpha\, w\, L^2}{4\, d\, (n+1)}}{AF \dfrac{\gamma_L\, \alpha\, \beta(L+I)\, L}{2\, d\, (n+1)}} \qquad (C5.9)$$

where AF is given by Eq. C5.6.

(Eq. C5.9 is to be compared with Eq. C2.19).

Required A_B to Assure Redundancy The required A_B to design for redundancy of the tension diagonals is found by setting Eq. C5.9 equal to unity, or

$$\text{Req'd. } A_B = AF \frac{\alpha\, L}{4\, d\, (n+1)\, F_y}\Big(\gamma_D\, w\, L + 2\, \gamma_L\, \beta(L+I)\Big) \qquad (C5.10)$$

where AF is given by Eq. C5.6.

(Eq. C5.10 is to be compared with Eq. C2.20).

Serviceability

The finite element investigation described in Chapter C2 was used to study the influence of alternate fracture locations on the girder deflection and deck slope. The study showed that midspan fracture produced the largest girder deflected and transverse deck slope, both of which occur at midspan. However, the largest longitudinal slope occurs when the fracture occurs in an end panel. In this case the longitudinal slope of the deck above the fractured girder and between the fracture and the end of the girder is somewhat larger than the longitudinal slope for midspan fracture. This may be a consideration when defining a limiting midspan deflection-to-span length, Δ/L, ratio. Further details are reported in Ref's. C-3 and C-4.

The results of this study are shown in Table C-6. The cross-section area of the bottom lateral diagonals is computed from Eq. C2.29 for each of the three study bridges shown in the table. For each bridge the specified Δ/L limit used in Eq. C2.29 varies from 1/100 to 1/300 as shown at the top of the table. The midspan deflection of the fractured girder is shown as Δ in the second column. For example, for $L = 100$ ft. and a specified $\Delta/L = 1/100$, Δ is 11.41 in. for midspan fracture and 3.92 in. for end panel fracture. The midspan deflection decreases considerably as the fracture occurs further from midspan.

In the second column, Δ/L is the computed midspan deflection-to-span ratio. For $L = 100$ ft. and a specified $\Delta/L = 1/100$, the computed Δ/L is 1/105. In every case the computed Δ/L is slightly less than the limiting Δ/L used in Eq. C2.29.

The term Δ_r in the second column is the ratio of the maximum deflection for end panel fracture to the maximum deflection for midspan fracture. Using the above examples, $\Delta_r = 3.92/11.41 = 0.34$, as shown in the table.

The term Θ_r in the second column is the ratio of the maximum longitudinal girder slope for end panel fracture to the maximum longitudinal girder slope for midspan fracture. Slope as used here is the slope of the straight line (chord) between the fracture and the end of the fractured girder. Using the above example, end panel fracture increases the longitudinal girder slope by a factor of 1.72. This slope occurs in the short distance between the end panel fracture and the end of the girder. The transverse deck slope at the end panel for an end panel fracture is less than the transverse slope at midspan for midspan fracture.

For the study bridges Θ_r varies from 1.57 to 1.79, as shown in the table. When selecting a limiting midspan Δ/L ratio for design or rating based on midspan deflection for midspan fracture, it should be recognized that the longitudinal slope of the deck above the fractured girder may increase substantially when the fracture is not at midspan. From the finite element study an estimate of the average value of Θ_r can be obtained from the following equation:

$$\Theta_r = 1.8 - \frac{0.8}{n} \qquad \text{(C5.11)}$$

where n is the number of panels in the span.

The maximum longitudinal deck slope $\Theta_{long.}$ is found by multiplying the deck slope $2\Delta/L$ for midspan fracture by Θ_r from Eq. C5.11,

$$\Theta_{long.} = (3.6 - \frac{1.6}{n}) \frac{\Delta}{L} \qquad \text{(C5.12)}$$

The maximum transverse deck slope $\Theta_{tran.}$ will occur at midspan for midspan fracture. For girder spacing S and midspan deflection Δ,

$$\Theta_{tran.} = \frac{\Delta}{S} \qquad \text{(C5.13)}$$

CROSS BRACING

Allowable Stress Method

Results of the finite element study reported in Refs. C-3 and C-4 indicate that, for the study bridges, maximum forces in the cross bracing members for the configurations shown in Fig. C-18 are generated for the midspan fracture case.

Load Factor Method

It was previously shown, in discussing Table C-5, that the critical fracture is located in the second panel from the end of a span. Figure C-22 shows the location of this fracture. For purposes of determining the forces in the cross bracing it is assumed that both tension diagonals in the panel containing the fracture are at their yield stress. The tension diagonal in the end panel is also assumed to be at the yield stress. The tension force in these three diagonals is therefore $(AF)F_B$ as shown in Fig. C-22 where $F_B = A_B F_y$ (Eq. C2.10), and the amplification factor, AF, is given by Eq. C5.6.

Cross Bracing AC - Fig. C-22 Referring to Fig's. C-18 and C-22 the forces U and F are therefore

$$U = 2(AF)\, F_B\, k_H \qquad \text{(C5.14)}$$

$$F = (AF)\, F_B\, k_H \qquad \text{(C5.15)}$$

Therefore, from Fig's. C-18(b), (c) and (d) the forces in the cross bracing horizontal and diagonal members as follows:

X-Bracing and K-Bracing (Type 1)

$$F_{CH} = 1.5\, (AF)\, F_B\, k_H \qquad \text{(C5.16)}$$

$$F_{CD} = 0.5\, (AF)\, F_B\, k_D \qquad \text{(C5.17)}$$

K-Bracing (Type 2)

$$F_{CH} = 2\, (AF)\, F_B\, k_H \qquad \text{(C5.18)}$$

$$F_{CD} = 0.5\, (AF)\, F_B\, k_D \qquad \text{(C5.19)}$$

Eq's. C5.16 to C5.19 are to be compared with Eq's. C3.29 to C3.32, where $k_D = k_d\ k_H$ as before.

Cross Bracing BD - Fig. C-22 Referring to Fig's. C22 and C26, the forces U and F are therefore

$$U = 0 \tag{C5.20}$$

$$F = (AF)\ F_B\ k_H \tag{C5.21}$$

Therefore, from Fig's. C-22(b), (c) and (d) the forces in the cross bracing horizontal and diagonal members are as follows.

X-Bracing and K-Bracing (Type 1)

$$F_{CH} = 0.5\ (AF)\ F_B\ k_H \tag{C5.22}$$

$$F_{CD} = 0.5\ (AF)\ F_B\ k_D \tag{C5.23}$$

K-Bracing (Type 2)

$$F_{CH} = (AF)\ F_B\ k_H \tag{C5.24}$$

$$F_{CD} = 0.5\ (AF)\ F_B\ k_D \tag{C5.25}$$

Eq's. C5.22 to C5.25 can also be compared to Eq's. C3.29 to C3.32.

Examination of Eq's. C5.16 through C5.25 indicates that for the three cross bracing configurations shown in Fig's. C-18(b), (c) and (d), the largest forces in the horizontal and diagonal members occur in the first interior cross bracing, or cross bracing AC in Fig. C-22.

Redundancy Rating Factor - RRF

The RRF's given by Eq's. C3.33 to C3.36 are therefore modified only by the amplification factor, AF, as follows:

X-Bracing and K-Bracing (Type 1)

Horizontal Member

$$RRF = \frac{0.85\ f_{cr}\ A_{CH} - AF\ \dfrac{\gamma_D\ \alpha\ w\ L^2\ k_H}{2.67\ d\ (n+1)}}{AF\dfrac{\gamma_L\ \alpha\ \beta(L+I)\ L\ k_H}{1.33\ d\ (n+1)}} \tag{C5.26}$$

Diagonal Member

$$RRF = \frac{0.85\ f_{cr}\ A_{CD} - AF\ \dfrac{\gamma_D\ \alpha\ w\ L^2\ k_D}{8\ d\ (n+1)}}{AF\dfrac{\gamma_L\ \alpha\ \beta(L+I)\ L\ k_D}{4\ d\ (n+1)}} \tag{C5.27}$$

K-Bracing (Type 2)

Horizontal Member

$$RRF = \frac{0.85\ f_{cr}\ A_{CH} - AF\ \dfrac{\gamma_D\ \alpha\ w\ L^2\ k_H}{2\ d\ (n+1)}}{AF\dfrac{\gamma_L\ \alpha\ \beta(L+I)\ L\ k_H}{d\ (n+1)}} \tag{C5.28}$$

Diagonal Member

$$RRF = \frac{0.85\ f_{cr}\ A_{CD} - AF\ \dfrac{\gamma_D\ \alpha\ w\ L^2\ k_D}{8\ d\ (n+1)}}{AF\dfrac{\gamma_L\ \alpha\ \beta(L+I)\ L\ k_D}{4\ d\ (n+1)}} \tag{C5.29}$$

Required A_{CH} and A_{CD} to Assure Redundancy

As before the required A_{CH}, and A_{CD} for redundancy design are determined by setting Eq's. C5.26 to C5.29 equal to unity.

X-Bracing and K-Bracing (Type 1)

Horizontal Bracing

$$\text{Req'd. } A_{CH} = AF\ \frac{\alpha\ L\ k_H}{2.67\ d\ (n+1)\ 0.85\ f_{cr}}\left(\gamma_D\ w\ L + 2\ \gamma_L\ \beta(L+I)\right) \tag{C5.30}$$

Diagonal Member

$$\text{Req'd. } A_{CD} = AF \frac{\alpha\, L\, k_D}{8\, d\, (n+1)\;\; 0.85\, f_{cr}} \left(\gamma_D\, w\, L + 2\, \gamma_L\, \beta(L+I)\right) \quad (C5.31)$$

K-Bracing (Type 2)

Horitzonal Member

$$\text{Req'd. } A_{CH} = AF \frac{\alpha\, L\, k_H}{2\, d\, (n+1)\;\; 0.85\, f_{cr}} \left(\gamma_D\, w\, L + 2\, \gamma_L\, \beta(L+I)\right) \quad (C5.32)$$

Diagonal Member

$$\text{Req'd. } A_{CD} = AF \frac{\alpha\, L\, k_D}{8\, d\, (n+1)\;\; 0.85\, f_{cr}} \left(\gamma_D\, w\, L + 2\, \gamma_L\, \beta(L+I)\right) \quad (C5.33)$$

where, in Eq's. C5.30 to C5.33, $0.85\ f_{cr}$ is the buckling stress as before (C-2).

Serviceability

Results of the finite element study reported in Ref's. C-3 and C-4 indicate that, for the study bridges, maximum forces in the cross bracing members for the configurations shown in Fig. C-18 are generated for the midspan fracture case, Allowable Stress Method.

TOP LATERAL BRACING

Allowable Stress Method

Results of the finite element study reported in Ref's. C3 and C4 indicate that, for the study bridges, maximum forces in the top lateral bracing compression diagonals are generated for midspan fracture.

Load Factor Method

As discussed previously the largest or near largest tension force in a bottom lateral diagonal is associated with fracture in the second panel from the end of a span. This

force is equal to the tension force in the midspan bottom diagonals, associated with midspan fracture, multiplied by the amplification factor, AF, given by Eq. C5.6. Each top lateral diagonal carries one half of this force, one in tension, one in compression as mentioned before:

Redundancy Rating Factor - RRF The RRF is obtained from Eq. C5.9 as follows:

$$RRF = \frac{0.85\, f_{cr}\, A_T - AF \dfrac{\gamma_D\, \alpha\, w\, L^2}{8\, d\, (n+1)}}{AF \dfrac{\gamma_L\, \alpha\, \beta(L+I)\, L}{4\, d\, (n+1)}} \quad (C5.34)$$

Required A_T to Assure Redundancy Setting Eq. C5.34 to unity

$$\text{Req'd. } A_T = AF \frac{\alpha\, L}{8\, d\, (n+1)\; 0.85\, f_{cr}} \left(\gamma_D\, w\, L + 2\, \gamma_L\, \beta(L+I)\right) \quad (C5.35)$$

Serviceability

Results of the finite element study reported in Ref's. C-3 and C-4 indicate that maximum forces in the top lateral bracing diagonals are generated for the midspan fracture case, Allowable Stress Method.

Table C-3 Maximum Tension Stress (ksi) in a Bottom Lateral Diagonal as a Function of Fracture Location

Fracture Location \ Bridge Span	100 ft, n=5	100 ft, n=7	200 ft, n=9
Midspan	25.2	25.8	26.6
Panel Adjacent to Midspan	25.6	26.7	26.0
Two Panels from Midspan	24.9	26.1	23.5
Three Panels from Midspan	----	25.6	19.4
Four Panels from Midspan	----	----	15.7

Table C-4 Effect of Variations in A_B on The Maximum Tension Stress (ksi) in a Bottom Lateral Diagonal

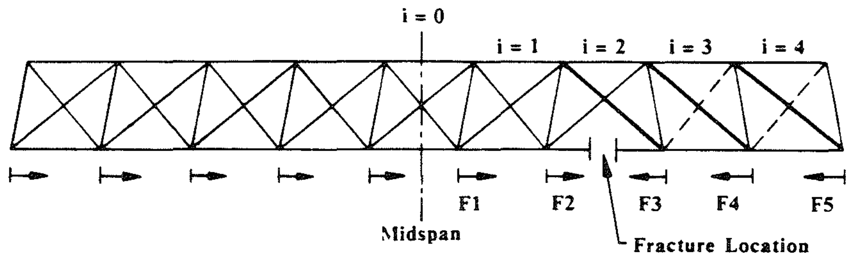
Fracture Location \ A_B	12.0 in^2 (RRF<1)	16.0 in^2 (RRF=1)	20.0 in^2 (RRF>1)
Midspan	31.0	25.8	22.4
Panel Adjacent to Midspan	33.0	26.7	22.8
Percent Increase	6.5	3.5	1.8

Table C-5 Amplification Factor as a Function of Fracture Location

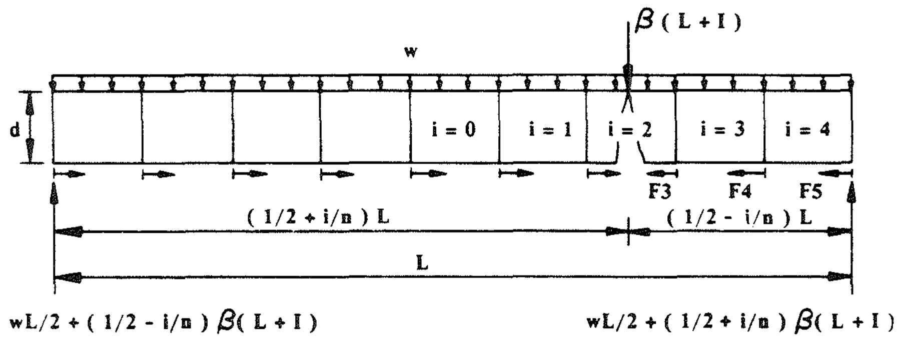
n \ i	1	2	3	4	5	6
5	1.26	1.08				
7	1.22	1.35	1.06			
9	1.19	1.34	1.39	1.05		
11	1.16	1.30	1.40	1.41	1.04	
13	1.14	1.27	1.38	1.45	1.43	1.04
15	1.12	1.24	1.34	1.43	1.48	1.44

Table C-6 Influence of End Panel Fracture on Deck Deflection and Slope

		$\Delta/L = 1/100$		$\Delta/L = 1/200$		$\Delta/L = 1/300$	
		midspan fracture	end panel fracture	midspan fracture	end panel fracture	midspan fracture	end panel fracture
L=100 ft n=5	Δ	11.41"	3.92"	5.79"	1.89"	3.80"	1.19"
	Δ/L	1/105		1/207		1/316	
	Δ_r	1.0	0.34	1.0	0.33	1.0	0.31
	Θ_r	1.0	1.72	1.0	1.63	1.0	1.57
L=150 ft n=9	Δ	15.59"	3.06"	8.27"	1.53"	5.69"	1.00"
	Δ/L	1/115		1/218		1/316	
	Δ_r	1.0	0.20	1.0	0.19	1.0	0.18
	Θ_r	1.0	1.77	1.0	1.67	1.0	1.58
L=200 ft n=9	Δ	19.90"	2.74"	10.74"	1.40"	7.51"	0.94"
	Δ/L	1/121		1/223		1/320	
	Δ_r	1.0	0.14	1.0	0.13	1.0	0.13
	Θ_r	1.0	1.79	1.0	1.69	1.0	1.63



(a) Bottom Lateral Bracing



(b) Forces and Reactions Acting on the Fractured Girder

Figure C-21 Mathematical Model for Computing Bottom Lateral Diagonal Forces as a Function of Fracture Location

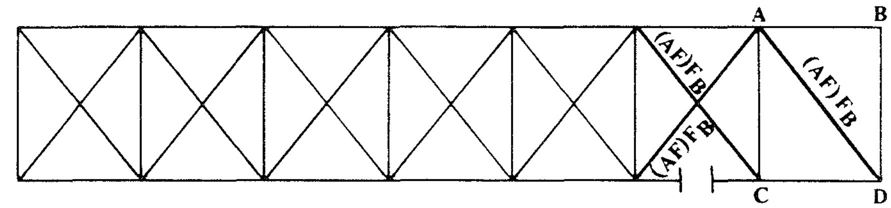


Figure C-22 Bottom Lateral Forces Assumed for the Load Factor Method with Fracture in the Second Panel

CHAPTER C6

DIAGONAL COMPRESSION MEMBERS

BOTTOM LATERAL BRACING

Allowable Stress Method

The finite element study described in Chapter C2 was used to investigate conditions which produce the maximum compression force in the bottom lateral compression members. It is shown that the largest compression force occurs in the end panel diagonal member when fracture occurs in the adjacent (second interior) panel. This compression force increases as the span length decrease and the ratio α increases. Further details are reported in Ref's. C-3 and C-4.

Let P_c be the maximum compression force in the end panel diagonal member which is associated with fracture in the adjacent panel. For the same span let P_t be the maximum tension force in the midspan diagonals which is associated with midspan fracture. The finite element study indicated that, conservatively

$$\frac{P_c}{P_t} = \alpha\,(0.6 - 0.0014L) \qquad \text{(C6.1)}$$

where α is the ratio of the length of a diagonal member to the length of a panel, as before, and L is the span length in feet. For the six study bridges cases P_c/P_t varies from about 0.3 for the 200 ft. span with n=9 to about 0.7 for the 100 ft. span with n=7.

Redundancy Rating Factor - RRF

The redundancy rating factor for compression diagonals is given by Eq. C2.8, multiplied by $m = 1.1\ \frac{P_c}{P_t}$, where the 1.1 factor is from Chapter C5. Therefore

$$m = \alpha\,(0.66 - 0.0015L) \qquad \text{(C6.2)}$$

and

$$RRF = \frac{f_{all} - \dfrac{m\,\alpha\,w\,L^2\,V_{DA}}{8\,d\,n\,A_B}}{\dfrac{m\,\alpha\,\beta(L+I)\,L\,V_{LA}}{4\,d\,n\,A_B}} \qquad \text{(C6.3)}$$

where, in this case, f_{all} is the allowable compression stress for a compression diagonal.

Required A_B to Assure Redundancy

Setting Eq. C6.3 equal to unity, the required A_B for redundancy design is

$$\text{Req'd. } A_B = \frac{m\,\alpha\,L}{8\,d\,n\,f_{all}}\left(w\,L\,V_{DB} + 2\,\beta(L+I)\,V_{LB}\right) \qquad \text{(C6.4)}$$

Allowable Compression Stress - f_{all}

Figure C-23 shows the buckling model for use in computing the allowable stress, f_{all}, for the diagonal compression members. Studies reported in Ref. C-8 show that when the two diagonals are of the same lengths and areas are of the same material, the tension member provides the equivalent of a brace at midlength of the compression member. The compression member will buckle into two half-waves, as shown in the figure. Buckling will occur either in the plane of the bottom lateral bracing or perpendicular to that plane depending on the relative slenderness ratios in these two directions. The effective length is therefore equal to one-half the length of the compression diagonal. The allowable stress is computed in accordance with AASHTO (Ref. C-2) but using the F.S. recommended in Chapter 3 of this report.

Load Factor Method

Since it is assumed that all compression diagonals of the bottom lateral bracing are buckled, then they do not participate when computing the RRF or the required cross-section area.

Serviceability

The results of the finite element study reported in Ref's. C-3 and C-4 indicate that the required cross-section area of bottom lateral diagonals is controlled by the critical tension diagonal for the Allowable Stress Method.

CROSS BRACING

The equations for computing redundancy rating factors, RRF, and required cross-section areas for the cross bracing configurations shown in Fig. C-18 are presented in Chapter C3 and C5. For all cross bracing members the RRF's and required cross-section areas are controlled by the compression member.

The allowable stress, f_{all}, or the AASHTO critical stress, 0.85 f_{cr}, for the X-Type cross bracing diagonal compression member can be computed using the model shown in Fig. C-23. In this case, however, buckling length which is one-half the length of the diagonal member is $\frac{1}{2}$ $k_d S$ where k_d is the ratio of the length of the X-Type diagonal member to the girder spacing S.

For K-Type cross-bracing, f_{all} and 0.85 f_{cr} are computed using the actual length of the diagonal member, $\frac{1}{2}$ $k_d S$, where k_d is the ratio of the length of the diagonal to its horizontal projection.

TOP LATERAL BRACING

The equations for computing RRF's and required cross-section areas for the top lateral bracing diagonals are presented in Chapters C4 and C5. For compression diagonals, f_{all} and 0.85 f_{cr} are computed as for the bottom lateral bracing compression diagonals using the buckling model shown in Fig. C-23.

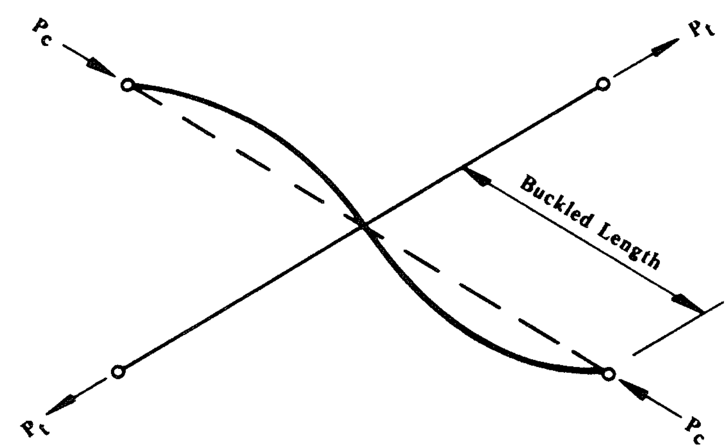


Figure C-23 Buckling Model for X-Type Diagonal Members

CHAPTER C7

CROSS FRAMES AND CROSS TRUSSES

CROSS FRAMES

Figure C-24(a) shows a redundant bracing system with cross frame type diaphragms. With X-type bottom lateral bracing no transverse member is required between the girder flanges. The forces applied to the legs of the cross frame are U and F as shown in the figure, where U and F are given by Eq's. C3.1 and C3.2 (see also Fig. C-19). Force U is always larger than F. The cross frame is therefore proportioned for U. The governing bending momen M_{CF} is taken as U d_{CF} where d_{CF} is the distance to the centroidal axis of the cross frame horizontal. The governing shear V_{CF} is U.

Allowable Stress Method

As discussed in Chapter C5 the maximum value of U is determined for the midspan fracture case.

Referring to Fig's. C-19 and C-24, taking $\phi_t = 0.85$ (Eq. C3.8)

$$M_{CF} = 1.85\ F_B\ k_H\ d_{CF} \tag{C7.1}$$

where F_B is given by Eq. C2.10 and k_H is the ratio of the girder spacing to the length of a bottom lateral diagonal member, as defined in Chapter C3. Combining Eq's. C2.10 and C7.1, the moment due to dead load, M_D, and the moment due to live load plus impact, M_L, are given by

$$M_D = \frac{1.85\ \alpha\ w\ L^2\ V_{DA}\ k_H\ d_{CF}}{8\ d\ n} \tag{C7.2}$$

$$M_L = \frac{1.85\ \alpha\ \beta(L+I)\ L\ V_{LA}\ k_H\ d_{CF}}{4\ d\ n} \tag{C7.3}$$

where V_{DA} and V_{LA} are given by Eq's. C2.12 and C2.13.

With the allowable moment equal to f_{all} S where f_{all} is the allowable stress and S is the section modulus then the redundancy rating factor, RRF, is given by

$$RRF = \frac{f_{all}\ S - \dfrac{\alpha\ w\ L^2\ V_{DA}\ k_H\ d_{CF}}{4.32\ d\ n}}{\dfrac{\alpha\ \beta(L+I)\ L\ V_{LA}\ k_H\ d_{CF}}{2.16\ d\ n}} \tag{C7.4}$$

The shear force, V_{CF}, corresponding to the above RRF is

$$V_{CF} = \frac{\alpha\ L\ k_H}{4.32\ d\ n}\left(w\ L\ V_{DA} + 2\ RRF\ \beta(L+I)\ V_{LA}\right) \tag{C7.5}$$

The required section modulus for design for redundancy is determined, as before, by setting Eq. C7.4 equal to unity

$$\text{Req'd. } S = \frac{\alpha\ L\ k_H\ d_{CF}}{4.32\ d\ n\ f_{all}}\left(w\ L\ V_{DB} + 2\ \beta(L+I)\ V_{LB}\right) \tag{C7.6}$$

where V_{DB} and V_{LB} are given by Eq's. C2.14 and C2.15.

The shear force corresponding to the required section modulus is

$$V_{CF} = \frac{\alpha\ L\ k_H}{4.32\ d\ n}\left(w\ L\ V_{DB} + 2\ \beta(L+I)\ V_{LB}\right) \tag{C7.7}$$

Load Factor Method

As discussed in Chapter C5 the maximum value of U is determined by fracture in the second panel from the end of the span. Referring to Fig. C-26, U is given by Eq. C5.14. Thus

$$M_{CF} = 2\,(AF)\,F_B\,k_H\,d_{CF} \qquad \text{(C7.8)}$$

where the amplification factor, AF, is given by Eq. C5.6.

The dead and live load moments are given by

$$M_D = \frac{4\,\alpha\,w\,L^2\,k_H\,d_{CF}}{8\,d\,(\,n+1\,)} \qquad \text{(C7.9)}$$

$$M_L = \frac{4\,\alpha\,\beta(L+I)\,L\,k_H\,d_{CF}}{4\,d\,(\,n+1\,)} \qquad \text{(C7.10)}$$

where for midspan fracture bottom lateral compression diagonals are assumed buckled and (n+1)/2 tension diagonals have yielded.

The redundancy rating factor, RRF, is therefore given by,

$$RRF = \frac{M_u - A_F\,\dfrac{\gamma_D\,\alpha\,w\,L^2\,k_H\,d_{CF}}{2\,d\,(n+1)}}{AF\,\dfrac{\gamma_L\alpha\,\beta(L+I)\,L\,k_H\,d_{CF}}{d\,(n+1)}} \qquad \text{(C7.11)}$$

where $M_u = F_y$ S for noncompact sections, $M_u = F_y$ Z for compact sections and F_y is the yield stress level. Factors γ_D and γ_L are the dead and live load factors, respectively, suggested in Chapter 3 of this report.

The shear force, V_{CF}, corresponding to the above RRF is

$$V_{CF} = AF\,\frac{\alpha\,L\,k_H}{2\,d\,(n+1)}\Big(\gamma_D\,w\,L + 2\,RRF\,\gamma_L\beta(L+I)\Big) \qquad \text{(C7.12)}$$

The required S or Z for design for redundancy is

$$\text{Req'd. S or Z} = AF\,\frac{\alpha\,L\,k_H\,d_{CF}}{2\,d\,(n+1)\,F_y}\Big(\gamma_D\,w\,L + 2\,\gamma_L\beta(L+I)\Big) \qquad \text{(C7.13)}$$

The shear force corresponding to Eq. C7.13 is

$$V_{CF} = AF\,\frac{\alpha\,L\,k_H}{2\,d\,(n+1)}\Big(\gamma_D\,w\,L + 2\,\gamma_L\beta(L+I)\Big) \qquad \text{(C7.14)}$$

CROSS TRUSSES

Figure C-24(b) shows a redundant bracing system with cross truss type diaphragms. Redundancy rating factors (RRF) and required areas of the cross truss members are derived in a manner similar to that shown in Chapters C3 and C5 for cross bracing. The resulting equations are only valid for cross trusses with three panels as shown in the figure. Similar derivations can be used for cross trusses with two, four or more panels.

Allowable Stress Method

As discussed in Chapter C5, the maximum values of U and F are determined for the midspan fracture case. The forces U and F are given by Eq's. C3.1 and C3.2.

Horizontal Member

Referring to Fig. C-24 and to Eq's. C3.8, C3.9 and C3.10, and observing that F_{CHU} is the largest forces in the bottom horizontal members, then

$$\frac{5\,U + F}{6} = \left(1 + \frac{4\,\phi_t + (\,\phi_t - \phi_c\,)}{6}\right) F_B\, k_H \tag{C7.15}$$

and

$$F_{CHU} = 1.68\; F_B\, k_H \tag{C7.16}$$

It is assumed that the horizontal member has a constant cross section throughout its length between the girder flanges.

As in Chapter C3, combining Eq's. C2.6, C2.7 and C7.18, where A_{CH} is the cross sectional area of the horizontal member, the dead and live load stresses f_D and f_L are

$$f_D = 1.68\, \frac{\alpha\; w\; L^2\; V_{DA}\; k_H}{8\; d\; n\; A_{CH}} \tag{C7.17}$$

$$f_L = 1.68\, \frac{\alpha\; \beta(L{+}I)\; L\; V_{LA} k_H}{4\; d\; n\; A_{CH}} \tag{C7.18}$$

Thus the redundancy rating factor, RRF, is given by

$$RRF = \frac{f_{all} - \dfrac{\alpha\; w\; L^2\; V_{DA}\; k_H}{4.76\; d\; n\; A_{CH}}}{\dfrac{\alpha\; \beta(L{+}I)\; L\; V_{LA}\; k_H}{2.38\; d\; n\; A_{CH}}} \tag{C7.19}$$

The required cross sectional area for design for redundancy corresponding to unit RRF is

$$\text{Req'd. } A_{CH} = \frac{\alpha\; L\; k_H}{4.76\; d\; n\; f_{all}} \left(w\; L\; V_{DB} + 2\; \beta(L{+}I)\; V_{LB}\right) \tag{C7.20}$$

In Eq's. C7.21 and C7.22, f_{all} is controlled by compression. Thus $f_{all} = f_{cr}/F.S.$ where f_{cr} is the critical buckling stress (see Chapter C6) and F.S. is the factor of safety (see Chapter 3 of this report). V_{DA}, V_{LA}, V_{DB}, V_{LB} and k_H are previously defined.

Diagonal Members

The axial force in each of the six diagonal members is shown in Fig. C-24. It is assumed that all diagonals are equal and that all carry equal force.

Referring to Fig. C-24 and to Eq. C3.10

$$F_{CD} = 0.25\; F_B\; k_D \tag{C7.21}$$

In a manner similar to that followed above, then

$$RRF = \frac{f_{all} - \dfrac{\alpha\; w\; L^2\; V_{DA}\; k_D}{32\; d\; n\; A_{CD}}}{\dfrac{\alpha\; \beta(L{+}I)\; L\; V_{LA}\; k_D}{16\; d\; n\; A_{CD}}} \tag{C7.22}$$

and the required area for design for redundancy corresponding to RRF of unit is

$$\text{Req'd. } A_{CD} = \frac{\alpha\; L\; k_D}{32\; d\; n\; f_{all}} \left(w\; L\; V_{DB} + 2\; \beta(L{+}I)\; V_{LB}\right) \tag{C7.23}$$

where A_{CD} is the area of a diagonal member, k_D is defined in Chapter C3 and $f_{all} = f_{cr}/F.S.$ as discussed above.

Load Factor Method

It's discussed in Chapter C5 the maximum values of U and F are determined by fracture in the second panel from the end of the span. As before

$$U = 2\,(AF)\; F_B\; k_H \tag{C7.24}$$

$$F = (AF)\; F_B\; k_H \tag{C7.25}$$

where the amplification factor is given by Eq. C5.6.

Horizontal Member

Referring to Fig. C-24 and to Eq's. C7.26 and C7.27 then

$$F_{CH} = 1.83\,(AF)\,F_B\,k_H \qquad (C7.26)$$

and

$$RRF = \frac{S_u - AF\,\dfrac{\gamma_D\,\alpha\,w\,L^2\,k_H}{2.19\,d\,(n+1)}}{AF\,\dfrac{\gamma_L\alpha\,\beta(L+I)\,L\,k_H}{1.09\,d\,(n+1)}} \qquad (C7.27)$$

where $S_u = 0.85\,f_{cr}\,A_{CH}$ as in Chapter C3.

The required area for design for redundancy corresponding to RRF unity is

$$\text{Req'd. } A_{CH} = AF\frac{\alpha\,L\,k_H}{2.19\,d\,(n+1)\,0.85\,f_{cr}}\left(\gamma_D\,w\,L + 2\,\gamma_L\,\beta(L+I)\right) \qquad (C7.28)$$

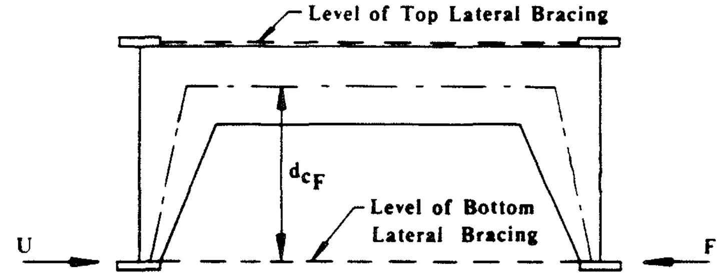
Diagonal Members

Similarly

$$RRF = \frac{S_u - AF\,\dfrac{\gamma_D\,\alpha\,w\,L^2\,k_D}{24\,d\,(n+1)}}{AF\,\dfrac{\gamma_L\alpha\,\beta(L+I)\,L\,k_D}{12\,d\,(n+1)}} \qquad (C7.29)$$

The required area for design for redundancy corresponding to RRF of unit is

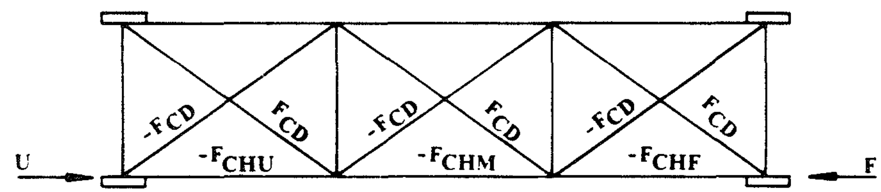
$$\text{Req'd. } A_{CD} = AF\frac{\alpha\,L\,k_D}{24\,d\,(n+1)\,0.85\,f_{cr}}\left(\gamma_D\,w\,L + 2\,\gamma_L\,\beta(L+I)\right) \qquad (C7.30)$$



$Mc_F = Ud_{cF}$

$Vc_F = U$

(a) Maximum Bending Moment and Shear Force of the Cross Frame.



$F_{CHU} = (5U + F)/6$

$F_{CHM} = (U + F)/2$

$F_{CHF} = (U + 5F)/6$

$F_{CD} = k_d\,(U - F)/6$

(b) Axial Forces of the Cross Truss

Figure C-24 Forces Induced in the Cross Frame and Cross Truss

CHAPTER C8

SUMMARY OF DEFINITIONS AND NOTATION

A_B = As built cross-section area of one X-type bottom lateral diagonal

Req'd. A_B = Cross-section area of a cross bracing horizontal or diagonal, respectively

A_{CH}, A_{CD} = Cross-section area of a cross bracing horizontal or diagonal, respectively

Req'd. A_{CH}, A_{CD} = Cross-section area of cross bracing horizontal or diagonal, respectively, when RRF = 1.0

$\bar{A}_F = A_f + \frac{1}{6} A_w$

A_F = weighted average area of girder bottom flange

A_T = As built cross-section area of one X-type top lateral diagonal

Req'd. A_T = Cross-section area of one X-type top lateral diagonal when RRF = 1.0

A_w = weighted average area of girder web

$AF = 0.75 \left(1 + \frac{1}{n} \right) \left(2 - \frac{3}{n} \right)$

E = Young's modulus for steel

L = span length

(L+I) = concentrated resultant tributary live load plus impact or, concentrated tributary lane load, on fractured girder.

M_{CF} = maximum moment of cross frame

M_u = maximum moment capacity of cross frame

$R = \dfrac{A_B}{\alpha^3 \bar{A}_f}$

RRF = Redundancy Rating Factor

S = section modulus of cross frame

V_{CF} = maximum shear force of cross frame

$V_{DA} = \dfrac{n^2 R + 2.4}{n R + 2.4}$

$V_{LA} = \dfrac{n^2 R + 7.0}{n R + 7.0}$

$V_{DB} = 0.8 + 0.36 \dfrac{L}{f_{all}} \geq 1.0$ where Lis in feet and f_{all} is the bottom lateral tension diagonal in ksi

$V_{LB} = 0.8 + 0.18 \dfrac{L}{f_{all}} \geq 1.0$ where Lis in feet and f_{all} is the bottom lateral tension diagonal in ksi

Z = plastic section modulus of cross frame

d = depth of fractured girder, center-to-center of flanges, at midspan

d_{CF} = distance from girder bottom flange to the centroidal axis of the cross frame horizontal

F_y = yield stress

k_d = ratio of the length of a cross bracing diagonal to its horizontal projection

k_H = ratio of girder spacing to the length of a top or bottom lateral diagonal

m = correction factor for bottom lateral diagonal force

n = number of panels of bracing in the span

w = distributed tributary dead load (plus distributed tributary lane load, if any) on fractured girder

$\frac{\Delta}{L}$ = desired midspan deflection-to-span length ratio selected by the engineer for total load conditions

$\left(\frac{\Delta}{L}\right)_D$ = desired midspan deflection-to-span length ratio selected by the engineer for total load conditions

θ_r = ratio of the maximum slope of the fractured girder for end panel fracture to the slope of the girder for midspan fracture

$\theta_{long.}$ = maximum longitudinal deck slope for end panel fracture

$\theta_{tran.}$ = maximum transverse deck slope for end panel fracture

α = ratio of the length of one X-type bottom or top lateral diagonal to the panel length, L/n

γ_D = dead load factor

γ_L = live load factor

ϕ = resistance factor

SUMMARY OF REDUNDANCY DESIGN AND RATING EQUATIONS

ALLOWABLE STRESS METHOD

Bottom Lateral Diagonals

$$RRF = \frac{f_{all} - \dfrac{m\ \alpha\ w\ L^2\ V_{DA}}{8\ d\ n\ A_B}}{\dfrac{m\ \alpha\ \beta(L+I)\ L\ V_{LA}}{4\ d\ n\ A_B}} \tag{C9.1}$$

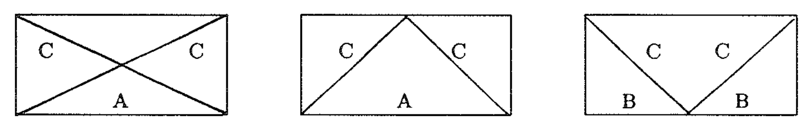
$$\text{Req'd. } A_B = \frac{m\ \alpha\ L}{8\ d\ n\ f_{all}}\ (\ w\ L\ V_{DB} + 2\ \beta(L+I)\ V_{LB}\ ) \tag{C9.2}$$

m= 1.1 when f_{all} is the allowable tension stress

m= α (0.66 − 0.0015L) when f_{all} is the allowable compression stress

Cross Bracing

In the following, f_{all} is the allowable compression stress.



X-Bracing K-Bracing (Type 1) K-Bracing (Type 2)

Member A

$$RRF = \frac{f_{all} - \dfrac{\alpha\ w\ L^2\ V_{DA}\ k_H}{5.93\ d\ n\ A_{CH}}}{\dfrac{\alpha\ \beta(L+I)\ L\ V_{LA}\ k_H}{2.96\ d\ n\ A_{CH}}} \tag{C9.3}$$

$$\text{Req'd. } A_{CH} = \frac{\alpha\ L\ k_H}{5.93\ d\ n\ f_{all}}\ (\ w\ L\ V_{DB} + 2\ \beta(L{+}I)\ V_{LB}\ ) \qquad (C9.4)$$

Member B

$$RRF = \frac{f_{all} - \dfrac{\alpha\ w\ L^2\ V_{DA}\ k_H}{4.32\ d\ n\ A_{CH}}}{\dfrac{\alpha\ \beta(L{+}I)\ L\ V_{LA}\ k_H}{2.16\ d\ n\ A_{CH}}} \qquad (C9.5)$$

$$\text{Req'd. } A_{CH} = \frac{\alpha\ L\ k_H}{4.32\ d\ n\ f_{all}}\ (\ w\ L\ V_{DB} + 2\ \beta(L{+}I)\ V_{LB}\ ) \qquad (C9.6)$$

Member C

$$RRF = \frac{f_{all} - \dfrac{\alpha\ w\ L^2\ V_{DA}\ k_d\ k_H}{10.67\ d\ n\ A_{CD}}}{\dfrac{\alpha\ \beta(L{+}I)\ L\ V_{LA}\ k_d\ k_H}{5.33\ d\ n\ A_{CD}}} \qquad (C9.7)$$

$$\text{Req'd. } A_{CD} = \frac{\alpha\ L\ k_d\ k_H}{10.67\ d\ n\ f_{all}}\ (\ w\ L\ V_{DB} + 2\ \beta(L{+}I)\ V_{LB}\ ) \qquad (C9.8)$$

Cross Frames

$$RRF = \frac{f_{all}\ S - \dfrac{\alpha\ w\ L^2\ V_{DA}\ k_H\ d_{CF}}{4.32\ d\ n}}{\dfrac{\alpha\ \beta(L{+}I)\ L\ V_{LA}\ k_H\ d_{CF}}{2.16\ d\ n}} \qquad (C9.9)$$

The shear force corresponding the above RRF is

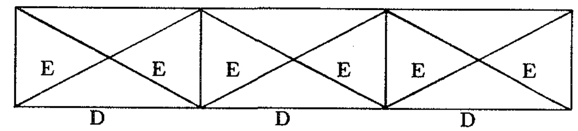
$$V_{CF} = \frac{\alpha\ L\ k_H}{4.32\ d\ n}\ (\ w\ L\ V_{DA} + 2\ RRF\ \beta(L{+}I)\ V_{LA}\ ) \qquad (C9.10)$$

$$\text{Req'd. } S = \frac{\alpha\ L\ k_H\ d_{CF}}{4.32\ d\ n\ f_{all}}\ (\ w\ L\ V_{DB} + 2\ \beta(L{+}I)\ V_{LB}\ ) \qquad (C9.11)$$

The shear force corresponding the required section modulus is

$$V_{CF} = \frac{\alpha\ L\ k_H}{4.32\ d\ n}\ (\ w\ L\ V_{DA} + 2\ \beta(L{+}I)\ V_{LA}\ ) \qquad (C9.12)$$

Cross Trusses



Member D

$$RRF = \frac{f_{all} - \dfrac{\alpha\ w\ L^2\ V_{DA}\ k_H}{4.76\ d\ n\ A_{CH}}}{\dfrac{\alpha\ \beta(L{+}I)\ L\ V_{LA}\ k_H}{2.38\ d\ n\ A_{CH}}} \qquad (C9.13)$$

$$\text{Req'd. } A_{CH} = \frac{\alpha\ L\ k_H}{4.76\ d\ n\ f_{all}}\ (\ w\ L\ V_{DB} + 2\ \beta(L{+}I)\ V_{LB}\ ) \qquad (C9.14)$$

Member E

$$RRF = \frac{f_{all} - \dfrac{\alpha\ w\ L^2\ V_{DA}\ k_d\ k_H}{32\ d\ n\ A_{CD}}}{\dfrac{\alpha\ \beta(L{+}I)\ L\ V_{LA}\ k_d\ k_H}{16\ d\ n\ A_{CD}}} \qquad (C9.15)$$

$$\text{Req'd. } A_{CD} = \frac{\alpha\ L\ k_d\ k_H}{16\ d\ n\ f_{all}}\ (\ w\ L\ V_{DB} + 2\ \beta(L{+}I)\ V_{LB}\ ) \qquad (C9.16)$$

Top Lateral Diagonals

In the following, f_{all} is the allowable compression stress.

$$RRF = \frac{f_{all} - \dfrac{\alpha\ w\ L^2\ V_{DA}}{10.67\ d\ n\ A_T}}{\dfrac{\alpha\ \beta(L+I)\ L\ V_{LA}}{5.33\ d\ n\ A_T}} \qquad (C9.17)$$

$$\text{Req'd. } A_T = \frac{\alpha\ L}{10.67\ d\ n\ f_{all}}\ (\ w\ L\ V_{DB} + 2\ \beta(L+I)\ V_{LB}\ ) \qquad (C9.18)$$

LOAD FACTOR METHOD

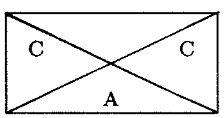
Bottom Lateral Diagonals

$$RRF = \frac{F_y\ A_B - AF\ \dfrac{\gamma_D\ \alpha\ w\ L^2}{4\ d\ (\ n + 1)}}{AF\ \dfrac{\gamma_L\ \alpha\ \beta(L+I)\ L}{2\ d\ (\ n + 1\ )}} \qquad (C9.19)$$

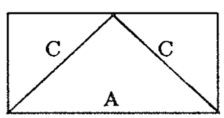
$$\text{Req'd. } A_B = AF\ \frac{\alpha\ L}{4\ d\ (\ n + 1\ )\ F_y}\ (\ \gamma_D\ w\ L + 2\ \gamma_L\ \beta(L+I)\ ) \qquad (C9.20)$$

Cross Bracing

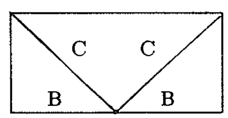
In the following, f_{cr} is the elastic or inelastic critical (buckling) stress.



X-Bracing



K-Bracing (Type 1)



K-Bracing (Type 2)

Member A

$$RRF = \frac{0.85\ f_{cr}\ A_{CH} - AF\ \dfrac{\gamma_D\ \alpha\ w\ L^2\ k_H}{2.67\ d\ (\ n + 1\ )}}{AF\ \dfrac{\gamma_L\ \alpha\ \beta(L+I)\ L\ k_H}{1.33\ d\ (\ n + 1\ )}} \qquad (C9.21)$$

$$\text{Req'd. } A_{CH} = AF\ \frac{\alpha\ L\ k_H}{2.67\ d\ (\ n + 1\ )\ 0.85 f_{cr}}\ (\ \gamma_D\ w\ L + 2\ \gamma_L\ \beta(L+I)\ ) \qquad (C9.22)$$

Member B

$$RRF = \frac{0.85\ f_{cr}\ A_{CH} - AF\ \dfrac{\gamma_D\ \alpha\ w\ L^2\ k_H}{2\ d\ (\ n + 1\ )}}{AF\ \dfrac{\gamma_L\ \alpha\ \beta(L+I)\ L\ k_H}{d\ (\ n + 1\ )}} \qquad (C9.23)$$

$$\text{Req'd. } A_{CH} = AF\ \frac{\alpha\ L\ k_H}{2\ d\ (\ n + 1\ )\ 0.85\ f_{cr}}\ (\ \gamma_D\ w\ L + 2\ \gamma_L\ \beta(L+I)\ ) \qquad (C9.24)$$

Member C

$$RRF = \frac{0.85\ f_{cr}\ A_{CD} - AF\ \dfrac{\gamma_D\ \alpha\ w\ L^2\ k_d\ k_H}{8\ d\ (\ n + 1\ )}}{A\ F\ \dfrac{\gamma_L\ \alpha\ \beta(L+I)\ L\ k_d\ k_H}{4\ d\ (\ n + 1\ )}} \qquad (C9.25)$$

$$\text{Req'd. } A_{CD} = AF\ \frac{\alpha\ L\ k_d\ k_H}{8\ d\ (\ n + 1\ )\ 0.85\ f_{cr}}\ (\ \gamma_D\ w\ L + 2\ \gamma_L\ \beta(L+I)\ ) \qquad (C9.26)$$

Cross Frames

$$RRF = \frac{M_u - AF\ \dfrac{\gamma_D\ \alpha\ w\ L^2\ k_H\ d_{CF}}{2\ d\ (\ n + 1\ )}}{AF\ \dfrac{\gamma_L\ \alpha\ \beta(L+I)\ L\ k_H\ d_{CF}}{d\ (\ n + 1\ )}} \qquad (C9.27)$$

The shear force corresponding the above RRF is

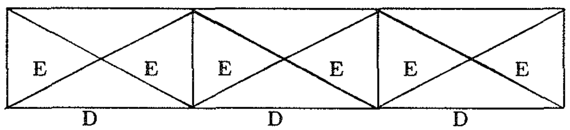
$$V_{CF} = AF\ \frac{\alpha\ L\ k_H}{2\ d\ (\ n + 1\ )}\ (\ \gamma_D\ w\ L + 2\ RRF\ \gamma_L\ \beta(L+I)) \qquad (C9.28)$$

$$\text{Req'd. S or Z } = AF\ \frac{\alpha\ L\ k_H\ d_{CF}}{2\ d\ (\ n + 1\ )\ F_y}\ (\ \gamma_D\ w\ L + 2\ \gamma_L\ \beta(L+I)) \qquad (C9.29)$$

The shear force corresponding the required section modulus is

$$V_{CF} = AF \frac{\alpha\ L\ k_H}{2\ d\ (\ n+1\ )} (\ \gamma_D\ w\ L + 2\ \gamma_L\ \beta(L{+}I)) \qquad (C9.30)$$

Cross Trusses



Member D

$$RRF = \frac{0.85\ f_{cr}\ A_{CH} - AF \dfrac{\gamma_D\ \alpha\ w\ L^2\ k_H}{2.19\ d\ (\ n+1\ )}}{AF \dfrac{\gamma_L\ \alpha\ \beta(L{+}I)\ L\ k_H}{1.09\ d\ (\ n+1\ )}} \qquad (C9.31)$$

$$\text{Req'd. } A_{CH} = AF \frac{\alpha\ L\ k_H}{2.19\ d\ (\ n+1\ )\ 0.85\ f_{cr}} (\ \gamma_D\ w\ L + 2\ \gamma_L\ \beta(L{+}I)) \qquad (C9.32)$$

Member E

$$RRF = \frac{0.85\ f_{cr}\ A_{CD} - AF \dfrac{\gamma_D\ \alpha\ w\ L^2\ k_d\ k_H}{24\ d\ (\ n+1\ )}}{AF \dfrac{\gamma_L\ \alpha\ \beta(L{+}I)\ L\ k_d\ k_H}{12\ d\ (\ n+1\ )}} \qquad (C9.33)$$

$$\text{Req'd. } A_{CD} = AF \frac{\alpha\ L\ k_d\ k_H}{24\ d\ (\ n+1\ )\ 0.85\ f_{cr}} (\ \gamma_D\ w\ L + 2\ \gamma_L\ \beta(L{+}I)) \qquad (C9.34)$$

Top Lateral Diagonals

In the following, f_{cr} is the elastic or inelastic critical (buckling) stress.

$$RRF = \frac{0.85\ f_{cr}\ A_T - AF \dfrac{\gamma_D\ \alpha\ w\ L^2}{8\ d\ (\ n+1\ )}}{AF \dfrac{\gamma_L \alpha\ \beta(L{+}I)\ L}{4\ d\ (\ n+1\ )}} \qquad (C9.35)$$

$$\text{Req'd. } A_T = AF \frac{\alpha\ L}{8\ d\ (\ n+1\ )\ 0.85\ f_{cr}} (\ \gamma_D\ w\ L + 2\ \gamma_L\ \beta(L{+}I)\ ) \qquad (C9.36)$$

Allowable Stress Method

$$\left(\frac{\Delta}{L}\right) = \frac{\alpha^3 L^2}{16 E d^2 n^2 A_B}\left(w L V_{DA} + 2 \beta(L+I) V_{LA}\right) \qquad (C9.37)$$

$$\left(\frac{\Delta}{L}\right)_D = \frac{\alpha^3 L^2}{16 E d^2 n^2 A_B}\left(w L V_{DA}\right) \qquad (C9.38)$$

$$\left(\frac{\Delta}{L}\right)_D = \frac{w L V_{DA}}{W L V_{DA} + 2 \beta(L+I) V_{LA}}\left(\frac{\Delta}{L}\right) \qquad (C9.39)$$

$$\Theta_{long.} = \left(3.6 - \frac{1.6}{n}\right) \frac{\Delta}{L} \qquad (C9.40)$$

$$\Theta_{tran.} = \frac{\Delta}{S} \qquad (C9.41)$$

Load Factor Method

Use above allowable stress equations when

$$\frac{\text{Req'd. } A_B \text{ by LFM}}{\text{Req'd. } A_B \text{ by ASM}} \geq \frac{1}{\gamma_D} \text{ or } \frac{1}{\gamma_L} \qquad (C9.42)$$

where $\gamma_D = \gamma_L$

REDUNDANCY DESIGN AND RATING EXAMPLES

EXAMPLE C1 - Allowable Stress, 110 ft. Span

Problem Statement

Evaluate the after-fracture redundancy of the alternate load path system of the as-built, simple span, noncomposite, two-girder bridge shown in Fig. C-25(C-5) for an AASHTO HS-20 truck with 30% impact. Use the Allowable Stress Method and Eq's. C9.1 to C9.18, as approproate. Use the allowable stresses given in Chapter 3. The evaluation consists of two parts: (1) computation of the RRF for each component of the as-built bracing system, and (2) determination of the required area of each component for the condition, RRF=1.0. Neglect the strength of the floor beam-deck system. Assume that all connections are retrofitted to develop member strengths.

Compute Parameters Common to Eq's. C9.1 to C9.10 and C9.17 and C9.18

Dead Load, w, on Fractured Girder

Slab - $\frac{10}{12}$ (0.15)(33.33)(0.5)	= 2.08 k/ft
Curb - Est. 5.0 ft^3 x 0.15	= 0.75
Girder Web - 120 x 0.75 x $\frac{0.49}{144}$	= 0.31
Girder Flanges - 2 $\left(\frac{18 \times 1.5}{144} \times \frac{66}{110} \right)$ 0.490	= 0.11
- 2 $\left(\frac{18 \times 1.0}{144} \times \frac{44}{110} \right)$ 0.490	= 0.05
Floor Beams, Bracing, Misc. - Est.	= 0.20
w	= 3.50 k/ft
or w	= 0.29 k/in

Live Load plus Impact on Fractured Girder

$\beta(L+I) = \left(\frac{15}{18} \times \frac{1}{2} + \frac{21}{18} \times \frac{1}{2} \right) 72 \times 1.3$ = 93.6 k α Factor

In the 20 ft. panels, the diagonals are 26.91 ft. long. For the 15 ft. panels they are 23.43 ft. long. Conservatively, use the largest α factor.

$$\alpha = 23.43/15.0 = 1.56$$

$\overline{A}_f$ and R

In the calculation of $\overline{A}_f$, A_f is the weighted average area of the girder bottom flange.

$$A_f = \frac{(18x1.5x66)+(18x1x44)}{110} = 23.4 \text{ in}^2$$

$$A_w = 120 \text{ x } 0.75 = 90 \text{ in}^2$$

$$\overline{A}_f = 23.4 \text{ x } 90/6 = 38.4 \text{ in}^2$$

$$R = \frac{3.97}{(1.56)^3 \text{x } 38.4} = 0.03$$

Bottom Lateral Diagonals

Refer to Fig. C-25 for details of the bottom lateral diagonals, and to Eq's. C9.1 and C9.2.

Tension Diagonals

The AASHTO allowable stress for rating is based on the smaller of the yield stress on the gross section and the ultimate stress on the net section. Assuming a 15% reduction in section holes, yield stress on the gross section governs. Thus $f_{all} = 0.85 \ F_y$ as discussed in Chapter 3.

$$f_{all} = 0.85 \text{ x } 36 = 30.6 \text{ ksi}$$

$$V_{DA} = \frac{6^2 (0.03) + 2.4}{6 (0.03) + 2.4} = 1.35$$

$$V_{LA} = \frac{6^2 (0.03) + 7.0}{6 (0.03) + 7.0} = 1.13$$

$$V_{DB} = 0.8 + (0.36) \frac{110}{30.6} = 2.09$$

$$V_{LB} = 0.8 + (0.18) \frac{110}{30.6} = 1.45$$

The as-built redundancy rating factor for the A36 tension diagonals is given by Eq. C9.1, as follows:

$$RRF = \frac{30.6 - \dfrac{1.1 \text{ x } 1.56 \text{ x } 0.29 \ (1{,}320)^2 \ 1.35}{8 \text{ x } 121.5 \text{ x } 6 \text{ x } 3.97}}{\dfrac{1.1 \text{ x } 1.56 \text{ x } 1.0 \ (93.6) \ (1{,}320) \ 1.13}{4 \text{ x } 121.5 \text{ x } 6 \text{ x } 3.97}}$$

$$= \frac{30.6 - 50.6}{20.7} < 0$$

This result shows that the as-built tension diagonals cannot carry the rated dead load and live loads.

The required cross-section area of the tensiom diagonal for A36 steel when the RRF=1.0, and with $f_{all} = 30.6$ ksi in given by Eq. C9.2, as follows:

$$\text{Req'd. } A_B = \frac{1.1 \text{ x } 1.56 \text{ x } 1{,}320}{8 \text{ x } 121.5 \text{ x } 6 \text{ x } 30.6} \ (0.29x1{,}320x2.09 + 2x93.6x1.45) = 13.6 \text{ in}^2$$

For 50 ksi steel and $f_{all} = 0.85 \text{ x } 50 = 42.5$ ksi, V_{DB} and V_{LB} are:

$$V_{DB} = 0.8 + 0.36 \left(\frac{110}{42.5} \right) = 1.73$$

$$V_{LB} = 0.8 + 0.18 \left(\frac{110}{42.5} \right) = 1.27$$

and the required area is

$$\text{Req'd. } A_B = \frac{1.1 \times 1.56 \times 1{,}320}{8 \times 121.5 \times 6 \times 42.5} \left(0.29 \times 1{,}320 \times 1.73 + 2 \times 93.6 \times 1.27 \right) = 8.2 \text{ in}^2$$

It is evident that the tension diagonal must be retrofitted from the as-built 3.97 in^2 in order to achieve after-fracture redundancy.

Compression Diagonals

The allowable compression stress is computed using the unbraced length shown in Fig. C-23 of Appendix C. The longest compression diagonals occur in the 20-ft panels. The unbraced length is therefore $0.5 \sqrt{(20^2 + 18^2)} = 13.45$ ft. or 161.44 in. For the as-built WT 6x13.5 the smallest radius of gyration is 1.52 in. Using the AASHTO value of K=0.75 for riveted, bolted and welded connections (Section 10.54 in Ref. 1 of the main report) the slenderness ratio is therefore (0.75 x 161.44)/1.52 = 79.7. For A36 steel, C_c =126.1 (Ref. 1). Thus, in accordance with the AASHTO rating provisions, with a F.S. = 1.4 as discussed in Chapter 3.

$$f_{all} = \frac{36}{1.4}\left(1 - \frac{(79.7)^2}{4\pi^2}\frac{36}{30{,}000}\right) = 20.7 \text{ ksi}$$

From Eq. C9.2,

$$r = 1.56 \,(0.66 - 0.0015 \times 110) = 0.77$$

The redundancy rating factor for the as-built, A36 compression diagonals is therefore :

$$\text{RRF} = \frac{20.7 - \dfrac{0.77 \times 1.56 \times 0.29\,(1{,}320)^2\, 1.35}{8 \times 121.5 \times 6 \times 3.97}}{\dfrac{0.77 \times 1.56 \times 1.0\,(93.6)\,(1{,}320)\, 1.13}{4 \times 121.5 \times 6 \times 3.97}}$$

$$= \frac{20.7 - 35.4}{14.5} < 0$$

The as-built compression diagonals cannot carry the rated dead and live loads.

The computation of the required area of a compression diagonal for a RRF=1.0 is a trial and error process requiring, as a first step, the selection of a trial section and an allowable compression stress. For a first guess, select a section that meets the area requirement of the tension diagonal. Also assume an unbraced length of 161.44 in., the same as for the as-built diagonal. Assuming 50 ksi steel, select a WT 8x28.5 with an area of 8.38 in^2 and r = 1.60 in. The slenderness ratio is (0.75x161.44)/1.60 = 75.7. The AASHTO allowable stress, with F.S. = 1.4 is

$$f_{all} = \frac{50}{1.4}\left(1 - \frac{(75.7)^2}{4\pi^2}\frac{50}{30{,}000}\right) = 27.1 \text{ ksi}$$

and refer to the previous calculations, $V_{DB} = 1.73$, and $V_{LB} = 1.27$.

Then, the required area of a compression diagonal, as given by Eq. C9.2 is,

$$\text{Req'd. } A_B = \frac{0.77 \times 1.56 \times 1{,}320}{8 \times 121.5 \times 6 \times 27.1} \left(0.29 \times 1{,}320 \times 1.73 + 2 \times 93.6 \times 1.27 \right) = 9.0 \text{ in}^2$$

The area provided is 8.38 in^2 or about 7 percent less than the required area. Therefore, use a WT 8x33.5 of 50 ksi steel. Note that, because of the assumption in Appendix C that all tension and compression diagonals of the bottom lateral bracing are identical, then the WT 8x33.5 in 50 ksi steel is to be used for both the tension and the compression diagonals.

Cross Bracing

Refer to Fig. C-25 for details of the X-type cross bracing, and to Eq's. C9.3, C9.4, C9.7, and C9.8.

Member A

Member A is always subjected to compression stress. The allowable compression stress is computed using the clear distance of 198 in. between girder flanges as the unbraced length and K = 0.75. For the WT 6x20, A_{CH} = 5.89 in^2, r = 1.56 in., and the slenderness ratio is (0.75x198)/1.56 = 95.2. Thus for the as-built A36 horozontal,

$$f_{all} = \frac{36}{1.4}\left(1 - \frac{(95.2)^2}{4\pi^2}\frac{36}{30{,}000}\right) = 18.6 \text{ ksi}$$

also, k_H = 18 / 26.9 = 0.67

With V_{DA} =1.35 and V_{LA} =1.13 as before, the as-built redundancy rating factor for the A36 horizontal is given by Eq. C9.3 as follows:

$$RRF = \frac{18.6 - \dfrac{1.56 \text{ x } 0.29\ (1{,}320)^2\ 1.35 \text{ x } 0.67}{5.93 \text{ x } 121.5 \text{ x } 6 \text{ x } 5.89}}{\dfrac{1.56 \text{ x } 1.0\ (93.6)\ (1{,}320)\ 1.13 \text{ x } 0.67}{2.96 \text{ x } 121.5 \text{ x } 6 \text{ x } 5.89}}$$

$$= \frac{18.6 - 28.0}{11.5} < 0$$

The as-built cross bracing horizontal cannot carry the rated dead and live loads.

The required area of the horizontal member when the RRF=1.0 is given by Eq. C9.4. As a first trial, assume that the as-built horizontal is replaced with a WT 8x33.5 section (same as is used for the bottom lateral diagonals) in 50 ksi steel and with r=2.22 in. Then, the slenderness ratio is (0.75 x 198) / 2.22 = 66.9.

$$f_{all} = \frac{50}{1.4}\left(1 - \frac{(66.9)^2}{4\pi^2}\frac{50}{30{,}000}\right) = 29.0 \text{ ksi}$$

Refer to the previous calculations, V_{DB} = 1.73, and V_{LB} = 1.27.

Then, the required area of a compression horizontal, as given by Eq. C9.4 is,

$$\text{Req'd. } A_{CH} = \frac{1.56 \text{ x } 1{,}320 \text{ x } 0.67}{5.93 \text{ x } 121.5 \text{ x } 6 \text{ x } 29.0}\ (\ 0.29\text{x}1{,}320\text{x}1.73 + 2\text{x}93.6\ \text{x}1.27) = 9.90 \text{ in}^2$$

The area provided is 9.84 in^2 or about 1 percent less than the required area. Thus, use a WT 8x33.5 section of 50 ksi steel. The same member is to be used for the horizontal in the plane of the top lateral bracing as discussed in Chapter C4.

Member C

For X-type cross bracing, one member is in tension, the other in compression. Since all cross bracing diagonal members are assumed identical and all are assumed to carry equal forces then the allowable compression stress will govern. The allowable stress of a diagonal is again computed based on the clear distance between girder flanges. Also, the unbraced length is computed as shown in Fig. C-23 of Appendix C.

For the cross section shown in Fig. C-25, the diagonal distance between the flange-web intersections is $\sqrt{(216^2 + 121.5^2)}$ = 247.83 in. The clear distance along the diagonal between vertical planes defined by the inside edges of the girder flanges is 247.83 x (216 – 18) / 216 = 227.18 in. The unbraced length of the diagonal is therefore 113.6 in. The slenderness ratio of the as-built WT 8x18 with r=1.52 in. is therefore (0.75 x 113.6) / 1.52 = 56.

$$f_{all} = \frac{36}{1.4}\left(1 - \frac{(56)^2}{4\pi^2}\frac{36}{30{,}000}\right) = 23.3 \text{ ksi}$$

The factor k_D can be computed using the actual diagonal member length, or equivalently using the girder spacing and depth as follows:

$$k_D = 247.83/216 = 1.15$$

With $k_H = 0.67$, $V_{DA} = 1.35$, $V_{LA} = 1.13$ and $A_{CD} = 5.30\ in^2$, the redundancy rating factor for an as-built cross bracing diagonal is given by Eq. C9.7 as follows:

$$RRF = \frac{23.3 - \dfrac{1.56 \times 0.29\ (1{,}320)^2\ 1.35 \times 1.15 \times 0.67}{10.67 \times 121.5 \times 6 \times 5.30}}{\dfrac{1.56 \times 1.0\ (93.6)\ (1{,}320)\ 1.13 \times 1.15 \times 0.67}{5.33 \times 121.5 \times 6 \times 5.30}}$$

$$= \frac{23.3 - 19.9}{8.2} = 0.41$$

The as-built cross bracing diagonals are therefore rated at 0.41 x HS-20 or about HS-8.

The required area of the cross bracing diagonal member for a RRF=1.0 is given by Eq. C9.8. As a first trial, assume that the as-built diagonals are replaced with a WT 8x22.5 section in 50 ksi steel, and with $A_{CD} = 6.63\ in^2$ and $r = 1.57$ in. Then, the slenderness ratio is (0.75 x 113.6) / 1.57 = 54.3.

$$f_{all} = \frac{50}{1.4}\left(1 - \frac{(54.3)^2}{4\pi^2}\ \frac{50}{30{,}000}\right) = 31.3 \text{ ksi}$$

Refer to the previous calculations, $V_{DB} = 1.73$, and $V_{LB} = 1.27$.

Then, the required area of a cross bracing diagonal, as given by Eq. C9.8 is,

$$\text{Req'd. } A_{CD} = \frac{1.56 \text{x} 1{,}320 \text{x} 1.15 \text{x} 0.67}{10.67 \text{x} 121.5 \text{x} 6 \text{x} 31.3}\ (0.29 \text{x} 1{,}320 \text{x} 1.73 + 2 \text{x} 93.6 \text{ x} 1.27) = 5.87\ in^2$$

The 6.63 in^2 area provided is sufficient.

Top Lateral Diagonals

Refer to Fig. C-25 for details of the top lateral diagonals and to Eq's. C9.17 and C9.18. Since it is assumed in Chapter C4 of Appendix C that all top lateral diagonals are identical and that each carries the same force in a panel, then the allowable compression stress will govern. Also since the top and bottom lateral diagonals are all WT 6x13.5 sections, referring to the previous calculations, the allowable stress is 19.4 ksi. With $A_T = 3.97\ in^2$, the redundancy rating factor for the as-built A36 diagonals is

$$RRF = \frac{20.7 - \dfrac{1.56 \times 0.29\ (1{,}320)^2\ 1.35}{10.67 \times 121.5 \times 6 \times 3.97}}{\dfrac{1.56 \times 1.0\ (93.6)\ (1{,}320)\ 1.13}{5.33 \times 121.5 \times 6 \times 3.97}}$$

$$= \frac{20.7 - 34.5}{14.2} < 0$$

The as-built top lateral diagonals cannot support the rated dead and live loads.

The computation of the required area of a top lateral diagonal for a RRF=1.0 is given by Eq. C9.18. Again assume that the as-built top lateral diagonals are replaced with WT 8x33.5 sections of 50 ksi steel with $A_T = 9.84\ in^2$ and r= 2.22 in., the same section used for the bottom lateral diagonals. The slenderness ratio is (0.75x161.44)/2.22 = 54.5.

$$f_{all} = \frac{50}{1.4}\left(1 - \frac{(54.5)^2}{4\pi^2}\ \frac{50}{30{,}000}\right) = 31.2 \text{ ksi}$$

Referring to the previous calculations, $V_{DB} = 1.73$, and $V_{LB} = 1.27$. Thus,

$$\text{Req'd. } A_T = \frac{1.56 \times 1{,}320}{10.67 \times 121.5 \times 6 \times 31.2}\ (0.29 \text{x} 1{,}320 \text{x} 1.73 + 2 \text{x} 93.6 \text{x} 1.27) = 7.64\ in^2$$

The 9.84 in^2 area provided is sufficient. Use WT 8x33.5 sections of 50 ksi for all top lateral diagonals.

Total Load and Dead Load Deflections

For midspan fracture the after-fracture deflection ratio of the fractured girder subjected to dead and live loads plus impact is given by Eq. C9.37 in terms of V_{DA} and V_{LA}. In the calculation of V_{DA} and V_{LA}, the A_B provided is used.

$$R = \frac{9.84}{(1.56)^3 \times 38.4} = 0.067$$

$$V_{DA} = \frac{(6)^2 \times 0.067 + 2.4}{6 \times 0.067 + 2.4} = 1.72$$

$$V_{LA} = \frac{(6)^2 \times 0.067 + 7.0}{6 \times 0.067 + 7.0} = 1.27$$

The total load deflection ratio is

$$\frac{\Delta}{L} = \frac{(1.56)^3 \times (1320)^2}{16 \times 30{,}000 \times (121.5)^2 \times (6)^2 \times 9.84}(0.29 \times 1{,}320 \times 1.72 + 2 \times 93.6 \times 1.27)$$

$$= \frac{1}{423}$$

Similarly, the dead load deflection ratio is given by Eq. C9.38.

$$\left(\frac{\Delta}{L}\right)_D = \frac{(1.56)^3 \times (1{,}320)^2}{16 \times 30{,}000 \times (121.5)^2 \times (6)^2 \times 9.84}(0.29 \times 1{,}320 \times 1.72)$$

$$= \frac{1}{576}$$

EXAMPLE C2 - Load Factor Method - 110 ft. Span

Problem Statement

Evaluate the after-fracture redundancy of the alternate load path system of the as-built, simple span, noncomposite, two-girder bridge shown in Fig. C-25 (C-5) for an AASHTO HS-20 truck with 30% impact. Use the Load Factor Method and Eq's. C10.19 to C10.36, as appropriate. Use the load factors given in Chapter 3. The evaluation consists of two parts: (1) computation of the RRF for each component of the as-built bracing system, and (2) determination of the required area of each component for the condition, RRF=1.0. Neglect the strength of the floor beam-deck system. Assume that all connections are retroffited to develop member strengths.

Compute Parameters Common to Eq's. C9.19 to C9.36

Dead Load, w, on Fractured Girder

Unfactored dead load is dentical to Example C1, and the dead load factor, $\gamma_D = 1.18$ as recommended in Chapter 3.

$$w = 0.29 \text{ k/in}$$

$$\text{or } \gamma_D D = 1.18 \times 0.29 = 0.34 \text{ k/in}$$

Live Load plus Impact on Fractured Girder

Unfactored live load plus impact is identical to Example C1, and the live load factor, $\gamma_L = 1.18$ as discussed in Chapter 3.

$$\beta(L+I) = 93.6 \text{ k}$$

$$\text{or } \gamma_L\beta(L+I) = 1.18 \times 93.6 = 110.4 \text{ k}$$

α Factor

Identical to Example C1

$\alpha = 1.56$

$\overline{A}_f$ and R

Identical to Example C1

$$\overline{A}_f = 38.4 \text{ in}^2$$

$$R = 0.03$$

AF Factor

For the six-panel bridge, the amplification factor for critical fracture location is,

$$AF = 0.75 \left(1 + \frac{1}{6} \right) \left(2 - \frac{3}{6} \right) = 1.31$$

Bottom Lateral Diagonals

Refer to Eq's. C9.19 and C9.20.

Tension Diagonals

The as-built redundancy rating factor for the A36 tension diagonals is given by Eq. C10.19, as follows:

$$RRF = \frac{36 \text{ x } 3.97 - 1.31 \dfrac{1.18 \text{ x } 1.56 \text{ x } 0.29 \ (1{,}320)^2}{4 \text{ x } 121.5 \text{ x } (6+1)}}{1.31 \dfrac{1.18 \text{ x } 1.56 \text{ x } 1.0 \ (93.6) \ (1{,}320)}{2 \text{ x } 121.5 \text{ x } (6+1)}}$$

$$= \frac{142.92 - 358.2}{175.2} < 0$$

This result shows that the as-built tension diagonals cannot carry the rated dead load and live loads.

The required cross-section area of the tension diagonal for A36 steel when the RRF=1.0, and with f_y = 36 ksi in given by Eq. C9.20, as follows:

$$\text{Req'd. } A_B = 1.31 \frac{1.56 \text{ x } 1{,}320}{4\text{x}121.5 \text{ x } (6+1) \text{ x } 36} (1.18\text{x}0.29\text{x}1{,}320 + 2\text{x}1.18\text{x}93.6) = 14.8 \text{ in}^2$$

For 50 ksi steel with f_y = 50 ksi, the required area is:

$$\text{Req'd. } A_B = 1.31 \frac{1.56 \text{ x } 1{,}320}{4\text{x}121.5 \text{ x } (6+1) \text{ x } 50} (1.18\text{x}0.29\text{x}1{,}320 + 2\text{x}1.18\text{x}93.6) = 10.7 \text{ in}^2$$

It is evident that the tension diagonal must be modified from the as-built 3.97 in^2 in order to achieve after-fracture redundancy. Thus, use a WT 8x38.5, with A_B=11.3 in^2, section of 50 ksi steel. The same member is to be used for all the bottom lateral diagonals.

Cross Bracing

Refer to Fig. C-25 for details of the X-type cross bracing, and to Eq's. C9.21, C9.22, C9.25, and C9.26.

Member A

Member A is always subjected to compression stress. The critical compression stress is computed using the clear distance of 198 in. between girder flanges as the unbraced length and K = 0.75. For the WT 6x20, A_{CH} = 5.89 in^2, r = 1.56 in., and the slenderness ratio is (0.75x198)/1.56 = 95.2. Thus for the as-built A36 horizontal,

$$f_{cr} = 36 \text{ x} \left(1 - \frac{(95.2)^2}{4\pi^2} \frac{36}{30{,}000} \right) = 26.1 \text{ ksi}$$

also, $k_H = 18 \ / \ 26.9 = 0.67$

The as-built redundancy rating factor for the A36 horizontal is given by Eq. C9.21 as follows:

$$\text{RRF} = \frac{0.85 \text{ x } 26.1 \text{ x } 5.89 - 1.31 \dfrac{1.18 \text{ x } 1.56 \text{ x } 0.29 \ (1{,}320)^2 \text{ x } 0.67}{2.67 \text{ x } 121.5 \text{ x } (6+1)}}{1.31 \dfrac{1.18 \text{ x } 1.56 \text{ x } 1.0 \ (93.6) \ (1{,}320) \text{ x } 0.67}{1.33 \text{ x } 121.5 \text{ x } (6+1)}}$$

$$= \frac{130.7 - 359.5}{176.5} < 0$$

The as-built cross bracing horizontal cannot carry the rated dead and live loads.

The required area of the horizontal member when the RRF=1.0 is given by Eq. C9.22. As a first trial, assume that the as-built horizontal is replaced with a WT 8x38.5 section (same as is used for the bottom lateral diagonals) in 50 ksi steel and with r=2.24 in. Then, the slenderness ratio is (0.75 x 198) / 2.24 = 66.3.

$$f_{cr} = 50 \text{ x} \left(1 - \frac{(66.3)^2 \ \ 50}{4\pi^2 \ \ 30{,}000} \right) = 40.7 \text{ ksi}$$

The required area of a compression horizontal, as given by Eq. C9.22 is,

$$\text{Req'd. } A_{CH} = 1.31 \frac{1.56 \text{ x } 1{,}320 \text{ x } 0.67}{2.67\text{x}121.5\text{x}(6+1)\text{x}0.85\text{x}40.7} (1.18\text{x}0.29\text{x}1{,}320 + 2\text{x}1.18\text{x}93.6)$$

$$= 15.5 \text{ in}^2$$

The area provided is 11.3 in^2 less than the required area. Thus, try a WT 12x52 section in 50 ksi steel and with A_{CH} = 15.3 in^2 and r=2.91 in. Then, the slenderness ratio is (0.75 x 198) / 2.91 = 51.0.

$$f_{cr} = 50 \text{ x} \left(1 - \frac{(51.0)^2 \ \ 50}{4\pi^2 \ \ 30{,}000} \right) = 44.7 \text{ ksi}$$

The required area of a compression horizontal, as given by Eq. C9.22 is,

$$\text{Req'd. } A_{CH} = 1.31 \frac{1.56 \text{ x } 1{,}320 \text{ x } 0.67}{2.67\text{x}121.5\text{x}(6+1)\text{x}0.85\text{x}44.7} (1.18\text{x}0.29\text{x}1{,}320 + 2\text{x}1.18\text{x}93.6)$$

$$= 14.1 \text{ in}^2$$

The area 15.3 in^2 provided is sufficient. Thus, use a WT 12x52 of 50 ksi steel. The same member is to be used for the horizontal in the plane of the top lateral bracing as discussed in Chapter C4.

Member C

For X-type cross bracing, one member is in tension, the other in compression. Since all cross bracing diagonal members are assumed identical and all are assumed to carry equal forces then the allowable compression stress will govern. The allowable stress of a diagonal is again computed based on the clear distance between girder flanges. Also, the unbraced length is computed as shown in Fig. C-23 of Appendix C.

For the cross section shown in Fig. C-25, the diagonal distance between the flange-web intersections is $\sqrt{(216^2 + 121.5^2)}$ = 247.83 in. The clear distance along the diagonal between vertical planes defined by the inside edges of the girder flanges is 247.83 x (216 − 18) / 216 = 227.18 in. The unbraced length of the diagonal is therefore 113.6 in. The slenderness ratio of the as-built WT 8x18 with r=1.52 in. is therefore (0.75 x 113.6) / 1.52 = 56.

$$f_{cr} = 36 \left(1 - \frac{(56)^2 \ \ 36}{4\pi^2 \ \ 30{,}000} \right) = 32.6 \text{ ksi}$$

The factor k_D can be computed using the actual diagonal member length, or equivalently using the girder spacing and depth as follows:

$$k_D = 247.83/216 = 1.15$$

With $k_H = 0.67$, and $A_{CD} = 5.30\ in^2$, the redundancy rating factor for an as-built cross bracing diagonal is given by Eq. C9.24 as follows:

$$RRF = \frac{0.85 \times 32.6 \times 5.30 - 1.31\ \dfrac{1.18 \times 1.56 \times 0.29\ (1{,}320)^2 \times 1.15 \times 0.67}{8 \times 121.5 \times (6+1)}}{1.31\ \dfrac{1.18 \times 1.56 \times 1.0\ (93.6)\ (1{,}320) \times 1.15 \times 0.67}{4 \times 121.5 \times (6+1)}}$$

$$= \frac{146.9 - 138.0}{67.5} = 0.13$$

The as-built cross bracing diagonals are therefore rated at 0.13 x HS-20 or about HS-3.

The required area of the cross bracing diagonal member for a RRF=1.0 is given by Eq. C9.25. As a first trial, assume that the as-built diagonals are replaced with a WT 8x22.5 section in 50 ksi steel, and with $A_{CD} = 6.63\ in^2$ and $r = 1.57$ in. Then, the slenderness ratio is (0.75 x 113.6) / 1.57 = 54.3.

$$f_{cr} = 50 \left(1 - \frac{(54.3)^2}{4\pi^2} \frac{50}{30{,}000} \right) = 43.8 \text{ ksi}$$

Then, the required area of a cross bracing diagonal, as given by Eq. C9.25 is,

$$\text{Req'd. } A_{CD} = 1.31\ \frac{1.56x1{,}320x1.15x0.67}{8x121.5x(6+1)x0.85x43.8}\ (1.18x0.29x1{,}320 + 2x1.18x93.6)$$

$$= 5.52\ in^2$$

The 6.63 in^2 area provided is sufficient.

Top Lateral Diagonals

Refer to Fig. C-25 for details of the top lateral diagonals and to Eq's. C9.35 and C9.36. Since it is assumed in Chapter C4 that all top lateral diagonals are identical and that each carries the same force in a panel, then the critical compression stress will govern. The top and bottom lateral diagonals are all WT 6x13.5 sections. The critical compression stress is computed using the unbraced length shown in Fig. C-23 of Appendix C. The longest compression diagonals occur in the 20-ft panels. The unbraced length is therefore $0.5 \sqrt{(20^2 + 18^2)} = 13.45$ ft. or 161.44 in. Since the as-built WT 6x13.5 the smallest radius of gyration is 1.52 in, the slenderness ratio is therefore (0.075 x 161.44)/1.52 = 79.7. For A36 steel,

$$f_{cr} = 36 \left(1 - \frac{(79.7)^2}{4\pi^2} \frac{36}{30{,}000} \right) = 29.0 \text{ ksi}$$

Thus the redundancy rating factor for the as-built A36 diagonals is

$$RRF = \frac{0.85 \times 29.0 \times 3.97 - 1.31\ \dfrac{1.18 \times 1.56 \times 0.29\ (1{,}320)^2}{8 \times 121.5 \times (6+1)}}{1.31\ \dfrac{1.18 \times 1.56 \times 1.0\ (93.6)\ (1{,}320)}{4 \times 121.5 \times (6+1)}}$$

$$= \frac{97.9 - 179.1}{87.6} < 0$$

The as-built top lateral diagonals cannot support the rated dead and live loads.

The computation of the required area of a top lateral diagonal for a RRF=1.0 is given by Eq. C9.36. Again assume that the as-built top lateral diagonals are replaced with WT 8x38.5 sections of 50 ksi steel with $A_T = 11.3\ in^2$ and r= 2.24 in., the same section used for the bottom lateral diagonals. The slenderness ratio is (0.75x161.44)/2.24 = 54.1.

$$f_{cr} = 50\left(1 - \frac{(54.1)^2}{4\pi^2}\,\frac{50}{30{,}000}\right) = 43.8 \text{ ksi}$$

Then, the required area of the top lateral diagonals, as given by Eq. C9.36 is,

$$\text{Req'd. } A_T = 1.31\,\frac{1.56 \times 1{,}320}{8\text{x}121.5\text{x}(6+1)\text{x}0.85\text{x}43.8}\,(1.18\text{x}0.29\text{x}1{,}320 + 2\text{x}1.18\text{x}93.6)$$

$$= 7.16 \text{ in}^2$$

The 11.3 in^2 area provided is sufficient. Use WT 8x38.5 sections of 50 ksi for all top lateral diagonals.

<u>Total Load and Dead Load Deflections</u>

Check Eq. C9.42 prior to use of Eq's. C9.37, C9.38, and C9.39 with the A_B provided from the Load Factor Method.

$$\frac{10.7}{9.0} > \frac{1}{1.18}$$

The condition is satisfied. Use the area provided of $A_B = 11.3 \text{ in}^2$ in the calculation of the after-fracture deflection of the fractured girder.

$$R = \frac{11.3}{(1.56)^3 \times 38.4} = 0.078$$

$$V_{DA} = \frac{(6)^2 \times 0.078 + 2.4}{6 \times 0.078 + 2.4} = 1.82$$

$$V_{LA} = \frac{(6)^2 \times 0.078 + 7.0}{6 \times 0.078 + 7.0} = 1.31$$

The total load deflection ratio, given by Eq. C9.37, is

$$\frac{\Delta}{L} = \frac{(1.56)^3 \times (1{,}320)^2}{16 \times 30{,}000 \times (121.5)^2 \times (6)^2 \times 11.3}\,(0.29 \times 1{,}320 \times 1.82 + 2 \times 93.6 \times 1.31)$$

$$= \frac{1}{463}$$

Similarly, the dead load deflection ratio is given by Eq. C9.38.

$$\left(\frac{\Delta}{L}\right)_D = \frac{(1.56)^3 \times (1{,}320)^2}{16 \times 30{,}000 \times (121.5)^2\,(6)^2 \times 11.3}\,(0.29 \times 1{,}320 \times 1.82)$$

$$= \frac{1}{625}$$

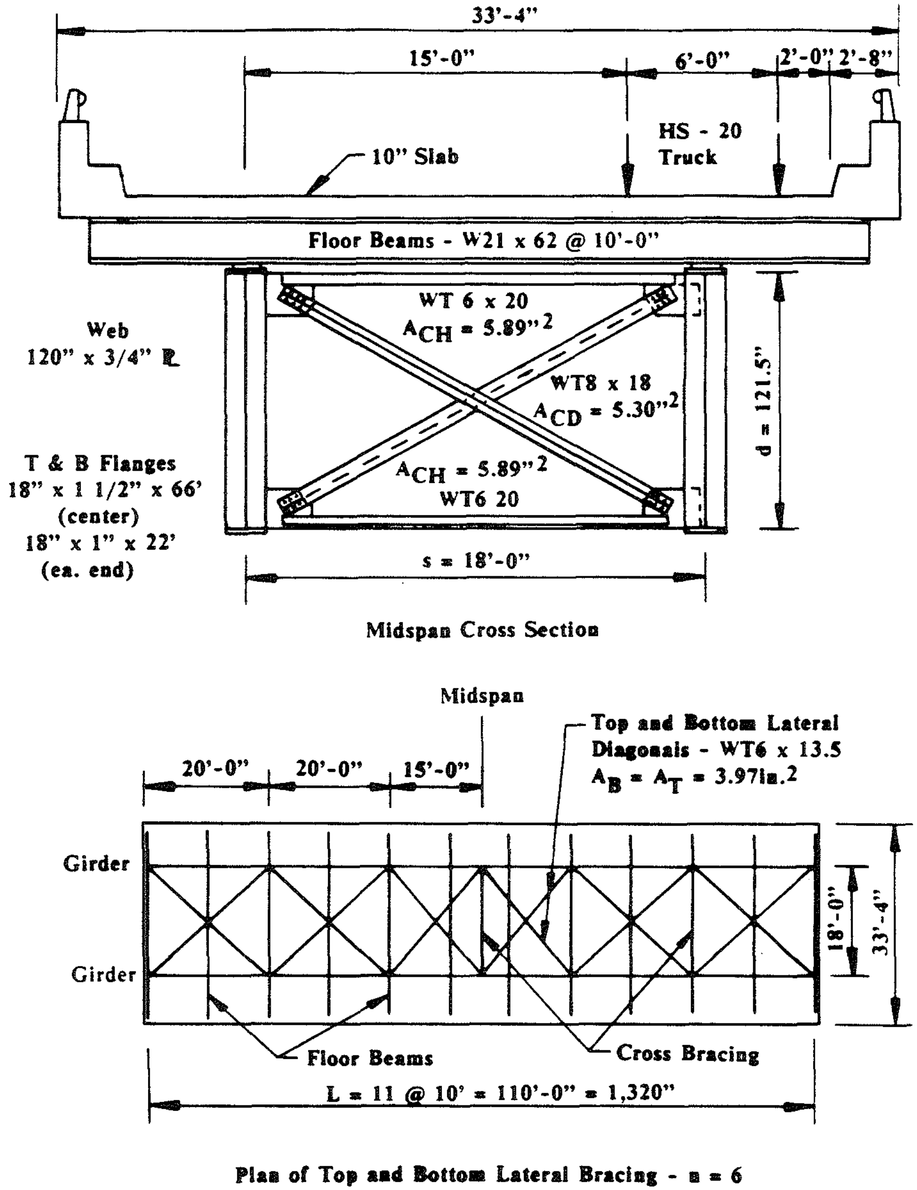


Figure C-25 Examples C1 and C2, Ref. C-5 (A36 Steel)

EXAMPLE C3 - Allowable Stress, 200 ft. Span

Problem Statement

Evaluate the after-fracture redundancy of the alternate load path system of the as-built, simple span, noncomposite, two-girder bridge shown in Fig. C-26 (C-5) for an AASHTO HS-20 truck with 30% impact. Use the Allowable Stress Method and Eq's. C9.1 to C9.18, as approproate. Use the allowable stresses given in Chapter 3. The evaluation consists of two parts: (1) computation of the RRF for each component of the as-built bracing system, and (2) determination of the required area of each component for the condition, RRF=1.0. Neglect the strength of the floor beam-deck system. Assume that all connections are retrofitted to develop member strengths.

Compute Parameters Common to Eq's. C9.1 to C9.10 and C9.17 and C9.18

Dead Load, w, on Fractured Girder

Slab - $\frac{10}{12}$ (0.15)(33.33)(0.5) $= 2.08$ k/ft

Curb - Est. 5.0 ft^3 x 0.15 $= 0.75$

Girder Web - 120 x 0.75 x $\frac{0.49}{144}$ $= 0.31$

Girder Flanges - $2\left(\frac{36 \times 1.5}{144} \times \frac{148}{200}\right) 0.490 = 0.63$

$-\,2\left(\frac{36 \times 1.75}{144} \times \frac{52}{220}\right) 0.490 = 0.11$

Floor Beams, Bracing, Misc. - Est. $= 0.20$

$w = 4.08$ k/ft
or $w = 0.34$ k/in

Live Load plus Impact on Fractured Girder

$\beta(L{+}I) = \left(\frac{15}{18} \times \frac{1}{2} + \frac{21}{18} \times \frac{1}{2}\right) 72 \times 1.3 \quad = 93.6$ k

α Factor

For the 20 ft. panels, the diagonals are 26.91 ft. long.

$\alpha = 26.91/20.0 = 1.35$

$\overline{A}_f$ and R

In the calculation of $\overline{A}_f$, A_f is the weighted average area of the girder bottom flange.

$$A_f = \frac{(36x3.5x148)+(36x1.75x52)}{200} = 109.6 \text{ in}^2$$

$$A_w = 120 \text{ x } 0.75 = 90 \text{ in}^2$$

$$\overline{A}_f = 109.6 \text{ x } 90/6 = 124.6 \text{ in}^2$$

$$R = \frac{3.97}{(1.35)^3 \text{x } 124.6} = 0.012$$

Bottom Lateral Diagonals

Refer to Fig. C-26 for details of the bottom lateral diagonals, and to Eq's. C9.1 and C9.2.

Tension Diagonals

The AASHTO allowable stress for rating is based on the smaller of the yield stress on the gross section and the ultimate stress on the net section. Assuming a 15% reduction in section holes, yield stress on the gross section governs. Thus $f_{all} = 0.85\ F_y$ as discussed in Chapter 3.

$$f_{all} = 0.85 \text{ x } 36 = 30.6 \text{ ksi}$$

$$V_{DA} = \frac{10^2\ (0.012) + 2.4}{10\ (0.012) + 2.4} = 1.43$$

$$V_{LA} = \frac{10^2\ (0.012) + 7.0}{10\ (0.012) + 7.0} = 1.15$$

$$V_{DB} = 0.8 + (0.36)\ \frac{200}{30.6} = 3.15$$

$$V_{LB} = 0.8 + (0.18)\ \frac{200}{30.6} = 1.98$$

The as-built redundancy rating factor for the A36 tension diagonals is given by Eq. C9.1, as follows:

$$RRF = \frac{30.6 - \dfrac{1.1 \text{ x } 1.35 \text{ x } 0.34\ (2{,}400)^2\ 1.43}{8 \text{ x } 123.5 \text{ x } 10 \text{ x } 3.97}}{\dfrac{1.1 \text{ x } 1.35 \text{ x } 1.0\ (93.6)\ (2{,}400)\ 1.15}{4 \text{ x } 123.5 \text{ x } 10 \text{ x } 3.97}}$$

$$= \frac{30.6 - 106.0}{19.6} < 0$$

This result shows that the as-built tension diagonals cannot carry the rated dead load and live loads.

The required cross-section area of the tensiom diagonal for A36 steel when the RRF=1.0, and with $f_{all} = 30.6$ ksi in given by Eq. C9.2, as follows:

$$\text{Req'd. } A_B = \frac{1.1 \text{ x } 1.35 \text{ x } 2{,}400}{8 \text{ x } 123.5 \text{ x } 10 \text{ x } 30.6}\ (\ 0.34\text{x}2{,}400\text{x}3.15 + 2\text{x}93.6\text{x}1.98\ ) = 34.7 \text{ in}^2$$

For 50 ksi steel and $f_{all} = 0.85 \text{ x } 50 = 42.5$ ksi, V_{DB} and V_{LB} are:

$$V_{DB} = 0.8 + 0.36\ (\ \frac{200}{42.5}\ ) = 2.49$$

$$V_{LB} = 0.8 + 0.18\ (\ \frac{200}{42.5}\ ) = 1.65$$

and the required area is

$$\text{Req'd. } A_B = \frac{1.1 \text{ x } 1.35 \text{ x } 2{,}400}{8 \text{ x } 123.5 \text{ x } 10 \text{ x } 42.5}\ (\ 0.34\text{x}2{,}400\text{x}2.49 + 2\text{x}93.6\text{x}1.65\ ) = 19.87 \text{ in}^2$$

It is evident that the tension diagonal must be retrofitted from the as-built 3.97 in^2 in order to achieve after-fracture redundancy.

Compression Diagonals

The allowable compression stress is computed using the unbraced length shown in Fig. C-23. The longest compression diagonals occur in the 20-ft panels. The unbraced length is therefore $0.5 \sqrt{(20^2 + 18^2)} = 13.45$ ft. or 161.44 in. For the as-built WT 6x13.5 the smallest radius of gyration is 1.52 in. Using the AASHTO value of K=0.75 for riveted, bolted and welded connections (Section 10.54 in Ref. 1 of the main report) the slenderness ratio is therefore (0.75 x 161.44)/1.52 = 79.7. For A36 steel, C_c =126.1 (Ref. 1). Thus, in accordance with the AASHTO rating provisions, with a F.S. = 1.4 as discussed in Chapter 3.

$$f_{all} = \frac{36}{1.4}\left(1 - \frac{(79.7)^2}{4\pi^2}\frac{36}{30{,}000}\right) = 20.7 \text{ ksi}$$

From Eq. C9.2,

$$r = 1.35\ (\ 0.66 - 0.0015 \times 200\ ) = 0.49$$

The redundancy rating factor for the as-built, A36 compression diagonals is therefore :

$$RRF = \frac{20.7 - \dfrac{0.49 \times 1.35 \times 0.34\ (2{,}400)^2\ 1.43}{8 \times 123.5 \times 10 \times 3.97}}{\dfrac{0.49 \times 1.35 \times 1.0\ (93.6)\ (2{,}400)\ 1.15}{4 \times 123.5 \times 10 \times 3.97}}$$

$$= \frac{20.7 - 47.6}{8.7} < 0$$

The as-built compression diagonals cannot carry the rated dead and live loads.

The computation of the required area of a compression diagonal for a RRF=1.0 is a trial and error process requiring, as a first step, the selection of a trial section and an allowable compression stress. For a first guess, select a section that meets the area requirement of the tension diagonal. Also assume an unbraced length of 161.44 in., the same as for the as-built diagonal. Assuming 50 ksi steel, select a WT 18x67.5 with an area of 19.9 in^2 and r = 2.38 in. The slenderness ratio is (0.75x161.44)/2.38 = 50.9. The AASHTO allowable stress, with F.S. = 1.4 is

$$f_{all} = \frac{50}{1.4}\left(1 - \frac{(50.9)^2}{4\pi^2}\frac{50}{30{,}000}\right) = 31.8 \text{ ksi}$$

and refer to the previous calculations, V_{DB} = 2.49, and V_{LB} = 1.65.

Then, the required area of a compression diagonal, as given by Eq. C9.2 is,

$$\text{Req'd. } A_B = \frac{0.49 \times 1.35 \times 2{,}400}{8 \times 123.5 \times 10 \times 31.8}\ (\ 0.34\text{x}2{,}400\text{x}2.49 + 2\text{x}93.6\ \text{x}1.65) = 11.83 \text{ in}^2$$

The area 19.9 in^2 provided is sufficient. Therefore, use a WT 18x67.5 of 50 ksi steel. Note that, because of the assumption in Chapters C2 and C4 that all tension and compression diagonals of the bottom lateral bracing are identical, then the WT 18x67.5 in 50 ksi steel is to be used for both the tension and the compression diagonals.

Cross Bracing

Refer to Fig. C-26 for details of the X-type cross bracing, and to Eq's. C9.3, C9.4, C9.7, and C9.8.

Member A

Member A is always subjected to compression stress. The allowable compression stress is computed using the clear distance of 180 in. between girder flanges as the unbraced length and K = 0.75. For the WT 6x20, A_{CH} = 5.89 in^2, r = 1.56 in., and the slenderness ratio is (0.75x180)/1.56 = 86.5. Thus for the as-built A36 horozontal,

$$f_{all} = \frac{36}{1.4}\left(1- \frac{(86.5)^2}{4\pi^2}\frac{36}{30{,}000}\right) = 19.9 \text{ ksi}$$

also, $k_H = 18 / 26.9 = 0.67$

With V_{DA} =1.43 and V_{LA} =1.15 as before, the as-built redundancy rating factor for the A36 horizontal is given by Eq. C9.3 as follows:

$$RRF = \frac{19.9 - \dfrac{1.35 \times 0.34\ (2{,}400)^2\ 1.43 \times 0.67}{5.93 \times 123.5 \times 10 \times 5.89}}{\dfrac{1.35 \times 1.0\ (93.6)\ (2{,}400)\ 1.15 \times 0.67}{2.96 \times 123.5 \times 10 \times 5.89}}$$

$$= \frac{19.9 - 58.7}{10.9} < 0$$

The as-built cross bracing horizontal cannot carry the rated dead and live loads.

The required area of the horizontal member when the RRF=1.0 is given by Eq. C9.4. As a first trial, assume that the as-built horizontal is replaced with a WT 18x67.5 section (same as is used for the bottom lateral diagonals) in 50 ksi steel and with r=2.38 in. Then, the slenderness ratio is (0.75 x 180) / 2.38 = 56.7.

$$f_{all} = \frac{50}{1.4}\left(1- \frac{(56.7)^2}{4\pi^2}\frac{50}{30{,}000}\right) = 30.9 \text{ ksi}$$

Refer to the previous calculations, V_{DB} = 2.49, and V_{LB} = 1.65.

Then, the required area of a compression horizontal, as given by Eq. C9.4 is,

$$\text{Req'd. } A_{CH} = \frac{1.35 \times 2{,}400 \times 0.67}{5.93 \times 123.5 \times 10 \times 30.9}\,(0.34\text{x}2{,}400\text{x}2.49 + 2\text{x}93.6\ \text{x}1.65) = 22.5 \text{ in}^2$$

The area provided is 19.9 in^2 or about 12 percent less than the required area. Thus, use a WT 18x75 section of 50 ksi steel. The same member is to be used for the horizontal in the plane of the top lateral bracing as discussed in Chapter C4.

Member C

For X-type cross bracing, one member is in tension, the other in compression. Since all cross bracing diagonal members are assumed identical and all are assumed to carry equal forces then the allowable compression stress will govern. The allowable stress of a diagonal is again computed based on the clear distance between girder flanges. Also, the unbraced length is computed as shown in Fig. C-23.

For the cross section shown in Fig. C-26, the diagonal distance between the flange-web intersections is $\sqrt{(216^2 + 121.5^2)}$ = 247.83 in. The clear distance along the diagonal between vertical planes defined by the inside edges of the girder flanges is 247.83 x (216 − 36) / 216 = 206.52 in. The unbraced length of the diagonal is therefore 103.3 in. The slenderness ratio of the as-built WT 8x18 with r=1.52 in. is therefore (0.75 x 103.3) / 1.52 = 51.0.

$$f_{all} = \frac{36}{1.4}\left(1- \frac{(51.0)^2}{4\pi^2}\frac{36}{30{,}000}\right) = 23.7 \text{ ksi}$$

The factor k_D can be computed using the actual diagonal member length, or equivalently using the girder spacing and depth as follows:

$$k_D = 247.83/216 = 1.15$$

With k_H = 0.67, V_{DA} = 1.43, V_{LA} = 1.15 and A_{CD} = 5.30 in^2, the redundancy rating factor for an as-built cross bracing diagonal is given by Eq. C9.7 as follows:

$$RRF = \frac{23.7 - \frac{1.35 \text{ x } 0.34 \text{ } (2{,}400)^2 \text{ } 1.43 \text{ x } 1.15 \text{ x } 0.67}{10.67 \text{ x } 123.5 \text{ x } 10 \text{ x } 5.30}}{\frac{1.35 \text{ x } 1.0 \text{ } (93.6) \text{ } (2{,}400) \text{ } 1.15 \text{ x } 1.15 \text{ x } 0.67}{5.33 \text{ x } 123.5 \text{ x } 10 \text{ x } 5.30}}$$

$$= \frac{23.7 - 41.7}{7.7} < 0$$

The as-built cross bracing diagonals can not carry the rated dead and live loads.

The required area of the cross bracing diagonal member for a RRF=1.0 is given by Eq. C9.8. As a first trial, assume that the as-built diagonals are replaced with a WT 8x44.5 section in 50 ksi steel, and with A_{CD} = 13.1 in^2 and r = 2.27 in. Then, the slenderness ratio is (0.75 x 103.3) / 2.27 = 34.1.

$$f_{all} = \frac{50}{1.4}\left(1 - \frac{(34.1)^2}{4\pi^2}\frac{50}{30{,}000}\right) = 34.0 \text{ ksi}$$

Refer to the previous calculations, V_{DB} = 2.49, and V_{LB} = 1.65.

Then, the required area of a cross bracing diagonal, as given by Eq. C9.8 is,

$$\text{Req'd. } A_{CD} = \frac{1.35\text{x}2{,}400\text{x}1.15\text{x}0.67}{10.67\text{x}123.5\text{x}10\text{x}34.0}\,(0.34\text{x}2{,}400\text{x}2.49 + 2\text{x}93.6\text{x}1.65) = 13.0 \text{ in}^2$$

The 13.1 in ^{2} area provided is sufficient.

Top Lateral Diagonals

Refer to Fig. C-26 for details of the top lateral diagonals and to Eq's. C9.17 and C9.18. Since it is assumed in Chapter C4 that all top lateral diagonals are identical and that each carries the same force in a panel, then the allowable compression stress will govern. Also since the top and bottom lateral diagonals are all WT 6x13.5 sections,

referring to the previous calculations, the allowable stress is 19.4 ksi. With A_T = 3.97 in^2, the redundancy rating factor for the as-built A36 diagonals is

$$RRF = \frac{20.7 - \frac{1.35 \text{ x } 0.34 \text{ } (2{,}400)^2 \text{ } 1.43}{10.67 \text{ x } 123.5 \text{ x } 10 \text{ x } 3.97}}{\frac{1.35 \text{ x } 1.0 \text{ } (93.6) \text{ } (2{,}400) \text{ } 1.15}{5.33 \text{ x } 123.5 \text{ x } 10 \text{ x } 3.97}}$$

$$= \frac{20.7 - 72.3}{13.3} < 0$$

The as-built top lateral diagonals cannot support the rated dead and live loads.

The computation of the required area of a top lateral diagonal for a RRF=1.0 is given by Eq. C9.18. Again assume that the as-built top lateral diagonals are replaced with WT 18x67.5 sections of 50 ksi steel with A_T = 19.9 in^2 and r= 2.38 in., the same section used for the bottom lateral diagonals. The slenderness ratio is (0.75x161.44)/2.38 = 50.9.

$$f_{all} = \frac{50}{1.4}\left(1 - \frac{(50.9)^2}{4\pi^2}\frac{50}{30{,}000}\right) = 31.8 \text{ ksi}$$

Referring to the previous calculations, V_{DB} = 2.49, and V_{LB} = 1.65. Thus,

$$\text{Req'd. } A_T = \frac{1.35 \text{ x } 2{,}400}{10.67 \text{ x } 123.5 \text{ x } 10 \text{ x } 31.8}\,(\,0.34\text{x}2{,}400\text{x}2.49 + 2\text{x}93.6\text{x}1.65) = 18.1 \text{ in}^2$$

The 19.9 in ^{2} area provided is sufficient. Use WT 18x67.5 sections of 50 ksi for all top lateral diagonals.

Total Load and Dead Load Deflections

For midspan fracture the after-fracture deflection ratio of the fractured girder subjected to dead and live loads plus impact is given by Eq. C9.37 in terms of V_{DA} and

V_{LA}. In the calculation of V_{DA} and V_{LA}, the A_B provided is used.

$$R = \frac{19.9}{(1.35)^3 \text{ x } 124.6} = 0.065$$

$$V_{DA} = \frac{(6)^2 \text{ x } 0.065 + 2.4}{6 \text{ x } 0.065 + 2.4} = 1.70$$

$$V_{LA} = \frac{(6)^2 \text{ x } 0.065 + 7.0}{6 \text{ x } 0.065 + 7.0} = 1.26$$

The total load deflection ratio is

$$\frac{\Delta}{L} = \frac{(1.35)^3 \text{ x } (2{,}400)^2}{16 \text{ x } 30{,}000 \text{ x } (123.5)^2 \text{ x } (10)^2 \text{ x } 19.9} (0.34 \text{ x } 2{,}440 \text{ x } 1.70 + 2 \text{ x } 93.6 \text{ x } 1.26)$$

$$= \frac{1}{663}$$

Similarly the dead load deflection ratio is given by Eq. C9.38.

$$\left(\frac{\Delta}{L}\right)_D = \frac{(1.35)^3 \text{ x } (2{,}400)^2}{16 \text{ x } 30{,}000 \text{ x } (123.5)^2 \, (10)^2 \text{ x } 19.9} (0.34 \text{ x } 2{,}400 \text{ x } 1.70)$$

$$= \frac{1}{741}$$

EXAMPLE C4 - Load Factor Method, 200 ft. Span

Problem Statement

Evaluate the after-fracture redundancy of the alternate load path system of the as-built, simple span, noncomposite, two-girder bridge shown in Fig. C-26 (C-5) for an AASHTO HS-20 truck with 30% impact. Use the Load Factor Method and Eq's. C9.19 to C9.36, as approproate. Use the load factors given in Chapter 3. The evaluation consists of two parts: (1) computation of the RRF for each component of the as-built bracing system, and (2) determination of the required area of each component for the condition, RRF=1.0. Neglect the strength of the floor beam-deck system. Assume that all connections are retrofitted to develop member strengths.

Compute Parameters Common to Eq's. C9.19 to C9.36

Unfactored dead load is dentical to Example C3, and the dead load factor, γ_D = 1.18 as recommended in Chapter 3.

$$w = 0.34 \text{ k/in}$$

$$\text{or } \gamma_D D = 1.18 \text{ x } 0.34 = 0.40 \text{ k/in}$$

Live Load plus Impact on Fractured Girder

Unfactored live load plus impact is identical to Example C3, and the live load factor, γ_L = 1.18 as discussed in Chapter 3.

$$\beta(L+I) = 93.6 \text{ k}$$

$$\gamma_L \beta(L+I) = 1.18 \text{ x } 93.6 = 110.4 \text{ k}$$

α Factor

Identical to Example C3

$$\alpha = 1.35$$

$\overline{A}_f$ and R

Identical to Example C3

$$\overline{A}_f = 124.6 \text{ in}^2$$

$$R = 0.012$$

AF Factor

For the ten-panel bridge, the amplification factor for critical fracture location is,

$$AF = 0.75\ (1 + \tfrac{1}{10})\ (2 - \tfrac{3}{10}) = 1.40$$

Bottom Lateral Diagonals

Refer to Eq's. C9.19 and C9.20.

Tension Diagonals

The as-built redundancy rating factor for the A36 tension diagonals is given by Eq. C9.19, as follows:

$$RRF = \frac{36 \text{ x } 3.97 - 1.40 \dfrac{1.18 \text{ x } 1.35 \text{ x } 0.34\ (2{,}400)^2}{4 \text{ x } 123.5 \text{ x } (10+1)}}{1.40 \dfrac{1.18 \text{ x } 1.35 \text{ x } 1.0\ (93.6)\ (2{,}400)}{2 \text{ x } 123.5 \text{ x } (10+1)}}$$

$$= \frac{142.92 - 803.8}{184.4} < 0$$

This result shows that the as-built tension diagonals cannot carry the rated dead load and live loads.

The required cross-section area of the tension diagonal for A36 steel when the RRF=1.0, and with $f_y = 36$ ksi in given by Eq. C9.20, as follows:

$$\text{Req'd. } A_B = 1.40 \frac{1.35 \text{ x } 2{,}400}{4\text{x}123.5 \text{ x}(10+1)\text{x } 36} (1.18\text{x}0.34\text{x}2{,}400 + 2\text{x}1.18\text{x}93.6) = 27.4 \text{ in}^2$$

For 50 ksi steel with $f_y = 50$ ksi, the required area is:

$$\text{Req'd. } A_B = 1.40 \frac{1.35 \text{ x } 2{,}400}{4\text{x}123.5 \text{ x}(10+1)\text{x } 50} (1.18\text{x}0.34\text{x}2{,}400 + 2\text{x}1.18\text{x}93.6) = 19.8 \text{ in}^2$$

It is evident that the tension diagonal must be modified from the as-built 3.97 in^2 in order to achieve after-fracture redundancy. Thus, use a WT 18x67.5, with A_B=19.9 in^2, section of 50 ksi steel. The same member is to be used for all the bottom lateral diagonals.

Cross Bracing

Refer to Fig. C-26 for details of the X-type cross bracing, and to Eq's. C9.21, C9.22, C9.25, and C9.26.

Member A

Member A is always subjected to compression stress. The critical compression stress is computed using the clear distance of 180 in. between girder flanges as the unbraced length and K = 0.75. For the WT 6x20, A_{CH} = 5.89 in^2, r = 1.56 in., and the slenderness ratio is (0.75x180)/1.56 = 86.5. Thus for the as-built A36 horozontal,

$$f_{cr} = 36 \text{ x} \left(1 - \frac{(86.5)^2}{4\pi^2} \frac{36}{30{,}000} \right) = 27.8 \text{ ksi}$$

also, $k_H = 18\ /\ 26.9 = 0.67$

The as-built redundancy rating factor for the A36 horizontal is given by Eq. C9.21 as follows:

$$\text{RRF} = \frac{0.85 \times 27.8 \times 5.89 - 1.40 \dfrac{1.18 \times 1.35 \times 0.34\,(2{,}400)^2 \times 0.67}{2.67 \times 123.5 \times (10+1)}}{1.40 \dfrac{1.18 \times 1.35 \times 1.0\,(93.6)\,(2{,}400) \times 0.67}{1.33 \times 123.5 \times (10+1)}}$$

$$= \frac{139.2 - 806.8}{185.8} < 0$$

The as-built cross bracing horizontal cannot carry the rated dead and live loads.

The required area of the horizontal member when the RRF=1.0 is given by Eq C9.22. As a first trial, assume that the as-built horizontal is replaced with a WT 18x67.5 section (same as is used for the bottom lateral diagonals) in 50 ksi steel and with r=2.38 in. Then, the slenderness ratio is (0.75 x 180) / 2.38 = 56.7.

$$f_{cr} = 50 \times \left(1 - \frac{(56.7)^2\ 50}{4\pi^2\ 30{,}000} \right) = 43.2 \text{ ksi}$$

Then, the required area of a compression horizontal, as given by Eq. C9.22 is,

$$\text{Req'd. } A_{CH} = 1.40 \frac{1.35 \times 2{,}400 \times 0.67}{2.67 \text{x} 123.5 \text{x} (10+1) \text{x} 0.85 \text{x} 43.2} (1.18 \text{x} 0.34 \text{x} 2{,}400 + 2 \text{x} 1.18 \text{x} 93.6)$$

$$= 27.0 \text{ in}^2$$

The area provided is 19.9 in^2 less than the required area. Thus, try a WT 18x91 section in 500 ksi steel and with A_H = 26.8 in^2 and r = 2.55 in. Then, the slenderness ratio is (0.75 x 180) / 2.55 = 52.9.

$$f_{cr} = 50 \times \left(1 - \frac{(52.9)^2\ 50}{4\pi^2\ 30{,}000} \right) = 44.1 \text{ ksi}$$

Then, the required area of a compression horizontal, as given by Eq. C9.22 is,

$$\text{Req'd. } A_{CH} = 1.40 \frac{1.35 \times 2{,}400 \times 0.67}{2.67 \text{x} 123.5 \text{x} (10+1) \text{x} 0.85 \text{x} 44.1} (1.18 \text{x} 0.34 \text{x} 2{,}400 + 2 \text{x} 1.18 \text{x} 93.6)$$

$$= 26.5 \text{ in}^2$$

The area 26.8 in^2 provided is sufficient. Thus, use a WT 18x91 of 50 ksi steel. The same member is to be used for the horizontal in the plane of the top lateral bracing as discussed in Chapter C4.

Member C

For X-type cross bracing, one member is in tension, the other in compression. Since all cross bracing diagonal members are assumed identical and all are assumed to carry equal forces then the critical compression stress will govern. The critical stress of a diagonal is again computed based on the clear distance between girder flanges. Also, the unbraced length is computed as shown in Fig. C-23.

For the cross section shown in Fig. C-26, the diagonal distance between the flange-web intersections is $\sqrt{(216^2 + 121.5^2)}$ = 247.83 in. The clear distance along the diagonal between vertical planes defined by the inside edges of the girder flanges is 247.83 x (216 − 36) / 216 = 206.52 in. The unbraced length of the diagonal is therefore 103.3 in. The slenderness ratio of the as-built WT 8x18 with r=1.52 in. is therefore (0.75 x 103.3) / 1.52 = 51.0.

$$f_{cr} = 36 \left(1 - \frac{(51.0)^2\ 36}{4\pi^2\ 30{,}000} \right) = 33.2 \text{ ksi}$$

The factor k_D can be computed using the actual diagonal member length, or equivalently using the girder spacing and depth as follows:

$$k_D = 247.83/216 = 1.15$$

With $k_H = 0.67$, and $A_{CD} = 5.30$ in^2, the redundancy rating factor for an as-built cross bracing diagonal is given by Eq. C9.24 as follows:

$$RRF = \frac{0.85 \text{ x } 33.2 \text{ x } 5.30 - 1.40 \dfrac{1.18 \text{ x } 1.35 \text{ x } 0.34 \ (2{,}400)^2 \text{ x } 1.15 \text{ x } 0.67}{8 \text{ x } 123.5 \text{ x } (10+1)}}{1.40 \dfrac{1.18 \text{ x } 1.35 \text{ x } 1.0 \ (93.6) \ (2{,}400) \text{ x } 1.15 \text{ x } 0.67}{4 \text{ x } 123.5 \text{ x } (10+1)}}$$

$$= \frac{150.0 - 309.6}{71.0} < 0$$

The as-built cross bracing diagonals can not carry the rated dead and live loads.

The required area of the cross bracing diagonal member for a RRF=1.0 is given by Eq. C9.25. As a first trial, assume that the as-built diagonals are replaced with a WT 8x44.5 section in 50 ksi steel, and with A_{CD} = 13.1 in^2 and r = 2.27 in. Then, the slenderness ratio is (0.75 x 103.3) / 2.27 = 34.1.

$$f_{cr} = 50 \left(1 - \frac{(34.1)^2}{4\pi^2} \frac{50}{30{,}000} \right) = 47.5 \text{ ksi}$$

Then, the required area of a cross bracing diagonal, as given by Eq. C9.25 is,

$$\text{Req'd. } A_{CD} = 1.40 \frac{1.35\text{x}2{,}400\text{x}1.15\text{x}0.67}{8\text{x}123.5\text{x}(10+1)\text{x}0.85\text{x}47.5} (1.18\text{x}0.34\text{x}2{,}400 + 2\text{x}1.18\text{x}93.6)$$

$$= 9.43 \text{ in}^2$$

The 9.43 in^2 area provided is sufficient.

Top Lateral Diagonals

Refer to Fig. C-26 for details of the top lateral diagonals and to Eq's. C9.35 and C9.36. Since it is assumed in Chapter C4 that all top lateral diagonals are identical and that each carries the same force in a panel, then the critical compression stress will govern. The top and bottom lateral diagonals are all WT 6x13.5 sections. The critical compression stress is computed using the unbraced length shown in Fig. C-23. The unbraced length is therefore $0.5 \sqrt{(20^2 + 18^2)}$ = 13.45 ft. or 161.44 in. Since the as-built WT 6x13.5 the smallest radius of gyration is 1.52 in, the slenderness ratio is therefore (0.075 x 161.44)/1.52 = 79.7. For A36 steel,

$$f_{cr} = 36 \left(1 - \frac{(79.7)^2}{4\pi^2} \frac{36}{30{,}000} \right) = 29.0 \text{ ksi}$$

Thus the redundancy rating factor for the as-built A36 diagonals is

$$RRF = \frac{0.85 \text{ x } 29.0 \text{ x } 3.97 - 1.40 \dfrac{1.18 \text{ x } 1.35 \text{ x } 0.34 \ (2{,}400)^2}{8 \text{ x } 123.5 \text{ x } (10+1)}}{1.40 \dfrac{1.18 \text{ x } 1.35 \text{ x } 1.0 \ (93.6) \ (2{,}400)}{4 \text{ x } 123.5 \text{ x } (10+1)}}$$

$$= \frac{97.9 - 401.9}{92.2} < 0$$

The as-built top lateral diagonals cannot support the rated dead and live loads.

The computation of the required area of a top lateral diagonal for a RRF=1.0 is given by Eq. C9.36. Again assume that the as-built top lateral diagonals are replaced with WT 8x38.5 sections of 50 ksi steel with A_T = 11.3 in^2 and r= 2.24 in. The slenderness ratio is (0.75x161.44)/2.24 = 54.1.

$$f_{cr} = 50 \left(1 - \frac{(54.1)^2}{4\pi^2} \frac{50}{30{,}000} \right) = 43.8 \text{ ksi}$$

Then, the required area of the top lateral diagonals, as given by Eq. C9.36 is,

$$\text{Req'd. } A_T = 1.40 \frac{1.35 \times 2{,}400}{8 \times 123.5 \times (10+1) \times 0.85 \times 43.8} (1.18 \times 0.34 \times 2{,}400 + 2 \times 1.18 \times 93.6)$$

$$= 9.48 \text{ in}^2$$

The 11.3 in ^{2} area provided is sufficient. Use WT 8x38.5 sections of 50 ksi for all top lateral diagonals.

Total Load and Dead Load Deflections

Check Eq. C9.42 prior to use Eq's. C9.37, C9.38 and C9.39 with the A_B provided from the Load Factor Method.

$$\frac{19.8}{19.9} > \frac{1}{1.18}$$

The condition is satisfied and both Load Factor and Allowable Stress Methods result in the same A_B. Therefore, the total load and dead load deflections from both methods are identical. From the evaluation of Example C3

$$\frac{\Delta}{L} = \frac{1}{633}$$

$$\left(\frac{\Delta}{L}\right)_D = \frac{1}{741}$$

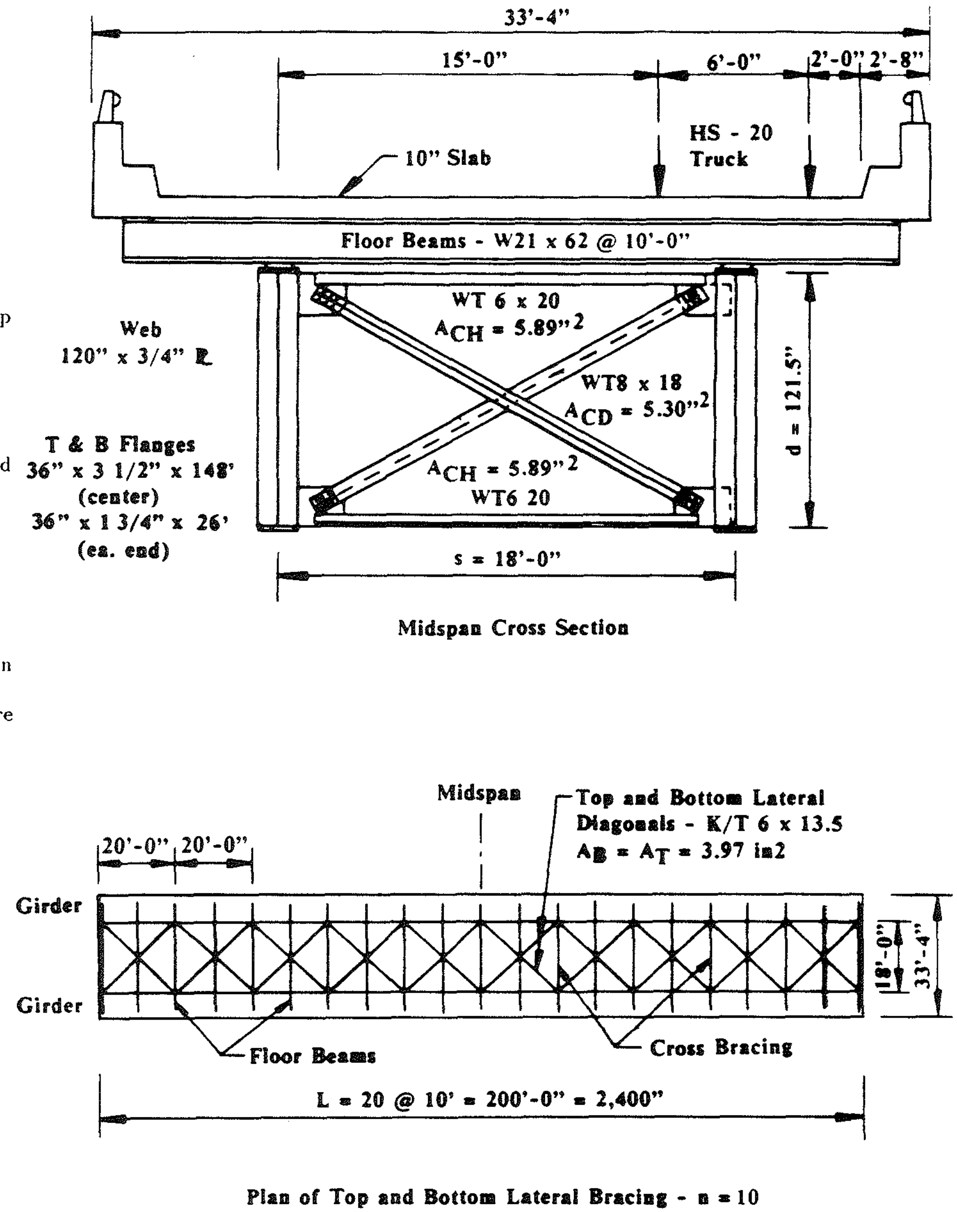


Figure C-26 Examples C3 and C4, Ref. C-5 (A36 Steel)

EXAMPLE C5 - Allowable Stress, 180 ft. Span, Retrofit

Problem Statement

Provide a retrofit design for the two-girder bridge shown in Fig. C-27(C-9) to conform to the requirements of a redundant bracing system. The after-fracture loading is an H-15 truck with 30% impact. Use the Allowable Stress Method and the allow stresses given in Chapter 3. Follow the retrofit guidelines in Chapter 4. The retrofit design consists of four parts: (1) determination of the required area of each component for the condition RRF = 1.0, (2) computation of $\left(\frac{\Delta}{L}\right)$ and $\left(\frac{\Delta}{L}\right)_D$ corresponding to the retrofitted structure, (3) check retrofit design to limit $\frac{\Delta}{L} = \frac{1}{300}$ and check $\left(\frac{\Delta}{L}\right)_D$.

Compute Parameters common to Eq's. C9.1 to C9.18

Dead Load, w, on Fractured Girder

From the summary of quantitities in Ref. C-9

Concrete - 0.5 (0.15)(147 yd^3)$(3)^3$/180	= 1.65
Reinf. Steel - 0.5 (35.5)/180	= 0.10
Struc. Steel - 0.5 (176.7 + 64.8 + 2.4)/180	= 0.68
Future Wearing Surface - 0.5 (0.022) (28)	= 0.31

w= 2.74 k/ft

or w= 0.23 k/in

Live Load plus Impact on Fractured Girder

$$\beta(L+I) = \left[\frac{23.5}{23} \times \frac{1}{2} + \frac{17.5}{23} \times \frac{1}{2}\right] 30 \times 1.3 = 34.8 \text{ k}$$

α Factor

For the 20 ft. panels, the diagonals are 30.48 ft. long.

$$\alpha = 30.48/20 = 1.52$$

Bottom Lateral Diagonals

No bottom lateral bracing is provided in Fig. C-27(a). The computation of cross-section areas in similar to that for X-type bottom lateral bracing diagonal in Examples C1 and C3. From Eq. C9.2 with F_y = 50 ksi.

$$V_{DB} = 2.32$$
$$V_{LB} = 1.56$$
and Req'd. $A_B = 12.5 \text{ in}^2$

Therefore, use a WT 9 x 43 with an area of 12.7 in^2.

Checking the compression diagonals, a WT 9 x 43 provides more area than required.

Interior Diaphragm

Referring to Fig. C-27(a) for details of the interior cross frame and using Eq. C9.11, the required section modulus of the interior cross frame is

$$\text{Req'd. } S = \frac{1.52 \times 2160 \times 0.75 \times 87.5}{4.32 \times 119 \times 9 \times 42.5} (0.23 \times 2160 \times 2.32 + 2 \times 34.8 \times 1.56)$$

$$= 1{,}382 \text{ in}^3$$

The section modulus provided by a W 27 x 94 is 243 in^3.

Since the section modulus is provided significantly less than the required, it is necessary to install cross bracing as shown in Fig. C-27(b).

Member A

The critical compression stress in member A is computed using a clear distance of 253 in. Assuming a WT 12 x 58.5 in 50 Ksi steel, A_{CH} = 17.2 in^2, r = 2.94 in., and the slenderness ratio is (0.75 x 253)/2.94 = 64.54. The allowable stress, with F.S. = 1.4, is

$$f_{all} = 29.4 \text{ ksi}$$

The required area of a compression horizontal, as given by Eq. C9.4, is

$$\text{Req'd. } A_{CH} = 16.6 \text{ in}^2$$

The 17.2 in^2 area provided is sufficient.

Member C

For K-bracing (Type 1) as shown in Chapter C9, one member is in tension, the other in compression. The allowable stress for a compression diagonal is again computed based on the clear distance of about 140 in. and k_d = 1.18. Assuming a W T 9 x 38, A_{CD} - 11.2 in^2, r = 2.54 in^2, and the slenderness ratio is (0.75 x 140)/2.54 = 41.34.

$$f_{all} = 33.1 \text{ Ksi}$$

The required area of a cross bracing diagonal, as given by Eq. C9.8, is

$$\text{Req'd. } A_{CD} = 9.69 \text{ in}^2$$

The 11.2 in^2 area provided is sufficient.

End Diaphragm

Refer to Fig. C-27(a) for the details of as-built end diaphragm with a geometric shape somewhat different from the retrofitted interior cross bracing as shown in Fig. C-27(b).

Since all diaphragms are assumed to carry equal forces, Eq's. C9.4 and C9.8 are used to compute the required areas of the cross bracing horizontal and diagonal in the end diaphragm.

Required member sizes for the end diaphragm are found to be the same as those for the interior diaphragms and are shown in Fig. C-27(b).

Top Lateral Bracing

The as-built top lateral bracing is not properly located at the level of girder top flange as shown in Fig. C-27(a). Thus, the as-built top lateral bracing does not meet the requirements of redundant bracing system as discussed in Chapter 3.

As a retrofit for the top lateral bracing, a drag strut could be provided between the floor beam and the deck. But a drag strut cannot be used in this bridge because the deck is noncomposite, as is discussed in Chapter 4.

Additional top lateral bracing must be provided at the level of girder top flange. However, it is difficult to install it with the deck in place. The top lateral bracing diagonal would cut through the stringers. Gusset plates would be difficult to weld to the underside of girder top flange.

Connection of the floor beam to the girder top flange is possible by adding a diagonal connection member as discussed in Chapter 4. However, it would also be difficult to install this member properly with the deck in place.

One possible solution is to remove removing the deck, instal top lateral bracing and replac the deck. Also during this retrofit shear connectors could be installed to make deck composite with girder. This is a viable option if the deck needs to be replaced anyway.

The required area of top lateral diagonals is computed by using Eq. C9.18. Computations indicate that a WT 9 x 43, which was used for the bottom lateral diagonals is satisfactory for the top lateral diagonals.

Total Load and Dead Load Deflections

For midspan fracture the after-fracture deflection of the fractured girder subjected to dead and live loads plus impact is given by Eq. C9.37 in terms of V_{DA} and V_{LA}.

$$R = \frac{12.7}{(1.52)^3 \times 50.3} = 0.072$$

$$V_{DA} = \frac{(9)^2 \times 0.072 + 2.4}{9 \times 0.072 + 2.4} = 2.70$$

$$V_{LA} = \frac{(9)^2 \times 0.072 + 7.0}{9 \times 0.072 + 7.0} = 1.68$$

The total load deflection is

$$\left(\frac{\Delta}{L}\right) = \frac{(1.52)^3 \times (2{,}160)^2}{16 \times 30{,}000 \times (19)^2 \times (9)^2 \times 12.7} (0.23 \times 2{,}160 \times 2.70 + 2 \times 34.8 \times 1.68)$$

$$= \frac{1}{293}$$

Similarly the dead load deflection ratio is given by Eq. C9.38.

$$\left(\frac{\Delta}{L}\right)_D = \frac{(1.52)^3 \times (2{,}160)^2}{16 \times 30{,}000 \times (119)^2 \times (9)^2 \times 12.7} (0.23 \times 2{,}160 \times 2.70)$$

$$= \frac{1}{318}$$

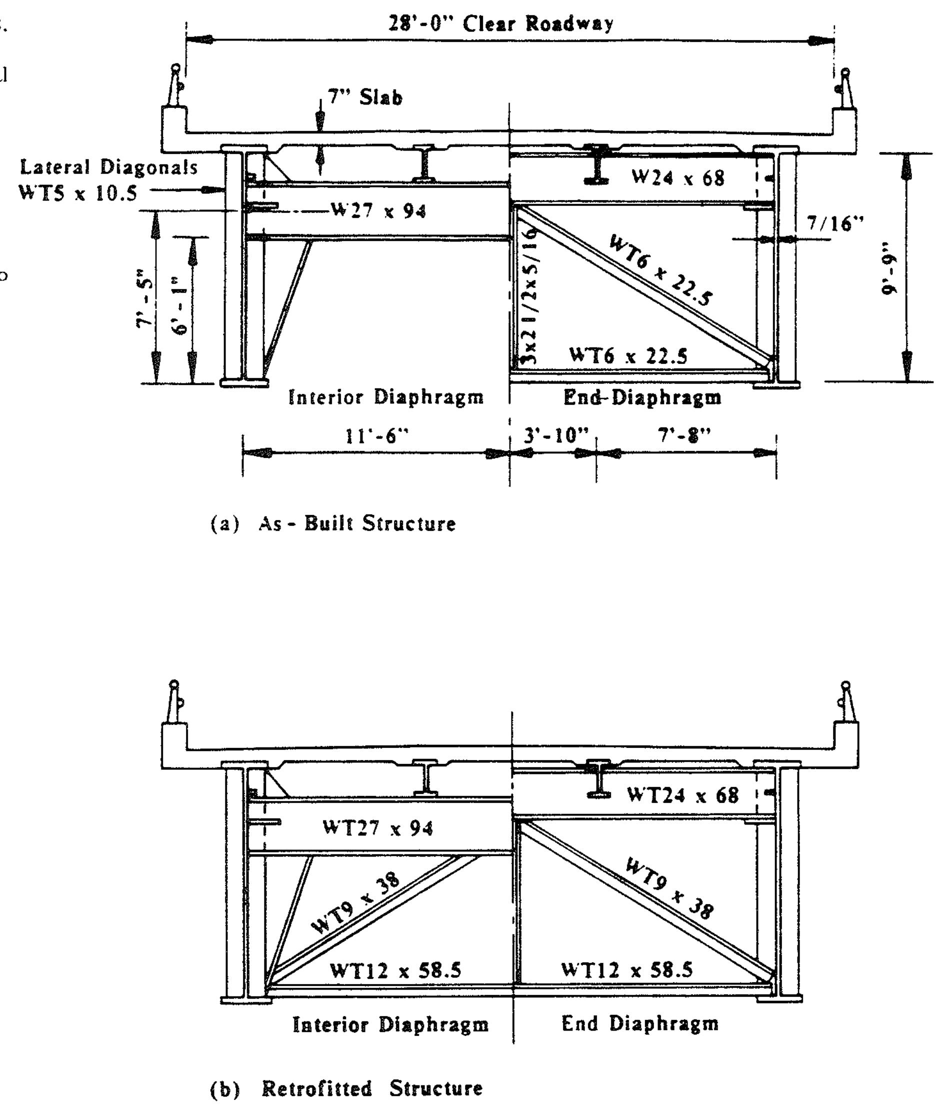


Figure C-27 Example C5, Ref. C-9 (A36 Steel)

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APPENDIX D

DEVELOPMENT OF REQUIREMENTS FOR REDUNDANT TENSION CABLES, RODS AND SHAPES

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CHAPTER D1

DEVELOPMENT OF REQUIREMENTS FOR REDUNDANT TENSION CABLES, RODS AND SHAPES

SCOPE

This Appendix provides the requirements of unprestressed tension cables, rods or shapes that can be attached to the tension flange of a girder to provide redundancy in the event of a near full depth fracture of the girder.

Figure D-1 shows the elevation view of a simple span girder with tension cables or rods attached to the bottom flange. Section A-A of Fig. D-1 provides a schematic view of the anchorage. The anchorage can be bolted or welded to the flange near the bearings in the case of a welded girder. For riveted girders it might be possible to remove rivets so that the anchorage can be bolted to the flange. Additional web stiffeners and flange plates may be necessary to strengthen the girder depending on the magnitude of the anchorage forces.

Figure D-2 shows the elevation view of the fractured girder as a free body. The after-fracture force at the anchorages is F as shown in the figure. Considering midspan fracture and zero bending moment at midspan (as in Chapter C2, Appendix C) F is given by

$$F = \frac{1}{d}\left(\frac{w\,L^2}{8} + \frac{\beta(L+I)\,L}{4} \right) \qquad (D.1)$$

where all other terms are defined in Appendix C. Note that equations D.1 and C2.1 are identical.

Equation D.1 can be used to compute the required total area of the tension cables

or rods as well as the anchorage force.

ALLOWABLE STRESS METHOD

The required total cross-section area, A_C, of the tension cables or rods is found from the condition of unit redundancy rating factor, as was previously shown in Appendix C. The redundancy rating factor, RRF, is given by Eq. C2.5 in Appendix C. Dividing Eq. D.1 by A_C, the dead load stress, f_D, and the live load plus impact stress, f_L, in the tension cables or rods are

$$f_D = \frac{w\,L^2}{8\,d\,A_C} \qquad (D.2)$$

$$f_L = \frac{\beta(L+I)\,L}{4\,d\,A_C} \qquad (D.3)$$

Setting RRF equal to unity in Eq. C2.5 and solving for the required A_C gives

$$\text{Req'd. } A_C = \frac{L}{8\,d\,f_{all}}\left(w\,L + 2\,\beta(L+I) \right) \qquad (D.4)$$

LOAD FACTOR METHOD

The redundancy rating factor, RRF, is given by Eq. C2.16 in Appendix C. The dead load force, F_D, and the live load plus impact force, F_L, in the tension cables or rods are

$$F_D = \frac{w\,L^2}{8\,d} \qquad (D.5)$$

$$F_L = \frac{\beta(L+I)\,L}{4\,d} \qquad (D.6)$$

Substituting Eq's. D.5 and D.6 into Eq. C2.16 and setting RRF equal to unity the required A_C for ϕ=1.0 gives

$$\text{Req'd. } A_C = \frac{L}{8\, d\, F_y} \left(\gamma_D\, w\, L + 2\, \gamma_L\, \beta(L+I) \right) \tag{D.7}$$

where γ_D and γ_L are the load factors for dead and live loads suggested in Chapter 3.

SERVICEABILITY

Deflection - Stress Relationship

For midspan fracture the after-fracture dead and live load deflection of the fractured girder is a function of the axial stress in the tension cables or rods. As the axial stress increases, the deflection increases.

It is conservatively assumed that the distance between anchorages is the same as the girder span. It is also assumed that the deflected shape of the fractured girder consists of two straight lines extending from each support to midspan and that the deflection of the deck above the fracture equals the deflection of the fractured girder. These assumptions are identical to those discussed in Appendix C and shown in Fig. C-17(a).

From similar triangles in Fig. C-17(a),

$$\frac{\Delta}{L} = \frac{h}{2\, d} \tag{D.8}$$

which is identical to Eq. C2.21, where h is one half of the crack opening width at the bottom of the fractured girder. Since h is also one half of the tension cable or rod elongation,

$$h = \frac{L\, f_C}{2\, E} \tag{D.9}$$

where f_C is the axial stress in the tension cables or rods. Substituting Eq. D.9 into Eq. D.8 gives

$$\frac{\Delta}{L} = \frac{L\, f_C}{4\, E\, d} \tag{D.10}$$

If f_C is the total dead and live load plus impact stress in the tension cables or rods, then from Eq. D.10

$$f_C = \frac{4\, E\, d}{L} \left(\frac{\Delta}{L}\right) \tag{D.11}$$

where $f_C < f_{all}$ and Δ/L is the desired deflection-to-span length ratio selected by the engineer for total load conditions.

Deflections for Allowable Stress Method

The dead load stress, f_D, and live load plus impact stress, f_L, are given in Eq's. D.2 and D.3. Substituting Eq's. D.2 and D.3 into Eq. D.10 the total load deflection is

$$\left(\frac{\Delta}{L}\right) = \frac{L^2}{32\, E\, d^2\, A_C} \left(w\, L + 2\beta(L+I) \right) \tag{D.12}$$

where A_C is the required area given by Eq. D.4.

Similarly, substituting Eq. D.2 only into Eq. D.10 the dead load deflection is

$$\left(\frac{\Delta}{L}\right)_D = \frac{L^2}{32\, E\, d^2\, A_C} (w\, L) \tag{D.13}$$

Also relating dead load to total load ratio in Eq's. D.12 and D.13 the dead load deflection can be expressed to

$$\left(\frac{\Delta}{L}\right)_D = \frac{w\,L}{w\,L + 2\beta(L+I)}\left(\frac{\Delta}{L}\right) \qquad (D.14)$$

Deflections for Load Factor Method

Equations D.12, D.13 and D.14 can be used to compute the service load deflections when using the Load Factor Method providing that

$$\frac{\text{Req'd. } A_B \text{ by LFM}}{\text{Req'd. } A_B \text{ by ASM}} \geq \frac{1}{\gamma_D} \text{ or } \frac{1}{\gamma_L} \qquad (D.15)$$

where it is assumed that γ_L is identical to γ_D as guided in Chapter 3 or dead load produces substantially larger deflection than live load. This will ensure that the tension stress in the highest stress diagonal does not exceed yield stress under the service loads.

Required A_C for Serviceability

As an alternative to the above, it may be desirable to determine the required area, A_C, for the tension cables or rods for a desired Δ/L ratio under the total dead and live load plus impact.

Referring to Eq. D.12 the required area is

$$\text{Req'd. } A_C = \frac{L^2}{32\,E\,d^2\left(\frac{\Delta}{L}\right)}\left(w\,L + 2\beta(L+I)\right) \qquad (D.16)$$

Equation D.16 provides the required total area of tension cables or rods as a function of an arbitrary $\frac{\Delta}{L}$ ratio. However, the total area provided by Eq. D.16 cannot be less than that given by Eq. D.4, otherwise the tension stress will exceed the allowable stress. Similarly, for the load factor method, the total area given by Eq. D.16 should not be less than that given by Eq. D.7.

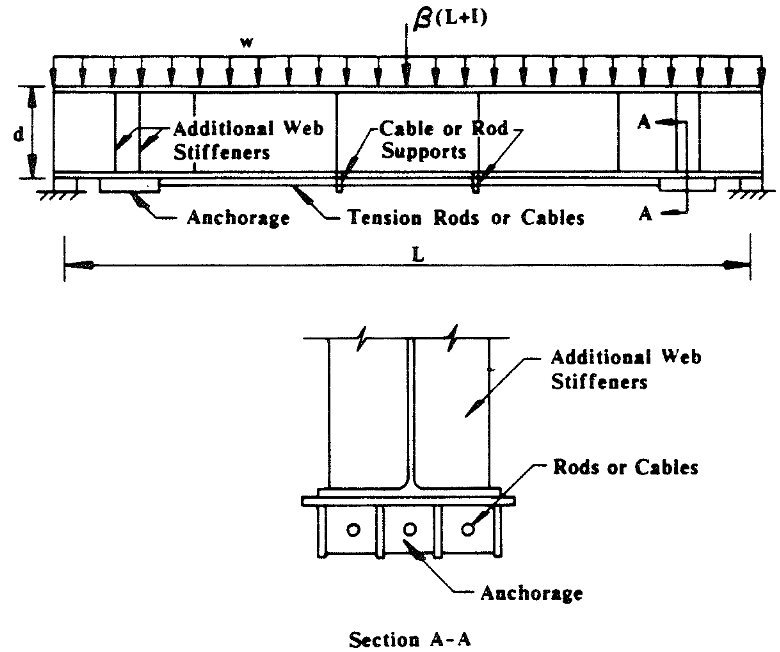

Figure D-1 Elevation View of Girder with Redundant Cables or Rods

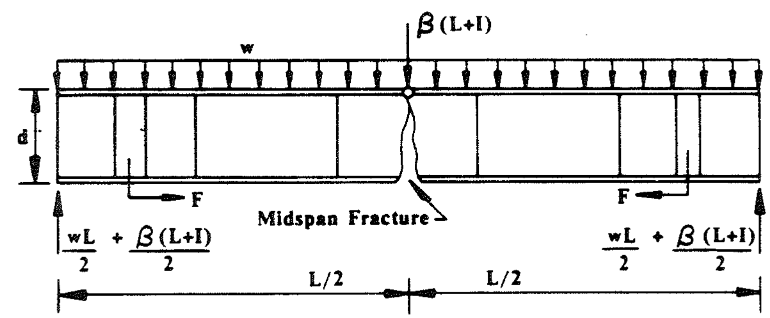

Figure D-2 Elevation View of Fractured Girder as a Free Body

CHAPTER D2

REDUNDANCY DESIGN EXAMPLES

EXAMPLE D1 - Allowable Stress, 110 ft. Span

Problem Statement

Evaluate the after-fracture redundancy of the simple span, two-girder bridge as shown in Fig. C-25 (C-8), Appendix C, which is to be retrofitted by tension rods as discussed in Chapter D1. The retrofitted bridge is required to carry an AASHTO HS-20 truck with 30% impact. Use the Allowable Stress Method with the allowable stresses in Chapter 3. Use A517 steel with a yield stress of 90 ksi. The evaluation consists of three parts: (1) determination of the required tension rod area for the condition, RRF = 1.0. (2) computation of $\left(\frac{\Delta}{L}\right)$ and $\left(\frac{\Delta}{L}\right)_D$ corresponding to the tension rod area provided by the ASM, and (3) determination of the required tension rod area to limit $\frac{\Delta}{L} = \frac{1}{200}$ and corresponding $\left(\frac{\Delta}{L}\right)_D$.

Compute Parameters

All the parameters involved in this evaluation are identical to those in Example C1.

$$w = 0.29 \text{ k/in}$$

$$\beta(L+I) = 93.6 \text{ k}$$

$$d = 121.5 \text{ in}$$

$$L = 1{,}320 \text{ in}$$

Required Tension Rod Area for RRF = 1.0

The required cross-section area of the tension rods for A517 steel when the RRF = 1.0, and with $f_{all} = 0.85\ F_y$ is given by Eq. D.4, as follows:

$$\text{Req'd. } A_C = \frac{1{,}320}{8 \times 121.5 \times 0.85 \times 90}(0.29 \times 1320 + 2 \times 93.6) = 10.12 \text{ in}^2$$

Use three 2 1/4 in. diameter rods, with $A_C = 11.93$ in^2.

Total Load and Dead Load Deflections

The midspan deflection ratios of the fractured girder retrofitted by the tension rods, with $A_C = 11.93$ in^2, as given by Eq's. D.12 and D.13 for total and dead load respectively, are

$$\frac{\Delta}{L} = \frac{(1{,}320)^2}{32 \times 30{,}000 \times (121.5)^2 \times 11.93}(0.29 \times 1{,}320 + 2 \times 93.6)$$

$$= \frac{1}{170} > \frac{1}{200} \text{ required}$$

$$\left(\frac{\Delta}{L}\right)_D = \frac{(1{,}320)^2}{32 \times 30{,}000 \times (121.5)^2 \times 11.93}(0.29 \times 1{,}320) = \frac{1}{253}$$

Required Tension Rod for Deflection Limit

The required cross-section area of the tension rods to limit $\frac{\Delta}{L} = \frac{1}{200}$ is given by Eq. D.16, as follows:

$$\text{Req'd. } A_C = \frac{(1{,}320)^2}{32 \times 30{,}000 \times (121.5)^2 \left(\frac{1}{200}\right)}(0.29 \times 1{,}320 + 2 \times 93.6)$$

$$= 14.02 \text{ in}^2$$

Use three 2 1/2 in. diameter rods with $A_C = 14.73$ in.^{2} with $A_C = 14.73$ in.^{2}, $\frac{\Delta}{L} = \frac{1}{210}$
Also, the dead load deflection ratio corresponding to the deflection limit, as given by Eq. D.14, is

$$\left(\frac{\Delta}{L}\right)_D = \frac{0.29 \times 1{,}320}{0.29 \times 1{,}320 + 2 \times 93.6}\left(\frac{1}{210}\right) = \frac{1}{313}$$

This dead load deflection ratio corresponds to a midspan deflection of 4.22 in. under the service loads. This is considered sufficient to provide an indication that damage has occurred.

EXAMPLE D2 - Load Factor, 110' ft. Span

Problem Statement

Evaluate the after-fracture redundancy of the simple span, two-girder bridge as shown in Fig. C-25 (C-8), Appendix C, which is to be retrofitted by tension rods as discussed in Chapter D1. The retrofitted bridge is required to carry an AASHTO HS-20 truck with 30% impact. Use the Load Factor Method with the load factors in Chapter 3. Use A517 steel with a yield stress of 90 ksi. The evaluation consists of three parts: (1) determination of the required tension rod area for the condition, RRF = 1.0, (2) computation of $\left(\frac{\Delta}{L}\right)$ and $\left(\frac{\Delta}{L}\right)_D$ corresponding to the tension rod area provided by the LFM, and (3) determination of the required tension rod area to limit $\frac{\Delta}{L} = \frac{1}{200}$ and the corresponding $\left(\frac{\Delta}{L}\right)_D$.

Compute Parameters

All the parameters involved in this evaluation are identical to those in Examples C1 and D1.

$$w = 0.29 \text{ k/in}$$

$$\beta(L+I) = 93.6 \text{ k}$$

$$d = 121.5 \text{ in}$$

$$L = 1{,}320 \text{ in}$$

Required Tension Rod Area for RRF = 1.0

The required cross-section area of the tension rods for A517 steel when the RRF = 1.0, and with $\gamma_D = \gamma_L = 1.18$ is given by Eq. D.7, as follows:

$$\text{Req'd. } A_C = \frac{1{,}320}{8 \times 121.5 \times 90} (1.18 \times 0.29 \times 1{,}320 + 2 \times 1.18 \times 93.6)$$
$$= 10.15 \text{ in}^2$$

Use three 2 1/4 in. diameter rods, with $A_C = 11.93 \text{ in}^2$.

Total Load and Dead Load Deflections

Check using Eq. D.15 whether Eq's. D.12, D.13 and D.14 are applicable to compute the total load and dead load deflection ratios. From Eq. D.15,

$$\frac{10.15}{10.12} = 1 \geq \frac{1}{1.18}$$

Since Eq. D.15 is satisfied, the midspan deflection ratios of the fractured girder retrofitted with tension rods having $A_C = 11.93 \text{ in}^2$, are identical to Example D1, or

$$\frac{\Delta}{L} = \frac{1}{170}$$

$$\left(\frac{\Delta}{L}\right)_D = \frac{1}{253}$$

Required Tension Rod for Deflection Limit

As in Example D1, the required area tension rods to limit $\frac{\Delta}{L} = \frac{1}{200}$ is identical to Example D1 since Eq. D.15 is satisfied with the larger area provided.

$$\text{Req'd. } A_C = 14.02 \text{ in}^2$$

Use three 2 1/2 in. diameter rods with $A_C = 14.73 \text{ in}^2$ as in Example D1.

As in Example D1 $\left(\frac{\Delta}{L}\right)_D = \frac{1}{313}$ which provides a midspan deflection of 4.22 in. under the service loads.

APPENDIX E

STEEL BRIDGE SUPERSTRUCTURE SUSCEPTIBILITY TO COMPLETE FAILURE DUE TO FATIGUE CRACKING AND BRITTLE FRACTURE

1. Suspended Spans with Two Girder:

   Links and hangers and the connecting pins are susceptible to fatigue cracking; pack rust can push hangers off pins.

2. Bar-Chain Suspension Bridge with Two Eyebars Per Panel:

   Fracture critical condition depends on the ability of a single eyebar and the joint at the panel point to resist loads.

3. Welded Tie Arches with Box Shaped Tie Girder:

   Partial length cover plates, diaphragm and floor beam connections, or details with low fatigue strength may be subject to fatigue crack growth and subsequent brittle fracture.

4. Simple Span Truss with Two Eyebars or Single Member Between Panel Points:

   If the floor system is rigidly connected to the verticals (hangers), redistribution of forces can take place, but bridge deflection may be excessive.

5. Simple Span Single Welded Box Girders with Category E Details:

   Fatigue cracks develop at details such as termination of longitudinal stiffeners or gusset plate could lead to fracture of the box, but fatigue crack growth would normally be at moderate rate, thus allowing time for inspection.

6. Simple Span Two Girder Bridges with Welded Partial Length Cover Plates on the Bottom Flange:

   The floor system, including the deck and the lateral bracing members, plus the diaphragms, provides alternative load paths and potential redundancy; deck deflection may be excessive for certain spans.

7. Continuous Span Two Girder System with Cantilever and Suspension Link Arrangement and Welded Partial Length Cover Plates

   Fracture of a girder will increase deflection; this may be excessive.

8. Simple Span Two Girder System with Lateral Bracing Connected to Horizontal Gusset Plates which are Attached to Webs:

   Differential forces in laterals could cause fatigue cracks in girder web at the ends of the gusset plate, particularly when the gusset plate is not attached to the floor beam or diaphragm connection plate. The vertical crack would grow toward the tension flange and may cause brittle fracture; little or no redundancy may be available.

9. Single Welded I-Girder or Box girder Pier Cap with Bridge Girders and Stringers Attached by Welding:

   Multigirders or stringers with bridge deck may permit redistribution of forces. Adverse details such as penetration of the pier cap web with a girder flange can result in cracking.

APPENDIX F

NEW YORK STATE OWNED STEEL BRIDGE SUPERSTRUCTURES RANKED BY ORDER OF SUSCEPTIBILITY TO COMPLETE FAILURE RESULTING FROM FATIGUE CRACKING AND BRITTLE FRACTURE

BRIDGE UPGRADE PROGRAM COMMITTEE

This is an estimate based upon assumptions made about material properties, predominant loading, anticipated details, existence of alternate load paths, quality of workmanship, relative movement of parts which bear upon each other, difficulty of inspection, etc., etc. The ranking is approximate, and reasons are given after the superstructure type description.

In the ranking of bridges supported by steel pins specially designed and constructed for that purpose, it is assumed that failure of the pin by stress corrosion cracking, fatigue cracking, brittle fracture, or shear is extremely unlikely. It is not unlikely that a suspended span hanger could fracture or be dislodged from its design location on the pin.

Three girder, truss, or arch superstructures are not ranked separately. They are obviously more fracture safe than two member systems but are not necessarily stable after the loss of a single member. No consideration is given to continuity even though it is agreed that continuity can provide redundancy.

RANK	TYPE AND REASON
1	2-Girders With Fixed Hanger Suspended Spans - Fixed Hanger Details are known to initiate weld and hanger cracking. There is no redundancy. Worksmanship quality poor.
2	2-Girders With Suspended Spans - Pin and Hanger Details are subject to crack initiation resulting from corrosion, design details, fretting, etc. Pack rust can push hangers off pins. Fifties era and earlier material properties and workmanship are often less than desirable. There is no redundancy.
4-2	Eyebar Trusses or Eyebar Suspension Chains Only 2-Eyebars Per Panel - Details, workmanship, and material properties often less than desirable. This superstructure type ranked after Expansion Hanger Details because there is no relative movement (wear and fretting) between the pins and eyebars. Most eyebar trusses have only 2-eyebars in some panels. The fracture criticalness of the structure will depend on the ability to survive the fracture of a single eyebar. When structural details fix the position of bridge pins against displacement and rotation, structures may survive the failure of a single eyebar in a 2-eyebar system. However, redundancy is generally very low.

5	2-Welded Plate Girders or 2-Rolled Beams With Partial Length Cover Plates or Welded Tied Arches Built Without A Fracture Control Plan - Partial Length cover plates and other Category e and E' details initiate fatigue cracks. These structures have no redundancy. If tensile stress and stress range are high, or material and workmanship quality are low, Rank 2 is more appropriate.
6	2-Welded Plate Girders - Less than desirable material properties, details, and soundness of welds in structures generally built prior to the evolution of modern workmanship and radiographic weld quality standards. Most structures of this type were built prior to 1968. There is no redundancy.
7-5	Steel Pier Caps Consisting of One Welded I-shape or Box Beam Attached to Intersecting Stringers by Welding - First designed in late sixties, most designs called for a minimum Charpy V-Notch toughness of 15 ft. lbs. at 40° F. Framing into intersecing members by welding makes these designs susceptible to fatigue cracking. However, care was taken to avoid fatigue -critical details and workmanship is generall good. There is no redundancy.
8	2-Welded Trusses or Arches (Employing Category D & E Details) - These structures are ranked after the fracture critical structures above because there may be some saving redundancy in the event of failure of a major component and because large structures have a greater proportion of deadload to total load. High tensile stress and stress range would justify lower ranking.
9	Single Welded Box Beam Ramp Structures - This type of structure, first constructed in the late sixties, has generally better workmanship standards than earlier construction. Charpy V-Notch toughness probably specfied - Not redundant.
10	On-Welded I-Shape or Box Beam Pier Cap, or Other Single Substructure Member - These members are rated better than the above since the effect of liveload and impact is less, reducing the probability of fatigue crack initiation and growth. Not redundant.
11	2-Riveted Plate Girders With Seated Spans - All web cuts necessary to seat spans concentrate stress and may be a fatigue crack initiation site, even though reinforced. Corrosion is often a problem at these locations due to joint leakage. These details require special attention during scheduled bridge inspections. Internal redundancy by parts.
12-2	2-Riveted Plate Girders - Rated more serious than riveted trusses due to a higher proportion of live load to total load and the absence of alternate load paths. Internal redundancy.
13-2	2-Trusses With Suspended or 2-Riveted Tied Arches (Depending on Suspender and Tie Girder Details) - Suspended span support details in trusses are generally more conservatively designed than in plate girders. Bronze bushings are often used to avoid steel-on-steel wear. Tied arches are often suspended spans. The tie girder is generally made up of redundant parts. Live load and impact are generally a minority portion of the total load. Nonredundant construction of suspenders and tie girders or poor steel-

on-steel suspender details justify Rank 2. Some intermediate truss spans are supported by long rocking members rather than suspenders. Members in compression are not subject to brittle fracture and, therefore, of less concern.

14 Suspension Bridges (State and Interstate bridges excluding bridges in New York metropolitan area) - This ranking is based upon age, difficulty of evaluating corrosion and fatigue damage to main cables and suspenders, ratio of live load and impact to total load and other somewhat nebulous factors. Poor details and high stress could place in Rank 2.

15 2-Riveted Trusses - Rated better than the above due to the redundancy of parts making up members, the redundancy of members that share the load after fracture of a major component, and ease of determining condition by visual inspection. Bottom laterals and stringers between floorbeams have prevented the collapse of trusses after failure of the bottom chord. Redundancy low.

16 2-Box Beams-Welded (Category D & E Details) - Better redundancy but poor details. Those structures built after the middle sixties generally have reasonable workmanship and material properties.

17 Redundant (Four or More) Stringers and Girders with Fixed Hanger Suspended Spans - Redundancy of this extremely bad detail makes total collapse remote. However, collapse of redundant superstructures is possible if there is no inspection and maintenance. The Kings Bridge failure in Australia is an example.

18 Redundant Welded (Four or More) Stringers or Girders With Partial Length Cover Plates - Category E and E' Details are likely candidates for fatigue cracking. Brittle fracture is probable if fatigue cracks are not discovered and repaired in a timely manner. However, collapse of a redundant superstructure, although possible, is extremely unlikely if there is an effective inspection program.

19 Eyebar Trusses or Eyebar Suspension Chains With Four or More Eyebars Per Panel - This ranking is based on the assumption that the pin will not fail and that failure of an isolated eyebar will not produce failure of the remaining members in the panel.

20 2-Welded Trusses (Category B & C Details) or Welded Tied Arches Built Under the Provisions of a Fracture Control Plan Employing Improved Toughness Steels - This ranking is based upon assumed progress in design, material properties, workmanship standards, and the elimination of details that shorten fatigue life. Welded tied arches are ranked lower than other trusses built under the provisions of a Fracture Control Plan because there is no viable alternate load path in the event of a tie girder failure.

21-20 2-Box Beams-Welded (Category B & C Details) - Assumed slightly better redundancy, good details, workmanship, and material properties.

22 2-Welded Box Girder Arches - 2 Hinge or 3 Hinge (Employing Category B & C Details) - This ranking is based on improved design and details and anticipated low tensile stress and stress range. High tensile stress and stress

range would make this structure type more appropriate for lower rank.

23 2-Welded Trusses Built Under the Provisions of a Fracture Control Plan Employing Improved Toughness Steels and Utilizing Only B & C Details - This ranking is based upon improved materials, details, workmanship, and a relatively low ratio of live load and impact to total load.

24 Short Radius Curved Superstructures Consisting of 3 or 4 Stringers - Short radius simple spans have a greater possibility of being fracture critical than continuous spans. Most designs of this type were built after 1967 using steel with a minimum specified Charpy V-Notch toughness of 15 ft. lbs. @ 40^{o} F. Workmanship and details generally good.

25 Redundant (Four or More) Stringers and Girders With Suspended Spans - This rank is based on redundancy. Properly designed and maintained suspension system provides safe and acceptable service.

26 2-Riveted Box Arches or Redundant (Four or More) Riveted Plate Girder Arches - 2 Hinge or 3 Hinge - This ranking is based upon an assumed low ratio of live load and impact to total load and the redundancy of parts and members.

27 Redundant (Four or More) Box Girders - This ranking based on redundancy and the knowledge that this type of structure is a recent design, employing generally good materials and workmanship.

28 Redundant (Four or More) Welded, Riveted, or Rolled I-Shape Stringers, Girders and Box Arches - Ranking of these categories is based upon redundancy, ease of inspection, and, in the case of Box Arches, the low ratio of live load and impact to the total load.