Steel Bridge Design Handbook

CHAPTER 9

Redundancy

February 2022

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Foreword

The Steel Bridge Design Handbook covers a full range of topics and design examples to provide bridge engineers with the information needed to make knowledgeable decisions regarding the selection, design, fabrication, and construction of steel bridges. The Handbook has a long history, dating back to the 1970s in various forms and publications. The more recent editions of the Handbook were developed and maintained by the Federal Highway Administration (FHWA) Office of Bridges and Structures as FHWA Report No. FHWA-IF-12-052 published in November 2012, and FHWA Report No. FHWA-HIF-16-002 published in December 2015. The previous development and maintenance of the Handbook by the FHWA, their consultants, and their technical reviewers is gratefully appreciated and acknowledged.

This current edition of the Handbook is maintained by the National Steel Bridge Alliance (NSBA), a division of the American Institute of Steel Construction (AISC). This Handbook, published in 2021, has been updated and revised to be consistent with the 9th edition of the AASHTO LRFD Bridge Design Specifications which was released in 2020. The updates and revisions to various chapters and design examples have been performed, as noted, by HDR, M.A. Grubb & Associates, Don White, Ph.D., and NSBA. Furthermore, the updates and revisions have been reviewed independently by Francesco Russo, Ph.D., P.E., Brandon Chavel, Ph.D., P.E., and NSBA.

The Handbook consists of 19 chapters and 6 design examples. The chapters and design examples of the Handbook are published separately for ease of use, and available for free download at the NSBA website, www.aisc.org/nsba.

The users of the Steel Bridge Design Handbook are encouraged to submit ideas and suggestions for enhancements that can be implemented in future editions to the NSBA and AISC at solutions@aisc.org.

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7. Supplementary Notes
The previous edition of this Handbook was published as FHWA-HIF-16-002 and was developed to be current with the 7th edition of the AASHTO LRFD Bridge Design Specifications. This edition of the Handbook was updated to be current with the 9th edition of the AASHTO LRFD Bridge Design Specifications, released in 2020.

8. Abstract
Redundancy is “the quality of a bridge that enables it to perform its design function in a damaged state” and it is considered a desired characteristic of good design. The consideration of redundancy affects the design, fabrication and in-service inspection of steel bridge members especially when they are classified as fracture critical member (a steel tension member whose failure would probably cause collapse). This chapter provides engineers with an explanation of redundancy and how it affects the design, fabrication, inspection, and management of steel girder bridges. The chapter also highlights some of the recent industry advancements related to redundancy considerations.

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Steel Bridge Design Handbook: Redundancy

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1.0 REDUNDANCY

1.1 Introduction

A typical dictionary defines redundant as "exceeding what is necessary or normal," and provides "superfluous" as a synonym. In the context of bridge engineering, redundancy is considered a characteristic of good design. The AASHTO *LRFD Bridge Design Specifications* 1.3.4 states "Multiple-load-path and continuous structure should be used unless there are compelling reasons not to use them" (1). There are cases where designs with non-redundant members are perfectly acceptable, and may clearly be the best value solution (e.g. single column piers, trusses, box girders, suspension bridges, etc.). This is often the case for major river crossings where the cost of providing complete redundancy in all members is prohibitive. Apart from these special cases, redundant design is preferred to the extent possible.

Historically, bridge members have been classified as redundant or non-redundant by the designer simply determining whether alternative load paths exist. If you were to poll a group of bridge designers, most would consider a bridge supported by four parallel members as redundant and one supported by two parallel members non-redundant. The redundancy of three parallel members is often viewed differently depending on the experience, criteria, and conservatism of the Engineer and/or Owner. The question of the sufficiency of these alternative load paths to carry the additional load and the system response was usually not a consideration.

1.2 Redundancy Classifications

The AASHTO LRFD *Bridge Design Specifications* (1) defines redundancy as "the quality of a bridge that enables it to perform its design function in a damaged state" and a redundant member as "a member whose failure does not cause failure of the bridge." Redundancy can be provided in one or more of the following ways:

1. load-path redundancy,
2. structural redundancy, and
3. internal redundancy.

1.2.1 Load-Path Redundancy

Load path redundancy is based on the number of main supporting members between points of support, usually parallel, such as girders or trusses. A member is considered load-path redundant if an alternative and sufficient load path is determined to exist. Load-path redundancy is the type of redundancy that designers consider when they count parallel girders or load paths. However, merely determining that alternative load paths exist is not enough. The alternative load paths must have sufficient capacity to carry the load redistributed to them in the event of a failed member. If the additional redistributed load overloads the alternative load path, progressive failure may occur, and the bridge may collapse.

1.2.2 Structural Redundancy

Structural redundancy can be provided by continuity in main members over interior supports or other 3-dimensional mechanisms available when the bridge is considered to behave as a system. A member is considered structurally redundant if its continuity or support conditions are such that failure of the member merely changes the system behavior but does not result in the collapse of the superstructure. Again, the member with modified support conditions must be sufficient to carry loads in its new configuration. For example, the failure of the negative-moment region of a two-span continuous girder is not critical to the survival of the superstructure if the positive-moment region is sufficient to carry the load as a simply-supported girder.

1.2.3 Internal Redundancy

Internal member redundancy can be provided by built-up member detailing that provides mechanical separation of elements (bolted or riveted) in an effort to prevent failure propagation across the entire member cross section. A member is considered internally redundant if a sufficient cross section exists within the member itself that can carry the load in the event of failure of one of the elements. To evaluate the sufficiency of the cross section in the damaged condition, the internal eccentricities and moments must be considered, but there is no need to quantify the global response of the bridge system.

1.3 Non-redundant Steel Tension Members

Any steel bridge member that is subjected to tension stress has the small possibility of developing cracks from discontinuities introduced in fabrication or by fatigue crack growth. Steel tension members that are also non-redundant are given the label “Fracture Critical Member” (FCM) which is used to identify a certain class of bridge members that require special treatment in their design, fabrication, and management to avoid fracture. The FCM label should not be misunderstood to be a reflection of the bridge’s structural safety. All new bridges, with FCMs or not, are designed to meet the current design standards of the AASHTO *LRFD Bridge Design Specifications*, so it can be said that they provide equal level of safety (as measured by the LRFD safety index) if designed properly. The FCM label triggers supplemental requirements referred to as the AASHTO/AWS Fracture Control Plan (FCP) in fabrication and Fracture Critical Inspection during in-service inspection to detect the presence of rejectable discontinuities, cracks, or other anomalous damage conditions which may lead to a safety concern.

The National Bridge Inspection Standards (NBIS) (2) define a fracture critical member (FCM) as “a steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse.”

The AASHTO Manual for Bridge Evaluation (MBE) (3), provides the following definition: “a fracture-critical member (FCM) is a steel member in tension, or with tension elements, whose failure would probably cause a portion of or the entire bridge to collapse.”

The AASHTO *LRFD Bridge Design Specifications*, 9th Edition (1), defines a fracture critical member as " a steel primary member or portion thereof subject to tension whose failure would probably cause a portion of or the entire bridge to collapse."

While the definitions of a fracture-critical member are nearly the same, there are sometimes different interpretations for "failure," "probably," and "collapse,", and therefore classifications of FCMs sometimes vary depending on the experience, criteria, and conservatism of the Engineer and/or Owner.

In 2012, the Federal Highway Administration (FHWA), Office of Bridges and Structures, issued a memorandum regarding the clarification of requirements for fracture critical members (4). In this memorandum, FHWA agrees with either of the FCM definitions published in the AASHTO MBE (3) and LRFD *Bridge Design Specifications* (1), but also recognizes the inconsistency of the language between the two. As such:

- FHWA interprets the LRFD's use of "component in tension" to be a steel member in tension, or sub-element within a built-up member that is in tension, and
- FHWA interprets the phrase from LRFD, "inability of the bridge to perform its function" to mean the inability of the bridge to safely carry some level of traffic (live load) in its damaged condition.

The live load for the damaged condition could be taken as less than the full design live load for the strength limit state load combination. However, load factors and combinations used to evaluate the damaged condition must be agreed upon between the Owner and Engineer, and reviewed by the FHWA (4).

Traditionally, for the purposes of identifying FCMs, redundancy has been defined primarily based on load-path redundancy alone, which was often determined by assessing the number of parallel main members provided, or the spacing of transverse members which could be utilized as a secondary load path around a damage section, without any additional investigations utilizing higher order analysis. However, experimental and analytical research has shown that bridges that used to be assumed non-redundant, actually may provide a certain level of redundancy through three-dimensional system behavior and lateral load redistribution. Additionally, the bridge engineering community has begun to discover through modern analytical techniques that system redundancy may often exist, even though few secondary load paths are readily apparent.

2.0 FRACTURE CONTROL

2.1 Historical Development of a Fracture Control Plan

The genesis of the steel bridge fracture requirements can be traced to the collapse of the Point Pleasant Bridge over the Ohio River between Point Pleasant, West Virginia, and Kanauga, Ohio, in 1967. (The bridge was more commonly called the Silver Bridge for its bright coating of aluminum paint.) This eyebar-chain suspension bridge collapsed due to the brittle fracture of one non-redundant eyebar supporting the bridge's main span.

Based upon concerns of the FHWA about the safety of non-redundant steel bridge members for brittle fracture, they and the American Iron and Steel Institute (AISI) sponsored research to address the issue. In 1973, after much debate and compromise, Charpy V-notch (CVN) toughness criteria were adopted into the AASHTO M 270/ASTM A 709 material specification to provide a minimum level of toughness; assuring that at the lowest service temperatures, the steel would exhibit toughness in the transition zone (i.e. not in the lower shelf). Note that this does not guarantee against brittle fracture, rather it is more of a quality assurance tool that reduces the susceptibility of the steel to fracture. Additionally, in 1974the AASHTO bridge design specifications were revised to comprehensively address fatigue design to assure that critical cracks would not develop during the service life of the bridge. These provisions introduced the six fatigue categories and figures to define the fatigue resistance of various details which still continue to this day. These fatigue design provision acknowledged a reduction in the fatigue allowable stress ranges for non-redundant members.

In 1978 AASHTO published the first edition of the *Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members* (5), and this became known as the AASHTO Fracture Control Plan (FCP). These guide specifications introduced the term of "fracture critical" and further distinguished such members to have more stringent CVN requirements than were published in AASHTO M 270/ASTM A 709. Second, for design the fatigue stress ranges were reduced for fracture critical members. Lastly, they introduced more stringent fabrication and weld quality requirements. These guide specifications are no longer published by AASHTO as the provisions within them have been fully integrated into ASTM A709, the AASHTO *LRFD Bride Design Specifications* (1), and Clause 12 of the AASHTO/AWS D1.5 *Bridge Welding Code* (6).

Since 1988, fracture critical members have been mandated to have enhanced in-service bridge inspection requirements. This was due to the collapse of the Mianus River Bridge carrying Interstate 95 in Greenwich Connecticut in 1983. Although not a caused by a fracture, this bridge failed dramatically when the suspended span of a pin-and-hanger girder system collapsed. Corrosion product accumulation behind hanger plates pushed them off the pin resulting in total failure of the non-redundant span. As a result of this, the National Bridge Inspection Standards (NBIS) (2) were revised in 1988 requiring biennial hands-on inspections of all fracture critical members.

A fracture control plan is often described as a three legged stool, with each leg representing requirements for material, fabrication, and inspection. Removing any one leg means the stool cannot stand, which is analogous to exposing a risk of failure by fracture. It has often been thought the provisions of the AASHTO FCP met the intent of the three-legged stool, but the "inspection" leg was limited to fabrication inspection only; not addressing in-service inspection. Not until 1988 was in-service inspection addressed with the enhanced inspection requirements for FCMs (which is dictated by FHWA, not AASHTO). From that point forward the AASHTO/FHWA Total Fracture Control Plan[1] was made complete and each leg of the stool is supported by: design and material selection in accordance with the AASHTO *LRFD Bridge Design Specifications* (1); fabrication and inspection of the elements in accordance with Clause 12 of the AASHTO/AWS D1.5 *Bridge Welding Code* (6); and in-service hands-on field inspections of the bridge as mandated by 23CFR650 (Code of Federal Regulations).

2.2 Materials and Fabrication

2.2.1 A Fracture Control Plan for Non-redundant Steel Bridge Members

FCMs are to be identified on design plans to ensure fabrication of these members is to a higher quality standard than typical members with load-path redundancy. The AASHTO Standard Specifications for Transportation Materials and Methods of Sampling and Testing (7) requires steels used for FCMs to meet higher CVN toughness requirements and contain fine-grained material. Additional fabrication and inspection procedures, and more strict shop certification is required to meet the AWS D1.5 *Bridge Welding Code* (6) requirements for fracture critical fabrication. The fracture critical fabrication requirements are intended to provide a lower likelihood of fatigue crack initiation by reducing the frequency and size of defects in fabrication. Material and fabrication requirements developed for the FCP also increase the tolerance to cracks and other discontinuities in members fully or partially in tension. It should be noted that CVN requirements do not necessarily guarantee uniform crack tolerances among the different grades of steel.

2.2.2 Identification of FCMs for Design

Article 6.6.2 of the AASHTO *LRFD Bridge Design Specifications* (1) states that the "Engineer shall have the responsibility for determining which, if any, component is a fracture critical member. Unless a rigorous analysis with assumed hypothetical cracked components confirms the strength and stability of the hypothetically damaged structure, the location of all FCMs shall be clearly delineated on the contract plans."

The FHWA expects that all members identified as FCMs according to load path redundancy be fabricated to meet the fracture critical requirements for quality (4).

In accordance with the FHWA memo (4), when identifying FCMs during design, it is not the failure of only the particular element in tension that needs to be considered with regard to performance of the damaged bridges, but rather the failure of the entire member containing that tension element. For example, a bridge girder in bending has two elements in tension, a flange and a portion of the web. For the purpose of the redundancy assessment, all three elements of

[1] The term Total Fracture Control Plan was actually created in 2015 in an AISC Modern Steel Construction article titled "Are You Sure That's Fracture Critical?" (9).

the girder cross-section, tension flange, web and compression flange should be considered fractured. However, for the purposes of fabrication, all three of the individual components would not necessarily be considered as fracture critical.

In accordance the FHWA memo (4), using a rigorous analysis as identified in AASHTO LRFD Article 6.6.2 to classify FCMs would not meet expectations of quality for materials and fabrication. Non-load path redundant members determined to be non-fracture critical through refined analysis will still be an important member for the structure. Therefore, regardless of any rigorous analysis performed, all non-load path redundant tension members shall be fabricated in accordance with the AASHTO FCP ((1) and (6)) to enhance safety and serviceability over the design life of the bridge.

2.3 In-Service Inspection

2.3.1 System Redundant Member

The FHWA memorandum (4) defines a new member classification called a System Redundant Member (SRM), which is a member that receives fabrication according to the AASHTO FCP, but need not be considered a fracture critical member for in-service inspection. SRMs are to be designated on the design plans with a note indicating that they shall be fabricated in accordance with AWS D1.5 *Bridge Welding Code* Clause 12 (6) and using steel meeting fracture critical toughness requirements. A refined analysis demonstrating sufficient structural system redundancy exists is to be used to determine when a member can be defined as an SRM. SRMs determined via refined analysis techniques are only applicable to in-service inspection protocol and required frequency of inspection, not for design and fabrication. The criteria and procedures for the refined analysis and subsequent evaluation should be agreed upon between the Engineer and Owner.

2.3.2 Identification of FCMs for In-Service Inspection

Currently available refined analysis techniques have provided a means to more accurately classify FCMs for new designs and to re-evaluate existing bridge members that were previously classified as fracture critical on the record design documents. If a refined analysis demonstrates that a structure has adequate strength and stability sufficient to avoid partial or total collapse and carry traffic in the presence of a completely fractured member (through structural redundancy) the member does not need to be considered fracture critical for in-service inspection protocol, and can thus the member can be classified as a System Redundant Member (SRM). The assumptions and analyses conducted to support this determination need to become part of the permanent inspection records or bridge file so that it can be revisited and adjusted as necessary to reflect changes in bridge conditions or loadings. This may include the loading used for the faulted condition, the type of refined analysis (including level of analysis and whether material on geometric non-linear analyses were utilized), and the deflection criteria.

Non-load path redundant tension members in existing bridges that were not fabricated to meet the AASHTO FCP are not eligible for relief from fracture critical in-service inspection based on such refined analysis. These bridge elements must always be treated as FCM for inspection purposes. Presently, the FHWA memorandum (4) does not include provisions for bridge elements not fabricated to the FCP introduced in 1978. The Owner should verify and document that the materials and fabrication specifications of any existing bridge being assessed for structural redundancy would meet the AASHTO FCP.

The FHWA classification of members for in-service inspection protocol provides recognition of structural redundancy that is demonstrated by system response only, and does not recognize redundancy from internal built-up details. Currently, the FHWA does not accept the approach of using internally redundant detailing to demonstrate that a non-load path redundant member is not fracture critical (4).

3.0 QUANTIFYING REDUNDANCY

3.1 Redundancy in Design

One of the stated objectives of the development of the *AASHTO LRFD Bridge Design Specifications* (1) was to enhance the redundancy and ductility of our nation's bridges. The consequences of redundancy are included in the basic LRFD equation of Article 1.3.2 of the LRFD Specifications.

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n$$

where:

η_i = load modifier, and is the product of factors relating to ductility, η_D, redundancy, η_R, and operational importance, η_I,

γ_i = load factor,

Q_i = force effect,

ϕ = resistance factor, and

R_n = nominal resistance.

Quantitative factors relating to the redundancy of a structural system were not available during the development of the first edition of the LRFD Specifications, so a "placeholder" was provided in the form of η_R. The specified values of η_R of Article 1.3.4 of the LRFD *Bridge Design Specifications* were subjectively chosen by the AASHTO Subcommittee on Bridges and Structures. For structural systems with conventional levels of redundancy, the factor is 1.0. For non-redundant systems, the factor is 1.05, thus increasing the force effect. Conversely, for systems with exceptional levels of redundancy, the factor is 0.95 resulting in slightly less force effect. The load modifiers relating to redundancy are summarized in Table 1 below.

Table 1 AASHTO LRFD load modifiers relating to redundancy

CLASSIFICATION	LOAD MODIFIER
Redundant (as designed in accordance with LRFD Specifications)	1.00
Non-redundant	1.05
Exceptionally redundant	0.95

Redundancy is an attribute of the structural system and thus theoretically should be on the resistance side of the equation. In the LRFD *Bridge Design Specifications*, the factors appear on the load side of the LRFD equation for practical purposes. When maximum load factors are applied to the permanent loads the load modifier is applied as shown in equation 1.3.2.1-2 of the

LRFD Specifications. When minimum load factors are chosen, the inverse of the load modifier is used as shown in equation 1.3.2.1-3 of the LRFD Specifications.

3.2 System Response

In support of the LRFD Specifications, the National Cooperative Highway Research Program (NCHRP) initiated NCHRP Project 12-36 which resulted in NCHRP Report 406, *Redundancy in Highway Bridge Superstructures* (11). This research developed system factors for girder bridges which reflect the redundancy of the structural system by assessing the safety and redundancy of the system. Tables of system factors are given for simple-span and continuous girder bridges with compact negative-moment sections (an uncommon practice), respectively. For this study, the researchers considered continuous steel bridges with noncompact sections in negative bending as non-redundant.

The system factors, ϕ_S, are given as a function of number of girders in the cross section and girder spacing. The proposed system factors replace the redundancy load modifier, η_R, used in Article 1.3.2. However, the system factor is applied to the resistance side of the LRFD equation as it is related to the resistance of the system. The load modifiers for ductility and operational importance are unaffected. The values of system factors range from a low of 0.80 to a high of 1.20. A system factor of greater than 1.0 rewards redundancy; a value less than 1.0 represents a penalty.

Table 2 and Table 3 below are adaptations of the tables in NCHRP Report 406 (11). With "a distributed set of diaphragms" throughout the span, the values of the tables may be increased by 0.10.

Table 2 System factors for simple-span I-girder bridges

GIRDER SPACING	4 GIRDERS	6 GIRDERS	8 GIRDERS	10 GIRDERS
4 feet	0.86	1.03	1.05	1.05
6 feet	0.97	1.01	1.01	1.01
8 feet	0.99	1.00	1.00	1.00
10 feet	0.98	0.99	0.99	-
12 feet	0.96	0.97	-	-

Table 3 System factors for continuous span I-girder bridges with compact negative moment sections

GIRDER SPACING	4 GIRDERS	6 GIRDERS	8 GIRDERS	10 GIRDERS
4 feet	0.83	1.03	1.04	1.03
6 feet	1.03	1.07	1.06	1.06
8 feet	1.06	1.07	1.07	1.07
10 feet	1.06	1.07	1.07	-
12 feet	1.04	1.05	-	-

The effects of girder spacing evident in the tables may appear to be counter-intuitive, but the researchers offer an explanation. They suggest that system factors tend to increase as the girder spacing increases from 4 feet to 8 feet since in narrower bridges the girders tend to be more equally loaded with little reserve strength available. For girder spacings above 8 feet, loads are not so equally distributed among the girders, and as the more heavily loaded girders go into the inelastic range, the more lightly loaded girders can pick up the load which is shed.

Further, the effects of continuity also appear to be counter-intuitive for the narrowest bridges (in other words, for girder spacings of 4 feet). For girder spacings above 4 feet, the system factors for continuous steel bridges are greater than those for simple-spans indicating more redundancy, on average 7% greater. Such is not the case for the steel bridges with girder spacings equal to 4 feet. While the authors discuss at length their opinion that continuous I-girders with non-compact negative-moment regions are non-redundant (in other words, they recommend applying a system factor of 0.80), they do not speak to this apparent inconsistency for continuous steel bridges with compact negative-moment regions. Most likely, it is a similar narrow-bridge effect as discussed earlier.

The values in the tables are presented in a manner suggesting more precision than is warranted based upon the inherent assumptions, and the assumptions themselves have been subject to debate (such as the need for compact negative-moment sections to consider continuous bridges redundant). The practicing bridge community has yet to embrace the systems factors of NCHRP Report 406 (11), and they have not been adopted by AASHTO for use in the LRFD Specifications.

More importantly, the Report developed criteria for redundancy and redefines redundancy as a damaged structure's ability to continue to carry load, safely and serviceably.

3.3 Redundancy in Evaluation

At their 2005 meeting, the AASHTO Subcommittee on Bridges and Structures (SCOBS) adopted the AASHTO *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* (12) with revisions elevating allowable stress (ASR) and load factor rating (LFR) to equal status with LRFR, as the AASHTO *Manual for Bridge Evaluation* (3). The Guide Manual was originally developed by a team including one of the authors of NCHRP Report 406 (11) and as such includes some aspects of that report. System factors, applied to the member resistance and similar to those of the NCHRP Report 406, are included as an alternative to system factors derived from the load modifiers of the LRFD Specifications. For most bridges, these alternative system factors are specified as 1.0, but for bridges deemed less redundant in NCHRP Report 406 (11), for example, two-girder bridges, three- and four-girder bridges with narrow girder spacing and widely spaced floorbeams supporting non-continuous stringers, the system factors are reduced to as low as 0.85. See Table 4 below.

Table 4 System factors from the AASHTO Manual for Bridge Evaluation (3)

SUPERSTRUCTURE TYPE	SYSTEM FACTOR
Welded members in two-girder/truss/arch	0.85
Riveted members in two-girder/truss/arch	0.90
Multiple eyebar members in truss bridge	0.90
Three-girder bridges with girder spacing ≤ 6 feet	0.85
Four-girder bridges with spacing ≤ 4 feet	0.95
All other girder bridges and slab bridges	1.00
Floorbeams with spacing ≥ 12 feet an non-continuous stringers	0.85
Redundant stringer subsystems between floorbeams	1.00

4.0 REDUNDANCY ANALYSIS

4.1 Theory

In the event of a member's brittle failure, the survival of the superstructure (and its classification as a redundant member) is contingent upon the system's ability to safely redistribute the existing applied and internal loads.

4.2 Applied Load

Based upon the working definition of redundancy provided earlier, an acceptable level of load-carrying capacity for the damaged superstructure must be agreed upon. Currently, the design literature does not provide a definitive answer. The Commentary to the LRFD Specifications (Article 6.6.2) provides some insight: "Relief from the full factored loads associated with the Strength I Load Combination should be considered, as should the number of loaded design lanes versus the number of striped traffic lanes" (1). Thus, this statement suggests that a two-tub girder cross section could be deemed system redundant by analysis if the superstructure with one fractured bottom flange can carry the factored live load in the lanes striped on the bridge, and not necessarily the factored live load of all of the design lanes that could be placed on the bridge. Additionally, the required load factors must also be re-visited for the reliability of the damaged bridge.

4.3 Internal Loads

The release of energy during the fracture should be considered in the modeling to determine if the superstructure can survive the event. Research at the University of Texas suggested that the gain in strength due to rapid loading may offset the increase in load due to impact. It was noted that during the simulated fracture test, an average dynamic increase factor of 1.30 was estimated from the data captured from different types of gauges at different locations (13). Additionally, research on the after-fracture performance of a two-line, simple steel truss bridge showed that the dynamic amplification from the induced blasts ranged from 1.08 to 1.41, depending on the instrumented member (14).

4.4 Analysis

The level of rigor required for a refined analysis to demonstrate that sufficient redundancy exists is addressed in a high-level form in the commentary to Article 6.6.2 of the LRFD specifications offers some general guidance regarding refined analysis for the demonstration of redundancy:

"The criteria for a refined analysis used to demonstrate that part of a structure is not fracture critical has not yet been codified. Therefore, the loading cases to be studied, location of potential cracks, degree to which the dynamic effects associated with a fracture are included in the analysis, and fineness of models and choice of element type should all be agreed upon by the Owner and the Engineer. The ability of a particular software product to adequately capture the complexity of the problem should also be considered and the choice of software should be mutually agreed upon by the Owner and the Engineer" (1).

Current analytical techniques can provide a means for Engineers to assess bridge redundancy and identify fracture critical members with the full consideration of three-dimensional system behavior in various damage scenarios. To demonstrate that a structure has adequate strength and stability sufficient to avoid partial or total collapse and carry a certain level of traffic in the presence of a completely fractured FCM, a criteria and procedure for the refined analysis and subsequent evaluation should be agreed upon between the Engineer and Owner. Additionally, the FHWA requires approval of the refined analysis and evaluation criteria that is used to conduct the study (4).

Again, in accordance with the FHWA, at time of this writing a refined analysis can only be used to demonstrate structural redundancy for in-service inspection protocol and frequency. Only load path redundancy may be considered for member design and fabrication.

4.5 AASHTO's *Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members*

AASHTO's *Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members* (16) (referred to hereafter as the SRM Guide Specification) a tool that allows engineers to take advantage of previously unexploited system-level redundancy, and owners to efficiently allocate resources to provide better infrastructural solutions to the public.

Released in 2018, the SRM Guide Specification tackles a complex problem: characterizing the demand and capacity of a structure in which a primary steel tension member has failed. For a system to be considered redundant, two fundamental concepts regarding load were followed: First, the bridge cannot be expected to operate as reliably in the faulted condition as in the pristine condition. Second, the bridge must be able to survive the failure event and provide service in the faulted state.

The SRM Guide Specifications introduce two load combinations, Redundancy I and Redundancy II. Redundancy I characterizes the loads experienced by the structure during the failure event, which is assumed to be sudden fracture of a primary steel tension member. This load combination is analogous to an extreme event load combination in which the event load includes the dynamic amplification of load due to the inertial effects of the member failure. Redundancy II basically warranties strength in the faulted condition against normal use until the member failure is detected.

The SRM Guide Specification contains guidelines to evaluate, via non-linear, detailed finite element models, the capacity of a steel bridge after the hypothetical failure of a primary tension member. Typical analysis procedures are not capable of reliably capturing the mechanisms that lead to redundancy without being overly conservative, so finite element analysis is needed to simultaneously consider and evaluate various load paths. The guide includes all necessary information for conducting a detailed finite element analysis, including material models for concrete and steel, meshing requirements, application of boundary conditions, and interactions and constraint modeling, as well as detailed provisions to model shear stud behavior. The guide also includes failure criteria intended to prevent the need for integrating stress data from a finite element analysis with sectional forces and moments.

Non-load-path redundant tension members evaluated and meeting the criteria of the SRM Guide Specifications will be deemed acceptable for consideration as SRMs in accordance with the 2012 FHWA memo (4). However, the owner is still required to submit the detailed analysis and evaluation conducted per the Guide Specifications for review by the FHWA Office of Bridges and Structures. While there may be an additional design cost associated with the required analysis and evaluation, a life cycle cost savings can be realized by the owner as SRMs do not need the calendar-based hands-on in-service inspections required for FCMs.

For more information regarding the SRM Guide Specification the reader is encouraged to review a February 2020 *Modern Steel Construction* article, “Revisiting Redundancy in Steel Bridges: Part Two,” (17) as well as an October 2020 *Modern Steel Construction* article, “That’s Not Fracture Critical,” (18) to learn more about the use of the SRM Guide Specification to satisfy the SRM classification in accordance with the FHWA memo (4).

4.6 AASHTO’s *Guide Specifications for Internal Redundancy of Mechanically Fastened Built-Up Steel Members*

The new AASHTO *Guide Specification for Internal Redundancy of Mechanically Fastened Built- Up Steel Members* (19) (referred to hereafter as the IRM Guide Specification) is a tool to help engineers better understand and leverage internal redundancy for built-up member structures and exploit their strength advantages and resistance to failure. The IRM Guide Specification shows how internal redundancy might be exploited in new designs and also includes a methodology to establish the interval for in-service inspection specifically intended to identify whether any of the tension components have failed. The guidelines are realistic about what can be reliably found during inspections and for what duration undiscovered damage may be safely tolerated.

The general basic steps for analyzing a built-up member for internal redundancy using the IRM Guide Specification are as follow:

- Screening criteria such as condition and remaining fatigue life
- Strength limit checks in the assumed faulted condition
- Fatigue life check in the assumed faulted condition
- Selection of a special inspection interval based on fatigue crack growth rates

The reader is cautioned however, that at the time if this writing, the FHWA does not accept the approach of using internally redundant detailing to demonstrate that a non-load path redundant member is not fracture critical (4). Further research and investigation may lead to changes in these restrictions, and adoption of the IRM Guide Specification.

For more information regarding the IRM Guide Specification the reader is encouraged to review an April 2020 *Modern Steel Construction* article, “Revisiting Redundancy in Steel Bridges: Part Three” (20).

5.0 ENHANCING REDUNDANCY

5.1 Design of New Bridges

The concept of bridge designs with varying levels of redundancy as championed by the LRFD Specifications has not found favor among practicing bridge engineers. Tradition has led to designers thinking of a bridge as redundant or non-redundant without varying degrees.

For bridges that have traditionally been labeled with fracture-critical members, the new AASHTO SRM Guide Specification and the AASHTO IRM Guide Specification can be utilized by designers to evaluate the redundancy of the member and overall structure. Both of these guide specifications offer a codified strategy to perform these evaluations.

Another manner to enhance the performance of non-redundant bridges is the selection of high-performance steels, such as A709 HPS70W, with their inherent enhanced fracture toughness. Designers should consider the use of steels with higher fracture toughness properties for use in non-redundant bridge members. Redundant members need not be fabricated from high-performance steel, unless warranted by unusually special conditions.

5.2 Rating and Retrofit of Existing Bridges

The application of the system factors suggested in the AASHTO *Manual for Bridge Evaluation* (3) (see Table 4) to the rating of existing bridges could lead to inadequate ratings for bridges with non-redundant members such as two-girder bridges. For example, a two-girder bridge designed without the application of system factors would be rated with a system factor of 0.85 reducing its resistance by 15 percent. If this bridge does not rate now, is it significant? The bridge has not changed, but our thoughts on reliability and safety have. Prior to posting or retrofitting, the bridge system (primary and secondary members including the deck and appurtenances) could be analyzed via a refined analysis to determine if system redundancy exists in the structure.

Two-girder bridges (or arches or trusses) designed in accordance with the LRFD *Bridge Design Specifications* will actually be more reliable or safer than those designed in accordance with the older AASHTO *Standard Specifications for Highway Bridges* (15). The calibration of the LRFD *Bridge Design Specifications* "set the bar" at the level of safety in multi-girder bridges where the increased load distribution of more refined lateral live-load distribution factors compensated for the increased live load of the HL-93 notional live-load model. Two-girder bridges do not enjoy the load distribution enhancement. This little-recognized fact should be factored into the considerations of rating a bridge with non-redundant members but designed to the LRFD Specifications.

6.0 ADDITIONAL READING AND RESOURCES

A Proposed Fracture Control Plan for New Bridges with Fracture Critical Members provides a more detailed discussion of fabrication issues that occurred in the 1970's along with commentary of where some provisions may have come from in the AASHTO Fracture Control Plan (10).

National Cooperative Highway Research Program (NCHRP) Synthesis 354, *Inspection and Management of Bridges with Fracture Critical Details* (8), provides detailed background on the fracture control plan for non-redundant welded steel bridge members that now appears in AASHTO/AWS D1.5 (6).

An article in AISC's *Modern Steel Construction*, "Are You Sure That's Fracture Critical?" provides additional guidance on the classification of Fracture Critical Members and Fracture Control Plans, and introduces the concept of a Total Fracture Control Plan (9).

An AISC *Engineering Journal* paper, "Transformative Approaches for Evaluating the Criticality of Fracture in Steel Members," summarizes some of the recent advancements related to the topic of the fracture critical members, system member redundancy, and internal member redundancy (21). The paper presents the concept of an integrated fracture control plan, encompassing material, design, fabrication, and inspection and how it can ensure fracture is no more likely than any other limit state, ultimately allowing for a better allocation of owner resources and increased steel bridge safety.

NCHRP Research Report 883 (22) presents the background research for the AASHTO *Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members* (16). The report describes the analysis methodology and provides application examples. The analysis methodology is based on comprehensive 3-D finite element analyses (FEA) and case studies to evaluate the redundancy of new and existing steel bridges with members traditionally designated as fracture critical members including simple- and continuous-span I-girder and tub-girder, through-girder, truss, and tied-arch steel bridges.

A series of three AISC *Modern Steel Construction* articles (23, 17, 20) review the historical consideration of redundancy and fracture critical members in steel bridges, introduce and explain the methodology of the AASHTO *Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members* (16), and introduce and explain the methodology AASHTO *Guide Specification for Internal Redundancy of Mechanically Fastened Built-Up Steel Members* (19), respectively.

Two AISC *Modern Steel Construction* articles (18 and 24) address how engineers and owners can, and have used the AASHTO *Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members* (16) to satisfy the SRM classification of certain members in accordance with the FHWA memo (4). Another AISC *Modern Steel Construction* article discusses how one owner developed a simplified method for evaluating system redundancy in two-tub girder span bridges, and added the methodology to the state's bridge design policy (25).

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