Steel Bridges: Redundancy in Design

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Steel Bridge Design Handbook: Redundancy

Table of Contents

List of Tables

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: “fracture critical members or member components are steel tension members or steel tension components of members whose failure would be expected to result in partial or full collapse of the bridge.”

The AASHTO LRFD *Bridge Design Specifications*, 7th Edition (1), defines a fracture critical member as a "component in tension whose failure is expected to result in the collapse of the bridge or the inability of the bridge to perform its function."

With multiple interpretations for "failure," "probably," "expected" and "collapse," just as for redundancy, 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 currently (2015), CVN requirements do not guarantee uniform crack tolerances among the different grades of steel, but on-going research is being performed in an effort to address this issue.

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

[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).

and a portion of the web. For the purpose of the redundancy assessment, all three elements of 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 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).

2.4 Additional Reading

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). A recent article in AISC modern Steel Construction provides additional guidance on the classification of Fracture Critical Members and Fracture Control Plans (9). Additionally, the designer is encouraged to review "*A Proposed Fracture Control Plan for New Bridges with Fracture Critical Members*" as it 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 FCP (10).

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

Some states using LRFR for rating and seeing the value of the approach of NCHRP Report 406 (11) are developing system factors for their own application using engineering judgment and analogies.

4.0 REDUNDANCY ANALYSIS

4.1 Application

4.1.1 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.

4.1.2 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.

NCHRP Report 406 (11) suggests that the required load be unfactored and consist of dead load plus two HS-20 trucks side-by-side. Using unfactored loads as suggested by the authors of NCHRP Report 406 (11) may be more reasonable if considering all design lanes loaded.

4.1.3 Internal Loads

The release of energy during the fracture should be modeled to determine if the superstructure can survive the event. In the design literature, an analogy exists for cable-stayed bridges which must be able to tolerate the loss of a cable. The Post-Tensioning Institute (PTI) suggests that the "lost" cable be replaced with an opposite force equal to 200 percent of the lost-cable force. This represents a dynamic load allowance (IM of the LRFD Specifications) or impact of 100 percent. One hundred percent impact is the extreme value and appropriate for the undamped cable-stay. Such an extreme value is not appropriate for the brittle fracture of an element of a steel member where damping is more significant. Further research and the resultant guidance is required for steel members. Research at the University of Texas suggests 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). Without definitive guidance, a conservative static analysis can be carried out using a dynamic amplification factor of 100 percent impact used as a test, realizing its extreme conservatism.

4.1.4 Analysis Requirements

The level of rigor required for a refined analysis to demonstrate that sufficient redundancy exists is currently not well defined via published procedures or guidelines. However, 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, 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.

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 lead to designers thinking of a bridge as redundant or non-redundant without varying degrees.

As demonstrated (though obtusely) by NCHRP Report 406 (11), bridges traditionally deemed redundant, multi-girder bridges, can be demonstrated to exhibit varying quantifiable degrees of redundancy based upon the number of girders and their spacing. Yet, if designers think of non-redundancy versus redundancy analogously with black versus white, the concept of enhancing redundancy equates to turning non-redundant bridges into redundant ones.

One manner to enhance the performance of non-redundant bridges is the selection of high-performance steels (in other words, ASTM A709 HPS50W, HPS70W or HPS100W) with their inherent enhanced fracture toughness. Non-redundant bridge members, those classified as such and those proven to be quasi-redundant by analysis should be fabricated from high-performance steel. 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 THE FUTURE

Work is currently under way within the steel-bridge industry, AASHTO, and the FHWA to better define redundancy criteria and fracture critical member requirements. This work includes the revisions to specifications and policies for identification, design, fabrication, and in-service inspection of those members currently classified as fracture critical. Also, research to better define the required load and analysis procedures to quantify redundancy and classify members are in process. In the meantime, engineering judgment must be used in the type selection, design, detailing, material selection, and evaluation of steel bridges to account for redundancy and assure bridge safety.

7.0 REFERENCES

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