

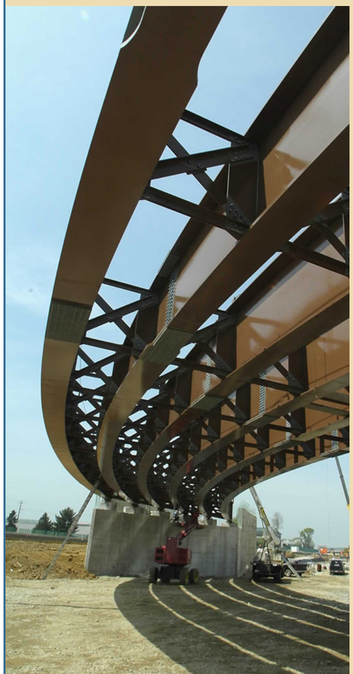


U.S. Department of Transportation
Federal Highway Administration

Steel Bridge Design Handbook

Limit States

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FOREWORD

This handbook covers a full range of topics and design examples intended to provide bridge engineers with the information needed to make knowledgeable decisions regarding the selection, design, fabrication, and construction of steel bridges. Upon completion of the latest update, the handbook is based on the Seventh Edition of the AASHTO LRFD Bridge Design Specifications. The hard and competent work of the National Steel Bridge Alliance (NSBA) and prime consultant, HDR, Inc., and their sub-consultants, in producing and maintaining this handbook is gratefully acknowledged.

The topics and design examples of the handbook are published separately for ease of use, and available for free download at the NSBA and FHWA websites: http://www.steelbridges.org, and http://www.fhwa.dot.gov/bridge, respectively.

The contributions and constructive review comments received during the preparation of the handbook from many bridge engineering processionals across the country are very much appreciated. In particular, I would like to recognize the contributions of Bryan Kulesza with ArcelorMittal, Jeff Carlson with NSBA, Shane Beabes with AECOM, Rob Connor with Purdue University, Ryan Wisch with DeLong's, Inc., Bob Cisneros with High Steel Structures, Inc., Mike Culmo with CME Associates, Inc., Mike Grubb with M.A. Grubb & Associates, LLC, Don White with Georgia Institute of Technology, Jamie Farris with Texas Department of Transportation, and Bill McEleney with NSBA.

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The previous version of this Handbook was published as FHWA-IF-12-052 and was developed to be current with the AASHTO LRFD Bridge Design Specifications, 5th Edition with the 2010 Interims. FHWA-HIF-16-002 was updated in 2015 by HDR, Inc., to be current with the AASHTO LRFD Bridge Design Specifications, 7th Edition.

16. Abstract

In the AASHTO LRFD Bridge Design Specifications, a limit state is defined as "a condition beyond which the bridge or component ceases to satisfy the provisions for which it was designed." Bridges designed using the limit-states philosophy of the LRFD Specifications must satisfy "specified limit states to achieve the objectives of constructability, safety and serviceability." These objectives are met through the strength, service, fatigue-and-fracture and extreme-event limit states. This module provides bridge engineers with the background regarding the development and use of the various limit states contained in the LRFD Specifications.

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Steel Bridge Design Handbook:
Limit States

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1.0 INTRODUCTION

1.1 General

In the *AASHTO LRFD Bridge Design Specifications, 7th Edition*, (referred to herein as the LRFD Specifications) (**Error! Reference source not found.**), a limit state is defined as "a condition beyond which the bridge or component ceases to satisfy the provisions for which it was designed." The LRFD Specifications essentially groups the traditional design criteria of the *AASHTO Standard Specifications for Highway Bridges* (referred to herein as the Standard Specifications) (**Error! Reference source not found.**) together, creating the various limit states.. The various limit states have load combinations assigned to them.

Section 1 of the LRFD Specifications briefly reviews the concept and philosophy of limit states design.

1.2 LRFD Equation

The limit states manifest themselves within the LRFD Specifications in the LRFD Equation (See Equation 1.3.2.1-1). Components and connections of a bridge are designed to satisfy the basic LRFD Equation for all specified force effects and limit-states combinations:

$$\sum_i \eta_i \gamma_i Q_i \leq \phi R_n = R_r \qquad \text{(LRFD Equation 1.3.2.1-1)}$$

where:

η_i = load modifier as defined in Equations 1.3.2.1-2 and 1.3.2.1-3 of the LRFD Specifications

γ_i = load factor

Q_i = load or force effect

ϕ = resistance factor

R_n = nominal resistance

R_r = factored resistance: ϕR_n

The LRFD Equation is, in effect, a generalized limit-states function. The left-hand side of LRFD Equation is the sum of the factored load (force) effects acting on a component; the right-hand side is the factored nominal resistance of the component for the effects. The LRFD Equation must be considered for all applicable limit state load combinations. "Considered" does not mean that a calculation is required. If it is evident that the limit-state load combination does not control, a calculation is not necessary. The designer may consider the limit-state load combination and logically dismiss it. The LRFD Equation is applicable to superstructures and substructures alike.

2.0 LIMIT STATE PHILOSOPHY

Bridges designed using the limit-states philosophy of the LRFD Specifications must satisfy "specified limit states to achieve the objectives of constructability, safety and serviceability." (See Article 1.3.1 of the LRFD Specifications.) These objectives are met through the strength, service, fatigue-and-fracture and extreme-event limit states.

Other less quantifiable design provisions address inspectability, economy and aesthetics. (See Article 2.5 of the LRFD Specifications.) However, these issues are not part of the limit-state design philosophy.

The strength and service limit states of the LRFD Specifications are calibrated, but the nature of the calibrations is quite different. The strength limit states are calibrated using the theory of structural reliability to achieve a uniform level of reliability or safety. This is achieved using the statistics available from laboratory and field experimentation for the strength limit states' associated loads and resistances. The service limit states, where the limit state functions are relatively subjective and thus not so well defined, are merely calibrated to yield member proportions comparable to those of the Standard Specifications. In addition, few experimental results, either laboratory or field based, exist for the service limit state functions.

3.0 STRENGTH LIMIT STATES

3.1 General

The strength limit states ensure strength and stability of the bridge and its components under the statistically predicted maximum loads during the 75-year life of the bridge. At the strength limit state (In other words, when the strength limit state is just satisfied, when the factored load exactly equals the factored resistance.), extensive structural distress and damage may result, but theoretically structural integrity will be maintained. The strength limit states are not based upon durability or serviceability.

Throughout the LRFD Specifications, the strength limit state functions are typically based upon load (for example; moments, shears, etc.) but in limited cases such as in the case of non-compact girders, stress is used in the strength limit state function. While contrary to LRFD philosophy where moments and shears are typically used as the nominal resistances for the strength limit states, the use of flange stress is more practical as these are the analytical results from the superposition of stresses on different sections; for example, short-term composite, long-term composite and non-composite sections. Converting the controlling flange stress to a moment would only add unnecessary complications.

For the strength limit states, the LRFD Specifications is basically a hybrid design code in that, for the most part, the force effect on the left-hand side of the LRFD Equation is based upon factored elastic structural response, while resistance on the right-hand side of the LRFD Equation is determined predominantly by applying inelastic response principles. (Again, this is not true for non-compact steel girders.) The LRFD Specifications has adopted the hybrid nature of strength design on the assumption that the inelastic component of structural performance will always remain relatively small because of non-critical redistribution of force effects. This non-criticality is assured by providing adequate redundancy and ductility of the structures, which is a general requirement for the design of bridges to the LRFD Specifications. The designer must provide adequate redundancy through design; the designer provides adequate ductility through material selection. Structural steel inherently exhibits relatively superior ductility.

3.2 Calibration of the Strength Limit States

The strength limit states are calibrated to achieve a uniform level of reliability for all bridges and components. This calibration takes the form of selecting the appropriate load and resistance factors.

Figure 1 demonstrates the application of load and resistance factors to the loads and nominal resistances used in the LRFD Equation. In the figure, load is treated as a single quantity when in fact it is the sum of the various components of load (for example, live load, dead load, etc.). As such the load factor, γ, shown in the figure is a composite load factor (in other words a weighted load factor based upon the magnitude of the various load components).

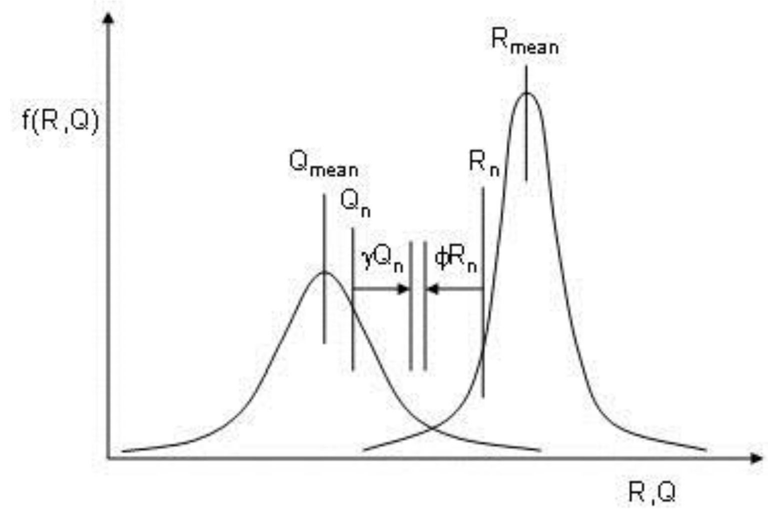

Figure 1 LRFD Equation Superimposed upon the Distributions of Load and Resistance

While the LRFD Specifications specifies that load and resistance be calculated as deterministically appearing single values, load and resistance are actually represented by multi-valued distributions as shown in the figure. The most likely values of load and resistance are shown as Q_{mean} and R_{mean}, respectively. These distributions are not apparent to the user of the LRFD Specifications. The user merely calculates the nominal values shown as Q_n and R_n. The code writers chose load factors, represented by γ, and resistance factors, represented by φ, such that when the limit state function is satisfied (in other words, $\gamma Q_n \leq \phi R_n$), the distributions of load and resistance are sufficiently apart to achieve a target level of safety.

The target level of safety or reliability cannot be shown in Figure 1, but the figure does provide the designer with an appreciation of how the deterministically appearing design process reflected probabilistic logic. The question of how far apart the distributions of Figure 1 are specified to be is answered by Figure 2.

Figure 2 graphically represents the target level of reliability. This figure shows the distribution of resistance minus load. Part of this distribution falls on the negative side of the vertical axis. This region represents the case when the calculated resistance is less than the calculated load. Points falling within this region represent a failure to satisfy the strength limit state function. It does not necessarily indicate that the bridge or component will actually fail, however, since the various design idealizations are relatively conservative.

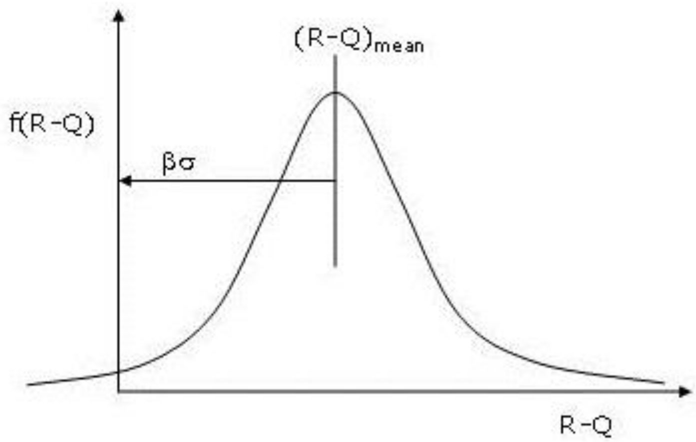

Figure 2 Graphical Representation of the Reliability Index

The area on the negative side of the vertical axis is equal to the probability of failure. Safety or reliability is defined by the number of standard deviations, σ, which the mean value of R-Q is from the origin. This number is called the reliability index and in the figure is shown as the variable, β. The greater the reliability index, β, the farther the distribution is away from the axis and the smaller the negative area or the probability of failure. The LRFD Specifications are calibrated (or in other words, the load and resistance factors chosen) such that in general the target reliability index is 3.5.

The concepts of structural reliability presented in this volume are invisible to the designer (i.e. the target reliability index is mentioned only briefly in the commentary to Sections 1 and 3 of the LRFD Specifications). Awareness of the calibration of the LRFD Specifications however leads to the designer's assurance that bridges designed to the LRFD Specifications will yield adequate and uniform reliability of safety at the strength limit states.

All five of the strength limit-state load combinations of the LRFD Specifications are potentially applicable to the design of steel bridges. The Loads and Load Combinations volume of this Steel Bridge Design Handbook discusses the applicability of each of the strength limit-state load combinations.

4.0 SERVICE LIMIT STATES

4.1 General

The service limit states ensure the durability and serviceability of the bridge and its components under typical "everyday" loads, traditionally termed service loads. The LRFD Specifications include four service limit state load combinations of which only two are applicable to steel bridges.

Currently, the service limit states for steel bridges are calibrated to result in section proportions comparable to those of the Standard Specifications.

4.2 Service I

The Service I limit-state load combination is applied in steel bridge design when the optional live-load deflection control of Article 2.5.2.6 of the LRFD Specifications is invoked by the owner. AASHTO has made this traditional limit-state optional. It is intended to control human perception of deflection but deflection control does not necessarily mitigate perception of deflection. Bridge frequency or period would be a better measure, but non-seismic bridge design does not typically include dynamic analysis. Nonetheless, the vast majority of States invoke live-load deflection control.

4.3 Service II

The Service II limit state load combination is applicable only to steel bridges. This service limit state ensures that objectionable permanent deformations due to localized yielding do not occur to impair rideability. Flexural members and slip-critical bolted connections must be checked. In fact in the case of flexural members, this limit state will govern only for compact steel girders, where the strength limit state is based upon moments in excess of the moment due to first yield where re-distribution of moments to other sections is possible. The LRFD Specifications are silent regarding the fact that it must only be checked for compact girders, but studying the Strength I and Service II limit state load combinations reveals that for girders governed by flange stresses at the strength limit state, the Strength I will always govern since its live-load load factor is greater.

The Service II limit state ensures that a girder that is allowed to plastically deform in resisting the largest load it is expected to experience in 75-years of service (i.e. Strength I, γ_{LL}=1.75), does not excessively deform under more typical loads (i.e. Service II, γ_{LL}=1.30).

Further, slip-critical bolted connections which are allowed to slip into bearing to resist the 75-year largest load under the Strength I combination must resist more typical loads, the factored Service II loads, as a friction connection. Bolted connections slipping back and forth under more typical loads are unacceptable, as fretting fatigue due to the rubbing of the faying surfaces may occur.

5.0 FATIGUE-AND-FRACTURE LIMIT STATES

5.1 General

The fatigue-and-fracture limit state is treated separately from the strength and service limit states since it represents a more severe consequence of failure than the service limit states, but not necessarily as severe as the strength limit states. Fatigue cracking is certainly more serious than loss of serviceability as unchecked fatigue cracking can lead to brittle fracture, yet many passages of trucks may be necessary to cause a critically-sized fatigue crack while only one heavy truck can lead to a strength limit state failure. The fatigue-and-fracture limit state is only applicable where the detail under consideration experiences a net applied tensile stress, as specified in Article 6.6.1.2.1 of the LRFD Specifications.

Further, the fatigue-and-fracture limit state has not been calibrated using the principles of structural reliability as the strength limit states, but has merely been moved into the LRFD Specifications from the Standard Specifications with formatting revisions. Designs satisfying the fatigue provisions of the Standard Specifications should equally satisfy the fatigue-and-fracture limit state of the LRFD Specifications. The fatigue provisions of the Standard Specifications were originally calibrated to be able to use the strength-based loads for fatigue design. In the LRFD Specifications, a specific fatigue load is specified in Article 3.6.1.4.

Figure 3 is an idealized S-N curve representing one of the AASHTO fatigue detail categories. The vertical axis is stress range, S_R, and the horizontal axis is the number of cycles to failure, N. Combinations of stress range and cycles below the curve represent safe designs. This region is not deemed “uncracked” as all welded steel details have inherent crack-like flaws, thus it is simply called the safe region. The region above the curve represents combinations of stress range and cycles that can be expected to result in cracks of length beyond an acceptable size. This region is not deemed “unsafe,” as the cracks are merely beyond the acceptable size. The curve itself represents combinations of stress range and cycles with equal fatigue damage (but on the verge of unacceptability). This demonstrates that higher stress ranges for fewer cycles will experience fatigue damage comparable to lower stress ranges for more cycles. The code writers who developed the fatigue provisions of the Standard Specifications used this fact to allow designers to use the higher strength load conditions to design for fatigue.

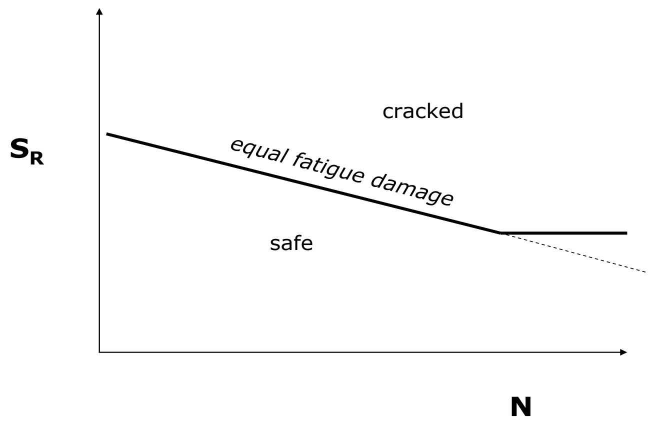

Figure 3 Idealized S-N Curve

Figure 4 graphically illustrates the relationship between the strength load of the Standard Specifications and the fatigue load of the LRFD Specifications. A simple calibration of true behavior as now represented by the LRFD Specifications to the strength load of the Standard Specifications allowed the code writers to specify that designers use a fictitiously lower number of design cycles with the higher strength load to design for the true fatigue resistance. Thus, the need to investigate a special load for fatigue design was avoided.

The problem with this approach to fatigue in the Standard Specifications is that designers did not realize that in actuality they were designing for many more actual cycles than the design cycles of the provisions. Thus, the simplification of the design effort resulted in designer confusion as the bridge experiences far more cycles than the specified number of design cycles at a fictitiously high stress range.

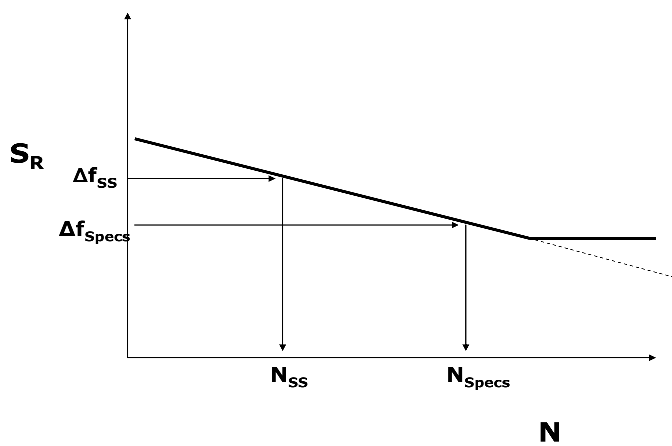

Figure 4 Relationship between the LRFD Specifications Fatigue Load and the Standard Specifications Strength and Fatigue Load

The LRFD Specifications require use of a fatigue load with a larger number of actual cycles for fatigue design. Thus, it is clear that design typically accounts for tens of millions of fatigue cycles for bridges with higher average daily truck traffic (ADTT) volumes.

The factored fatigue load (in other words, the stress range of the LRFD fatigue truck times the appropriate load factor) represents the cube-root of the sum of the cubes of the stress-range distribution that a bridge is expected to experience. This weighed average characterizes the fatigue damage due to the entire distribution through a single value of effective stress range that is assumed to occur the total number of cycles in the distribution.

5.2 Infinite Life versus Finite Life

While the fatigue-and-fracture limit state is a single limit state, it actually represents two distinct limit states: infinite fatigue life and finite fatigue life.

Equation 6.6.1.2.2-1 of the LRFD Specifications represents the general fatigue design criteria, in which the factored fatigue stress range, $\gamma(\Delta f)$, must be less than the nominal fatigue resistance, $(\Delta F)_n$.

$$\gamma(\Delta f) \leq (\Delta F)_n \qquad \text{(LRFD Equation 6.6.1.2.2-1)}$$

The load factor, γ, is dependent on whether the designer is checking for infinite fatigue life (Fatigue I load combination, $\gamma = 1.5$) or finite fatigue life (Fatigue II load combination, $\gamma = 0.75$). Which fatigue load combination to use is dependent on the detail or component being designed and the projected 75-year single lane Average Daily Truck Traffic, $(ADTT)_{SL}$. Except for fracture critical members, as stated in Article 6.6.1.2.3, when the $(ADTT)_{SL}$ is greater than the value specified in Table 6.6.1.2.3-2 of the LRFD Specifications, the component or detail should be designed for infinite fatigue life using the Fatigue I load combination. Otherwise the component or detail shall be designed for finite fatigue life using the Fatigue II load combination. The values in Table 6.6.1.2.3-2 were determined by equating infinite and finite fatigue life resistances with due regard to the difference in load factors used with Fatigue I and Fatigue II load combinations.

For the Fatigue I load combination and infinite fatigue life, Equation 6.6.1.2.5-1 defines the nominal fatigue resistance as:

$$(\Delta F)_n = (\Delta F)_{TH} \qquad \text{(LRFD Equation 6.6.1.2.5-1)}$$

For the Fatigue II load combination and finite fatigue life, Equation 6.6.1.2.5-2 defines the nominal fatigue resistance as:

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}} \qquad \text{(LRFD Equation 6.6.1.2.5-2)}$$

where:

A = an experimentally determined constant specified for each detail category, and is taken from Table 6.6.1.2.5-1 of the LRFD Specifications

N = anticipated cycles during 75-year life calculated by the designer as a function of $(ADTT)_{SL}$, and is computed per Equation 6.6.1.2.5-3 of the LRFD Specifications

$(\Delta F)_{TH}$ = constant-amplitude fatigue threshold specified for each detail category, and is taken from Table 6.6.1.2.5-3 of the LRFD Specifications

Actually, a designer can save some time by first checking whether the stress range due to the Fatigue I load combination is less than the constant-amplitude fatigue threshold (LRFD Equation 6.6.1.2.5-1). If so, the designer is finished as infinite life has been provided for the detail. Otherwise, the designer must determine the finite life resistance (LRFD Equation 6.6.1.2.5-2) by using an estimate of the single lane average daily truck traffic $(ADTT)_{SL}$ to determine N.

Satisfying the Equation 6.6.1.2.5-1 provides infinite life with no estimation of the ADTT of the 75-year life required. This can be satisfied in the majority of typical steel girder designs. Failing this, the designer can provide the necessary finite life by satisfying the second limit state given by Equation 6.6.1.2.5-2.

6.0 EXTREME-EVENT LIMIT STATES

6.1 General

The extreme-event limit states for earthquakes (Extreme-Event I), and vessel, vehicle or ice-floe collisions and certain hydraulic events (Extreme-Event II), while strength-type provisions, are very different from the strength limit states as the return period of these extreme events far exceeds the design life of the bridge. The strength limit states are calibrated for events with 75-year return periods, in other words the design life of the bridge. The extreme-event limit states of the LRFD Specifications are essentially carried over from the Standard Specifications.

These limit states represent loads or events of such great magnitude that to design for the levels of reliability or failure rates of the strength limit states would be economically prohibitive. Thus, at these limit states more risk is accepted along with more potential structural damage. The return period of the extreme-event is typically much greater than the 75-year design life of the bridge. For example, bridges are designed for earthquakes with specified return periods of as much as 2500 years.

7.0 REFERENCES

1. AASHTO. *LRFD Bridge Design Specifications, 7th Edition*. Washington, D.C.: American Association of State Highway and Transportation Officials, 2014.

2. AASHTO. *Standard Specifications for Highway Bridges, 17th Edition*. Washington, D.C.: American Association of State Highway and Transportation Officials, 2002.