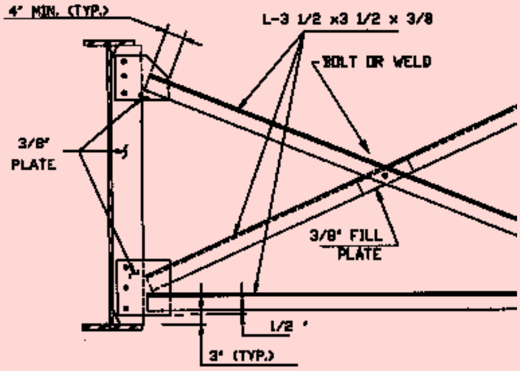

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Practical Information For The Bridge Industry

LFD vs. LRFD — What's Up With the Letter 'R' Anyway?

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Over the past three decades, different disciplines of structural engineering practice have been gradually following a trend toward design for maximum strength under factored loads. In line with this growing trend, two ultimate strength or limit states design approaches have evolved to improve steel-bridge design by providing more uniform levels of safety than are possible using the more traditional allowable or working stress design approach. The first of these limit-states design approaches (LFD) was developed in the 1960s. The second limit-states approach was introduced to bridge design in the early 1990s (LRFD) and parallels a reliability-based approach that has been available for the design of steel building frames since 1986. This article examines the basic philosophy of each of these design approaches in an attempt to elucidate the important differences and similarities between the two methods.

Load Factor Design (LFD)

In the mid 1960s, an advisory committee was formed by the American Iron and Steel Institute (AISI) to review bridge-design practices and develop new design recommendations that would yield a more consistent and effective use of steel in highway bridges. The efforts of this committee resulted in the publication in 1969 of the "Tentative Criteria for Load Factor Design of Steel Highway Bridges" or AISI Bulletin No. 15. After a year of study and some modifications, the tentative criteria were adopted by AASHTO in 1970 as an alternate method and published in the 1971 AASHTO Interim Specifications. Since that time, the use of Load Factor Design (LFD) for steel bridges has continued to increase; it is estimated that LFD is being used, either all or in part, by at least 40 State DOTs.

In recognition of the inherent ductility and reserve strength of steel and an improved understanding of the structural behavior of steel bridges, LFD was developed as a method for proportioning structural members for multiples of the design loads to satisfy specified structural performance requirements. A structural performance requirement indicates what is required from a bridge at a given load level. With properly selected multiples of the load, LFD can ensure a design allowing:

1. the expected number of passages of ordinary vehicles during the life of the bridge,
2. occasional passages of overload vehicles without permanent damage, and
3. in an extreme emergency, very few passages of exceptionally heavy vehicles. The underlying philosophy is to ensure both safe and serviceable performance, while at the same time providing a consistent live-load carrying capacity for all bridges on the system. In Allowable Stress Design (ASD), attention is focused on performance under service conditions only. LFD considers performance in a broader context in that it deals with serviceability and safety separately.

LFD recognizes three basic and distinct load levels — Service Load, Overload, and Maximum Load. Service Load represents ordinary vehicles that may operate on the highways without special permit. For design purposes, Service Load is represented in AASHTO as the sum of the dead loads *D* and the standard live loads plus impact *L+I*. The primary structural performance requirements at Service Load are to provide adequate fatigue life and to control live-load deflections and concrete deck cracking. If the design is adequate for fatigue and deflection under normal traffic loads, the absolute maximum stress due to these loads — used for design in ASD — is of little concern. Overload is defined as the maximum live load that can be allowed on the structure on infrequent occasions. Infrequent implies that the stresses caused by these loads are not subject to fatigue requirements. The single structural performance requirement at Overload is control of permanent deformations caused by localized yielding and connection slip to ensure good riding quality. For design purposes, Overload is taken as the load factor β_D times the dead load plus the load factor β_L times the live load plus impact. The load factor β_D is to allow for possible increases in the dead load and is usually taken as 1.0 on the assumption that the designer will allow for future additions to the dead load. The load factor β_L allows for possible overloads and is usually taken as 5/3 or 1.67 for live loadings

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greater than or equal to AASHTO H20 loading.

Service Load and Overload address serviceability requirements. To ensure adequate safety, the Maximum Load level is introduced. The single structural performance requirement at Maximum Load is that the bridge be able to safely resist the load. In LFD, this performance requirement is satisfied at Maximum Load through the following relationship:

$$\phi\,(\text{Maximum strength}) \geq \gamma\,[\beta_D D + \beta_L(L+I)]$$

The load factor γ recognizes uncertainties that exist in the loads and load analysis. The resistance factor ϕ represents several sources of uncertainty such as variations in materials and section size, variations in workmanship, and approximations made in strength calculations. In LFD, a value of ϕ equal to 1.0 was selected for members in flexure and shear since the maximum strength equations in LFD for flexure and shear represent the lower bounds of the test data. Lower values of ϕ are specified for column and connection design because of the greater consequences of failure of these elements.

For flexure and shear design in LFD, ϕ is shifted to the right-hand side of the preceding equation. The resulting γ/ϕ term, together with the load factors β_D and β_L, establishes the margin of safety inherent in LFD for flexural members. The value of the γ/ϕ term was established based on past experience using ASD practice as a guide. The safety of the ASD approach has been well established, but the live-load margin of safety is known to vary with the span because a single safety factor of 1/0.55 or 1.82 is applied to both dead and live loads in ASD. The minimum margin of safety in ASD is associated with short spans. Therefore, it was decided that in order to provide both safe and economical designs in LFD, a value of γ/ϕ would be selected that would yield the same steel section by ASD and LFD for a short simple-span bridge. It was determined that a value of γ/ϕ equal to 1.3 would yield about the same minimum level of safety by ASD and LFD for an approximately 45-foot long noncomposite simple-span bridge. As the span length increases, the live-load margin of safety increases slightly in LFD since different load factors are applied to the dead and live loads, while the margin of safety remains nearly constant in ASD. Since ϕ is equal to 1.0 for flexural members, ϕ does not explicitly appear in the strength equations for flexural members in LFD.

Load and Resistance Factor Design (LRFD)

In 1993, AASHTO adopted the Load and Resistance Factor Design (LRFD) specifications for bridge design, which were developed under NCHRP Project 12-33. The LRFD specifications were approved by AASHTO for use as alternative specifications to the AASHTO *Standard Specifications for Highway Bridges*, which contain both the ASD and LFD provisions. The LRFD specifications were developed in response to a high level of interest amongst the members of the AASHTO Subcommittee on Bridges and Structures in developing updated bridge specifications along with an accompanying commentary. The goal was to develop more comprehensive specifications that would eliminate any gaps and inconsistencies in the Standard Specifications, incorporate the latest in bridge research, and achieve more uniform margins of safety or reliability across a wide variety of structures. The decision was then made to develop these new specifications in a probability based LRFD format.

In the LRFD method, load and resistance factors are determined through statistical studies of the variability of loads and resistances, which is considered to be a more realistic approach than the application of judgment-based deterministic factors. In the calibration process, load and resistance factors are calculated to provide a target level of reliability for a wide variety of structure types and configurations.

The reliability theory on which the LRFD method is based has been well-documented elsewhere and will not be expanded on in depth here. Essentially, the level of reliability is measured through the use of a reliability index. Although not strictly correct, the reliability index can be thought of in simple terms as a statistical indicator of the fraction of times that a particular design criteria will be met or exceeded over the design life of the structure. For example, according to this simple definition, a reliability index of 3.5 indicates that a particular design criteria may be exceeded in 2 out of 10,000 cases. The reliability index is currently based only on the design of individual components of the bridge and does not represent a system reliability, which will typically be higher. In the calibration, a target reliability index is selected to provide a minimum acceptable margin of safety. Following the lead of LFD, past practice was used as a guide in establishing the target reliability index. The primary goal during the development of the LRFD specifications was not to cause a radical departure from the basic level of safety inherent in the current highway system. Rather, the primary objective was ***to increase the uniformity of the margin of safety across the various structure types that are utilized on the system***.

Reliability indices were calculated for a number of sample bridge designs extracted from plans supplied by State DOTs using:

1. the AASHTO HS live loading and lateral distribution factors given in the Standard

Specifications in conjunction with the current load and resistance factors specified in LFD, and

2. a new HL live-load model and new lateral distribution factors introduced in the AASHTO LRFD specifications in conjunction with new load and resistance factors determined from the calibration process.

The LFD designs were clustered around a reliability index of 3.5 with a large amount of scatter. The use of the selected LRFD load and resistance factors in conjunction with the new live-load model and distribution factors again resulted in a clustering of the indices around the target value of 3.5, but with a greatly reduced amount of scatter indicating the attainment of a more uniform reliability than provided by LFD procedures.

Similarities and Differences

Both LFD and LRFD are limit-states design approaches that strive to achieve more uniform live-load margins of safety for steel bridges, while still meeting established structural performance criteria for serviceability and safety. While traditional ASD considers performance under service conditions only, LFD and LRFD treat serviceability and safety separately. In LFD, limit-state criteria are specified to satisfy performance criteria at three distinct load levels: Service Load, Overload and Maximum Load. In LRFD, similar limit-state criteria are specified to satisfy similar performance objectives at four distinct limit states: the Service Limit State, the Fatigue and Fracture Limit State, the Strength Limit State and the Extreme Event Limit State.

Load and resistance factors are specified in each method to account for various sources of uncertainty. In each method, lower load factors are applied to the dead loads. The values of the load and resistance factors are different in the two methods. Also, in LRFD, a single load factor is applied to each load component rather than applying separate β_D and β_L factors. The resistance factors are always explicitly applied in LRFD, while the resistance factor of 1.0 for flexural members is implicitly applied in LFD. In the LRFD specifications, an attempt is also made to treat redundancy, ductility and importance more explicitly in the design by applying subjective modifiers to the load side of the equation.

In the LRFD specifications, the load and resistance factors are determined from a probability-based calibration process to achieve a more uniform reliability index for the various components of the system than LFD. In the LFD specifications, the load and resistance factors are determined using a simpler calibration process based on judgment and experience to achieve a more uniform live-load carrying capacity than is possible using ASD.

While more specific differences between the AASHTO LRFD and LFD specifications — such as differences in the live-load models, impact factors, lateral distribution factors, load combinations and the design for fatigue — and their effects on the overall design could be discussed and debated at some length, the purpose of this discussion is to emphasize that the differences in the basic underlying philosophies of the two methods are not all that great. The primary difference in philosophy boils down to the procedures used to perform the calibration in order to provide the minimum desired level of safety. LRFD calibration procedures allow for an improvement in the uniformity of the margin of safety across the system and also provide a more realistic and rational framework for performing future calibrations as more is learned about loads and material resistances. While probabilistic theories are employed in the LRFD calibration process, it should be kept in mind that the user of the LRFD specification provisions need not be well-grounded in probability theory in order to apply the provisions. Most of the LRFD resistance equations for the design of steel-bridge components are in fact very similar to the resistance equations given in the current LFD provisions. Thus, designers who are proficient with LFD procedures for steel bridges should have little trouble converting to LRFD, once some level of familiarity is attained with the revisions to the load side of the basic design equation that are presented in the LRFD specifications.

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