Impact of Load and Resistance Factor Design Specifications on Short- to Medium-Span Steel Bridges

Dennis R. Mertz, *University of Delaware*
John M. Kulicki, *Modjeski and Masters, Inc.*

In 1993, AASHTO adopted the *Load and Resistance Factor Design Bridge Design Specifications* (LRFD specifications) as an alternative to the *Standard Specifications for Highway Bridges* (standard specifications). Its adoption raises many questions regarding the specification's impact on the resultant bridge members' proportions and the design process itself. The implication of the provisions of the LRFD specifications on the design of steel highway bridges relative to those of the load factor design (LFD) provisions of the standard specifications is investigated through a dissection of the specifications into the load and resistance sides of the LRFD equation. A simple design example illustrates the impact of the LRFD specifications. Finally, the design process and effort required to apply each set of provisions, LRFD and LFD, are discussed on the basis of the example. Through the dissection of the LRFD specifications into the load and resistance sides of the LRFD equation and the discussion of the design process, the general impact of the specifications on the economy of short- to medium-span steel bridges and on the design community in general is assessed.

At the spring meeting of 1993 the AASHTO Subcommittee on Bridges and Structures adopted the *Load and Resistance Factor Bridge Design Specifications* (*1*) (LRFD specifications) as an equal alternative to the *Standard Specifications for Highway Bridges* (*2*) (standard specifications).

The LRFD specifications are the product of NCHRP Project 12-33, Development of Comprehensive Bridge Design Specifications and Commentary, a 5-year, 50-person research effort led by Modjeski and Masters, Inc. The intent of NCHRP Project 12-33 was to develop a structural reliability-based, technically state-of-the-art bridge design code to replace the standard specifications.

Bridges designed to the standard specifications were not deemed to be performing unsatisfactorily. The provisions of the standard specifications, however, were deemed to contain gaps, in which coverage is missing, and inconsistencies, in which internal conflicts or contradictions in wording or philosophy exist. Bridges designed to the code developed by NCHRP Project 12-33 are not to be specifically stronger or weaker but more rationally designed.

Furthermore, NCHRP Project 12-33 was charged with integrating existing research findings into the code but not to develop new findings, because funding was not adequate to do both.

Reliability-based design methodologies seek to account for the statistical variations in loads and resistances in the design process. The theory of structural reliability can be used to directly compute the level of safety, quantified as the reliability index, inherent to a given set of nominal loads, the designer's estimate of the nominal resistance of the component being designed, and statistical data quantifying the variation of load and resistance. Thus, it is possible to vary the nominal resistance to achieve the level of safety for the component (or system) that is acceptable to society.

Alternatively, the process can be worked backwards to calibrate a combination of the load and resistance factors required to achieve a general targeted reliability index. In this manner the combination of load and resistance factors was derived for the LRFD specifications. The design process then proceeds analogously to load factor design (LFD) in the standard specifications, and the designer needs to know little, if anything, of structural reliability theory. Bridges designed to the LRFD specifications are not to be specifically stronger or weaker than those designed to the standard specifications but more uniformly reliable.

Reliability indexes inherent in the standard specifications were calculated for existing bridges and a supplemental set of virtual bridges studied during the calibration of load and resistance factors. The resultant ranges of the reliability indexes for both moment and shear are given in Figure 1. The wide range of values indicates the inconsistent levels of safety inherent in the standard specifications.

Reliability indexes were recalculated for each of the bridges on the basis of the provisions of the LRFD specifications. The resulting ranges of the reliability index are given in Figure 2.

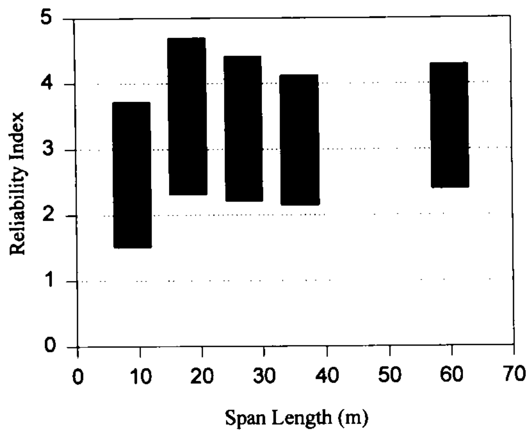

FIGURE 1 Reliability indexes by standard specifications.

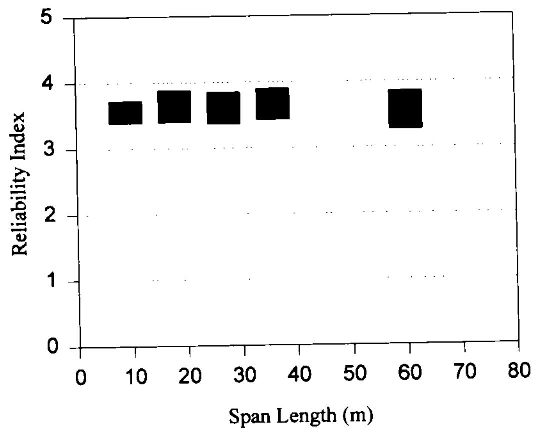

FIGURE 2 Reliability indexes by LRFD specifications.

Figure 1 suggests that a reliability index of 3.5 is indicative of past practice. Hence, this value was selected as a target for the calibration of the LRFD specifications.

Figure 2 suggests that the LRFD specifications achieve this level of reliability or safety consistently, with considerable improvement in the clustering of reliability indexes around the target value. This is a direct result of the integration of the load factors, resistance factors, accurate load models, and suitable resistance models of the LRFD specifications.

NCHRP Project 12-33 was not charged with making a wholesale readjustment of the inherent safety in the nation's highway system but to achieve a more uniform level of safety. The resultant uniformity of structural safety is evident in Figure 2.

The provisions of the LRFD specifications can be subdivided according to the general LRFD equation given in Article 1.3.2.1 of these specifications:

$$\eta \Sigma_i \gamma_i Q_i \leq \phi R_n \quad (1)$$

where

η = a factor relating to ductility, redundancy, and operational importance;
γ_i = load factor;
Q_i = force effect;
ϕ = resistance factor; and
R_n = nominal resistance.

The discussion of the impact of the LRFD specifications on the design of steel bridges is divided into two parts:

- Those provisions relating to the load side of the LRFD equation (the left-hand side), and

• Those relating to the resistance side of the LRFD equation (the right-hand side).

LOAD SIDE OF LRFD EQUATION

The provisions relating to the load side of the LRFD equation that have an impact on the design of steel bridges are those relating to

- The live-load model,
- The dynamic load allowance,
- Lateral live-load distribution, and
- The load factors.

Live-Load Model

As a result of the evolution of the vehicles traveling on the nation's highways, the HS 20 loading of the standard specifications, originally developed in 1944, no longer bears a uniform relationship to these vehicles. In developing a new design specification providing more uniform and consistent safety for bridges, a new live load is necessary.

Figure 3 compares the various moment-type force effects for span lengths of 6 to 45 m generated by a set of vehicles currently using the nation's highways with those of the HS 20 live-load model. This set of vehicles was determined to be that which routinely produces the most severe moments and shears. These vehicles represent state grandfather exceptions to the federal axle weight limits or gross vehicle weight limits and travel without special permits (these are referred to here as grandfather vehicles). The ratio of the force effect from the envelope of grandfather vehicles divided by the corresponding force effect from the HS 20 model is plotted on the verticle axis versus the span length on the horizontal axis. Thus, a complete match of force effects, suggesting that the HS 20 model is an accurate and representative model of the loads of grandfather vehicles, would be indicated by a horizontal line passing through the vertical axis at a value of 1.0. Corresponding information for the shear force effects has also been developed. These comparisons illustrate that the HS 20 model is not representative of current loads on the nation's highways.

A combination of the various elements of the live-load model of the standard specifications adequately represents the current loads on the nation's highways and was chosen as the new notional design live load in the LRFD specifications. These elements are

• The HS 20 truck, or
• A slight variation of the Alternate Military Loading in combination with
• The HS 20 uniform load without the concentrated loads.

For the case of negative moment over a pier a special provision is included for two closely spaced HS 20 trucks in one lane. The live-load model is specified in Article 3.6.1.2 of the LRFD specifications and is termed the HL 93 loading.

A summary of the moment-type force effect ratios for the grandfather vehicles divided by that for the new LRFD live-load model is shown in Figure 4. The results for moment-type force effects are tightly clustered and form bands of data that are essentially horizontal. This tight clustering of data results in a comparison of shear-type force effects also, indicating that one notional

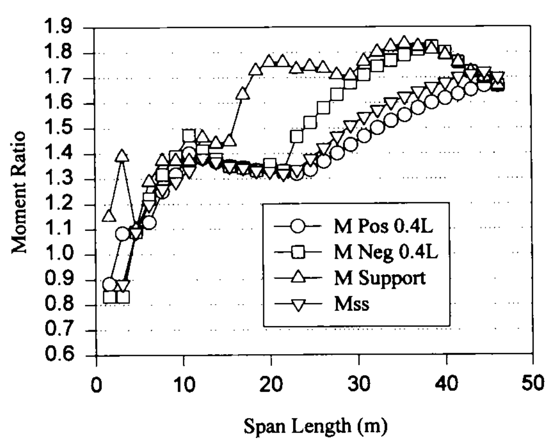

FIGURE 3 Moment ratios comparing grandfather vehicle loads versus HS 20 loads.

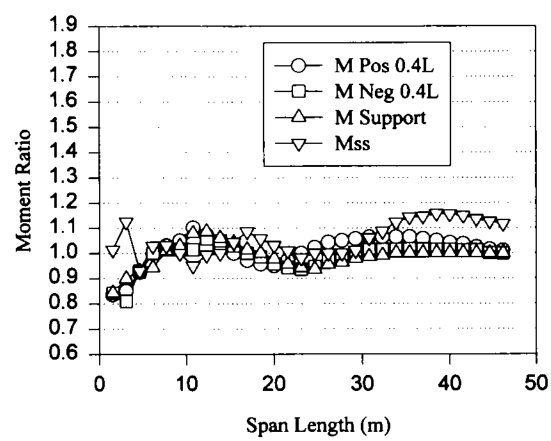

FIGURE 4 Moment ratios comparing grandfather vehicle loads versus HL 93 loads.

model can be developed for both moment and shear. The fact that the data are essentially horizontal indicates that both the model and the load factor applied to live load are independent of span length. The tight clustering of all the data for all force effects further indicates that a single live-load factor will also be sufficient for both moment and shear.

A comparison of Figures 3 and 4 reveals the change in live-load requirements inherent in the implementation of the LRFD specifications. Since, for the most part, the curves of Figure 4 all fall below those of Figure 3, the unfactored live-load requirements of the LRFD specifications are greater than those of the standard specifications.

In the special case of the fatigue limit state, a single HS 20 truck is specified as the live load to be considered on the bridge (i.e., multiple lanes of loading are not considered). Neither the tandem nor the design lane load is applied in this limit state. Furthermore, the distance between the two large axles on the truck is a constant 9 m rather than the variable spacing used for strength design. The reason for this simplification is that the majority, and therefore the statistically significant number, of vehicles on the road are relatively long 3S-2 configurations. It would be unduly severe to assume that all of the fatigue stress ranges result from the smaller numbers of relatively short trucks. In addition to the size of the vehicle, the number of single-lane occurrences is necessary to predict the number of cycles of fatigue loading. This is referred to in the LRFD specifications as a single-lane average daily truck traffic. In the absence of site-specific information, this may be related to the typically tabulated average daily truck traffic volume through a factor ranging from 100 percent for a single lane available to truck traffic to as low as 80 percent when three or more lanes are available.

Calculated stress ranges by the LRFD specifications are lower than those by the standard specifications because of the reduced fatigue live-load model, but the numbers of cycles by the LRFD specifications are much increased. The net result is that both specifications are essentially equivalent. The cumulative fatigue damage due to the lower stress range for a larger number of cycles according to the LRFD specifications is essentially equal to that due to the higher stress range for a smaller number of cycles according to the standard specifications.

Since multilane loading and lane loading are not considered for fatigue design, the fatigue limit state controls less often in the LRFD specifications than in the standard specifications. This is not a liberalization of the fatigue requirements but an acknowledgment that the requirements of the standard specifications are not rational.

Dynamic Load Allowance

In the standard specifications the specified amplification of static load to replicate dynamic response is termed *impact* and is a function of span length alone.

The LRFD specifications terms the amplification the *dynamic load allowance* and in general simply requires that a constant amplification of 33 percent be applied to the design vehicle only. Initially, this may seem like a step backwards, but research suggests that this simple approach is warranted.

The simple approach of the LRFD specifications is based on a study of dynamic effects reported by Nowak (*3*). In that study the dynamic effect was quantified by investigating deflection. The study concludes that

- As the gross vehicle weight increases, naturally the static deflection increases,
- As the gross vehicle weight increases, the dynamic amplification as a percentage of the gross vehicle weight decreases, yet
- Throughout the range of gross vehicle weights, the magnitude of the increment between static and dynamic deflections remains constant.

The study revealed that the most influential factor on dynamic load allowance is roadway surface roughness.

Since roadway surface roughness during the service life of the bridge is beyond the control of the designer, it is foolhardy to specify a precise value of dynamic load allowance including the functionalities beyond surface roughness.

In consideration of that study the general dynamic load allowance in the LRFD specifications is taken as one-third of the weight of the design truck or tandem, with no dynamic load allowance applied to the design lane load. Nowak's study (*3*) indicates that the dynamic load allowance for a 325-kN truck is about one-fourth. Since the specified superposition of the design truck and lane loads is intended to represent a truck with a weight greater than 325 kN, the dynamic load allowance is taken as one-third of the weight of the design truck. Furthermore, as the span length of the bridge increases, the lane load models not only a single truck heavier than the design truck but also the presence of other traffic around the design truck. The study indicates that increased traffic decreases the dynamic effects; therefore, no dynamic load allowance is applied to the design lane portion of the live-load model.

For comparisons with the standard specifications, the dynamic load allowance should be considered one-fourth, since the one-third on the truck is to replicate one-fourth on the whole live load, as discussed earlier. Therefore, for the strength and service limit states the LRFD specifications require

- Less impact or dynamic load allowance for span lengths less than 22.9 m, and
- More impact for span lengths greater than 22.9 m.

For the fatigue limit state the LRFD specifications require 15 percent impact, a more average value, resulting in

- Less impact or dynamic load allowance for span lengths less than 63.5 m, and
- More impact for span lengths greater than 63.5 m.

Lateral Live-Load Distribution

New lateral live-load distribution factors have been developed by Imbsen and Associates, Inc., under NCHRP Project 12-26 (*4*). The distribution factors developed in that study are reported to be typically accurate to within 5 percent of the results obtained by more refined methods of analysis. These new distribution factors form the basis for those included in the LRFD specifications and are included in the recently published AASHTO *Guide Specifications for Distribution of Loads for Highway Bridges* (*5*).

Their study (*4*) reveals that beam spacing is the most significant parameter. However, span length, longitudinal stiffness, and transverse stiffness also affect the load distribution factor. Ignoring the effects of parameters other than beam spacing can result in highly inaccurate results. Even when they are properly applied, the simple distribution factors of the standard specifications can result in both highly unconservative and highly conservative designs.

Thus, new load distribution factors are provided for the design of bridges. These factors are more realistic than the traditional values of the standard specifications. The distribution factors in the LRFD specifications yield less moment per girder for larger girder spacings, say greater than 2.75 m, for all span lengths and more moment per girder for very short span lengths, say less than 9 m, when the girder spacing is less than 2.5 m.

Load Factors

The basic strength load combination in the LRFD specifications, called Strength Load Combination I, is

$$\Sigma_i \ \gamma_i Q_i \ \ 1.25DC + 1.50DW + 1.75LL \tag{2}$$

where

DC = dead load due to components and attachments,
DW = dead load due to wearing surfaces and utilities, and
LL = live load.

In the standard specifications, the comparable load combination appears as

$$\Sigma_i \ \gamma_i Q_i = 1.3D + 2.17LL \tag{3}$$

where D is equal to all components of the dead load.

The differences in dead-load factor are insignificant, because the slight decrease in load factor for components and attachments compensates for the larger increase in the load factor for wearing surfaces and utilities.

The change in the live-load factor is much more significant, representing an across-the-board decrease of 24 percent for the LRFD specifications.

Resistance Side of LRFD Equation

Unlike other sections of the LRFD specifications such as that dealing with concrete structures, Section 5 (Concrete Structures), the section dealing with steel structures, Section 6 (Steel Structures), offers no major changes. For the most part this is due to the steel industry's involvement in the AASHTO Bridge Subcommittee's Technical Committee system. This involvement ensures that recent research findings quickly get adopted as interim changes to the standard specifications.

Nonetheless, new provisions that do not appear in the standard specifications are included in the LRFD specifications. They relate to

- Replacement of deflection limitations with flange-stress control of permanent deformation at overload,
- Removal of the arbitrary 7.6-m diaphragm spacing,
- Inclusion of reduction factor on tensile resistance to account for shear lag,
- New equations for combined flexure and compressive resistance,
- Updating of provisions for flexural resistance of I-sections,
- Updating of shear resistance provisions,
- New constructability provisions, and
- Inelastic analysis procedures.

The changes represented by this list, other than the inclusion of resistance factors, evolve from the LFD provisions of the standard specifications and do not consist of the implementation of new design philosophies or methodologies. With time these changes need to be incorporated into the standard specifications. As such their impact should not be considered as part of the implementation of the LRFD methodology.

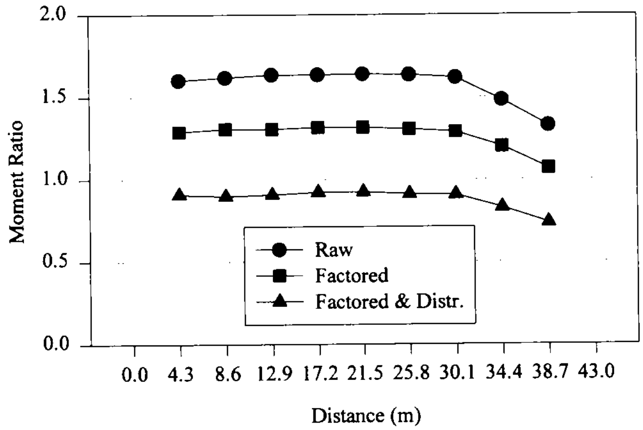

FIGURE 5 Positive live-load moment ratios.

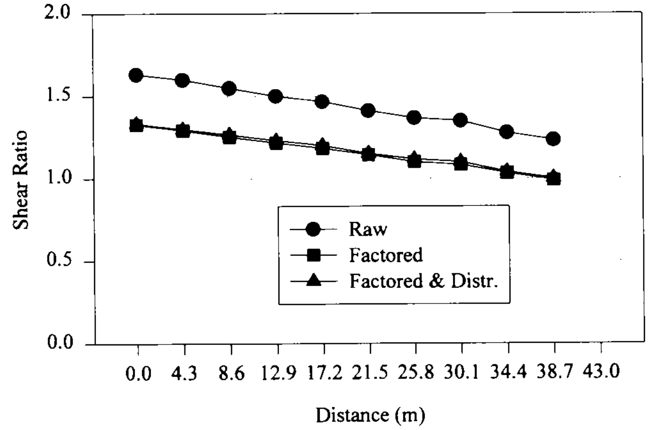

FIGURE 7 Positive live-load shear ratios.

Resistance Factors

The explicit specification of resistance factors does represent a part of the implementation of the LRFD methodology. The resistance factors specified in the LRFD specifications for flexure and shear of steel members are both specified as 1.0. Although the LFD provisions of the standard specifications do not contain resistance factors or capacity reduction factors, since the specified values are unity, these represent no net change between the two specifications.

Conclusions

If it is accepted that the resistance side of the LRFD equation for steel bridges represents an evolution of the LFD provisions of the standard specifications and not new design methodologies, the impact of the LRFD specifications on short- to medium-span steel bridges can be assessed on the basis of mainly the load side of the LRFD equation.

A typical design example illustrates the impacts that changes on the load side of the LRFD equation have on the design of steel bridges. The results shown in Figures 5 through 8 are based on a two-span unit of equal 43-m span lengths. The girder spacing is 3.7 m. An interior girder loaded with two or more lanes of live load is chosen for comparison.

The figures show comparisons between the load provisions of the LRFD specifications and those of the standard specifications. The vertical axes of the figures represent the ratio of moments or shears obtained by applying the provisions of the LRFD specifications to those obtained by applying the standard specifications. The horizontal axes represent longitudinal distance along the girder. Datum points representing values at the ten 0.1 points along the girder have been plotted. The moments or shears, including the dynamic load allowance or impact, indicated as raw data are unfac-

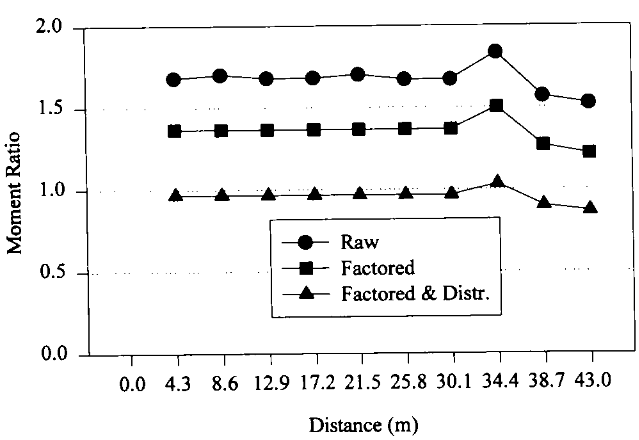

FIGURE 6 Negative live-load moment ratios.

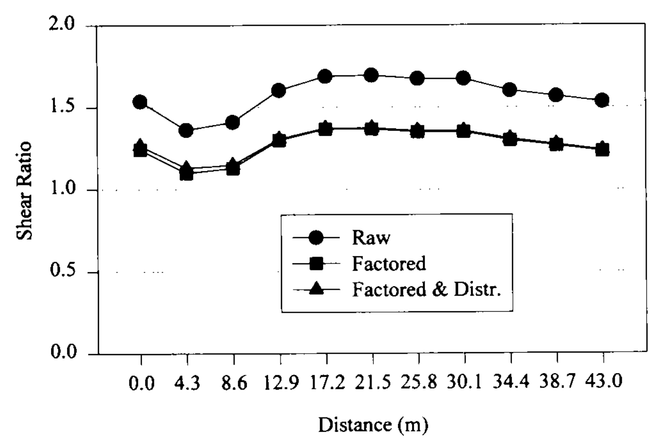

FIGURE 8 Negative live-load shear ratios.

tored. The factored data are the product of the raw data and the appropriate load factors. The factored and distributed data are the product of the factored data and the appropriate distribution factors. Since the resistance factors for moment and shear of steel members are both unity, this final ratio, factored and distributed data, represents the change in live-load demand required on the basis of the implementation of the LRFD specifications.

Figure 5 shows the positive live-load moment ratios. The impact of combining the truck and lane loads in the HL 93 loading of the LRFD specifications in comparison with considering them independently according to the standard specifications is indicated by the datum points representing unfactored moment and is labeled raw data. Most of the live-load moments at the various 0.1 points according to the LRFD specifications are approximately 1.6 times the moments according to the standard specifications. Once the unfactored moments are factored by the respective live-load factors, 1.75 for the LRFD specifications and 2.17 for the standard specifications, the factored moments are somewhat closer, with typical ratios of about 1.3. Distribution of these factored moments per lane to the girders brings the factored moments per girder to approximately the same level in the two specifications, with the demand according to the LRFD specification being approximately 90 percent of that according to the standard specifications for this steel bridge example. The refined distribution factor of the LRFD specification is significantly lower than the distribution factor of the standard specifications. The close comparison in factored and distributed moments is very dependent on the individual structure type. For the same span length and girder spacing configuration, a prestressed concrete girder example resulted in a demand of 110 percent of that of the standard specifications.

Figure 6 shows similar ratios of negative live-load moment, with the spike in raw data at the 0.8 point indicating the place where the two closely spaced trucks in one lane of the HL 93 loading began to govern negative moment.

Figures 7 and 8 show positive and negative live-load shear ratios, respectively. The refined distribution factor for shear according to the LRFD specifications is relatively larger than that for bending moment in this example. The effect of this can be seen in Figures 7 and 8. There is almost no difference between the ratios of the factored shears and those of the factored and distributed shears according to the LRFD specifications and the standard specifications; in others words the distribution factors were essentially the same for each specification. The increased live load of the LRFD specifications results in more shear demand than the shear demand from the live load of the standard specifications for this steel bridge example.

The development of this design example illustrated the relative effort required in the application of each specification. This design example suggests that a small amount of increased design effort is required when applying the LRFD specifications. The calculation of the live-load distribution factor is more complex. Although the HL 93 load is merely a superposition of existing HS 20 loads, additional bookkeeping is required. The special provision for two closely spaced trucks for negative moment near a support definitely requires additional effort. Furthermore, the fatigue load with its fixed rear axle spacing requires additional bookkeeping. The increased effort, however, brings with it increased designer confidence in the relative precision of the calculations.

As was intended by the subcommittee, the LRFD specifications do not alter the basic safety or reliability inherent in the standard specifications. As Figure 1 indicated, however, the inherent safety is not uniform. The implementation of the LRFD specifications provides more uniform safety, as indicated in Figure 2. Some bridges require slightly additional strength, such as very short spans with close girder spacings, whereas others require less strength. In general, the impact on short- to medium-span steel superstructures is minimal.

ACKNOWLEDGMENTS

AASHTO, in cooperation with the FHWA, sponsored the effort to develop a new probability-based bridge design code, which became the *Load and Resistance Factor Bridge Design Specifications*, 1st ed. TRB of the National Research Council administered the effort, conducted in NCHRP.

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