CHAPTER 1

BRIDGE DESIGN SPECIFICATIONS

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1.1 INTRODUCTION

The main purpose of bridge design specifications is to ensure bridge safety such that minimum resistances or capacities, in terms of strength, stiffness, and stability of each bridge component and the whole bridge structural system exceed the potential maximum demands or force effects due to various loads during its design life. The first national standard for highway bridge design and construction in the United States, "*Standard Specifications for Highway Bridges and Incidental Structures*" was published by the American Association of State Highway Officials (AASHO) in 1927, the predecessor to the present day American Association of State Highway and Transportation Officials (AASHTO). Design theory and practice have evolved significantly due to increased understanding of structural behavior and loading phenomena gained through research. Before 1970, the sole design philosophy was allowable stress design (ASD). Beginning in early 1970, a new design philosophy referred to as load factor design (LFD) was introduced. The latest version entitled "*Standard Specifications for Highway Bridges*" (Standard Specifications) is the final 17th Edition (AASHTO, 2002) and includes both ASD and LFD philosophies. Reliability-based and probability-based load and resistance factor design (LRFD) philosophy was first adopted in "*AASHTO LRFD Bridge Design Specifications*" (LRFD Specifications) (AASHTO 1994) and continues to this day. The LRFD Specifications had not been widely used until AASHTO discontinued updating its ASD and LRD Standard Specifications in 2003.

ASD, LFD, and LRFD are distinct design philosophies and methods. ASD does not recognize that some loads are more variable than others. LFD brings the major philosophical change of recognizing that some loads are more accurately represented than others. LRFD is a logical extension of the LFD procedure and provides a mechanism to select the load and resistance factors more systematically and rationally with uniform margins of safety.

The LRFD Specifications with California Amendments have been implemented for all new bridge designs in the State of California since 2006. The latest version of the *California Amendments* to *AASHTO LRFD Bridge Design Specifications* – 8th Edition (AASHTO, 2017) was published in 2019 (Caltrans, 2019). This chapter will briefly describe the general concepts and backgrounds of ASD and LFD but primarily discuss the LRFD philosophy. A detailed discussion may be found in Kulicki (2014).

It should be pointed out that highway bridges shall also satisfy requirements specified in Caltrans *Structure Technical Policies* (*STPs*) (Caltrans, 2021). STPs supplement design specifications by addressing areas not covered; expanding requirements based on successful past practices; and implementing more recent innovations.

1.2 ALLOWABLE STRESS DESIGN (ASD)

ASD, also known as working stress design (WSD) or service load design, is based on the concept that the maximum applied stress in a structural component does not exceed the certain allowable stress under normal services or working conditions. The general ASD design equation can be expressed as:

$$\sum Q_i \leq \frac{R_n}{FS} \tag{1.2-1}$$

where Q_i is a load effect; R_n is the nominal resistance and FS is a factor of safety.

The left side of Equation 1.2-1 represents working stress or service load effects. The right side of Equation 1.2-1 means allowable stress. The load effect Q_i is obtained by an elastic structural analysis for a specified load, while the allowable stress (R_n/FS) is the nominal limiting stress such as yielding, instability, or fracture divided by a safety factor. The magnitude of a factor of safety is primarily based on past experience and engineering judgment. For example, the factors of safety for axial tension and axial compression in structural steel are 1.82 and 2.12, respectively in the Standard Specifications (AASHTO, 2002).

The ASD treats each load in a given load combination as equal from the viewpoint of statistical variability. It does not consider the probability of both a higher than expected load and a lower than expected strength occurring simultaneously. They are both taken care of by the factor of safety. Although there are some drawbacks to ASD, bridges designed based on ASD have served very well with safety inherent in the system.

1.3 LOAD FACTOR DESIGN (LFD)

LFD, also known as ultimate or strength design, mainly recognizes that the live load such as vehicular loads and wind forces, in particular, is more variable than the dead load. This concept is achieved by using different multipliers, i.e., load factors on dead and live loads. The general LFD design equation can be expressed as:

$$\sum \gamma_i Q_i \leq \phi R_n \tag{1.3-1}$$

where γ_i is a load factor and ϕ is the strength reduction factor.

The nominal resistance is usually based on either loss of stability of a component or inelastic cross-sectional strength. In some cases, the resistance is reduced by a “strength reduction factor”, ϕ, which is based on the possibility that a component may be undersized, the material may be under strength, or the method of calculation may be less accurate. It should be pointed out, however, that the probability of a joint occurrence of higher than expected loads and less than the expected resistance is not considered.

One major disadvantage of LFD is that the load factors and resistance factors were not calibrated in a manner that takes into account the statistical variability of design parameters in nature, although the calibration for a simple span of a 40-foot steel girder was performed by Vincent (1969).

1.4 LIMIT STATES

The design specifications are written to establish an acceptable level of safety for different loading cases. “Limit states” is terminology for treating safety issues in modern specifications. A limit state is a condition beyond which the bridge or component ceases to satisfy the provisions for which it was designed. Limit states may be expressed by functional requirements such as limiting deformation, stress, or cracks, or by safety requirements such as the maximum strength. The design provisions make certain that the probability of exceeding a limit state is acceptably small by stipulating combinations of nominal loads and load factors, as well as resistances and resistance factors that are consistent with the design assumptions. The following four limit states are specified in the LRFD Specifications (AASHTO, 2017):

- Service Limit State: Deals with restrictions on stress, deformation, and crack width under regular service conditions. These provisions are intended to ensure the bridge performs acceptably during its design life.
- Fatigue and Fracture Limit State: The fatigue limit state deals with restrictions on stress range under specified truck loading reflecting the number of expected stress range cycles. The fracture limit state is to establish a set of material toughness requirements. These provisions are intended to limit crack growth under repetitive loads to prevent fracture during its design life.
- Strength Limit State: Ensures that strength and stability, both local and global, are provided to resist the statistically significant load combinations during the life of a bridge. The overall structural integrity is expected to be maintained.
- Extreme Event Limit State: Ensures the structural survival of a bridge during a major earthquake, collision by a vessel, vehicle, ice flow, or floods. These provisions deal with circumstances considered to be unique occurrences whose return period is significantly greater than the design life of a bridge. The probability of a simultaneous occurrence of these extreme events is extremely low; and, therefore, they are applied separately. Under these extreme conditions, the structure is expected to undergo considerable inelastic deformation.

1.5 LOAD AND RESISTANCE FACTOR DESIGN (LRFD)

1.5.1 Probability-Based Design

Probability-based design is to ensure that the probability of failure of a structure is less than a level acceptable to society. It directly takes into account the statistical mean resistance, the statistical mean loads, the nominal or notional value of resistance, the nominal or notional value of the loads, and the dispersion of resistance and loads as measured by either the standard deviation or the coefficient of variation. Direct probability-based design that computes the probability of failure for a given set of loads, statistical data, and the estimate of the nominal resistance of the component has been used in numerous engineering disciplines but has not been widely used in bridge engineering.

1.5.2 Probabilistic Basis of the LRFD Specifications

The probability-based LRFD Specifications center around the load effects Q and the resistances R modeled as statistically independent random variables (Ravindra and Galambos, 1978; Ellingwood, et. al., 1982; Kulicki, et. al., 1994). Figure 1.5-1 shows the relative frequency distributions for Q and R as separate curves. The mean value of the load effects ($\bar{Q}$) and the mean value of the resistance ($\bar{R}$) are also shown. Q_n and R_n are the nominal value of the load effects and the resistance, respectively. γ and ϕ are the load factor and resistance factor, respectively.

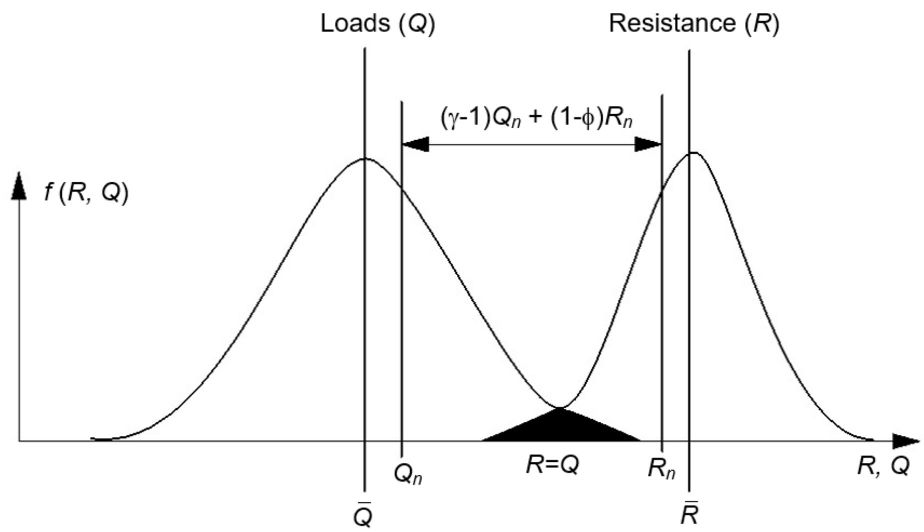

Figure 1.5-1 Relative Frequency Distribution of Load Effect *Q* and Resistance *R*

As long as the resistance R is greater than the load effects Q, a margin of safety for a particular limit state exists. However, since Q and R are random variables in reality, there is a small probability that R may be less than Q. In other words, the probability of $R < Q$ as shown as the overlap shadow area in Figure 1.5-1 is related to the relative positioning of $\bar{R}$ and $\bar{Q}$, and their dispersions. For both the load effect and the resistance, a second value somewhat offset from the mean value, is the “nominal” value. Designers calculate these values for the load effect and the resistance. The objective of the reliability-based or probability-based design philosophy is to separate the distribution of resistance from the distribution of load effect, such that the area of overlap, i.e., the area where load effect is greater than resistance, is acceptably small. In the LRFD Specifications, load factors and resistance factors were developed together in a way that forces the relationship between the resistance and load effect to be such that the area of overlap in Figure 1.5-1 is less than or equal to the value that AASHTO accepts.

The probability of "exceedance" or "achievement of a limit state" can be examined by comparing *R* and *Q* as shown in Figure 1.5-2. Potential structural failure is represented by the left side region. The distance between the "exceedance" line and the mean value of the function of *R*-*Q* is defined as $\beta\sigma$, where σ is the standard deviation of the function of *R*-*Q* and β is called the "reliability index" or "safety index". The larger β is, the greater the margin of safety.

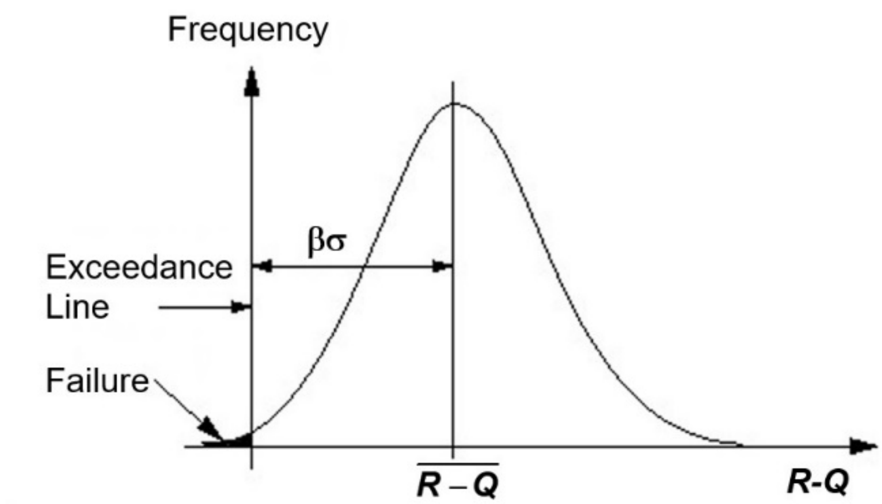

Figure 1.5-2 Reliability Index

The probability of $R < Q$ depends on the distribution shapes of each of many variables (material, loads, etc.). Usually, the mean values and the standard deviations or the coefficients of variation of many variables involved in *R* and *Q* can be estimated. By applying the simple advanced first-order second-moment method (Ravindra and Galambos, 1978; Kulicki et. al., 1994) and assuming that both the resistance and load effect are normal random variables, the following reliability index equation can be obtained:

$$\beta = \frac{\bar{R} - \bar{Q}}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \tag{1.5-1}$$

where σ_R and σ_Q are the coefficients of variation of the resistance *R* and the load effect *Q*, respectively.

Considering variations of both the load effect and the resistance, and introducing x_i as the *i*th load effect, the basic design equations can be expressed as:

$$\phi R = Q = \sum \gamma_i x_i \tag{1.5-2}$$

Introducing λ as the ratio of the mean value divided by the nominal value called the "bias" leads to:

$$\lambda R = \frac{1}{\phi}\lambda \sum \gamma_i x_i \tag{1.5-3}$$

Solving for the resistance factor ϕ from Equations 1.5-3 and 1.5-1 yields:

$$\phi = \frac{\lambda \sum \gamma_i x_i}{\overline{Q} + \beta\sqrt{\sigma_R^2 + \sigma_Q^2}} \tag{1.5-4}$$

Examining the equations shows that there are three unknowns, i.e., the resistance factor, ϕ, the reliability index, β, and the load factor, γ.

The reliability index is very useful. It can give an indication of the consistency of safety of a bridge designed using traditional methods. It also can be used to establish new methods which will have consistent margins of safety. Most importantly, it is a comparative indicator. One group of bridges having a reliability index that is greater than another group of bridges has more inherent safety. A group of existing bridges designed by either ASD or LFD formed the basis for determining the target, or code-specified reliability index and the load and resistance factors in the LRFD Specifications (Kulicki et. al., 1994).

1.5.3 Calibration of Load and Resistance Factors

A target value of the reliability index β, usually denoted β_T, is chosen by a code-writing body. Equation 1.5-4 still indicates that both the load and resistance factors must be found. One way to solve this problem is to select the load factors and then calculate the resistance factors. This process has been used by several code-writing authorities (AASHTO, 1994; OMTC, 1994; CSA, 1998). The basic steps of calibration (Nowak, 1993) of the load and resistance factors for the LRFD Specifications were:

- Develop a database of sample current bridges
- Extract load effects as a percentage of the total load
- Estimate the reliability indices implicit in current designs
- Quantify variability in loads and materials by deciding on coefficients of variation
- Assume load factors
- Vary resistance factors until suitable reliability indices result

Approximately 200 representative bridges (Nowak, 1993) were selected from various regions of the United States by requesting sample bridge plans from various states. The selection was based on structural type, material, and geographic location to represent a full range of materials and design loads and practices as they vary around the country.

Statistically projected live load and the notional values of live load effects were calculated. Resistance was calculated in terms of the moment and the shear capacity for each structure according to the prevailing requirements, in this case, the *AASHTO Standard Specifications* (AASHTO, 1989) for load factor design. Based on the relative amounts of the loads identified for each of the combinations of span and spacing and type of construction indicated by the database, a simulated set of 175 bridges was developed. The simulated group was comprised of non-composite steel girder bridges, composite steel girder bridges, reinforced concrete T-beam bridges, and prestressed concrete I-beam bridges.

The reliability indices were calculated for each simulated and each actual bridge for both the shear and the moment. The range of reliability indices that resulted from this calibration process is presented in Figure 1.5-3 (Kulicki, et. al., 1994). It can be seen that a wide range of values was obtained using the Standard Specifications, but this was anticipated based on previous calibration work done for the *Ontario Highway Bridge Design Code* (Nowak and Lind, 1979).

These calculated reliability indices, as well as past calibration of other specifications, served as a basis for the selection of the target reliability index, β_T. A target reliability index of 3.5 was selected for the *Ontario Highway Bridge Design Code* (OMTC, 1994) and other reliability-based specifications. A consideration of the data shown in Figure 1.5-3 indicates that a β of 3.5 is representative of past LFD practice. Hence, this value was selected as a target for the calibration of the LRFD Specifications.

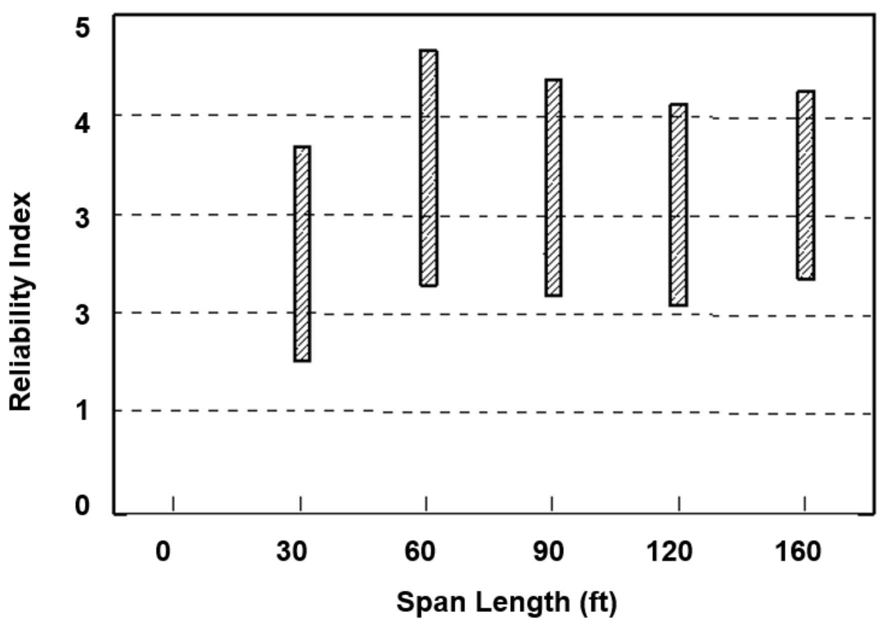

Figure 1.5-3 Reliability Indices Inherent in the 1989 AASHTO Standard Specifications

1.5.4 Load and Resistance Factors

The recommended values of load factors are simplified to be practical for bridge design. One factor is specified for the weight of both shop-built and field-built components: γ = 1.25. For the weight of asphalt and utilities, γ = 1.50, a higher value is used. For live load, the calibrated value of the load factor was 1.60. However, a more conservative value of γ = 1.75 is utilized in the LRFD Specifications. A brief discussion of load factors and load combinations is in Chapter 3.

The acceptance criterion, in the selection of resistance factors, is how close the calculated reliability indices are to the target value of the reliability index, β_T. Calculations were performed using the load components for each of the 175 simulated bridges using the range of resistance factors shown in Table 1.5-1 (Nowak, 1993).

Reliability indices were recalculated for each of the 175 simulated cases and each of the actual bridges from which the simulated bridges were produced. The range of values obtained using the new load and resistance factors are indicated in Figure 1.5-4 (Kulicki, et. al., 1994). It is seen from Figure 1.5-4 that the new calibrated load and resistance factors, and new load models and load distribution techniques work together to produce very narrowly clustered reliability indices. This was the objective of developing the new factors. Correspondence to a reliability index of 3.5 can now be altered by AASHTO when either a higher level of safety or taking more risk is appropriate. If the target reliability index is to be raised or lowered, the factors need to be recalculated accordingly. This ability to adjust the design parameters in a coordinated manner is one of the benefits of a probability-based reliability design.

Table 1.5-1 Considered Resistance Factors in LRFD Calibration

MATERIAL	LIMIT STATE	RESISTANCE FACTOR ϕ	
		LOWER	UPPER
Non-Composite Steel	Moment	0.95	1.00
	Shear	0.95	1.00
Composite Steel	Moment	0.95	1.00
	Shear	0.95	1.00
Reinforced Concrete	Moment	0.85	0.90
	Shear	0.90	0.90
Prestressed Concrete	Moment	0.95	1.00
	Shear	0.90	0.95

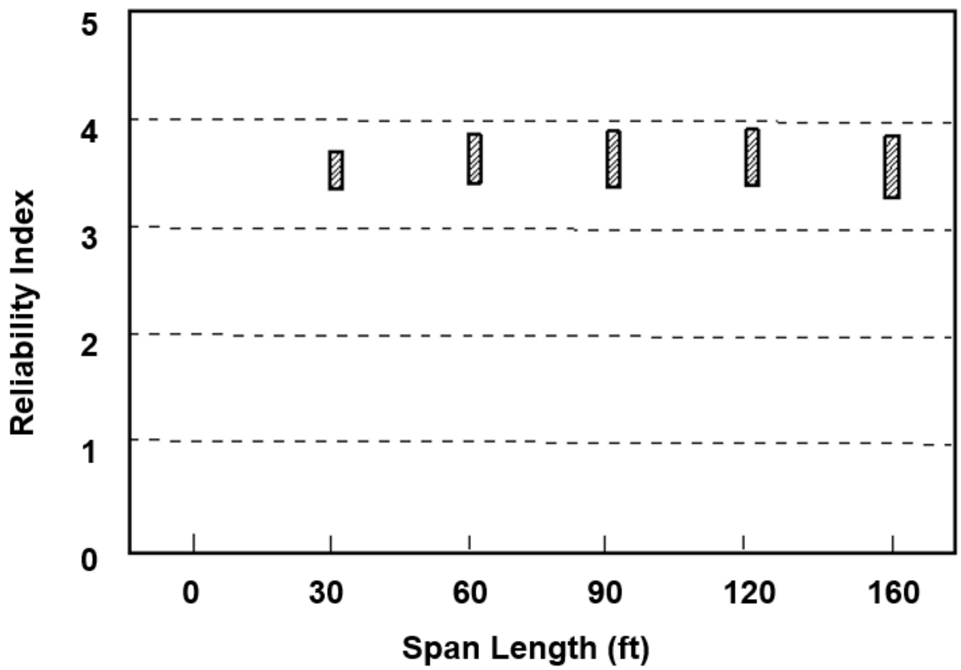

Figure 1.5-4 Reliability Indices Inherent in LRFD Specifications

1.5.5 General Design Requirements

Public safety is the primary responsibility of the design engineer. All other aspects of design, including serviceability, maintainability, economics, and aesthetics are secondary to the requirement for safety. The LRFD Specifications specify that each component and connection shall satisfy the following equation for each limit state:

$$\sum \eta_i \gamma_i Q_i \leq \varphi R_n = R_r \qquad \text{(AASHTO 1.3.2.1-1)}$$

where η_i is a load modifier relating to ductility, redundancy, and operational importance and R_r is the factored resistance.

For loads for which a maximum value of γ_i is appropriate:

$$\eta_i = \eta_D \eta_R \eta_I \geq 0.95 \qquad \text{(AASHTO 1.3.2.1-2)}$$

For loads for which a minimum value of γ_i is appropriate:

$$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0 \qquad \text{(AASHTO 1.3.2.1-3)}$$

where η_D, η_R, and η_I are factors relating to ductility, redundancy, and operational importance, respectively. The *California Amendments* (Caltrans, 2019) specify that the value of 1.0 shall be used for η_D, η_R and η_I until their applications are further studied.

1.5.6 Serviceability Requirements

The LRFD Specifications address serviceability from the viewpoints of durability, restriction of stresses, cracking, corrosion, and deformation - all in conjunction with contract documents to achieve the desired design life. Bridge designers also need to be mindful of the ease in inspection and maintainability, as addressed in *The Manual for Bridge Evaluation,* 3rd Edition (AASHTO, 2018) with the 2019 and 2020 Interims (AASHTO, 2019 and 2020), and Caltrans SM&I (Structures Maintenance & Investigations) *Bridge Load Rating Manual* (Caltrans, 2020).

Durability is to be assured through contract documents calling for high quality materials and requiring that those materials that are subject to deterioration from moisture content and/or salt attack be protected. Good workmanship is also important for good durability.

Maintainability is treated in the specifications in a similar manner to durability; a list of desirable attributes to be considered is provided.

Inspectability is to be assured by providing adequate means for inspectors to view all parts of the structure which have structural or maintenance significance. Bridge inspection can be very expensive and is a recurring cost. Therefore, the cost of providing walkways and other means of access and adequate room for people and inspection equipment to be moved about on the structure is usually a good investment.

Rider comfort is often rationalized as a basis for deflection control. As a compromise between the need for establishing comfort levels and the lack of compelling evidence that deflection was the cause of structural distress, the deflection criteria, other than those pertaining to relative deflections of ribs of orthotropic decks and components of some wood decks, were written as voluntary provisions to be activated by those states that so choose. Deflection limits, stated as span length divided by some number, were established for most cases, and additional provisions of absolute relative displacement between planks and panels of wooden decks and ribs of orthotropic decks were also added. Similarly, optional criteria were established for a span-to-depth ratio for guidance primarily in starting preliminary designs, but also as a mechanism for checking when a given design deviates significantly from past successful practice.

User comfort on pedestrian bridges is addressed in the *LRFD Guide Specifications for the Design of Pedestrian Bridges* (AASHTO, 2009) and its 2015 Interim Revisions (AASHTO, 2015).

1.5.7 Constructability Requirements

The following provisions in the LRFD Specifications Article 2.5.3 (AASHTO, 2017) are related to constructability:

- Design bridges so that they can be fabricated and built without undue difficulty and with control over locked in construction force effects.
- Document one feasible method or a particular sequence of construction in the contract documents unless the type of construction is self-evident.
- Indicate the need to provide strength and/or temporary bracing or support during erection, but not requiring the complete design thereof.
- Avoid details that require welding in restricted areas or placement of concrete through congested reinforcing.
- Consider climatic and hydraulic conditions that may affect the construction of the bridge.

NOTATIONS

FS	=	factor of safety
Q	=	load effect
Q_n	=	nominal load effect
$\overline{Q}$	=	mean value of the load effect
Q_i	=	a load effect
R	=	resistance
R_n	=	nominal resistance
$\overline{R}$	=	mean value of the resistance
R_r	=	factored resistance
x_i	=	ith load effect
γ_i	=	a load factor
γ	=	load factor
λ	=	ratio of the mean value divided by the nominal value, called the “bias”
β	=	reliability index
β_T	=	target reliability index
ϕ	=	resistance factor or strength reduction factor
η_D	=	a factor relating to ductility
η_R	=	a factor relating to redundancy

η_I = a factor relating to operational importance

η_i = a load modifier, a factor relating to ductility, redundancy, and operational importance

σ = standard deviation

σ_R = coefficient of variation of the resistance R

σ_Q = coefficient of variation of the load effect Q

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