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5

Design Philosophies for Highway Bridges

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5.1 Introduction

Several bridge design specifications will be referred to repeatedly herein. In order to simplify the references, the "Standard Specifications" means the *AASHTO Standard Specifications for Highway Bridges* [1], and the sixteenth edition will be referenced unless otherwise stated. The "LRFD Specifications" means the *AASHTO LRFD Bridge Design Specifications* [2], and the first edition will be referenced, unless otherwise stated. This latter document was developed in the period 1988 to 1993 when statistically based probability methods were available, and which became the basis of quantifying safety. Because this is a more modern philosophy than either the load factor design method or the allowable stress design method, both of which are available in the Standard Specifications, and neither of which have a mathematical basis for establishing safety, much of the chapter will deal primarily with the LRFD Specifications.

There are many issues that make up a design philosophy — for example, the expected service life of a structure, the degree to which future maintenance should be assumed to preserve the original resistance of the structure or should be assumed to be relatively nonexistent, the ways brittle behavior can be avoided, how much redundancy and ductility are needed, the degree to which analysis is expected to represent accurately the force effects actually experienced by the structure, the extent to which loads are thought to be understood and predictable, the degree to which the designers' intent will be upheld by vigorous material-testing requirements and thorough inspection during construction, the balance between the need for high precision during construction in terms of alignment and positioning compared with allowing for misalignment and compensating for it in the design, and, perhaps most fundamentally, the basis for establishing safety in the design specifications. It is this last issue, the way that specifications seek to establish safety, that is dealt with in this chapter.

5.2 Limit States

All comprehensive design specifications are written to establish an acceptable level of safety. There are many methods of attempting to provide safety and the method inherent in many modern bridge design specifications, including the LRFD Specifications, the Ontario Highway Bridge Design Code [3], and the Canadian Highway Bridge Design Code [4], is probability-based reliability analysis. The method for treating safety issues in modern specifications is the establishment of "limit states" to define groups of events or circumstances that could cause a structure to be unserviceable for its original intent.

The LRFD Specifications are written in a probability-based limit state format requiring examination of some, or all, of the four limit states defined below for each design component of a bridge.

- The *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.
- The *fatigue and fracture limit state* deals with restrictions on stress range under regular service conditions reflecting the number of expected stress range excursions. These provisions are intended to limit crack growth under repetitive loads to prevent fracture during the design life of the bridge.
- The *strength limit state* is intended to ensure that strength and stability, both local and global, are provided to resist the statistically significant load combinations that a bridge will experience in its design life. Extensive distress and structural damage may occur under strength limit state conditions, but overall structural integrity is expected to be maintained.
- The *extreme event limit state* is intended to ensure the structural survival of a bridge during a major earthquake, or when collided by a vessel, vehicle, or ice flow, or where the foundation is subject to the scour that would accompany a flood of extreme recurrence, usually considered to be 500 years. These provisions deal with circumstances considered to be unique occurrences whose return period is significantly greater than the design life of the bridge. The joint probability of these events is extremely low, and, therefore, they are specified to be applied separately. Under these extreme conditions, the structure is expected to undergo considerable inelastic deformation by which locked-in force effects due to temperature effects, creep, shrinkage, and settlement will be relieved.

5.3 Philosophy of Safety

5.3.1 Introduction

A review of the philosophy used in a variety of specifications resulted in three possibilities, allowable stress design (ASD), load factor design (LFD), and reliability-based design, a particular application of which is referred to as load and resistance factor design (LRFD). These philosophies are discussed below.

5.3.2 Allowable Stress Design

ASD is based on the premise that one or more factors of safety can be established based primarily on experience and judgment which will assure the safety of a bridge component over its design life; for example, this design philosophy for a member resisting moments is characterized by design criteria such as

$$\Sigma M/S \leq F_y/1.82 \tag{5.1}$$

where

ΣM = sum of applied moments

F_y = specified yield stress

S = elastic section modules

The constant 1.82 is the factor of safety.

The "allowable stress" is assumed to be an indicator of the resistance and is compared with the results of stress analysis of loads discussed below. Allowable stresses are determined by dividing the elastic stress at the onset of some assumed undesirable response, e.g., yielding of steel or aluminum, crushing of concrete, loss of stability, by a safety factor. In some circumstances, the allowable stresses were increased on the basis that more representative measures of resistance, usually based on inelastic methods, indicated that some behaviors are stronger than others. For example, the ratio of fully yielded cross-sectional resistance (no consideration of loss of stability) to elastic resistance based on first yield is about 1.12 to 1.15 for most rolled shapes bent about their major axis. For a rolled shape bent about its minor axis, this ratio is 1.5 for all practical purposes. This increased plastic strength inherent in weak axis bending was recognized by increasing the basic allowable stress for this illustration from 0.55 F_y to 0.60 F_y and retaining the elastic calculation of stress.

The specified loads are the working basis for stress analysis. Individual loads, particularly environmental loads, such as wind forces or earthquake forces, may be selected based on some committee-determined recurrence interval. Design events are specified through the use of load combinations discussed in Section 5.4.1.4. This philosophy treats each load in a given load combination on the structure as equal from the viewpoint of statistical variability. A "commonsense" approach may be taken to recognize that some combinations of loading are less likely to occur than others; e.g., a load combination involving a 160 km/h wind, dead load, full shrinkage, and temperature may be thought to be far less likely than a load combination involving the dead load and the full design live load. For example, in ASD the former load combination is permitted to produce a stress equal to four thirds of the latter. There is no consideration of the probability of both a higher-than-expected load and a lower-than-expected strength occurring at the same time and place. There is little or no direct relationship between the ASD procedure and the actual resistance of many components in bridges, or to the probability of events actually occurring.

These drawbacks notwithstanding, ASD has produced bridges which, for the most part, have served very well. Given that this is the historical basis for bridge design in the United States, it is important to proceed to other, more robust design philosophies of safety with a clear understanding of the type of safety currently inherent in the system.

5.3.3 Load Factor Design

In LFD a preliminary effort was made to recognize that the live load, in particular, was more highly variable than the dead load. This thought is embodied in the concept of using a different multiplier on dead and live load; e.g., a design criteria can be expressed as

$$1.30 M_D + 2.17\left(M_{L+I}\right) \leq \phi M_u \tag{5.2}$$

where

M_D = moment from dead loads

M_{L+I} = moment from live load and impact

M_u = resistance

ϕ = a strength reduction factor

Resistance is usually based on attainment of either loss of stability of a component or the attainment of inelastic cross-sectional strength. Continuing the rolled beam example cited above, the distinction between weak axis and strong axis bending would not need to be identified because the cross-sectional resistance is the product of yield strength and plastic section modulus in both cases. In some cases, the resistance is reduced by a “strength reduction factor,” which is based on the possibility that a component may be undersized, the material may be understrength, or the method of calculation may be more or less accurate than typical. In some cases, these factors have been based on statistical analysis of resistance itself. The joint probability of higher-than-expected loads and less-than-expected resistance occurring at the same time and place is not considered.

In the Standard Specifications, the same loads are used for ASD and LFD. In the case of LFD, the loads are multiplied by factors greater than unity and added to other factored loads to produce load combinations for design purposes. These combinations will be discussed further in Section 5.4.3.1.

The drawback to load factor design as seen from the viewpoint of probabilistic design is that the load factors and resistance factors were not calibrated on a basis that takes into account the statistical variability of design parameters in nature. In fact, the factors for steel girder bridges were established for one correlation at a simple span of 40 ft (12.2 m). At that span, both load factor design and service load design are intended to give the same basic structure. For shorter spans, load factor design is intended to result in slightly more capacity, whereas, for spans over 40 feet, it is intended to result in slightly less capacity with the difference increasing with span length. The development of this one point calibration for steel structures is given by Vincent in 1969 [5].

5.3.4 Probability- and Reliability-Based Design

Probability-based design seeks to take into account directly 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, i.e., the standard deviation divided by the mean. This process can be used directly to compute probability of failure for a given set of loads, statistical data, and the designer’s estimate of the nominal resistance of the component being designed. Thus, it is possible to vary the designer’s estimated resistance to achieve a criterion which might be expressed in terms, such as the component (or system) must have a probability of failure of less than 0.0001, or whatever variable is acceptable to society. Design based on probability of failure is used in numerous engineering disciplines, but its application to bridge engineering has been relatively small. The AASHTO “Guide Specification and Commentary for Vessel Collision Design of Highway Bridges” [6] is one of the few codifications of probability of failure in U.S. bridge design.

Alternatively, the probabilistic methods can be used to develop a quantity known as the “reliability index” which is somewhat, but not directly, relatable to the probability of failure. Using a reliability-based code in the purest sense, the designer is asked to calculate the value of the reliability index provided by his or her design and then compare that to a code-specified minimum value. Through a process of calibrating load and resistance factors to reliability indexes in simulated trial designs, it is possible to develop a set of load and resistance factors, so that the design process looks very much like the existing LFD methodology. The concept of the reliability index and a process for reverse-engineering load and resistance factors is discussed in Section 5.3.5.

In the case of the LRFD Specifications, some loads and resistances have been modernized as compared with the Standard Specifications. In many cases, the resistances are very similar. Most of the load and resistance factors have been calculated using a statistically based probability method which considers the joint probability of extreme loads and extreme resistance. In the parlance of the LRFD Specifications, “extreme” encompasses both maximum and minimum events.

5.3.5 The Probabilistic Basis of the LRFD Specifications

5.3.5.1 Introduction to Reliability as a Basis of Design Philosophy

A consideration of probability-based reliability theory can be simplified considerably by initially considering that natural phenomena can be represented mathematically as normal random variables,

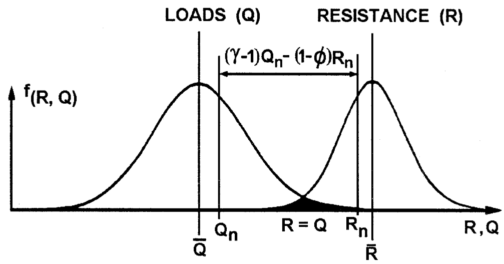

FIGURE 5.1 Separation of loads and resistance. (*Source:* Kulicki, J.M., et al., NH, Course 13061, Federal Highway Administration, Washington, D.C., 1994.)

as indicated by the well-known bell-shaped curve. This assumption leads to closed-form solutions for areas under parts of this curve, as given in many mathematical handbooks and programmed into many hand calculators.

Accepting the notion that both load and resistance are normal random variables, we can plot the bell-shaped curve corresponding to each of them in a combined presentation dealing with distribution as the vertical axis against the value of load, Q, or resistance, R, as shown in Figure 5.1 from Kulicki et al. [7]. The mean value of load, $\overline{Q}$, and the mean value of resistance, $\overline{R}$, are also shown. For both the load and the resistance, a second value somewhat offset from the mean value, which is the "nominal" value, or the number that designers calculate the load or the resistance to be, is also shown. The ratio of the mean value divided by the nominal value is called the "bias." The objective of a design philosophy based on reliability theory, or probability theory, is to separate the distribution of resistance from the distribution of load, such that the area of overlap, i.e., the area where load is greater than resistance, is tolerably small. In the particular case of the LRFD formulation of a probability-based specification, load factors and resistance factors are developed together in a way that forces the relationship between the resistance and load to be such that the area of overlap in Figure 5.1 is less than or equal to the value that a code-writing body accepts. Note in Figure 5.1 that it is the nominal load and the nominal resistance, not the mean values, which are factored.

A conceptual distribution of the difference between resistance and loads, combining the individual curves discussed above, is shown in Figure 5.2. It now becomes convenient to define the mean value of resistance minus load as some number of standard deviations, $\beta\sigma$, from the origin. The variable β is called the "reliability index" and σ is the standard deviation of the quantity $R - Q$. The problem with this presentation is that the variation of the quantity $R - Q$ is not explicitly known. Much is already known about the variation of loads by themselves or resistances by themselves, but the difference between these has not yet been quantified. However, from the probability theory, it is known that if load and resistance are both normal and random variables, then the standard deviation of the difference is

$$\sigma_{(R-Q)} = \sqrt{\sigma_R^2 + \sigma_Q^2} \tag{5.3}$$

Given the standard deviation, and considering Figure 5.2 and the mathematical rule that the mean of the sum or difference of normal random variables is the sum or difference of their individual means, we can now define the reliability index, β, as

$$\beta = \frac{\overline{R} - \overline{Q}}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \tag{5.4}$$

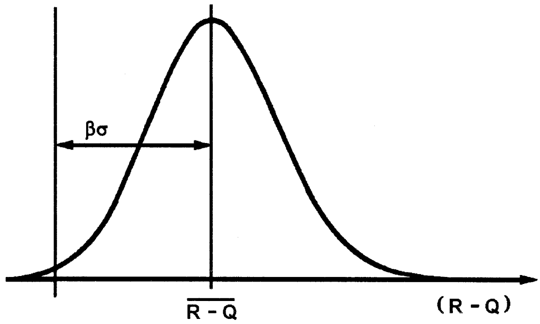

FIGURE 5.2 Definition of reliability index, β. (*Source:* Kulicki, J.M., et al., NH, Course 13061, Federal Highway Administration, Washington, D.C., 1994.)

Comparable closed-form equations can also be established for other distributions of data, e.g., log-normal distribution. A "trial-and-error" process is used for solving for β when the variable in question does not fit one of the already existing closed-form solutions.

The process of calibrating load and resistance factors starts with Eq. (5.4) and the basic design relationship; the factored resistance must be greater than or equal to the sum of the factored loads:

$$\phi R = Q = \Sigma \gamma_i x_i \tag{5.5}$$

Solving for the average value of resistance yields:

$$\overline{R} = \overline{Q} + \beta\sqrt{\sigma_R^2 + \sigma_Q^2} = \lambda R = \frac{1}{\phi}\lambda \Sigma \gamma_i x_i \tag{5.6}$$

By using the definition of bias, indicated by the symbol λ, Eq. (5.6) leads to the second equality in Eq. (5.6). A straightforward solution for the resistance factor, ϕ, is

$$\phi = \frac{\lambda \Sigma \gamma_i x_i}{\overline{Q} + \beta\sqrt{\sigma_R^2 + \sigma_Q^2}} \tag{5.7}$$

Unfortunately, Eq. (5.7) contains three unknowns, i.e., the resistance factor, ϕ, the reliability index, β, and the load factors, γ.

The acceptable value of the reliability index, β, must be chosen by a code-writing body. While not explicitly correct, we can conceive of β as an indicator of the fraction of times that a design criterion will be met or exceeded during the design life, analogous to using standard deviation as an indication of the total amount of population included or not included by a normal distribution curve. Utilizing this analogy, a β of 2.0 corresponds to approximately 97.3% of the values being included under the bell-shaped curve, or 2.7 of 100 values not included. When β is increased to 3.5, for example, now only two values in approximately 10,000 are not included.

It is more technically correct to consider the reliability index to be a comparative indicator. One group of bridges having a reliability index that is greater than a second group of bridges also has more safety. Thus, this can be a way of comparing a new group of bridges designed by some new process to a database of existing bridges designed by either ASD or LFD. This is, perhaps, the most correct and most effective use of the reliability index. It is this use which formed the basis for determining the target, or code specified, reliability index, and the load and resistance factors in the LRFD Specifications, as will be discussed in the next two sections.

The probability-based LRFD for bridge design may be seen as a logical extension of the current LFD procedure. ASD does not recognize that various loads are more variable than others. The introduction of the load factor design methodology brought with it the major philosophical change of recognizing that some loads are more accurately represented than others. The conversion to probability-based LRFD methodology could be thought of as a mechanism to select the load and resistance factors more systematically and rationally than was done with the information available when load factor design was introduced.

5.3.5.2 Calibration of Load and Resistance Factors

Assuming that a code-writing body has established a target value reliability index β, usually denoted β_T, Eq. (5.7) still indicates that both the load and resistance factors must be found. One way to deal with this problem is to select the load factors and then calculate the resistance factors. This process has been used by several code-writing authorities [2–4]. The steps in the process follow:

- Factored loads can be defined as the average value of load, plus some number of standard deviation of the load, as shown as the first part of Eq. (5.6) below.

$$\gamma_i x_i = \bar{x}_i + n\sigma_i = \bar{x}_i + nV_i\bar{x}_i \tag{5.8}$$

  Defining the "variance," V_i, as equal to the standard deviation divided by the average value leads to the second half of Eq. (5.8). By utilizing the concept of bias one more time, Eq. (5.6) can now be condensed into Eq. (5.9).

$$\gamma_i = \lambda\left(1 + nV_i\right) \tag{5.9}$$

  Thus, it can be seen that load factors can be written in terms of the bias and the variance. This gives rise to the philosophical concept that load factors can be defined so that all loads have the same probability of being exceeded during the design life. This is not to say that the load factors are identical, just that the probability of the loads being exceeded is the same.

- By using Eq. (5.7) for a given set of load factors, the value of the resistance factor can be assumed for various types of structural members and for various load components, e.g., shear, moment, etc. on the various structural components. Computer simulations of a representative body of structural members can be done, yielding a large number of values for the reliability index.
- Reliability indexes are compared with the target reliability index. If close clustering results, a suitable combination of load and resistance factors has been obtained.
- If close clustering does not result, a new trial set of load factors can be used and the process repeated until the reliability indexes do cluster around, and acceptably close to, the target reliability index.
- The resulting load and resistance factors taken together will yield reliability indexes close to the target value selected by the code-writing body as acceptable.

The outline above assumes that suitable load factors are assumed. If the process of varying the resistance factors and calculating the reliability indexes does not converge to a suitable narrowly grouped set of reliability indexes, then the load factor assumptions must be revised. In fact, several sets of proposed load factors may have to be investigated to determine their effect on the clustering of reliability indexes.

The process described above is very general. To understand how it is used to develop data for a specific situation, the rest of this section will illustrate the application to calibration of the load and resistance factors for the LRFD Specifications. The basic steps were as follows:

- Develop a database of sample current bridges.
- Extract load effects by percentage of total load.
- Develop a simulation bridge set for calculation purposes.
- Estimate the reliability indexes implicit in current designs.
- Revise loads-per-component to be consistent with the LRFD Specifications.
- Assume load factors.
- Vary resistance factors until suitable reliability indexes result.

Approximately 200 representative bridges 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 practices as they vary around the country. Anticipated future trends should also be considered. In the particular case of the LRFD Specifications, this was done by sending questionnaires to various departments of transportation asking them to identify the types of bridges they are expecting to design in the near future.

For each of the bridges in the database, the load indicated by the contract drawings was subdivided by the following characteristic components:

- The dead load due to the weight of factory-made components;
- The dead load of cast-in-place components;
- The dead load due to asphaltic wearing surfaces where applicable;
- The dead weight due to miscellaneous items;
- The live load due to the HS20 loading;
- The dynamic load allowance or impact prescribed in the 1989 AASHTO Specifications.

Full tabulations for all these loads for the full set of bridges in the database are presented in Nowak [8].

Statistically projected live load and the notional values of live load force effects were calculated. Resistance was calculated in terms of moment and shear capacity for each structure according to the prevailing requirements, in this case the AASHTO Standard Specifications for load factor design.

Based on the relative amounts of the loads identified in the preceding section for each of the combination of span and spacing and type of construction indicated by the database, a simulated set of 175 bridges was developed, comprising the following:

- In all; 25 noncomposite steel girder bridge simulations for bending moments and shear with spans of 9, 18, 27, 36, and 60 m and, for each of those spans, spacings of 1.2, 1.8, 2.4, 3.0, and 3.6 m;
- Representative composite steel girder bridges for bending moments and shear having the same parameters as those identified above;
- Representative reinforced concrete T-beam bridges for bending moments and shear having spans of 9, 18, 27, and 39 m, with spacings of 1.2, 1.8, 2.4, and 3.6 m in each span group;
- Representative prestressed concrete I-beam bridges for moments and shear having the same span and spacing parameters as those used for the steel bridges.

Full tabulations of these bridges and their representative amounts of the various loads are presented in Nowak [8].

The reliability indexes were calculated for each simulated and each actual bridge for both shear and moment. The range of reliability indexes which resulted from this phase of the calibration process is presented in Figure 5.3 from Kulicki et al. [7]. It can be seen that a wide range of values was obtained using the current specifications, but this was anticipated based on previous calibration work done for the Ontario Highway Bridge Design Code (OHBDC) [9].

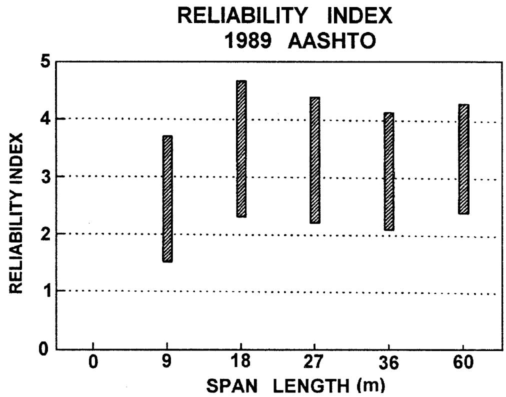

FIGURE 5.3 Reliability indexes inherent in the 1989 AASHTO Standard Specifications. (*Source:* Kulicki, J.M., et al., NH, Course 13061, Federal Highway Administration, Washington, D.C., 1994.)

TABLE 5.1 Parameters of Bridge Load Components

			Load Factor		
Load Component	Bias Factor	Coefficient of Variation	$n = 1.5$	$n = 2.0$	$n = 2.5$
Dead load, shop built	1.03	0.08	1.15	1.20	1.24
Dead load, field built	1.05	0.10	1.20	1.25	1.30
Dead load, asphalt and utilities	1.00	0.25	1.375	1.50	1.65
Live load (with impact)	1.10–1.20	0.18	1.40–1.50	1.50–1.60	1.60–1.70

Source: Nowak, A.S., Report UMCE 92-25, University of Michigan, Ann Arbor, 1993. With permission.

These calculated reliability indexes, as well as past calibration of other specifications, serve as a basis for selection of the target reliability index, β_T. A target reliability index of 3.5 was selected for the OHBDC and is under consideration for other reliability-based specifications. A consideration of the data shown in Figure 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.

5.3.5.3 Load and Resistance Factors

The parameters of bridge load components and various sets of load factors, corresponding to different values of the parameter n in Eq. (5.9) are summarized in Table 5.1 from Nowak [8].

Recommended values of load factors correspond to $n = 2$. For simplicity of the designer, one factor is specified for shop-built and field-built components, $\gamma = 1.25$. For D_3, weight of asphalt and utilities, $\gamma = 1.50$. For live load and impact, the value of load factor corresponding to n = 2 is $\gamma = 1.60$. However, a more conservative value of $\gamma = 1.75$ is utilized in the LRFD Specifications.

The acceptance criterion in the selection of resistance factors is how close the calculated reliability indexes are to the target value of the reliability index, β_T. Various sets of resistance factors, ϕ, are considered. Resistance factors used in the code are rounded off to the nearest 0.05.

Calculations were performed using the load components for each of the 175 simulated bridges using the range of resistance factors shown in Table 5.3. For a given resistance factor, material, span, and girder spacing, the reliability index is computed. Values of β were calculated for live-load factors, $\gamma = 1.75$. For comparison, the results are also shown for live-load factor, $\gamma = 1.60$. The calculations are performed for the resistance factors, ϕ, listed in Table 5.2 from Nowak [8].

Reliability indexes 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 is indicated in Figure 5.4.

TABLE 5.2 Considered Resistance Factors

		Resistance Factors, ϕ	
Material	Limit State	Lower	Upper
Noncomposite 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

Source: Nowak, A.S., Report UMCE 92-25, University of Michigan, Ann Arbor, 1993. With permission.

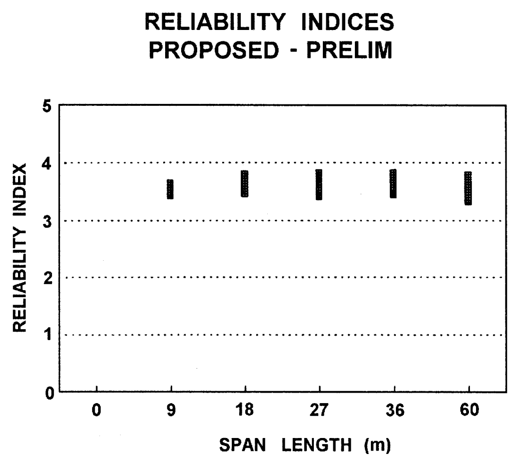

FIGURE 5.4 Reliability indexes inherent in LRFD Specifications. (*Source:* Kulicki, J.M., et al., NH, Course 13061, Federal Highway Administration, Washington, D.C., 1994.)

Figure 5.4 from Kulicki et al. [7] shows that the new calibrated load and resistance factors and new load models and load distribution techniques work together to produce very narrowly clustered reliability indexes. This was the objective of developing the new factors. Correspondence to a reliability index of 3.5 is something which can now be altered by AASHTO. The target reliability index could be raised or lowered as may be advisable in the future and the factors can be recalculated accordingly. This ability to adjust the design parameters in a coordinated manner is one of the strengths of a probabilistically based reliability design.

5.4 Design Objectives

5.4.1 Safety

5.4.1.1 Introduction

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. This does not mean that other objectives are not important, but safety is paramount.

5.4.1.2 The Equation of Sufficiency

In design specifications the issue of safety is usually codified by an application of the general statement the design resistances must be greater than, or equal to, the design load effects. In ASD, Eq. (5.1) can be generalized as

$$\Sigma Q_i \leq R_E/\mathrm{FS} \tag{5.10}$$

where
Q_i = a load
R_E = elastic resistance
FS = factor of safety

In LFD, Eq. (5.2) can be generalized as

$$\Sigma \gamma_i Q_i \leq \phi R \tag{5.11}$$

where
γ_i = a load factor
Q_i = a load
R = resistance
ϕ = a strength reduction factor

In LRFD, Eq. (5.2) can be generalized as

$$\Sigma \eta_i y_i Q_i \leq \phi R_n = R_r \tag{5.12}$$

where
η_i = $\eta_D\ \eta_R\ \eta_I$: $\eta_i = \eta_D\ \eta_R\ \eta_I \geq 0.95$ for loads for which a maximum value of γ_i is appropriate and $\eta_i = 1/(\eta_I \eta_D \eta_R) \leq 1.0$ for loads for which a minimum value of γ_i is appropriate
γ_i = load factor: a statistically based multiplier on force effects
ϕ = resistance factor: a statistically based multiplier applied to nominal resistance
η_i = load modifier
η_D = a factor relating to ductility
η_R = a factor relating to redundancy
η_I = a factor relating to operational importance
Q_i = nominal force effect: a deformation, stress, or stress resultant
R_n = nominal resistance: based on the dimensions as shown on the plans and on permissible stresses, deformations, or specified strength of materials
R_r = factored resistance: ϕR_n

Eq. (5.12) is applied to each designed component and connection as appropriate for each limit state under consideration.

5.4.1.3 Special Requirements of the LRFD Specifications

Comparison of the equation of sufficiency as it was written above for ASD, LFD, and LRFD shows that, as the design philosophy evolved through these three stages, more aspects of the component under design and its relation to its environment and its function to society must be expressly considered. This is not to say that a designer using ASD necessarily considers less than a designer using LFD or LRFD. The specification provisions are the minimum requirements, and prudent designers often consider additional aspects. However, as specifications mature and become more reflective of the real world, additional criteria are often needed to assure adequate safety which may have been provided, albeit nonuniformly, by simpler provisions. Therefore, it is not surprising to find that the LRFD Specifications require explicit consideration of ductility, redundancy, and operational importance in Eq. (5.12), while the Standard Specifications does not.

Ductility, redundancy, and operational importance are significant aspects affecting the margin of safety of bridges. While the first two directly relate to the physical behavior, the last concerns the consequences of the bridge being out of service. The grouping of these aspects is, therefore, arbitrary; however, it constitutes a first effort of codification. In the absence of more precise information, each effect, except that for fatigue and fracture, is estimated as ±5%, accumulated geometrically, a clearly subjective approach. With time, improved quantification of ductility, redundancy, and operational importance, and their interaction, may be attained.

Ductility

The response of structural components or connections beyond the elastic limit can be characterized by either brittle or ductile behavior. Brittle behavior is undesirable because it implies the sudden loss of load-carrying capacity immediately when the elastic limit is exceeded. Ductile behavior is characterized by significant inelastic deformations before any loss of load-carrying capacity occurs. Ductile behavior provides warning of structural failure by large inelastic deformations. Under cyclic loading, large reversed cycles of inelastic deformation dissipate energy and have a beneficial effect on structure response.

If, by means of confinement or other measures, a structural component or connection made of brittle materials can sustain inelastic deformations without significant loss of load-carrying capacity, this component can be considered ductile. Such ductile performance should be verified by experimental testing.

Behavior that is ductile in a static context, but that is not ductile during dynamic response, should also be avoided. Examples of this behavior are shear and bond failures in concrete members and loss of composite action in flexural members.

The ductility capacity of structural components or connections may either be established by full- or large-scale experimental testing, or with analytical models that are based on realistic material behavior. The ductility capacity for a structural system may be determined by integrating local deformations over the entire structural system.

Given proper controls on the innate ductility of basic materials, proper proportioning and detailing of a structural system are the key consideration in ensuring the development of significant, visible, inelastic deformations, prior to failure, at the strength and extreme event limit states.

For the fatigue and fracture limit state for fracture-critical members and for the strength limit state for all members:

η_D ≥ 1.05 for nonductile components and connections,
$= 1.00$ for conventional designs and details complying with these specifications
≥ 0.95 for components and connections for which additional ductility-enhancing measures have been specified beyond those required by these specifications

For all other limit states:

$\eta_D = 1.00$

Redundancy

Redundancy is usually defined by stating the opposite, e.g., a nonredundant structure is one in which the loss of a component results in collapse or a nonredundant component is one whose loss results in complete or partial collapse. Multiple load path structures should be used, unless there are compelling reasons to the contrary. The LRFD Specifications require additional resistance in order to reduce probability of loss of nonredundant component and to provide additional resistance to accommodate load redistribution.

For the strength limit state:

$$\eta_R \geq 1.05 \text{ for nonredundant members}$$
$$= 1.00 \text{ for conventional levels of redundancy}$$
$$\geq 0.95 \text{ for exceptional levels of redundancy}$$

For all other limit states:

$$\eta_R = 1.00$$

The factors currently specified were based solely on judgment and were included to require more explicit consideration of redundancy. Research is under way by Ghosn and Moses [10] to provide more rational requirements based on reliability indexes thought to be acceptable in damaged bridges which must remain in service for a period of about 2 years. The "reverse engineering" concept is being applied to develop values similar in intent to η_R.

Operational Importance
The concept of operational importance is applied to the strength and extreme event limit states. The owner may declare a bridge, or any structural component or connection, thereof, to be of operational importance. Such classification should be based on social/survival and/or security/defense requirements. If a bridge is deemed of operational importance, η_I is taken as ≥1.05. Otherwise, η_I is taken as 1.0 for typical bridges and may be reduced to 0.95 for relatively less important bridges.

5.4.1.4 Design Load Combinations in ASD, LFD, and LRFD

The following permanent and transient loads and forces are considered in the ASD and LFD using the Standard Specifications, and in LRFD using the LRFD Specifications.

The load factors for various loads, making up a design load combination, are indicated in Table 5.4 and Table 5.5 for LRFD and Table 5.6 for ASD and LFD. In the case of the LRFD Specifications, all of the load combinations are related to the appropriate limit state. Any, or all, of the four limit states may be required in the design of any particular component and those which are the minimum necessary for consideration are indicated in the specifications where appropriate. Thus, a design might involve any load combination in Table 5.4.

In the case of ASD or LFD, there is no direct relationship between the load combinations specified in Table 5.6 and limit states, as the design requirements in the Standard Specifications are not organized in that manner. A design by ASD uses those combinations in Table 5.5 indicated for the allowable stress design method as appropriate for the component under consideration. The load combinations indicated for LFD are not used in conjunction with allowable stress design. The opposite is true for LFD.

The application of the load combinations in Table 5.6 for ASD and LFD has been available to bridge designers for decades and is relatively well understood. Numerous textbooks have dealt with these subjects. For this reason, the remainder of this section will deal primarily with the relatively newer LRFD Specifications.

All relevant subsets of the load combinations in Table 5.4 should be investigated. The factors should be selected to produce the total factored extreme force effect. For each load combination, both positive and negative extremes should be investigated. In load combinations where one force effect decreases the effect of another, the minimum value should be applied to load reducing the force effect. For each load combination, every load that is indicated, including all significant effects due to distortion, should be multiplied by the appropriate load factor.

It can be seen in Table 5.4 that some of the load combinations have a choice of two load factors. The larger of the two values for load factors shown for TU, TG, CR, SH, and SE are to be used when calculating deformations; the smaller value should be used when calculating all other force

TABLE 5.3 Load Designations

Name of Load	LRFD Designation	Standard of Specification Designation
	Permanent Loads	
Downdrag	DD	
Dead load of structural components attachments	DC	D
Dead load of wearing surfaces and utilities	DW	D
Dead load of earth fill	EF	D
Horizontal earth pressure	EH	E
Earth surcharge load	ES	E
Vertical earth pressure	EV	D
	Transient Loads	
Vehicular braking force	BR	LF
Vehicular centrifugal force	CE	CF
Creep	CR	R
Vehicular collision force	CT	—
Vessel collision force	CV	—
Earthquake	EQ	EQ
Friction	FR	—
Ice load	IC	ICE
Vehicular dynamic load allowance	IM	I
Vehicular live load	LL	L
Live–load surcharge	LS	L
Pedestrian live load	PL	L
Settlement	SE	—
Shrinkage	SH	S
Temperature gradient	TG	—
Uniform temperature	TU	T
Water load and stream pressure	WA	SF
Wind on live load	WL	WL
Wind load on structure	WS	W

TABLE 5.4 Load Combinations and Load Factors in LRFD

Limit State Load Combinations	DC DD DW EH EV ES	LL IM CE BR PL LS	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These at a Time EQ	IC	CT	CV
Strength I	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Strength III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Strength IV													
EH, EV, ES, DW	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—
DC only	1.5									—	—	—	—
Strength V	γ_p	1.35	1.00	0.40	0.40	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Extreme Event I	γ_p	γ_{EQ}	1.00	—	—	1.00	—	—	—	1.00	—	—	—
Extreme Event II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	0.30	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—
Service III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Fatigue LL, IM and CE only	—	0.75	—	—	—	—	—	—	—	—	—	—	—

TABLE 5.5 Load Factors for Permanent Loads, γ_p in LRFD

Type of Load	Load Factor Maximum	Load Factor Minimum
DC: Component and attachments	1.25	0.90
DD: Downdrag	1.80	0.45
DW: Wearing surfaces and utilities	1.50	0.65
EH: Horizontal earth pressure		
• Active	1.50	0.90
• At rest	1.35	0.90
EV: Vertical earth pressure		
• Overall stability	1.35	N/A
• Retaining structure	1.35	1.00
• Rigid buried structure	1.30	0.90
• Rigid frames	1.35	0.90
• Flexible buried structures other than metal box culverts	1.95	0.90
• Flexible metal box culverts	1.50	0.90
ES: Earth surcharge	1.50	0.75

effects. Where movements are calculated for the sizing of expansion dams, the design of bearing, or similar situations where consideration of unexpectedly large movements is advisable, the larger factor should be used. When considering the effect of these loads on forces that are compatibility generated, the lower factor may be used. This latter use requires structural insight.

Consideration of the variability of loads in nature indicates that loads may be either larger or smaller than the nominal load used in the design specifications. While the concept of variability of permanent loads receives little coverage in ASD, it is codified expressly in LFD. Note that in Table 5.6 the LFD load combinations contain a dead load modifier, indicated as β_E or β_D. These β terms are not to be confused with the reliability index, heretofore referred to as β. The purpose of the modifying factors β_E and β_D is to account for conditions where it is inadvisable to consider either that all of the dead load exists all of the time or that the dead load may be less than the nominal values indicated in the specifications. Thus, for example, the use of the β_D factor 0.75 when checking members for minimum axial load maximum moment means when designing columns and those fixtures which abut the columns, such as footings, it is necessary to evaluate not just the maximum bending moment and the maximum axial load, based on assuming that all the elements of a load combination are thought to obtain their maximum values, but also a load combination in which it is assumed that the dead load is lighter than the nominal load. In the case where the majority of the axial load comes from the dead load and the majority of the bending moment comes from lateral load or live load, this modified combination will tend to produce a maximum eccentricity and hence could control the design of columns and footings.

The specified values of β_E are given below:

- β_E 1.00 for vertical and lateral loads on all other structures
- β_E 1.3 for lateral earth pressure for retaining walls and rigid frames, excluding rigid culverts; for lateral at-rest earth pressures, $\beta_E = 1.15$
- β_E 0.5 for lateral earth pressure when checking positive moments in rigid frames; this complies with Section 3.20
- β_E 1.0 for vertical earth pressure
- β_D 0.75 when checking member for minimum axial load and maximum moment or maximum eccentricity — for column design
- β_D 1.0 when checking member for maximum axial load and minimum moment — for column design
- β_D 1.0 for flexural and tension members
- β_E 1.0 for rigid culverts
- β_E 1.5 for flexible culverts

TABLE 5.6 Table of Coefficients γ and β in ASD and LFD

		Col. No. 1	2	3	3A	4	5	6	7	8	9	10	11	12	13	14
			β Factors													
	Group	γ	D	$(L+I)_n$	$(L+I)_p$	CF	E	B	SF	W	WL	LF	R + S + T	EQ	Ice	%
SERVICE LOAD	I	1.0	1	1	0	1	β_E	B	1	0	0	0	0	0	0	100
	IA	1.0	1	2	0	0	0	1	0	0	0	0	0	0	0	150
	IB	1.0	1	0	1	1	β_E	0	1	0	0	0	0	0	0	[b]
	II	1.0	1	0	0	0	1	1	1	1	0	0	0	0	0	125
	III	1.0	1	1	0	1	β_E	1	1	0.3	1	1	0	0	0	125
	IV	1.0	1	1	0	1	β_E	1	1	0	0	0	1	0	0	125
	V	1.0	1	0	0	0	1	1	1	1	0	0	1	0	0	140
	VI	1.0	1	1	0	1	β_E	1	1	0.3	1	1	1	0	0	140
	VII	1.0	1	0	0	0	1	1	1	0	0	0	0	1	0	133
	VIII	1.0	1	1	0	1	1	1	1	0	0	0	0	0	1	140
	IX	1.0	1	0	0	0	1	1	1	1	0	0	0	0	1	150
	X	1.0	1	1	0	0	β_E	0	0	0	0	0	0	0	0	100
LOAD FACTOR DESIGN	I	1.3	β_D	1.67[a]	0	1.0	β_E	1	1	0	0	0	0	0	0	NOT APPLICABLE
	IA	1.3	β_D	2.20	0	0	0	0	0	0	0	0	0	0	0	
	IB	1.3	β_D	0	1	1.0	β_E	1	1	0	0	0	0	0	0	
	II	1.3	β_D	0	0	0	β_E	1	1	1	0	0	0	0	0	
	III	1.3	β_D	1	0	1	β_E	1	1	0.3	1	1	0	0	0	
	IV	1.3	β_D	1	0	1	β_E	1	1	0	0	0	1	0	0	
	V	1.25	β_D	0	0	0	β_E	1	1	1	0	0	1	0	0	
	VI	1.25	β_D	1	0	1	β_E	1	1	0.3	1	1	1	0	0	
	VII	1.3	β_D	0	0	0	β_E	1	0	0	0	0	0	1	0	
	VIII	1.3	β_D	1	0	1	β_E	1	1	0	0	0	0	0	1	
	IX	1.20	β_D	0	0	0	β_E	1	1	1	0	0	0	0	1	
	X	1.30	1	1.67	0	0	β_E	0	0	0	0	0	0	0	0	

- $(L + I)_n$ = Live load plus impact for AASHTO Highway H or HS loading.
- $(L + I)_p$ = Live load plus impact consistent with the overload criteria of the operation agency.
- % (col. 14) = percentage of basic unit stress.
- No increase in allowable unit stresses shall be permitted for members or connections carrying wind loads only.

[a] 1.25 may be used for design of outside roadway beam when combination of sidewalk live load, and traffic live load plus impact governs the design, but the capacity of the section should not be less than required for highway traffic live load only, using a β factor of 1.67. 1.00 may be used for design of deck slab with combination of loads as described in Article 3.24.2.2.

[b] $$\text{Percentage} = \frac{\text{Maximum Unit Stress (Operating Rating)}}{\text{Allowable Basic Unit Stress}} \times 100.$$

The LRFD Specifications recognize the variability of permanent loads by providing both maximum and minimum load factors for the permanent loads, as indicated in Table 5.5. For permanent force effects, the load factor that produces the more critical combination should be selected from Table 5.4. In the application of permanent loads, force effects for each of the specified six load types should be computed separately. Assuming variation of one type of load by span, length, or component within a bridge is not necessary. For each force effect, both extreme combinations may need to be investigated by applying either the high or the low load factor, as appropriate. The algebraic sums of these products are the total force effects for which the bridge and its components should be designed. This reinforces the traditional method of selecting load combinations to obtain realistic extreme effects.

When the permanent load increases the stability or load-carrying capacity of a component or bridge, the minimum value of the load factor for that permanent load should also be investigated. Uplift, which is treated as a separate load case in past editions of the AASHTO Standard Specifications for Highway Bridges, becomes a Strength I load combination. For example, when the dead-load reaction is positive and live load can cause a negative reaction, the load combination for maximum uplift force would be 0.9DC + 0.65DW + 1.75(LL+IM). If both reactions were negative, the load combination would be 1.25DC + 1.50DW + 1.75(LL+IM).

The load combinations for various limit states shown in Table 5.4 are described below.

Strength I	Basic load combination relating to the normal vehicular use of the bridge without wind.
Strength II	Load combination relating to the use of the bridge by permit vehicles without wind. If a permit vehicle is traveling unescorted, or if control is not provided by the escorts, the other lanes may be assumed to be occupied by the vehicular live load herein specified. For bridges longer than the permit vehicle, addition of the lane load, preceding and following the permit load in its lane, should be considered.
Strength III	Load combination relating to the bridge exposed to maximum wind velocity which prevents the presence of significant live load on the bridge.
Strength IV	Load combination relating to very high ratios of dead load to live load force effect. This calibration process had been carried out for a large number of bridges with spans not exceeding 60 m. Spot checks had also been made on a few bridges up to 180 m spans. For the primary components of large bridges, the ratio of dead and live load force effects is rather high and could result in a set of resistance factors different from those found acceptable for small- and medium-span bridges. It is believed to be more practical to investigate one more load case, rather than requiring the use of two sets of resistance factors with the load factors provided in Strength I, depending on other permanent loads present. This Load Combination IV is expected to govern when the ratio of dead load to live load force effect exceeds about 7.0.
Strength V	Load combination relating to normal vehicular use of the bridge with wind of 90 km/h velocity.
Extreme Event I	Load combination relating to earthquake. The designer-supplied live-load factor signifies a low probability of the presence of maximum vehicular live load at the time when the earthquake occurs. In ASD and LFD the live load is ignored when designing for earthquake.
Extreme Event II	Load combination relating to reduced live load in combination with a major ice event, or a vessel collision, or a vehicular impact.
Service I	Load combination relating to the normal operational use of the bridge with 90 km/h wind. All loads are taken at their nominal values and extreme load conditions are excluded. This combination is also used for checking deflection of certain buried structures and for the investigation of slope stability.
Service II	Load combination whose objective is to prevent yielding of steel structures due to vehicular live load, approximately halfway between that used for Service I and Strength I limit state, for which case the effect of wind is of no significance. This load combination corresponds to the overload provision for steel structures in past editions of the AASHTO Standard Specifications for the Design of Highway Bridges.
Service III	Load combination relating only to prestressed concrete structures with the primary objective of crack control. The addition of this load combination followed a series of trial designs done by 14 states and several industry groups during 1991 and early 1992. Trial designs for prestressed concrete elements indicated significantly more prestressing would be needed to support the loads specified in the proposed specifications. There is no nationwide physical evidence that these vehicles used to develop the notional live loads have caused detrimental cracking in existing prestressed concrete components. The statistical significance

Fatigue — of the 0.80 factor on live load is that the event is expected to occur about once a year for bridges with two design lanes, less often for bridges with more than two design lanes, and about once a day for the bridges with a single design lane.

Fatigue: Fatigue and fracture load combination relating to gravitational vehicular live load and dynamic response, consequently BR and PL need not be considered. The load factor reflects a load level which has been found to be representative of the truck population, with respect to large number of return cycles.

5.4.2 Serviceability

The LRFD Specification treats serviceability from the view points of durability, inspectibility, maintainability, rideability, deformation control, and future widening.

Contract documents should call for high-quality materials and require that those materials that are subject to deterioration from moisture content and/or salt attack be protected. Inspectibility is to be assured through adequate means for permitting inspectors to view all parts of the structure which have structural or maintenance significance. The provisions related to inspectibility are relatively short, but as all departments of transportation have begun to realize, bridge inspection can be very expensive and is a recurring cost due to the need for biennial inspections. Therefore, the cost of providing walkways and other access means and adequate room for people and inspection equipment to be moved about on the structure is usually a good investment.

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

The subject of live-load deflections and other deformations remains a very difficult issue. On the one hand, there is very little direct correlation between live-load deflection and premature deterioration of bridges. There is much speculation that "excessive" live-load deflection contributes to premature deck deterioration, but, to date (late 1997), no causative relationship has been statistically established.

Rider comfort is often advanced as a basis for deflection control. Studies in human response to motion have shown that it is not the magnitude of the motion, but rather the acceleration that most people perceive, especially in moving vehicles. Many people have experienced the sensation of being on a bridge and feeling a definite movement, especially when traffic is stopped. This movement is often related to the movement of floor systems, which are really quite small in magnitude, but noticeable nonetheless. There being no direct correlation between magnitude (not acceleration) of movement and discomfort has not prevented the design profession from finding comfort in controlling the gross stiffness of bridges through a deflection limit. As a compromise between the need for establishing comfort levels and the lack of compelling evidence that deflection was a 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 chose. Deflection limits, stated as span 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 deviated significantly from past successful practice.

5.4.3 Constructibility

Several new provisions were included in the LRFD Specification related to:

- The need to design bridges so that they can be fabricated and built without undue difficulty and with control over locked-in construction force effects;

- The need to document one feasible method of construction in the contract documents, unless the type of construction is self-evident; and
- A clear indication of the need to provide strengthening and/or temporary bracing or support during erection, but not requiring the complete design thereof.

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