2.2 AMERICAN INSTITUTE OF STEEL CONSTRUCTION SPECIFICATION

Because the emphasis of this book is on the design of structural steel building members and their connections, the Specification of the American Institute of Steel Construction is the design specification of most importance here. It is written and kept current by an AISC committee comprising structural engineering practitioners, educators, steel producers, and fabricators. New editions are published periodically, and supplements are issued when interim revisions are needed. Allowable stress design has been the primary method used for structural steel buildings since the first AISC Specification was issued in 1923, although plastic design was made part of the Specification in 1963. In 1986, AISC issued the first specification for load and resistance factor design along with a companion *Manual of Steel Construction*. The purpose of these two documents was to provide an alternative to allowable stress design, much as plastic design is an alternative. The current specification (AISC, 2010a) incorporates both LRFD and ASD.

The LRFD provisions are based on research reported in eight papers published in 1978 in the *Structural Journal of the American Society of Civil Engineers* (Ravindra and Galambos, Yura, Galambos, and Ravindra; Bjorhovde, Galambos, and Ravindra; Cooper, Galambos, and Ravindra; Hansell et al.; Fisher et al.; Ravindra, Cornell, and Galambos; Galambos and Ravindra, 1978).

Although load and resistance factor design was not introduced into the AISC Specification until 1986, it is not a recent concept; since 1974, it has been used in Canada, where it is known as *limit states design*. It is also the basis of most European building codes. In the United States, LRFD has been an accepted method of design for reinforced concrete for years and is the primary method authorized in the American Concrete Institute’s Building Code, where it is known as *strength design* (ACI, 2008). Current highway bridge design standards also use load and resistance factor design (AASHTO, 2010).

The AISC Specification is published as a stand-alone document, but it is also part of the *Steel Construction Manual*, which we discuss in the next section. Except for such specialized steel products as cold-formed steel, which is covered by a different specification (AISI, 2007), the AISC Specification is the standard by which virtually all structural steel buildings in this country are designed and constructed. Hence the student of structural steel design must have ready access to his document. The details of the Specification will be covered in the chapters that follow, but we discuss the overall organization here.

The Specification consists of three parts: the main body, the appendixes, and the Commentary. The body is alphabetically organized into Chapters A through N. Within each chapter, major headings are labeled with the chapter designation followed by a number. Furthermore, subdivisions are numerically labeled. For example, the types of structural steel authorized are listed in Chapter A, “General Provisions,” under Section A3, “Material,” and, under it, Section 1, “Structural Steel Materials.” The main body of the Specification is followed by appendixes 1–8. The Appendix section is followed by the Commentary, which gives background and elaboration on many of the provisions of
the Specification. Its organizational scheme is the same as that of the Specification, so material applicable to a particular section can be easily located.

The Specification incorporates both U.S. customary and metric (SI) units. Where possible, equations and expressions are expressed in non-dimensional form by leaving quantities such as yield stress and modulus of elasticity in symbolic form, thereby avoiding giving units. When this is not possible, U.S. customary units are given, followed by SI units in parentheses. Although there is a strong move to metrification in the steel industry, most structural design in the United States is still done in U.S. customary units, and this textbook uses only U.S. customary units.

2.3 LOAD FACTORS, RESISTANCE FACTORS, AND LOAD COMBINATIONS FOR LRFD

Equation 2.4 can be written more precisely as

\[ \sum_{i} Q_i \gamma_i \leq \phi R_n \]

where

- \( Q_i \) = a load effect (a force or a moment)
- \( \gamma_i \) = a load factor
- \( R_n \) = the nominal resistance, or strength, of the component under consideration
- \( \phi \) = resistance factor

The factored resistance \( \phi R_n \) is called the **design strength**. The summation on the left side of Equation 2.5 is over the total number of load effects (including, but not limited to, dead load and live load), where each load effect can be associated with a different load factor. Not only can each load effect have a different load factor but also the value of the load factor for a particular load effect will depend on the combination of loads under consideration. Equation 2.5 can also be written in the form

\[ R_n \leq \phi R_n \]

where

- \( R_n \) = required strength = sum of factored load effects (forces or moments)

Section B2 of the AISC Specification says to use the load factors and load combinations prescribed by the governing building code. If the building code does not give them, then ASCE 7 (ASCE, 2010) should be used. The load factors and load combinations in this standard are based on extensive statistical studies and are prescribed by most building codes.

ASCE 7 presents the basic load combinations in the following form:

- **Combination 1:** \( 1.4D \)
- **Combination 2:** \( 1.2D + 0.6L + 0.5(L_r \text{ or } S \text{ or } R) \)
- **Combination 3:** \( 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W) \)
- **Combination 4:** \( 1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R) \)
Combination 5: $1.2D + 1.0E + L + 0.2S$
Combination 6: $0.9D + 1.0W$
Combination 7: $0.9D + 1.0E$

where

$D =$ dead load  
$L =$ live load due to occupancy  
$L_r =$ roof live load  
$S =$ snow load  
$R =$ rain or ice load*  
$W =$ wind load  
$E =$ earthquake (seismic load)

In combinations 3, 4, and 5, the load factor on $L$ can be reduced to 0.5 if $L$ is no greater than 100 pounds per square foot, except for garages or places of public assembly. In combinations with wind or earthquake loads, you should use a direction that produces the worst effects.

The ASCE 7 basic load combinations are also given in Part 2 of the AISC Steel Construction Manual (AISC 2011a), which will be discussed in Section 2.6 of this chapter. They are presented in a slightly different form as follows:

Combination 1: $1.4D$
Combination 2: $1.2D + 1.6L + 0.5(L_r$ or $S$ or $R$)
Combination 3: $1.2D + 1.6(L_r$ or $S$ or $R$) + (0.5$L$ or 0.5$W$)
Combination 4: $1.2D + 1.0W + 0.5L + 0.5(L_r$ or $S$ or $R$)
Combination 5: $1.2D \pm 1.0E + 0.5L + 0.2S$
Combinations 6 and 7: $0.9D \pm (1.0W$ or $1.0E)$

Here, the load factor on $L$ in combinations 3, 4, and 5 is given as 0.5, which should be increased to 1.0 if $L$ is greater than 100 pounds per square foot or for garages or places of public assembly. ASCE 7 combinations 6 and 7 arise from the expression shown by considering combination 6 to use 1.0$W$ and combination 7 to use 1.0$E$. In other words,

Combination 6: $0.9D \pm 1.0W$
Combination 7: $0.9D \pm 1.0E$

Combinations 6 and 7 account for the possibility of dead load and wind or earthquake load countering each other; for example, the net load effect could be the difference between 0.9$D$ and 1.0$W$ or between 0.9$D$ and 1.0$E$. (Wind or earthquake load may tend to overturn a structure, but the dead load will have a stabilizing effect.)

As previously mentioned, the load factor for a particular load effect is not the same in all load combinations. For example, in combination 2 the load factor for the live load $L$ is 1.6, whereas in combination 3, it is 0.5. The reason is that the live load

*This load does not include ponding, a phenomenon that we discuss in Chapter 5.
Solution

Even though a load may not be acting directly on a member, it can still cause a load effect in the member. This is true of both snow and roof live load in this example. Although this building is subjected to wind, the resulting forces on the structure are resisted by members other than this particular column.

a. The controlling load combination is the one that produces the largest factored load. We evaluate each expression that involves dead load, \( D \), live load resulting from equipment and occupancy, \( L \), roof live load, \( L_r \), and snow, \( S \).

Combination 1: \[ 1.4D = 1.4(109) = 152.6 \text{ kips} \]

Combination 2: \[ 1.2D + 1.6L + 0.5(L_r \text{ or } S) + R \] Because \( S \) is larger than \( L_r \) and \( R = 0 \), we need to evaluate this combination only once, using \( S \).

\[ 1.2D + 1.6L + 0.5S = 1.2(109) + 1.6(46) + 0.5(20) = 214.4 \text{ kips} \]

Combination 3: \[ 1.2D + 1.6(L_r \text{ or } S) + R \] In this combination, we use \( S \) instead of \( L_r \), and both \( R \) and \( W \) are zero.

\[ 1.2D + 1.6S + 0.5L = 1.2(109) + 1.6(20) + 0.5(46) = 185.8 \text{ kips} \]

Combination 4: \[ 1.2D + 1.6W + 0.5L + 0.5(L_r \text{ or } S) \] This expression reduces to \( 1.2D + 0.5L + 0.5S \), and by inspection, we can see that it produces a smaller result than combination 3.

Combination 5: \[ 1.2D + 1.0E + 0.5L + 0.2S \] As \( E = 0 \), this expression reduces to \( 1.2D + 0.5L + 0.2S \), which produces a smaller result than combination 4.

Combination 6: \[ 0.9D \pm (1.6W \text{ or } 1.0E) \] This expression reduces to \( 0.9D \), which is smaller than any of the other combinations.

Answer

Combination 2 controls, and the factored load is 214 kips.

b. If the factored load obtained in part (a) is substituted into the fundamental LRFD relationship, Equation 2.3, we obtain

\[ \sum \lambda_i Q_i \leq \phi R_n \]

\[ 214.4 \leq 0.85R_n \]

\[ R_n \geq 252.2 \text{ kips} \]

Answer

The required nominal strength is 252 kips.
is being taken as the dominant effect in combination 2, and one of the three effects, $L_r$, $S$, or $R$, will be dominant in combination 3. In each combination, one of the effects is considered to be at its "lifetime maximum" value and the others at their "arbitrary point in time" values.

The resistance factor $\phi$ for each type of resistance is given by AISC in the Specification chapter dealing with that resistance, but in most cases, one of two values will be used: 0.90 for limit states involving yielding or compression buckling and 0.75 for limit states involving rupture (fracture).

2.4 SAFETY FACTORS AND LOAD COMBINATIONS FOR ASD

For allowable strength design, the relationship between loads and strength (Equation 2.1) can be expressed as

$$R_a \leq \frac{R_n}{\Omega}$$  \hspace{1cm} (2.7)

where

- $R_a$ = required strength
- $R_n$ = nominal strength (same as for LRFD)
- $\Omega$ = safety factor
- $R_n/\Omega$ = allowable strength

The required strength $R_a$ is the sum of the service loads or load effects. As with LRFD, specific combinations of loads must be considered. Load combinations for ASD are also given in ASCE 7. These combinations, as presented in the AISC Steel Construction Manual (AISC 2011a), are:

- **Combination 1:** $D$
- **Combination 2:** $D + L$
- **Combination 3:** $D + (L_r \text{ or } S \text{ or } R)$
- **Combination 4:** $D + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$
- **Combination 5:** $D \pm (0.6W \text{ or } 0.7E)$
- **Combination 6a:** $D + 0.75L + 0.75(0.6W \text{ or } 0.7E) + 0.75(L_r \text{ or } S \text{ or } R)$
- **Combination 6b:** $D + 0.75L \pm 0.75(0.7E) + 0.75S$
- **Combinations 7 and 8:** $0.6D \pm (0.6W \text{ or } 0.7E)$

The factors shown in these combinations are not load factors. The 0.75 factor in some of the combinations accounts for the likelihood that all loads in the combination will be at their lifetime maximum values simultaneously. The 0.7 factor applied to the seismic load effect $E$ is used because ASCE 7 uses a strength approach (i.e., LRFD) for computing seismic loads, and the factor is an attempt to equalize the effect for ASD.

Corresponding to the two most common values of resistance factors in LRFD are the following values of the safety factor $\Omega$ in ASD: For limit states involving yielding...
or compression buckling, $\Omega = 1.67$. For limit states involving rupture, $\Omega = 2.00$. The relationship between resistance factors and safety factors is given by

$$\Omega = \frac{1.5}{\phi} \quad (2.8)$$

For reasons that will be discussed later, this relationship will produce similar designs for LRFD and ASD, under certain loading conditions.

If both sides of Equation 2.7 are divided by area (in the case of axial load) or section modulus (in the case of bending moment), then the relationship becomes

$$f \leq F$$

where

$f = \text{applied stress}$

$F = \text{allowable stress}$

This formulation is called *allowable stress design*.

### EXAMPLE 2.1

A column (compression member) in the upper story of a building is subject to the following loads:

- Dead load: 109 kips compression
- Floor live load: 46 kips compression
- Roof live load: 19 kips compression
- Snow: 20 kips compression

**SOLUTION**

Even though a load may not be acting directly on a member, it can still cause a load effect in the member. This is true of both snow and roof live load in this example. Although this building is subjected to wind, the resulting forces on the structure are resisted by members other than this particular column.

a. The controlling load combination is the one that produces the largest factored load. We evaluate each expression that involves dead load, $D$; live load resulting from occupancy, $L$; roof live load, $L_r$; and snow, $S$.

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*The value of $\Omega$ is actually $1\frac{2}{3} = 5/3$ but has been rounded to 1.67 in the AISC specification.*
**Chapter 2  Concepts in Structural Steel Design**

*Predominant LOAD*

**DEAD**
Combination 1: \[1.4D = 1.4(109) = 152.6 \text{kips} \times 1.2D + 1.6L + 0.5S + 0.5R \] Because \( S \) is larger than \( L \) and \( R = 0 \), we need to evaluate this combination only once, using \( S \).

\[1.2D + 1.6L + 0.5S = 1.2(109) + 1.6(46) + 0.5(20) = 214.4 \text{kips} \]

**LIVE**
Combination 2: \[1.6L + 0.5S \]

**SNOW**
Combination 3: \[1.2D + 1.6(L_n \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.5W) \] In this combination, we use \( S \) instead of \( L_n \) and both \( R \) and \( W \) are zero.

\[1.2D + 1.6S + 0.5L = 1.2(109) + 1.6(20) + 0.5(46) = 185.8 \text{kips} \]

**WIND**
Combination 4: \[1.2D + 1.0W + 0.5L + 0.5(S \text{ or } R) \] This expression reduces to \(1.2D + 0.5L + 0.5S \), and by inspection, we can see that it produces a smaller result than combination 3.

\[1.2D \pm 1.0E + 0.5L + 0.2S. \] As \( E = 0 \), this expression reduces to \(1.2D + 0.5L + 0.2S \), which produces a smaller result than combination 4.

**EARTHQUAKE**
Combination 5: \[0.9D \pm (1.0W \text{ or } 1.0E) \] These combinations do not apply in this example, because there are no wind or earthquake loads to counteract the dead load.

**ANSWER**
Combination 2 controls, and the factored load is 214.4 kips.

b. If the factored load obtained in part (a) is substituted into the fundamental LRFD (24) relationship, Equation 2.6, we obtain

\[R_u \leq \phi R_n\]

\[214.4 \leq 0.90 R_n\]

\[R_n \geq 238 \text{kips}\]

**ANSWER**
The required nominal strength is 238 kips.

c. As with the combinations for LRFD, we will evaluate the expressions involving \( D, L, L_n, \) and \( S \) for ASD.

Combination 1: \[D = 109 \text{kips}. \] (Obviously this case will never control when live load is present.)

Combination 2: \[D + L = 109 + 46 = 155 \text{kips}\]

Combination 3: \[D + (L_n \text{ or } S \text{ or } R) \] Since \( S \) is larger than \( L_n \) and \( R = 0 \), this combination reduces to \( D + S = 109 + 20 = 129 \text{kips} \)

Combination 4: \[D + 0.75L + 0.75(L_n \text{ or } S \text{ or } R) \] This expression reduces to \( D + 0.75L + 0.75S = 109 + 0.75(46) + 0.75(20) = 158.5 \text{kips} \)

Combination 5: \[D \pm (0.6W \text{ or } 0.7E) \] Because \( W \) and \( E \) are zero, this expression reduces to combination 1.
Combination 6a: \[ D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R). \] Because \( W \) and \( E \) are zero, this expression reduces to combination 4.

Combination 6b: \[ D + 0.75L \pm 0.75(0.7E) + 0.75S. \] This combination also gives the same result as combination 4.

Combinations 7 and 8: \[ 0.6D \pm (0.6W \text{ or } 0.7E). \] These combinations do not apply in this example, because there are no wind or earthquake loads to counteract the dead load.

**Answer** Combination 4 controls, and the required service load strength is 158.5 kips.

d. From the ASD relationship, Equation 2.7,

\[ R_n \leq \frac{R_n}{\Omega} \]

\[ 158.5 \leq \frac{R_n}{1.67} \]

\[ R_n \geq 265 \text{ kips} \]

**Answer** The required nominal strength is 265 kips.

Example 2.1 illustrates that the controlling load combination for LRFD may not control for ASD.

When LRFD was introduced into the AISC Specification in 1986, the load factors were determined in such a way as to give the same results for LRFD and ASD when the loads consisted of dead load and a live load equal to three times the dead load. The resulting relationship between the resistance factor \( \phi \) and the safety factor \( \Omega \), as expressed in Equation 2.8, can be derived as follows. Let \( R_n \) from Equations 2.6 and 2.7 be the same when \( L = 3D \). That is,

\[ \frac{R_n}{\phi} = R_n \Omega \]

\[ \frac{1.2D+1.6L}{\phi} = (D + L)\Omega \]

or

\[ \frac{1.2D+1.6(3D)}{\phi} = (D + 3D)\Omega \]

\[ \Omega = \frac{1.5}{\phi} \]