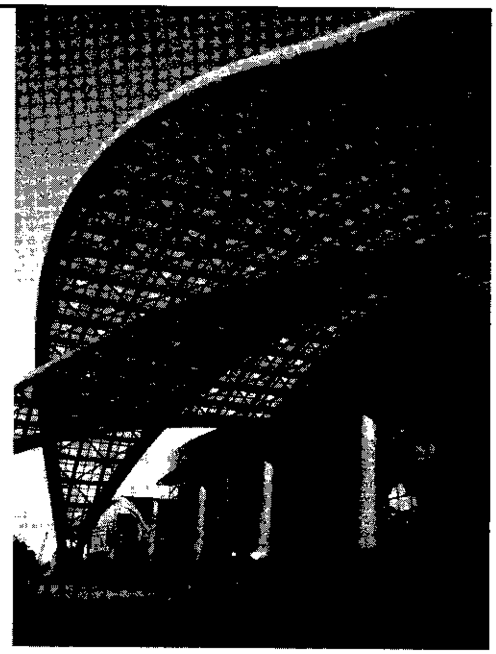
Chapter 2



Puerto Rico Convention Center.
Photo courtesy Walter P Moore.

Loads, Load Factors, and Load Combinations

2.1 INTRODUCTION

Material design specifications, like the AISC Specification, do not normally prescribe the magnitudes of loads that are to be used as the basis for design. These loads vary based on the usage or type of occupancy of the building, and their magnitudes are dictated by the applicable local, regional, or state laws, as prescribed through the relevant building code.

Building code loads are given as nominal values. These values are to be used in design, even though it is well known that the actual load magnitude will differ from these specified values. This is a common usage of the term *nominal*, the same as will be used for the nominal depth of a steel member to be discussed later. These nominal values are determined on the basis of material properties for dead load, load surveys for live loads, weather data for snow and wind load, and geological data for earthquake or seismic loads. These loads are further described in Section 2.2. To be reasonably certain that these loads are not exceeded in a given structure, code load values have tended to be higher or conservative, compared to the loads on a random structure at an arbitrary point in time. This somewhat higher load level also accounts for the fact that all structural loads will exhibit some random variations as a function of time and load type.

To properly address this random variation of load, an analysis reflecting time and space interdependence should be used. This is called a *stochastic analysis*. Many studies have dealt with this highly complex phenomenon, especially as it pertains to live load in buildings. However, the use of time-dependent loads is cumbersome and does not add significantly to the safety or economy of the final design. For most design situations the building code will specify the magnitude of the loads as if they were constant or unchanging. Their time and space variations are accounted for through the use of the maximum load occurring over a certain reference or return period. As an example, the American live load criteria are based on a reference period of 50 years, whereas the Canadian criteria use a 30-year interval.

The geographical location of a structure plays an important role for several load types, such as those from snow, wind, or earthquake.

2.2 BUILDING LOAD SOURCES

Many types of loads may act on a building structure at one time or another and detailed data for each is given later. Loads of primary concern to the building designer include:

1. Dead load
2. Live load
3. Snow load
4. Wind load
5. Seismic load
6. Special loads

Each of these primary load types are characterized as to their magnitude and variability by the building code, and are described in the ensuing paragraphs.

2.2.1 Dead Load

Theoretically, the dead load of a structure remains constant throughout its lifespan. The dead load includes the self-weight of the structure, as well as the weight of any permanent construction materials such as stay-in-place formwork, partitions, floor and ceiling materials, machinery, and other equipment. The dead load may potentially vary from the magnitude used in the design, even in cases where actual element weights are accurately calculated.

The weight of all dead load elements can be exactly determined only by actually weighing and/or measuring the various pieces that compose the structure. This is almost always an impractical solution and the designer therefore usually relies on published data of building material properties to obtain the nominal dead loads to be used in design. These data can be found in such publications as ASCE 7, the model building codes, and product literature. Some variation will thus likely occur in the real structure. Similarly, differences are bound to occur between the weight of otherwise identical structures, representing another source of dead load variability. However, compared to other structural loads, dead load variations are relatively small and the actual mean values are quite close to the published data.

2.2.2 Live Load

Live load is the load on the structure that occurs from all of the non-permanent installations. It includes the weight of the occupants, the furniture and moveable equipment, plus anything else that the designer could possibly anticipate might occur in the structure. The fluctuations in live load are potentially quite substantial. They vary from being essentially

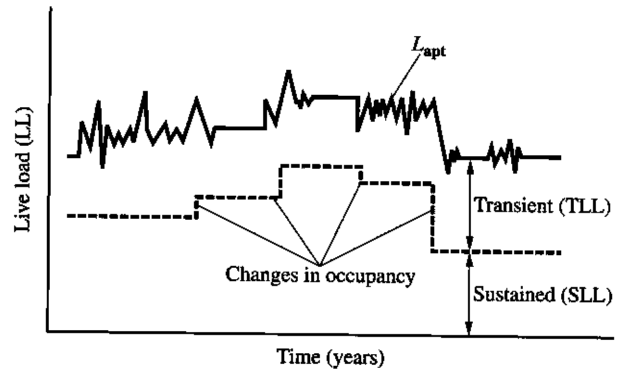


Figure 2.1 Variation of Live Load with Time.

zero immediately before the occupants take possession to a maximum value at some arbitrary point in time during the life of the structure. The magnitude of the live load to be used in a design is obtained from the appropriate building code. The actual live load on the structure at any given time may differ significantly from that specified by the building code. This is one reason why numerous attempts have been made to model live load and its variation and why measurements in actual buildings continue to be made. Although the nominal live loads found in modern building codes have not changed much over the years, the actual use of buildings has, and load surveys continue to show that the specified load levels are still an adequate representation of the loads the structure should be designed to resist.

The actual live load on a structure at any given point in time is called the *arbitrary point-in-time live load*. Figure 2.1 shows the variation of the live load on a structure as might be obtained from a live load survey. The load specified by the building code is always higher than the actual load found in the building survey. In addition, a portion of the live load remains constant. This load comes from the relatively permanent fixtures and furnishings and can be referred to as the *sustained live load (SSL)*. The occupants who enter and leave the space form another part of the live load, raising and lowering the overall live load magnitude with time. This varying live load is called a *transient live load (TLL).*

2.2.3 Snow Loads

Although snow might be considered a form of live load, unique conditions govern its magnitude and distribution. It is the primary roof load in many geographical areas and heavily depends on local climate, building exposure, and building geometry.

Snow load data are normally based on surveys that result in isoline maps showing areas of equal depth of ground snowfall. Using this method, annual extreme snowfalls have been determined over a period of many years. These data have been analyzed through statistical models and the expected lifetime maximum snow loads estimated. The reference period is again the 50-year anticipated life of the structure.

A major difficulty is encountered in translating the ground snow load into a roof snow load. This is accomplished through a semiempirical relationship whereby the ground snow load is multiplied by factors to account for such things as roof geometry and the thermal characteristics of the roof. Work continues to be done to improve the method of snow load computation and to collect snowfall data.

2.2.4 Wind Load

By its very nature, wind is a highly dynamic natural phenomenon. For this reason it is also a complex problem from a structural perspective. Wind forces fluctuate significantly, and are also influenced by the geometry of the structure, including the height, width, depth, plan and elevation shape, and the surrounding landscape. The basic building code approach to wind load analysis is to treat wind as a static load problem, using the Bernoulli equation to translate wind speed into wind pressure. In an approach similar to that used for snow load, a semi-empirical equation is used to give the wind load at certain levels as a function of a number of factors representing such effects as wind gusts, topography, and structural geometry.

The data used for determining wind loads are based on measured wind speeds. Meteorological data for 3-second wind speed gusts have been accumulated over the contiguous United States and corrected to a standard height of 33 ft. These data are then used to model the long-term characteristics for a mean recurrence interval of 50 years. ASCE 7 and the model codes provide maps to be used as the foundation of wind force calculations. Because local site characteristics often dictate wind behavior, there are locations for which special attention must be given to wind load calculation. In addition, some buildings require special attention in determining wind load magnitude. In these cases it might be valuable to conduct wind tunnel tests before the structural design for wind is carried out.

2.2.5 Seismic Load

The treatment of seismic load effects is extremely complicated because of the highly variable nature of this natural phenomenon and the many factors that influence the impact of an earthquake on any particular structure. In addition, because the force the building feels is the result of the ground moving, inertia effects must be considered.

For most buildings it is sufficient to treat seismic effects through the use of an equivalent static load, provided that the magnitude of this equivalent static load properly reflects the dynamic characteristics of the seismic event. Many characteristics of the problem must be quantified in order to establish the correct magnitude of this static load. These include such factors as the ground motion and response spectra for the seismic event and the structural and site characteristics for the specific project. At the present time, there are many more approaches to earthquake load determination than there are current model building codes, because many jurisdictions still use out-of-date model codes. In addition, the extension of seismic design requirements to all areas of the country through the current model building codes is making seismic design a requirement for many more structures than had been the case just a few years ago.

2.2.6 Special Loads

Several other loads will become important for particular structures in particular situations. These include impact, blast, and thermal effects.

Impact

Most building loads are static or essentially so, meaning that their rate of application is so slow that the kinetic energy associated with their motion is insignificant. For example, a person entering a room is actually exerting a dynamic load on the structure by virtue of their motion. However, because of the small mass and slow movement of the individual, their kinetic energy is essentially zero.

When loads are large and/or their rate of application is very high, the influence of the energy brought to bear on the structure as the movement of the load is suddenly restrained must be taken into account. This phenomenon, known as *impact*, occurs as the kinetic energy of the moving mass is translated into a load on the structure. Depending on the rate of application, the effect of the impact is that the structure experiences a load that may be as large as twice the static value of the same mass.

Impact is of particular importance for structures where machinery and similar actions occur. Cranes, elevators, and equipment such as printing presses could all produce impact loads that would need to be considered in a design. In addition, vibrations may be induced into a structure either by these high magnitude impact loads or the normally occurring occupancy loads. Although normal live load occupancy, such as walking, is not likely to produce increased design load magnitudes, the potential for vibration from these activities should be addressed in any design.

Blast

Blast effects on buildings have become a more important design consideration during the first years of the twenty-first century. Prior to that time, when blast effects were considered they were normally the accidental kind. These types of blast do not occur as often as impact for normal structures, but should be considered under certain circumstances. Many structures designed for industrial installations, where products of a volatile nature are manufactured, are designed with resistance to blast as a design consideration. When the structure is called upon to resist the effects of blast, a great deal of effort must be placed on determining the magnitude of the blast to be resisted.

The threat of terrorism has been increasingly recognized since the attacks on the World Trade Center and Pentagon on September 11, 2001. In order to take that threat into account, owners must determine the level of threat to be designed for and design engineers must establish the extent to which a particular threat will influence a particular structure. Generally speaking, analysis and design data for blast effects is somewhat limited. Work is currently being done to establish design guidelines that help determine blast effects and member strength in response to blast.

Thermal Effects

Steel expands or contracts under changing temperatures, and in so doing may exert considerable forces on the structure if the members are restrained from moving. For most building structures, the thermal effects are less significant than other loads for structural strength. Because the movement of the structure results from the total temperature change and is directly proportional to the length of the member experiencing the change, the use of expansion joints becomes important. When expansion or contraction is not permitted, the resulting forces must be accommodated in the members.

The AISC Specification includes guidance on the design of steel structures exposed to fire. Appendix 4 provides criteria for the design and evaluation of structural steel components, systems, and frames for fire conditions. In the current building design environment, design for fire is usually accomplished by a prescriptive approach defined in the Specification as design by qualification testing. If the actual thermal effects of a fire are to be addressed, the Specification permits design by engineering analysis.

2.3 BUILDING LOAD DETERMINATION

Once the appropriate building load sources are identified, their magnitudes must be determined. Methods to determine these magnitudes are set by the applicable building code for each load source. The following sections provide general guidance to determine the building load magnitudes but for specific details, the applicable building code must be consulted.

2.3.1 Dead Load

Building dead load determination can be either quite straightforward or very complex. If the sizes of all elements of the structural system are known before an analysis is conducted, actual material weights may be determined and applied in the structural analysis. Selected unit weights of typical building materials are given in Table 2.1. Manual Table 17.13 provides the weights of building materials and product catalogs provide weights of things such as building mechanical equipment.

If the final sizes are not known, as would normally be the case in the early stages of design, assumptions need to be made to estimate the self-weight of the structure. This then necessitates an iterative process of refinement as the design and its corresponding weight are brought together.

2.3.2 Live Load

As discussed earlier, live load magnitudes are established by the applicable building code. Table 2.2 provides values for the minimum uniformly distributed live loads for buildings for selected occupancies.

In design, a simplified approach for determining the load on a particular element is through the use of the tributary area, A_T. The tributary area method is a way to visualize the load on a structural element without performing the actual equilibrium calculations. It does, however, provide the same result because it is fundamentally based on an equilibrium analysis. Simplified tributary areas for some structural members are given in Figure 2.2.

Although the concept of the tributary area can be used to determine the load on a member, an equally important concept is the *influence area*, A_I. The influence area is significant because it reflects the area over which any applied load would have an influence on the member of interest. The member under consideration would feel no portion of the load applied outside of the influence area. Table 2.3 provides the relationship between tributary area and influence area for several specific structural elements where $A_I = K_{LL} A_T$. Several of the values in this table are simplified from the actual relationship and it is always permissible to calculate the actual influence area.

Table 2.1 Unit Weights of Typical Building Materials

Material	(lb/ft^3)
Aluminum	165
Brick	120
Concrete	
Reinforced, with stone aggregate	150
Block, 60 percent void	87
Steel, rolled	490
Wood	
Fir	32–44
Plywood	36

Table 2.2 Minimum Uniformly Distributed Live Loads for Building Design[a]

Occupancy or use	(lb/ft^2)
Residential dwellings, apartments, hotel rooms, school classrooms	40
Offices	50
Auditoriums (fixed seats)	60
Retail stores	73–100
Bleachers	100
Library stacks	150
Heavy manufacturing and warehouses	250

[a]Data are taken from ASCE 7.

As the influence area increases for a particular member, the likelihood of the full code specified nominal live load actually occurring on the structure decreases. Because the code does not know ahead of time the likelihood of that full area being loaded, the magnitude of the specified load is set without consideration of loaded area. Thus, the tabulated values are referred to as the unreduced nominal live load. To account for the size of the influence area and thereby provide a more realistic predictor of the actual live load on the structure, a live load reduction factor is introduced. For influence areas greater than 400 ft^2 the live load may be reduced according to the live load reduction equation:

$$L = L_o\left(0.25 + \frac{15}{\sqrt{A_I}}\right) \tag{2.1}$$

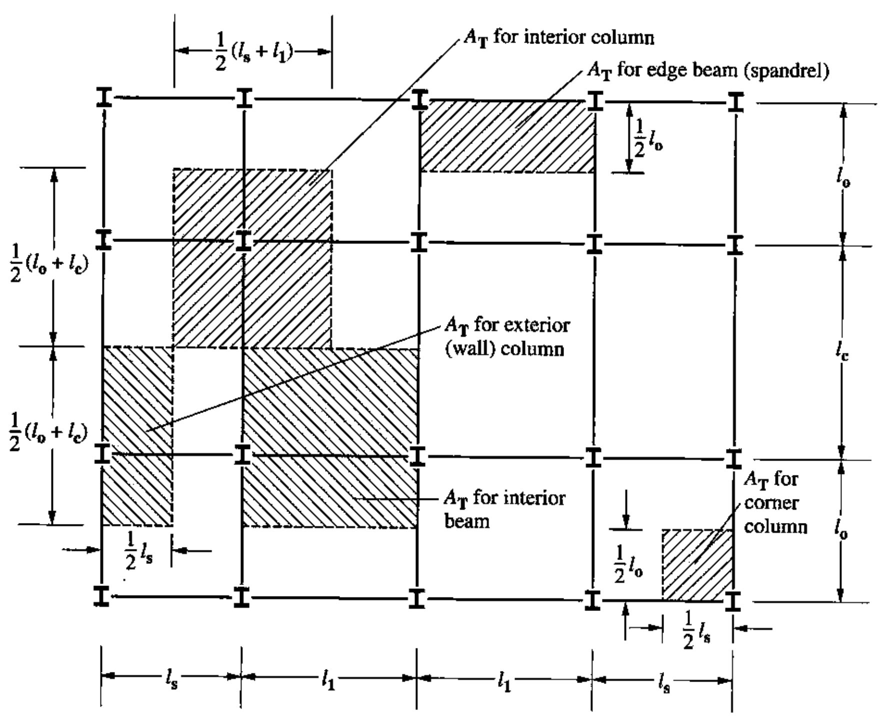


Figure 2.2 Simplified Tributary Areas for Some Structural Members.

Table 2.3 Live Load Element Factor, K_{LL}[a]

Element	K_{LL} (note 1)
Interior columns	4
Exterior columns without cantilever slabs	4
Edge columns with cantilever slabs	3
Corner columns with cantilever slabs	2
Edge beams without cantilever slabs	2
Interior beams	2
All other members not identified above, including:	1
Edge beams with cantilever slabs	
Cantilever beams	
One-way slabs	
Two-way slabs	
Members without provisions for continuous shear transfer normal to their span	

Note 1. In lieu of the values above, K_{LL} is permitted to be calculated.
[a]Data are taken from ASCE 7.

where

L = reduced live load

L_o = code specified design live load

A_I = influence area = $K_{LL}A_T$

Limitations on the use of this live load reduction are spelled out in ASCE 7.

2.3.3 Snow Load

Roof snow load calculations start with determination of the ground snow load for the building site. Table 2.4 provides typical ground snow load values for selected locations. The complete picture of ground snow load is provided in the appropriate building code. In many locations, however, the snowfall depth is a very localized phenomena and the variability is such that it is not appropriate to map those values. In these situations, local building officials should be consulted to determine what the local requirements are.

Table 2.4 Typical Ground Snow Loads, p_g[a]

Location	(lb/ft^2)
Portland, Maine	60
Minneapolis, Minnesota	50
Hartford, Connecticut	30
Chicago, Illinois	25
St. Louis, Missouri	20
Raleigh, North Carolina	15
Atlanta, Georgia	5

[a]Data are taken from ASCE 7.

The determination of roof snow load is complex and there are many acceptable approaches. Roof snow load on an unobstructed flat roof, as given in ASCE 7, is

$$p_f = 0.7C_eC_tIp_g \tag{2.2}$$

where p_g is the ground snow load determined from the appropriate map, C_e is the exposure factor, C_t is the thermal factor, and I is the importance factor. Numerous other factors enter into the determination of roof snow load, including roof slope, roof configuration, snowdrift, and additional load due to rain on the snow. The applicable building code or ASCE 7 should be referred to for the complete provisions regarding snow load determination.

2.3.4 Wind Load

As with snow load and other geographically linked environmental loads, the starting point for wind load calculation is the map of 3-second gusts provided in the building code. Table 2.5 provides the wind speed data for several selected locations with varying wind velocities. These data must be transformed into wind pressure on a given building to determine the appropriate design wind loads. This transformation must take into account such factors as the importance of the building, height above the ground, relative sheltering of the site, topography, and the direction of the dominate winds. ASCE 7 provides the following equation to convert the mapped data to velocity pressure:

$$q_z = 0.00256K_zK_{zt}K_dV^2I \tag{2.3}$$

where

q_z = velocity pressure at a specific height above ground
K_z = exposure coefficient
K_{zt} = topography factor
K_d = directionality coefficient
V = wind speed
I = importance factor

Once the velocity pressure is determined through Equation 2.3, it must be converted to the external design wind pressure. For the main wind force resisting system, this is given by

$$p = qGC_p - q_h(GC_{pi}) \tag{2.4}$$

Table 2.5 Representative Wind Velocities and Resulting Dynamic Pressures[a]

Location	Wind velocity (mph)	Velocity pressure (lb/ft^2)
Miami, Florida	145	53.8
Houston, Texas	120	36.9
New York, New York	105	28.2
Chicago, Illinois	90	20.7
San Francisco, California	85	18.5

[a]Data are taken from ASCE 7.

where

$p =$ design wind pressure

$q =$ velocity pressure from Equation 2.3

$G =$ gust factor

$C_p =$ pressure coefficient

$GC_{pi} =$ internal pressure coefficient

The actual forces applied to the structure are then determined by multiplying the design wind pressure by the tributary area. Because each building code has potentially different requirements for wind load determination, the designer must review the provisions specified in the controlling code. If there is no building code, ASCE 7 should be used.

2.3.5 Seismic Loads

Perhaps the most rapidly fluctuating area of building load determination is that for seismic design. Although there have been many advances in the use of dynamic analysis for earthquake response, common practice is still to model the phenomenon using a static load. For those cases where it applies, ASCE 7 permits the determination of the building base shear through the expression

$$V = C_s W \tag{2.5}$$

where

$V =$ base shear

$C_s =$ seismic response coefficient

$W =$ total building weight

The seismic response coefficient need not be greater than

$$C_s = \frac{S_{D1}}{T(R/I)} \tag{2.6}$$

where

$C_s =$ seismic response coefficient

$S_{D1} =$ design spectral response acceleration

$T =$ building period

$R =$ response modification factor

$I =$ Importance factor

For the design of steel structures to resist seismic forces, the designer must select an appropriate value for the response modification factor, R. In cases where appropriate, the selection of $R = 3$ permits the structure to be designed according to the AISC Specification without using the seismic provisions. If a value of R greater than 3 is used, the design must proceed according to the additional provisions of *ANSI/AISC 341-05 Seismic Provisions for Structural Steel Buildings*. This is discussed further in Chapter 13.

As with the other environmental loads discussed here, the details of load determination for seismic response must be found in the appropriate building code.

2.4 LOAD COMBINATIONS FOR ASD AND LRFD

In addition to specifying the load magnitudes for which building structures must be designed, building codes specify how the individually defined loads should be combined to obtain the maximum load effect. Care must be exercised in combining loads to determine the most critical combination because all loads are not likely to be at their maximum magnitude at the same time. For instance, it is unlikely that the maximum snow load and maximum wind load would occur simultaneously because the wind would undoubtedly blow some of the snow off the structure. Another unlikely occurrence would be a design earthquake occurring at the same time as the maximum design wind. Thus, building codes specify which loads are to be combined and at what magnitude they should be considered. The designer must exercise judgment when combining loads in situations where the normal expectations of the building code might not be satisfied or where some particular combination would result in a greater demand than previously identified.

The two design philosophies addressed in the AISC Specification are the direct result of the two approaches to load combinations presented in current building codes. ASD uses load combinations defined in ASCE 7 as being for allowable stress design, and LRFD uses load combinations defined as being for strength design. The provisions in ASCE 7 for allowable stress design combine loads normally at their nominal or serviceability levels, the load magnitudes discussed in Section 2.3. These load combinations were historically used to determine the load effect under elastic stress distributions and those stresses were compared to the allowable stresses established at some arbitrary level below failure, indicated by either the yield stress or the ultimate stress. Considering load combinations that include only dead, live, wind, snow, and seismic loads, the load combinations presented in ASCE 7-05 Section 2.4 for ASD are:

1. Dead
2. Dead + Live
3. Dead + Roof Live
4. Dead + 0.75 Live + 0.75 Roof Live
5. Dead + Wind
6. Dead + 0.7 Earthquake
7. Dead + 0.75 (Wind or 0.7 Earthquake) + 0.75 Live + 0.75 Roof Live
8. 0.6 Dead + Wind
9. 0.6 Dead + 0.7 Earthquake

As used with the current AISC Specification, these load combinations are not restricted to an elastic stress distribution as done in the past. The current Specification is a strength-based specification, not a stress-based one, and the requirement for elastic stress distribution is no longer applicable. This has no impact on the use of these load combinations but may have some historical significance to those who were educated primarily with this former interpretation.

The second approach available in ASCE 7 combines loads at an amplified level. These combinations, referred to as strength load combinations, permits one to investigate the ability of the structure to resist loads at its ultimate strength. In this approach, loads are multiplied by a load factor that incorporates both the likelihood of the loads occurring simultaneously at their maximum level and the margin against which failure of the structure is measured. Again, considering load combinations that include only dead, live, wind, snow, and seismic loads, the load combinations presented in ASCE 7 Section 2.3, if the live load is not greater than 100 psf, for LRFD are:

1. 1.4 Dead
2. 1.2 Dead + 1.6 Live + 0.5 Roof Live
3. 1.2 Dead + 1.6 Roof Live + 0.5 Live
4. 1.2 Dead + 0.5 Live + 0.5 Roof Live + 1.6 Wind
5. 1.2 Dead + 0.5 Live + 1.0 Earthquake + 0.2 Snow
6. 0.9 Dead + 1.6 Wind
7. 0.9 Dead + 1.0 Earthquake

The design method to be used, and thus the load combinations, are at the discretion of the designer. All current building codes permit either ASD or LRFD, and the AISC Specification provisions address all limit states for each approach. As discussed in Chapter 1, the resulting design may differ for each design philosophy, because the approach taken to assure safety is different, but safety is assured when following the appropriate building code and the AISC Specification regardless of the design approach.

2.5 LOAD CALCULATIONS

In order to understand the impact of these two approaches on analysis, it is helpful to compute the load effect for a variety of structural members according to both ASD and LRFD load combinations. The floor plan of a moderate-height multistory building is given in Figure 2.3. Load case 2 for dead plus live load is considered for several beams and columns. The building is an office building with a nominal live load of 50 pounds per square foot (psf) and a calculated dead load of 70 psf.

1. *Girder AB on line 2-2 if the floor deck spans from line 1-1 to 2-2 to 3-3*:
   Tributary area: $A_T = (40)(20) = 800\ \text{ft}^2$
   Influence area: $A_I = 2A_T = 1600\ \text{ft}^2$
   Live load reduction:

$$0.25 + \frac{15}{\sqrt{1600}} = 0.625$$

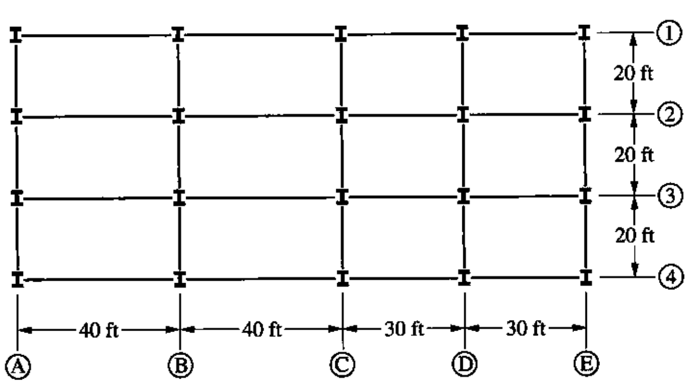


Figure 2.3 Floor Plan of High-Rise Office Building.

LRFD
Amplified loads per lineal foot:

Dead load = 1.2 (70 psf) (20 ft) = 1680 plf
Live load = 1.6 (0.625) (50 psf) (20 ft) = 1000 plf

1.2 Dead + 1.6 Live = 1680 + 1000 = 2680 lbs/ft = 2.68 kips/ft

Required Moment (LRFD), $M_u = \frac{wl^2}{8} = \frac{2.68(40)^2}{8} = 536$ ft-kips

ASD
Nominal loads per lineal foot:
Dead load = 70 psf (20 ft) = 1400 pounds per lineal foot (plf)
Live load = 0.625 (50 psf) (20 ft) = 625 plf

Dead + Live = 1400 + 625 = 2030 lbs/ft = 2.03 kips/ft

Required Moment (ASD), $M_a = \frac{wl^2}{8} \frac{2.03(40)^2}{8} = 406$ ft-kips

2. *Floor beam 2-3 on line D-D if the floor deck spans from line C-C to D-D to E-E*:
   Tributary area: $A_T = (20)(30) = 600\,\text{ft}^2$
   Influence area: $A_I = 2A_T = 1200\,\text{ft}^2$
   Live load reduction:

$$0.25 + \frac{15}{\sqrt{1200}} = 0.683$$

LRFD
Amplified loads per lineal foot:

Dead load = 1.2 (70 psf) (30 ft) = 2520 plf
Live load = 1.6 (0.683) (50 psf) (30 ft) = 1640 plf

1.2 Dead + 1.6 Live = 2520 + 1640 = 4160 plf = 4.16 kips/ft

Required Moment (LRFD), $M_u = \frac{wl^2}{8} = \frac{4.16(20)^2}{8} 208$ ft-kips.

ASD
Nominal loads per lineal foot:
Dead load = 70 psf (30 ft) = 2100 plf
Live load = 0.683 (50 psf) (30 ft) = 1020 plf

Dead + Live = 2100 + 1020 = 3120 plf = 3.12 kips/ft

Required Moment (ASD), $M_a = \frac{wl^2}{8} = \frac{3.12(20)^2}{8} = 156$ ft-kips

3. *Interior column D-2 regardless of deck span direction*:

Tributary area: $A_T = (30)(20) = 600\,\text{ft}^2$

Influence area: $A_I = 4A_T = 2400\,\text{ft}^2$

Live load reduction:

$$0.25 + \frac{15}{\sqrt{2400}} = 0.556$$

LRFD

Amplified load entering column at this level:

Dead load = 1.2 (70 psf) (600 ft^2) = 50,400 lbs

Live load = 1.6 (0.556) (50 psf) (600 ft^2) = 26,700 lbs

Dead + Live = 50,400 + 26,700 = 77,100 lbs = 77.1 kips

Required axial force (LRFD), $P_u = 77.1$ kips

ASD

Nominal load entering column at this level:

Dead load = 70 psf (600 ft^2) = 42,000 lbs

Live load = 0.556 (50 psf) (600 ft^2) = 16,700 lbs

Dead + Live = 42,000 + 16,700 = 58,700 lbs = 58.7 kips

Required axial force (ASD), $P_a = 58.7$ kips

4. *Exterior column D-4 regardless of deck span direction*:

Tributary area: $A_T = (30)(10) = 300\,\text{ft}^2$

Influence area: $A_I = 4A_T = 1200\,\text{ft}^2$

Live load reduction:

$$0.25 + \frac{15}{\sqrt{1200}} = 0.683$$

LRFD

Amplified load entering column at this level:

Dead load = 1.2 (70 psf) (300 ft^2) = 25,200 lbs

Live load = 1.6 (0.683) (50 psf) (300 ft^2) = 16,400 lbs

1.2 Dead + 1.6 Live = 25,200 + 16,400 = 41,600 lbs = 41.6 kips

Required axial force (LRFD), $P_u = 41.6$ kips

ASD

Nominal load entering column at this level:

Dead load = 70 psf (300 ft^2) = 21,000 lbs

Live load = 0.683 (50 psf) (300 ft^2) = 10,200 lbs

Dead + Live = 21,000 + 10,200 = 31,200 lbs = 31.2 kips

Required axial force (ASD), $P_a = 31.2$ kips

2.6 CALIBRATION

The basic requirements of the ASD and LRFD design philosophies were presented in Sections 1.6 and 1.7 and Equations 1.1 and 1.2. The required load combinations for ASD and LRFD, as found in ASCE 7, have been presented earlier in this chapter. This section establishes the relationship between the resistance factor, ϕ, and the safety factor, Ω.

Early development of the LRFD approach to design concentrated on the determination of resistance factors and load factors that would result in a level of structural reliability consistent with previous practice but more uniform for different load combinations. Because the design of steel structures before that time had no particular safety-related concerns, the LRFD approach was calibrated to the then-current ASD approach. This calibration was carried out for the live load plus dead load combination at a live-to-dead load ratio, L/D = 3.0. It was well known that for any other load combination or live load-to-dead load ratio, the two methods could give different answers for the same design situation.

The current Specification has been developed with this same calibration, which results in a direct relationship between the resistance factor of LRFD and the safety factor of ASD. For the live load plus dead load combination in ASD, using Equation 1.1, and representing the load effect simply in terms of L and D,

$$(D + L) \leq \frac{R_n}{\Omega}$$

This same combination in LRFD, using Equation 1.2, yields

$$(1.2D + 1.6L) \leq \phi R_n$$

If it is assumed that the load effect is equal to the available strength and each equation is solved for the nominal strength, the results for ASD are:

$$\Omega(D + L) = R_n$$

and for LRFD:

$$\frac{(1.2D + 1.6L)}{\phi} = R_n$$

With L/D taken as 3, the above equations are set equal. They are then solved for the safety factor, which gives:

$$\Omega = \frac{1.5}{\phi}$$

The resistance factors in the Specification were developed through a stochastic analysis to be consistent with the specified load factors and result in the desired reliability for each limit state. More detail on the development of these resistance factors can be found in Section B3.3 of the Commentary to the Specification. Once the resistance factors were established, the corresponding safety factors were determined. This relationship has been used throughout the Specification to set the safety factor for each limit state.

Although the relationship is simple, there is actually no reason to use it to determine safety factors, because the Specification explicitly defines resistance factors and safety factors for every limit state.