1 Steel and Steel Structures

1.1 PROPERTIES OF STEEL

The stress-strain relationship for mild structural steel is shown graphically in Fig.1.1. This curve is obtained from a tensile test in which a prismatic bar of cross-sectional area A and length L is subjected to two equal but opposite force P at its two ends. In Fig.1.1 (a), elongation, strain, and stress are shown by δ, ε, and f, respectively. Several important points and stress limits can be identified on this figure.

Point A is the proportional limit, the largest stress for which Hook's law applies. Point B is the elastic limit (no permanent deformation). Point C is the yield point with yield stress F_y. Point E is the ultimate limit with ultimate stress F_u. The three points A, B, C are usually very close to each other. Plastic strains are usually 10 to 15 times the elastic strain. The strain-hardening portion of the stress-strain diagram is not used in structural steel design. Yielding of steel with practically no stress increase is a significant property of the material for resisting dynamic loads. A mild structural steel undergoes substantial deformation before failure by fracture. In contrast, failure of a brittle material such as concrete or glass is sudden (Fig. 1.2) without any advance warning. The strain at failure [point F in Fig. 1.1(b)] for mild steels is 150 to 200 times the elastic strain.

When the normal stress *f* is found by dividing the axial force *P* by original cross-sectional area *A*, the stress-strain diagram *OABCDEF* will be obtained. Beyond approximately point *D*, however, a lateral contraction or *necking* occurs, as shown in Fig. 1.1(b). If the narrow cross section of the bar at the neck is used in calculating the normal stress, the dashed curve *DG* will be obtained instead of *DF*.

There is an inverse relationship between ductility and strength of steel. In other words, high-strength steels have low ductility. As a result, the designer may have to compromise between these two properties.

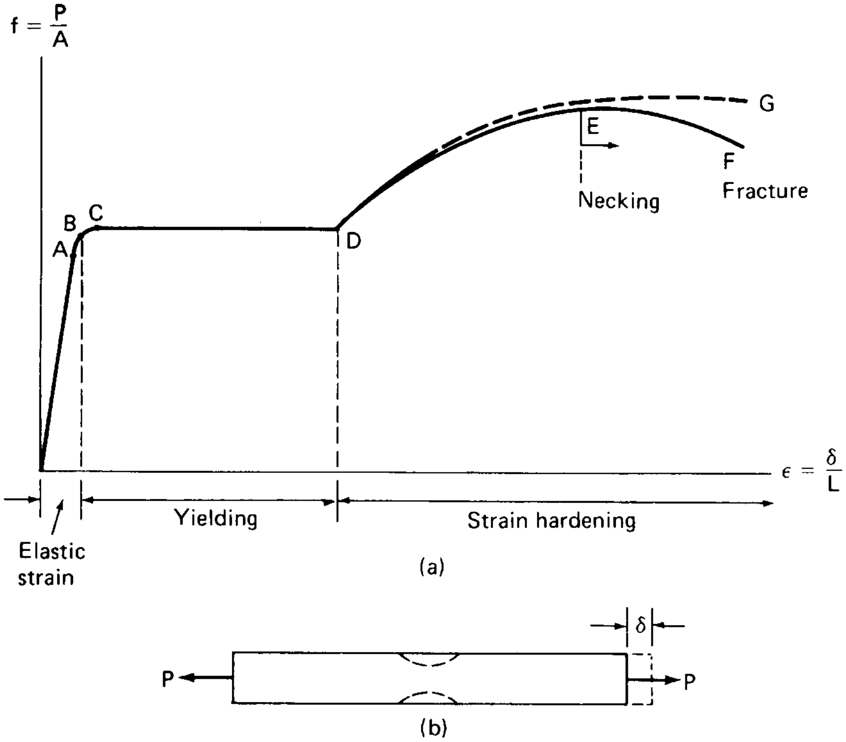

Figure 1.1 Stress-strain relationship for mild steel.

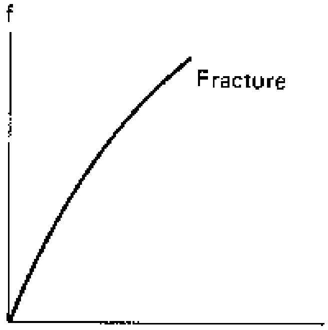

Figure 1.2 Stress-strain relationship for a brittle material.

1.2 ADVANTAGES AND DISADVANTAGES OF STEEL AS A STRUCTURAL DESIGN MATERIAL

The following advantages in general may be credited to steel as a structural design material:

1. High strength/weight ratio. Steel has a high strength/weight ratio. Thus, the dead weight of steel structures is relatively small. This property makes steel a very attractive structural material for
    a. High-rise buildings
    b. Long-span bridges
    c. Structures located on soft ground
    d. Structures located in highly seismic areas where forces acting on the structure due to an earthquake are in general proportional to the weight of the structure.
2. Ductility. As discussed in the previous section, steel can undergo large plastic deformation before failure, thus providing a large reserve strength. This property is referred to as *ductility*. Properly

designed steel structures can have high ductility, which is an important characteristic for resisting shock loading such as blasts or earthquakes. A ductile structure has energy-absorbing capacity and will not incur sudden failure. It usually shows large visible deflections before failure or collapse.

3. Predictable material properties. Properties of steel can be predicted with a high degree of certainty. Steel in fact shows elastic behavior up to a relatively high and usually well-defined stress level. Also, in contrast to reinforced concrete, steel properties do not change considerably with time.
4. Speed of erection. Steel structures can be erected quite rapidly. This normally results in quicker economic payoff.
5. Quality of construction. Steel structures can be built with high-quality workmanship and narrow tolerances.
6. Ease of repair. Steel structures in general can be repaired quickly and easily.
7. Adaptation of prefabrication. Steel is highly suitable for prefabrication and mass production.
8. Repetitive use. Steel can be reused after a structure is disassembled.
9. Expanding existing structures. Steel buildings can be easily expanded by adding new bays or wings. Steel bridges may be widened.
10. Fatigue strength. Steel structures have relatively good fatigue strength.

The following may be considered as disadvantages of steel in certain cases:

1. General cost. Steel structures may be more costly than other types of structures.

2. Fireproofing. The strength of steel is reduced substantially when heated at temperatures commonly observed in building fires. Also, steel conducts and transmits heat from a burning portion of the building quite fast. Consequently, steel frames in buildings must have adequate fireproofing.
3. Maintenance. Steel structures exposed to air and water, such as bridges, are susceptible to corrosion and should be painted regularly. Application of weathering and corrosion-resistant steels may eliminate this problem.
4. Susceptibility to buckling. Due to high strength/weight ratio, steel compression members are in general more slender and consequently more susceptible to buckling than, say, reinforced concrete compression members. As a result, considerable materials may have to be used just to improve the buckling resistance of slender steel compression members.

1.3 TYPES OF STEEL STRUCTURES

1.3.1 Framed Structures

These are the most common types of steel structures. This book concentrates on the design of framed structures. Framed structures in general consist of tension members, beams, columns, beam-columns, and members acted upon by combined bending and torsion. Framed structures may be divided into six categories: beams, plane trusses, space trusses, plane frames, space frames, and grids. Many three-dimensional structures such as buildings may be considered as consisting of two-dimensional or planar structures in two perpendicular directions.

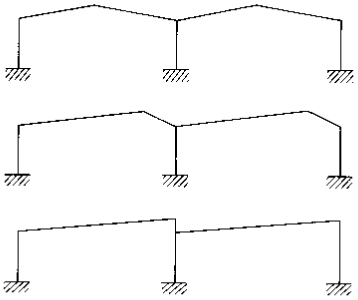

Figure 1.3 Single-story rigid frames.

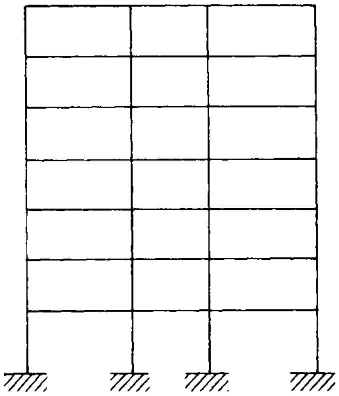

Figure 1.4 Multistory rigid frames.

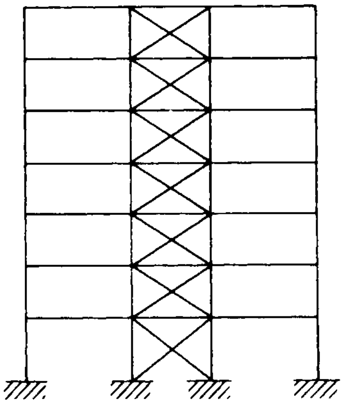

Figure 1.5 Braced frame.

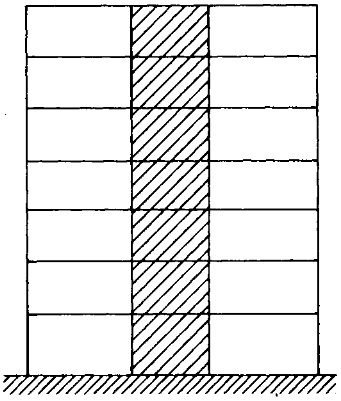

Figure 1.6 Frame with shear wall.

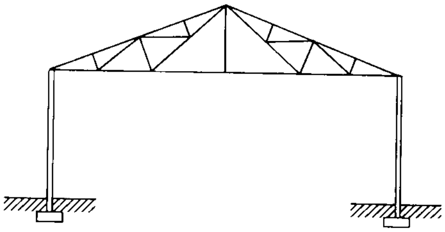

Figure 1.7 Truss on vertical walls.

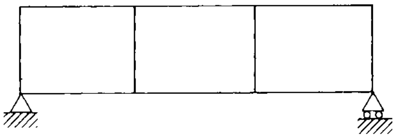

Figure 1.8 Vierendeel frame.

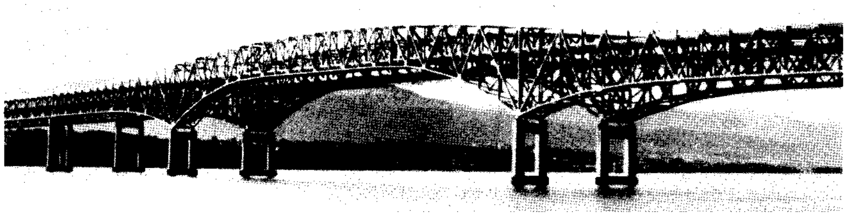

Figure 1.9 North and South Newburgh-Beacon Bridges over the Hudson River in New York. (From AISC, 1985, printed by permission of the AISC.)

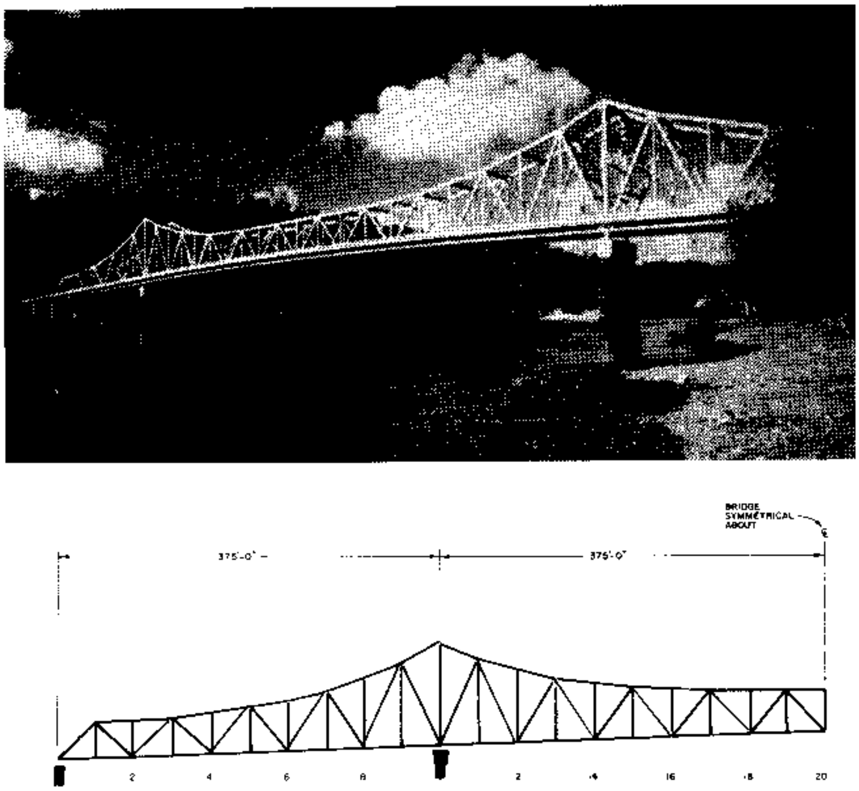

Figure 1.10 Sewickley Bridge over the Ohio River near Pittsburgh. (From AISC, 1985, printed by permission of the AISC.)

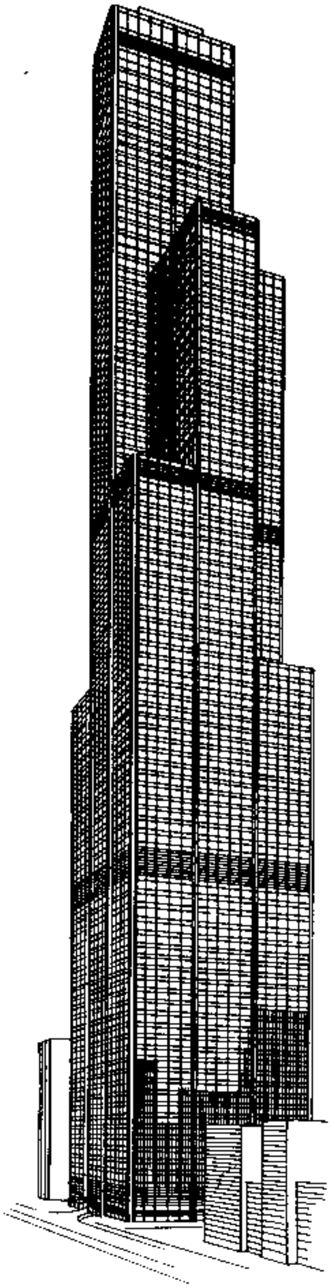

Figure 1.11 One-hundred-nine-story, 445-m-high Sears Building in Chicago. (Reprinted from Hart et al.,1985, by permission of Collins Professional and Technical Books.)

Common planar steel structures are simple and continuous beams (such as the plate girder of Fig. 9.1), single-story rigid frames (Fig. 1.3), multistory rigid frames (Fig. 1.4), braced frames (Fig. 1.5), frames with shear walls (Fig. 1.6), trusses (Fig. 1.7), and Vierendeel frames (Fig. 1.8).

Steel truss bridges are often used for 400-1200-ft spans. Figure 1.9 shows the north and south Newburgh-Beacon bridges over the Hudson River in the state of New York, completed in 1963 and 1980, respectively. Both of them have a three-span cantilever through-truss and a number of continuous deck trusses.

Figure 1.10 shows the prize-winning Sewickley bridge over the Ohio river near Pittsburgh. This bridge is a three-span continuous Warren truss with a main span of 750 ft.

Figure 1.11 shows the 109-story, 445-m-high Sears building in Chicago. The amount of steel used in this building (total weight divided by total floor area) is 33 psf.

1.3.2 Tensile Structures

These structures are also called *cable structures* or *suspension-type structures*. Tension cables play an important role in design of these structures. Tension is in general the most efficient means of supporting loads. Very light structures can be built by using high-strength cables. Providing sufficient stiffness for tensile structures is, however, a problem particularly for buildings located in high-wind or seismic areas.

Examples of bridge tensile structures are suspension bridges and cable-stayed bridges (Fig. 1.12). Figure 1.12 shows the Weirton-Steubenville asymmetrical cable-stayed bridge with a main span of 820 ft and a back span of 688 ft in West Virginia.

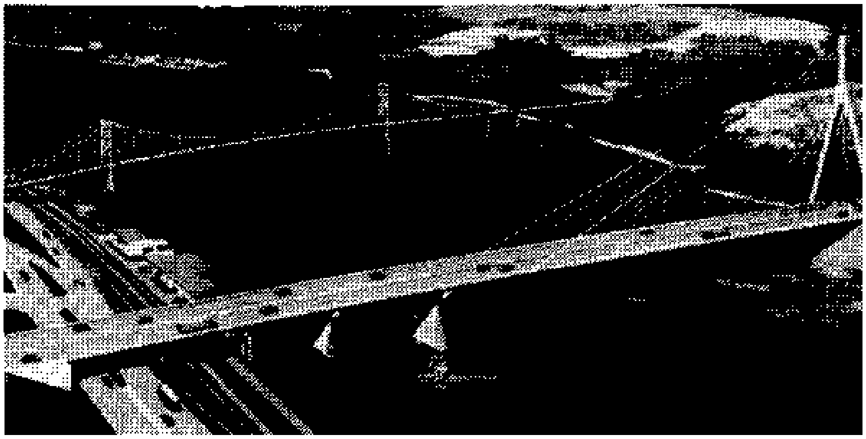

Figure 1.12 Suspension and cable-stayed bridges. (From AISC, 1985, printed by permission of the AISC.)

Cable structures are also used as roof structures. Figure 1.13 shows a suspension roof built in Raleigh, North Carolina, in 1953 with a span of 325 ft (99m). The main cables are suspended from two intersecting arches. In order to increase the stiffness of the roof, prestressed secondary cables are used at right angles to the main cables. Figure 1.14 shows a stadium with a span of 308ft (94m) built in Uruguay. The roof consists of a single layer of cables, an outer

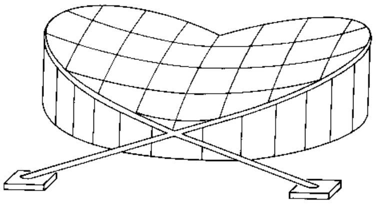

Figure 1.13 Suspension roof built in Raleigh, North Carolina.

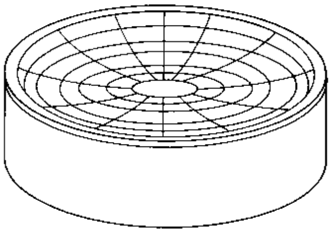

Figure 1.14 Suspension roof built in Uruguay.

reinforced concrete compression ring, and an inner steel tension ring. The roof is covered by precast concrete slabs. The Oakland-Alameda County Auditorium, built with a span of 420 ft (128m) in 1967 has a similar construction. In this structure the rainwater is collected off the roof through pumping.

Figure 1.15 shows the cable-suspended elliptical roof of Terminal 3 at John F. Kennedy International Airport in New York. This roof consists of radial beams supporting reinforced concrete slabs. The beams cantilever 150 ft over an outer ring and are supported by cable

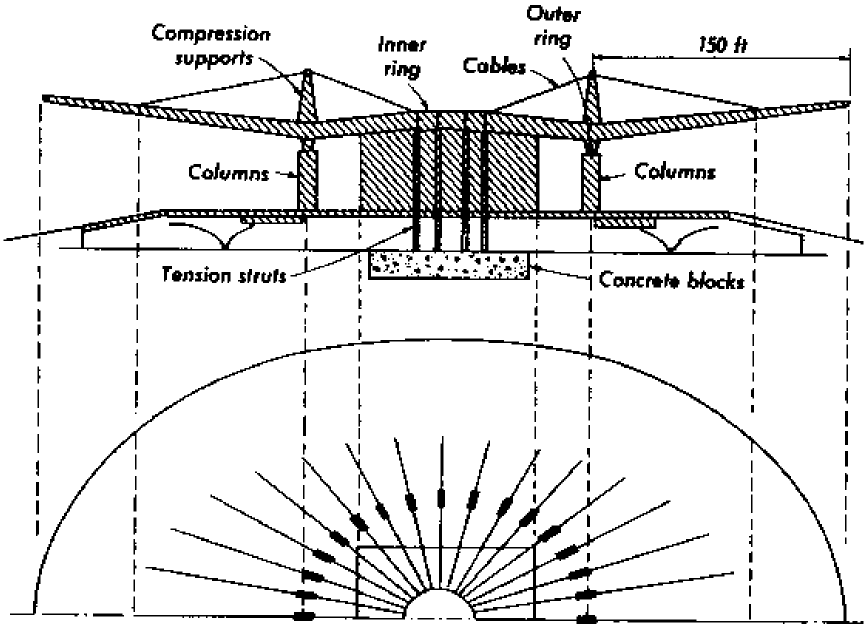

Figure 1.15 Cable-suspended elliptical roof of Terminal 3 at Kennedy Airport in New York. (Mario Salvadori and Robert Heller, *Structure in Architecture*: The Building of Buildings, © 1986 3rd ed., p. 121. Reprinted by permission of Prentice-Hall, Inc., Englewood Cliffs, New Jersey.)

anchored at an inner ring. The outer ring is supported by columns. The inner ring is anchored to tension members connected to a massive concrete foundation in the ground.

1.3.3 Thin-Plate Structures

Examples of this type of structure are liquid storage tanks (such as elevated water tanks, storage bins and silos) and shell roofs.

1.4 STRUCTURAL SHAPES

Common structural shapes available in the American Institute of Steel Construction Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD) Manuals (AISC, 1995, 1998) are shown in Fig. 1.16. The most widely used type of section is wide-flange shape or W shape [Fig. 1.16(a)]. The inner surface of the flange of a W shape has very little slope (from 0 to 5 percent). In contrast, the inner flange surfaces of American Standard or S shapes [Fig. 1.16(b)] and American standard channels or C shapes [Fig. 1.16(c)] have a slope of roughly 16 2/3 percent (or 2 in 12 inches).

Shapes W, S, and C are designated by two numbers such as W14x132, S24x121, and C15x50. The first number in W and S designations indicates the "nominal" (not the actual) depth of cross section. The actual depth of W14x132 is d = 14.66 in. and the actual depth of S24x121 is 24.50 in. On the other hand, the first number in the C designation indicates the actual depth of the cross section. The second number in W, S, and C designations indicates the weight (in pounds) per unit length (in feet) of the shape.

The web thickness of W and S shapes in the AISCM is always smaller than the flange thickness. The HP (bearing pile) shapes are similar to W shapes, but their web thickness is the same as their flange thickness. The increased web thickness of HP shapes is necessary to resist the impact of pile driving. The M shapes are miscellaneous shapes

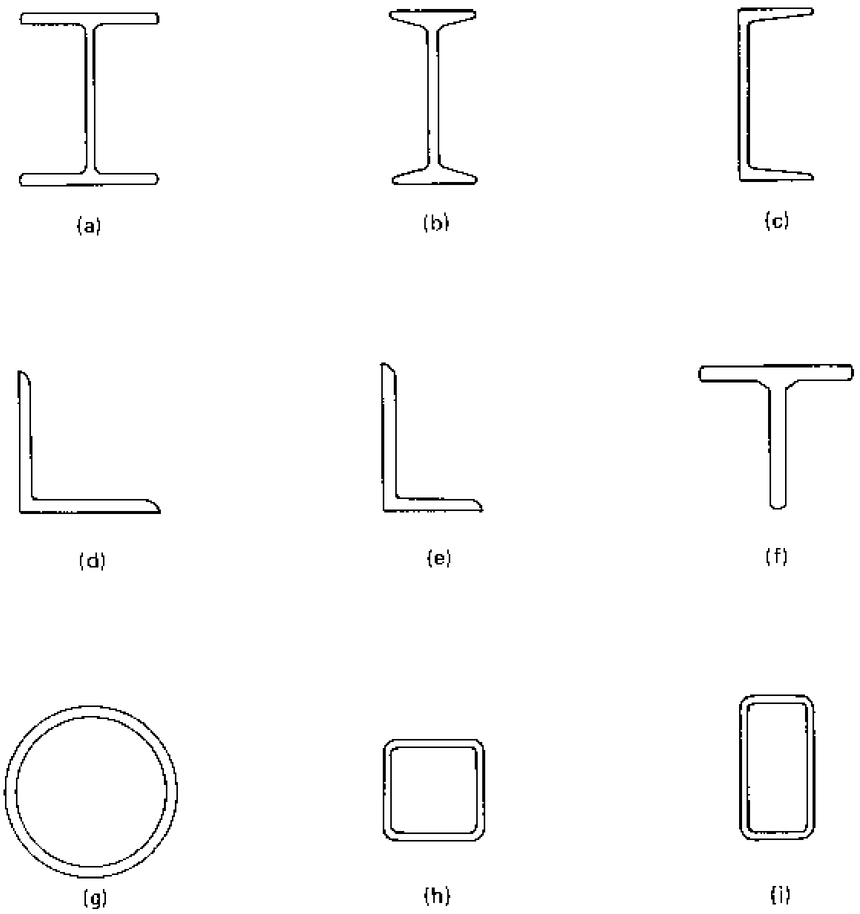

Figure 1.16 Structural shapes

that cannot be classified as W, S, or HP shapes. Similarly, MC designates miscellaneous channel. In the AISC ASD Manual (AISC, 1995), one finds 295 W shapes, 31 S shapes, 15 HP shapes, 10 M shapes, 29 C shapes, and 35 MC shapes.

Figure 1.16(d) shows an equal-legs angle, and Fig. 1.16(e) shows an unequal-legs angle. An angle is designated by three numbers. For example, L9x4x5/8 indicates an angle with leg size of 9 and 4 inches and thickness of 5/8 in. There are 137 angles in the AISC ASD Manual (AISC, 1995).

Figure 1.16(f) shows a Tee (or simply T) shape. The T shapes are made by splitting the webs of W, S, and M shapes. For example, a WT7x66 is a T shape obtained by splitting W14x132 shape.

Other shapes sometimes used in steel structures are pipes [Fig. 1.16(g)], square tubes [Fig. 1.16(h)], and rectangular tubes [Fig. 1.16(i)]. Dimensions and geometric properties of different structural shapes have been tabulated in Part One of the ASD and LRFD Manuals (AISC, 1995, 1998). The flange thickness in these tables for S, C, M and MC shapes is the average flange thickness.

1.5 DESIGN APPROACHES

1.5.1 Allowable Stress Design

The majority of steel structures are designed according to the elastic design philosophy. In this approach, known as *allowable stress design* or *working stress design* method, the designer estimates the working loads that the structure must safely carry during its lifetime. Then, the structure subjected to the working loads is proportioned so that nowhere in the structure the stress exceeds the allowable stress. The allowable stress for steel is usually defined in terms of the yield stress F_y as follows:

$$F_{all} = \frac{F_y}{\text{F.S.}} \tag{1.1}$$

where F.S. is the factor of safety; this factor is always greater than one.

The AISC ASD specifications are based on this approach (AISC, 1995).

1.5.2 Plastic Design

In this approach, also known as *limit design*, *collapse design*, or *ultimate strength design*, the working loads are multiplied by factors greater than unity called *load factors* (L.F.). Then the structure is

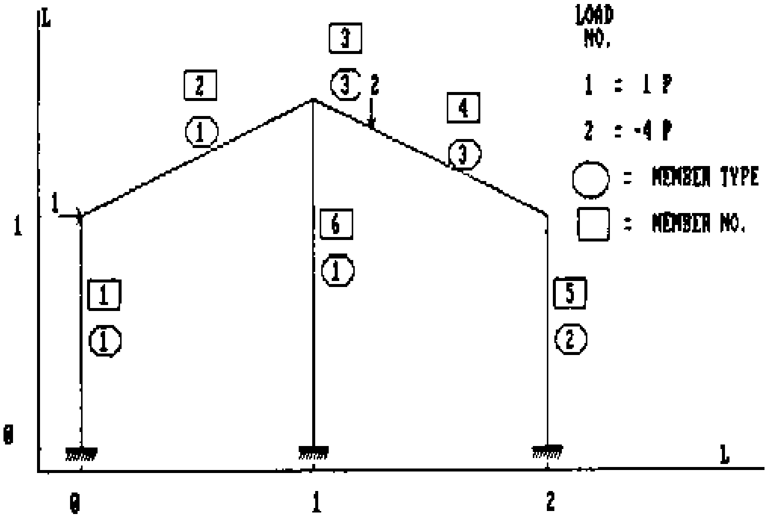

Figure 1.17

proportioned so that its ultimate load capacity is at least equal to the factored working load. Thus, in this approach it is necessary to perform a limit or collapse analysis based on postulated failure mechanisms. For example, the failure mechanism of the frame shown in Fig. 1.17 is shown in Fig. 1.18 (Adeli and Chyou, 1986)

Although this approach has not been used extensively in the past, it is only logical to observe its more widespread used in the future. This approach takes into account the ductility and plastic reserve strength of steel structures. For certain types of structures, it results in a more efficient structure. Chapter N of the AISC ASD code (AISC, 1995) covers the plastic design specifications.

The factor of safety in the allowable stress design approach and load factor in the plastic design approach, sometimes called "factor of ignorance", take into account different uncertainties involved in load and

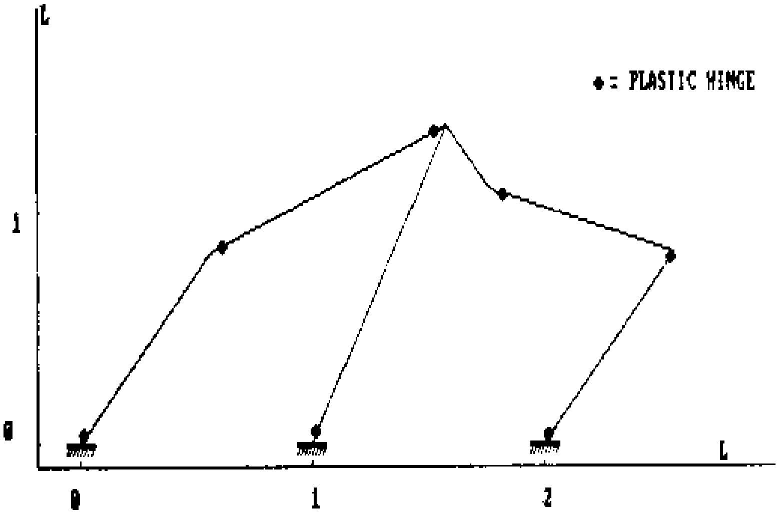

Figure 1.18

strength evaluation. Some of these uncertainties are as follows:

1. In order to make structure amenable to analysis, idealized conditions are usually assumed.
2. There are tolerances in the evaluation of material strength. Also, material strength may change during the lifetime of a structure due to corrosion or fatigue, the exact prediction of which is not possible.
3. Loading on the structure can only roughly be estimated. This is especially true for geophysical loading such as those due to earthquakes and winds.
4. Stresses developed in the structure during fabrication and erection are often ignored.
5. Residual stresses are developed in the structural shape during the manufacturing process. They may result from (Salmon and Johnson, 1980):
    a. Uneven cooling after hot-rolling of structural shapes. (In W shapes, for example, after hot-rolling, the flange tips and the mid-portion of the web are thinner and consequently cool faster than the other portions, this results in compressive residual stresses at flange tips and mid-depth of the web, and tensile residual stresses at the intersections of the flange and web.)
    b. Cold bending or cambering during fabrication.
    c. Welding.
    d. Punching of holes.
    e. Cutting operations during fabrication.
6. There are tolerances in dimensions of the structural members.

1.5.3 Load and Resistance Factor Design

Another design code has been published by AISC for structural steel buildings which is based on limit stated philosophy and called Load and Resistance Factor Design (LRFD) specifications (AISC, 1998). It is based on the limit states of strength and serviceability combined with a first-order probability analysis for determination of load and resistance factors. The advantage of such a probability-based design is more uniform and consistent approach to the load and strength evaluation.

LRFD is a method of designing structural components so that no applicable limit state is exceeded when the component is subjected to all the appropriate load combination. In this approach equations of the following type should be satisfied (Cooper et al., 1978, Yura et al., 1978).

Factored strength of a component ≥ factored nominal load effect

or

$$\phi R_n \geq \lambda_A \sum_{i=1}^{n_L} \lambda_i Q_i \tag{1.2}$$

where R_n is a nominal resistance and Q_i is the load effect i. Greek letter ϕ is a resistance factor for taking into account the uncertainties in the calculation of resistance. λ_i is the load factor for the load effect i for taking into account the uncertainties in determining the load effect i, λ_A is an analysis factor for taking into account the uncertainties of the structural analysis, and n_L is the number of load combinations.

It should be noted that for design according to the AISC LRFD code, the designer does not have to be involved in the application of the probability theory and statistics for determining the load and resistance factors. These factors have already been estimated for different conditions and are included in the LRFD code.