12.12 SADDLE-SHELL ACTION

The behavior of the most common saddle shell, the hyperbolic paraboloid, shows the dependence of its structural action on support conditions. This surface was shown in Section 12.4 to be generated either by a vertical parabola with downward curvature sliding on a vertical parabola with upward curvature or by a straight-line segment the ends of which slide on two straight-line segments askew in space (see Figs. 12.16 and 12.17).

When the shell is supported on two parabolic arches or stiffeners, it transfers the load to the support by shears (Figure 12.58). Its action is similar to that of a stiffener-supported barrel, but the upward curvature in the longitudinal direction gives the shell additional strength, particularly against buckling. If the shell tends to buckle, the parabolas with downward curvature tend to buckle downward and flatten; this deformation is resisted by the parabolas with upward curvature, since their tension stabilizes the compressed parabolas.

If the paraboloid reaches the ground, its intersections with the ground consist of two outward-curved boundaries (the two branches of a hyperbola), so that the area covered by the shell has two straight sides and two curved sides (see Figure 12.16). When the paraboloid does not reach the ground, the shell may be supported at its four corners, but the end stiffeners must then carry the vertical and horizontal loads on the shell as arches. Bending distortions, similar to those at the curved boundaries of barrels, are encountered at the intersection of the paraboloid with the stiffeners. Their values and the width of the disturbed band are of the same magnitude as those of the paraboloid.

It is easy to see that even when the hyperbolic paraboloid is supported on its straight-line generatrices, the directions of principal stress coincide with those of principal curvature, that is, the parabolas (Figure 12.7). Since the straight lines have no curvature, no cable or arch stresses can be developed along these lines by the shell that can only react in shear. Hence, tension along the upward parabolas must combine with compression along the downward parabolas to create a state of pure shear along the straight lines (Figure 12.59). The load is thus carried to the supporting boundaries by pure shear directed along the straight lines, and these shears accumulate along the boundary supports. The supporting elements are usually cables, but less than being loaded vertically, are loaded by the shears accumulating along their lengths, so that they behave like compresive struts (Figure 12.60) or tensile bars (Figure 12.61), except for the action of their own dead load.
In this case, the outer horizontal stiffeners are tensioned by the accumulation of shears, and the tension in one of them balances the tension in the stiffener of the adjoining segment. The roof is inwardly equilibrated and its boundary stiffeners are its tie-rods (Figure 12.61). In either combination, the weight of the horizontal stiffeners (the boundary stiffeners for the umbrella, and the inner cross for the gabled roof) is carried in part by the shell itself; the shell, therefore, develops some bending stresses. The roof supported at the corners may also buckle in certain regions where it is flat, that is, where no appreciable curvature increases the buckling resistance of the thin shell.

A variety of other combinations of hyperbolic paraboloids may be used to cover areas of varied shape. Each one of these combinations must be carefully analyzed to determine whether it is inwardly balanced. A typically unbalanced combination, consisting of two segments, is used as a northlight roof (see Figure 12.31). The thrust of the inclined boundary struts is equilibrated by a tie-rod, but the two compressive forces are not equilibrated in the inclined strut common to the two segments are not equilibrated (Figure 12.62). Hence, the horizontal boundary beam is acted upon by a concentrated load at midspan and is subject to bending. The shell behavior, in this case, is a bending action in horizontal projection, with compression at the upper-boundary segments and tension at the lower horizontal boundary, due to a load concentrated at midspan; the two tensile forces at the outer lateral boundaries of the shell equilibrate the concentrated load, just like beam reactions (Figure 12.62).

Hyperbolic paraboloids with a very high rise have also been used as roof elements; others as almost vertical elements. In such cases, their behavior is totally different from that of the shallow paraboloids considered above and similar to the behavior of a thin plate loaded in its own plane. It may be often approximated by the behavior of deep beams (see Section 7.1).

The different mechanisms by which the hyperbolic paraboloid carries loads are one more example of the dependence of shell behavior on support conditions.

12.13 STRESSES IN SCALLOPED AND OTHER TYPES OF SHELLS

Interesting shell shapes may be obtained by adding local curvatures to a simple surface, according to structural needs or aesthetic requirements. An ellipsoid intersected by a horizontal plane is a smooth surface covering an elliptic area. In order to enhance its appearance and stiffness its boundary, it may be scalloped, thus creating local curvatures that point toward the interior of the shell (see Figure 12.27). The introduction of such curvatures substantially changes the shell behavior. The smooth ellipsoid carries load by stresses similar to those encountered in a rotational shell: compression along the meridians and compression or tension along the elliptical parallels. The introduction of local curvatures destroys the parallels' stiffness. The bending of the shell is similar to that of a series of arches with variable cross section, hinged at the supports and at the crown, where the lack of curvature reduces the bending resistance of the shell (see Section 8.5). Curved domes of large diameter have also been built by scalloping a spherical surface with the purpose of increasing its buckling resistance (see Figure 12.28). The waving of the perimeter boundary rather than vertical loads, retaining walls loaded by the thrust of the earth may be built as vertical cylinders or cones (Figure 12.64). Cylindrical tanks containing water, oil, or other liquids are built as thin steel or prestressed concrete shells to withstand the outward pressure of the liquid. Dams with heights of over 1000 feet (300m), have been built with curvatures both in the vertical and the horizontal sections and a thickness of only a few feet (meters) (Figure 12.65). The ratio of radius of curvature to thickness in these dams is of the order of 400 or more, so that such shells are to be considered thin even though they may be several feet thick at their foot.

12.14 THIN-SHELL FORMWORK

The aesthetic, architectural, and structural possibilities of thin-shell construction are practically unlimited; only their cost may at times hamper their diffusion. The problem of building expensive curved forms on which to pour reinforced-concrete shells has made their use uneconomical in countries with high labor costs. The solution of the forming problem has been attacked in a variety of ways. Roofing consisting of identical shell elements are built by pouring an element on a moveable form, which is then lowered, shifted, and reused again to pour the next. Prefabrication of shell elements and their erection on a light scaffold has become a standard procedure, which eliminates expensive forms. The joining of the prefabricated elements must be carried out with great care, and often requires welding the reinforcing bars sticking out of the elements and pouring the joints at a later date. Some of the exceptional thin-shell structures designed by Neuf were built by this procedure.

Prefabrication of shells by elements is often used in conjunction with posttensioning. It was observed in previous sections that certain types of shells developed tensile stresses over large portions of their area. These tensile stresses may be eliminated by introducing in the thickness of the shell steel cables or tendons which are tensioned after the concrete has hardened (posttensioned); they compress the shell so as to eliminate altogether the tensile stresses due to the loads (see Section 3.5). The thin shells of cylindrical tanks are usually posttensioned by spirals of steel cables, thus guaranteeing the waterproofness of the tanks under all conditions. Northlight shells with large spans may also be posttensioned and pre-stressed tendons so as to eliminate tensile stresses in the shell portions below the inclined neutral axis (see Figure 12.63).

The use of a concrete gun allows the spraying of layers of concrete (Shokrete or Gunite) on the reinforcing bars with a minor amount of formwork. The bars are supported on simple scaffolding, in which the insulation panels constitute the actual surface against which the concrete is shot. Finally, balloons have been used as inflated forms to support the steel and against which to spray the concrete or, as in the Eifel process, as an inflatable form to lift both the reinforcing steel and the concrete (see Section 11.3).
12.15 RETICULATED DOMES

Space frames (see Section 10.9) behave like structures in which the distributed material of a plate and, hence the stresses in it, are lumped in the frame bars. Their load carrying mechanisms can be understood by analogy with those of plates. Similarly, the behavior of reticulated or ribbed domes may be grasped readily by analogy, with that of thin-shell domes.

For example, if the material in a spherical dome is lumped in bars lying along the two families of curves created by the meridians and the parallels (Figure 12.66), the meridional bars will absorb the compressive membrane stress from the top to the boundary of the dome, and the parallel bars will be in tension or in compression depending on the value of the opening angle (see Section 12.7). The continuous thin shell has been transformed into a discrete structure of the same shape.

In order to use bars that do not differ too much in area, the meridional bars often branch off from the top of the dome down. The steel structures of some of the largest domes in the United States, like those of the 642-foot-diameter (195 m) Astrodome in Houston, Texas (Figure 12.67), and of the 681-foot-diameter (207 m) Louisiana Superdome in New Orleans, Louisiana, follow this scheme (Figure 12.68).

The meridional-parallel lattice presents the disadvantage of using bars of different lengths. Other schemes have been invented to avoid this difficulty and simplify the connections between the bars, but since the surface of a sphere cannot be entirely covered by a lattice of regular polygons, the bars of a spherical dome cannot all be identical. In the geodesic domes, designed by Buckminster Fuller, triangles and hexagons are used to obtain a subdivision in terms of bars of equal length (Figure 12.69). An irregular triangular lattice of ribs is used instead in the classical Schwedler dome (Figure 12.70), which presents the design advantages of being statically determinate (see Section 4.2), and in the Zeiss-Dwyig dome (Figure 12.71).

Ribbed domes spanning up to three or four hundred feet have been built in Europe with standard connectors, using bars made out of steel pipes on a triangular grid. These structures require only 4 to 5 pounds of steel per square foot (192-239 Pa), as against 10 to 20 pounds per square foot (479-958 Pa) for domes built of rolled sections, but are very sensitive to local behavior. In their context this behavior is analogous to that of a thin shell in which the steel of the bars, uniformly distributed over the dome surface, has a much smaller apparent modulus of elasticity and a smaller apparent density. In other words, these domes behave like a thin shell made out of a thicker, spongier material, just as space frames do in relation to plates. As the buckling load is proportional to the elastic modulus of the material, the lower apparent modulus lowers the buckling capacity of these latticed domes.

The analogy between latticed roofs and thin shells persists for roofs of other than spherical shape. Latticed barrel roofs can be built with bars parallel and perpendicular to the barrel axis (Figure 12.72) and can be supported on end diaphragms. The forces in their discrete bars are, to a good approximation, the resultants of the stresses in the barrel shell area "contributory" to the bars, and can be obtained by considering the barrel as a beam (see Section 12.9). When the barrel roof springs from the ground and, hence, behaves like a series of parallel arches, a skew grid of discrete arches leads to a so-called Lamella roof (see Figure 12.73), which is used to span hundreds of feet in wood, concrete or steel (see Section 8.5). Zerman built domed Lamella roofs spanning up to 300 feet (91 m), by erecting prefabricated steel-pipe sections with standard bolted connections. The structure of these roofs, also, weighs not more than 5 pounds per square foot (239 Pa).

Regular and irregular doubly curved surfaces called gridshells consist of a latticework of light members that follow the shell surface curvature. These have been constructed of wood or steel, such as the Savill Building outside of London, England (Figure 12.74). Such structures have shell-like behavior, but forces are restricted to axial tension and compression along the member length. They have been sheathed in wood, glass and fabric membranes. One of the first such structures was by Frei Otto, the 1974 Muhlhalle in Mannheim, Germany. It was constructed with the gridwork horizontally on the ground, and then hoisted into final position as a complete unit, and then secured at the foundation. Steel cables were used diagonally to laterally stiffen the structure.
Saddle roofs, in the shape of hyperbolic paraboloids made out of plywood supported by aluminum pipes (oriented along the straight line generatrices of these surfaces), were used by Candela as units to plug the openings between the steel truss arches of the 1968 Olympic Stadium in Mexico City (Figure 12.75). The bending disturbances at the shell boundary (see Section 12.8) correspond to local bending of the bars of the laminated roof near its supports. All other bars are only stressed by the axial forces corresponding to the membrane shell stresses (compressive or tensile) and hence use the material to great efficiency. As is the case for the thickness of thin shells, limitations in the bar sizes may be due to the requirements of buckling.

The erection of laminated domes is facilitated by the Binistar system patented by the Italian architect Dante Brini and first designed in 1985 for a triangulated dome on a hexagonal base of 131-foot diameter (40 m) in Bari, Italy. The "telescopic" steel bars of a Binistar dome consist of outer pipes and two inner rods sliding in each pipe (Figure 12.76). Short lugs, set into the inner rods in correspondence with holes in the outer pipes, can be pushed out radially by springs. The inner rods are attached to spherical connectors that allow their ends to rotate freely. To erect the dome the pipe-rods bars are mounted, over a deflated plastic balloon, into a flat trapezoidal lattice, anchored to a base ring of reinforced concrete. As the balloon is inflated to the required dome shape, the inner rods slide inside the pipes until the lugs are shot into the pipe holes by the springs. The lugs, thus engaged into the pipes, transform the telescopic bars into rigid bars and freeze the shape of the triangulated dome (Figure 12.77). The balloon is then attached to the external laminated dome and becomes its permanent surface. As Geiger's tensile domes (see Section 6.2), Binistars may be considered membrane balloon supported by reticulated domes rather than by air pressure.

In countries with high material costs and low labor costs, thin shells of prestressed, prefabricated or prestressed concrete are usually economical in comparison to similar laminated steel domes. The opposite is usually true in countries with high labor costs and low material costs. The same considerations of relative materials and labor costs explain the favor encountered by the laminated pipe domes outside the United States. While rolled-section domes can be built in sections, using a single center support for their erection, lightweight pipe domes require more complicated scaffolds and greater amounts of labor. Since, moreover, in the United States steel pipe is, pound per pound, more expensive than rolled steel sections, the fabrication and erection costs of pipe domes do not, in general, make these structures competitive.

**KEY IDEAS DEVELOPED IN THIS CHAPTER**

- Membranes, shells, and domes are form-resistant structures that can carry loads because they are curved surfaces.
- If a membrane that carries loads by tension stresses is turned upside down, it becomes a shell or a dome carrying loads by compression just like an upside down cable has the shape of an arch. The materials of shells, domes, and arches must be able to withstand compressive stresses.
CHAPTER TWELVE: Thin Shells and Reticulated Domes

- As was the case for membranes, shells and domes have curvatures at every point, with maximum and minimum values in principal directions.
- Surfaces can be developable, that is, they may be flattened without cutting them, or undevelopable that have to be cut in order to be flattened.
- Curvatures may be downward (positive), dome or cylindrical shaped or upward (negative), bowl shaped at very point on the surface.
- Others may have an upward curvature in one principal direction and downward in the other.
- Curved surfaces may be generated by rotating a plane curve around an axis, by moving a line along a curve or a curve along a line.
- Shallow circular domes have arch action along both meridians and parallels. They are in compression.
- High-rise domes are compressed along meridians, but their parallels are mostly in tension. Just like for arches their base needs buttressing to prevent it from moving outward. As a consequence bending stresses are also present along with shear.
- Cylindrical shells have only one curvature. Along the curved surface, it has arch action, but a long cylinder actually acts similarly to a beam.
- Shedde or flat surfaces have positive curvatures along one principal direction and negative in the other. Along the positive curvatures, arch action and compression prevails while the negative cable action supports the arches in tension.

Reticulated domes are similar to space frames and are constructed of triangulated or hexagonal elements.

QUESTIONS AND EXERCISES

1. Find a large orange preferably with a thick skin. Cut it in half and remove the fruit without damaging the pulpy or the skin. You now have a high-rise dome. Place it on a sheet of paper and draw the outline of the base. Gently apply pressure to the top and observe the outward movement of the base. What kinds of stresses are present in the circumferential directions along the parallels? How about along the meridians?

2. Carefully cut a circular hole, about ⅛ inches (12 mm) in diameter, on top of the dome. Put a drinking glass with the open end on the dome and apply pressure. What kinds of stresses are present along the meridians and along the parallels at the edge of the hole?

3. Take the second half of the orange skin and cut a cap, about ⅛ in. tall (12 mm), to create a shallow dome. Repeat the previous two experiments.

4. Cut construction paper into a 2 foot (60 cm) long by 1 foot (30 cm) wide piece. Bend the paper into a long cylindrical barrel. Support it along the long edges with a few books. Gently apply point loads to the barrel and observe the type of deformations. Now put a small pillow on the barrel to distribute the load and put a book on the pillow. Again observe the deformations. What type of stresses can explain these deformations?

5. If you have access to "Tinker Toys" or a child's construction set, construct a Geodesic Dome similar to one shown in Figure 12.69.

FURTHER READING


CHAPTER THIRTEEN

STRUCTURAL FAILURES

13.1 HISTORICAL FAILURES

The moment humanity started erecting structures, structures began to fail. In prehistoric times, corbelled domed houses of stone were built all along a band running uninterrupted through Asia Minor, Greece, Crete, Sardinia, southern France, and England, but only a few survive intact after 2,000 to 5,000 years. The pyramids of Egypt stand alone among the Seven Wonders of the Ancient World: the Colossus of Rhodes, an over-100-foot-high (30.5 m) bronze statue to the sun god Helios, vanished, and an earthquake destroyed the second longest lived, the lighthouse of Pharaohs at Alexandria (Egypt), said by some historians to be 200 feet (61m), by others 600 feet (183 m) tall.

Collapses and failures plagued some of the most magnificent monuments of history. The dome of Saint Sophia in Constantinople began showing signs of weakness during construction, and tradition has it that it was saved by the intervention of the emperor Justinian, who urged the architect Anthemius to complete one of its main arches that was falling, because "when it rests upon itself it will no longer need the uprights under it." Yet, after two earthquakes, the eastern arch collapsed in 557, the western in 985, and the eastern for the second time in 1346; the dome was made stable only in 1847 by placing iron chains around the base. The masterpiece of Gothic architecture, the Cathedral of Saint Pierre at Beauvais, France, had the main vaults of the choir collapse in 1224, only 12 years after it was completed, and its 502-foot tower collapsed 13 days after its erection in 1573. The great domes of Renaissance Italy, those of Santa Maria degli Angeli in Florence and of Saint Peter's in Rome, cracked, and few, if any, of the monuments of the past still exist do not show signs of weakness today.

We accept such failures because of the lack of sound structural knowledge by the builders of the times, but we are shocked and puzzled by the rash of collapses of some of our large buildings and bridges. How does our scientific age explain and contain such disasters? If we are interested in learning why most of our buildings stand up, we also want to know why some fall down.

13.2 MAIN CAUSES OF STRUCTURAL FAILURE

In the following sections, the main causes of contemporary failures are discussed using the intuitive approach of this book, but it must be pointed out at the outset that such failures occur in a small percentage of our increasingly large, tall, and complex structures. As should be expected, our record is far superior to that of our predecessors, although the number of exceptional structures being erected grows yearly all over the world.

In the last analysis all structural failures are caused by human error, that is, are due to lack of knowledge or judgment, but for purposes of classification they may be attributed to deficiencies in design, in fabrication and erection, or in materials. One could put failures due to unexpected events, natural or human-made, in a separate category, but even these are due, most of the time, to our incomplete knowledge or lack of caution. Different causes often conspire and result in collapses. Technical investigations should try to explain them so that their repeated occurrence might be avoided, but should waste no time assigning blame, since this is the province of the law. Yet, it might interest the reader to learn the professional and legal consequences of engineering and architectural failures, which is the subject of Section 13.6.

13.3 FAULTS IN STRUCTURAL DESIGN

13.3.1 SOURCES

Design deficiencies may occur for a number of reasons: pure and simple mistakes in calculations, undetected errors in computer inputs, incomplete or mistaken interpretation of building codes, unfamiliarity with dynamic or "burst loads," defective detailing, lack of redundancy, lack of coordination between the members of the construction team, and many more.

Good design can only result from experience, since design is learned through many years of continued practice. Hence, the young practitioner should not be expected to assess the relative importance of the many facets of design and should not be given responsibility for the design of
CHAPTER THIRTEEN: Structural Failures

essential structural components. The organization of a professional office requires clear assignment of tasks and careful supervision at all levels. Engineers, architects, and other members of the construction team. Unfortunately, under the time and economic pressures of our culture, such supervision and coordination are not always given the consideration they deserve and often become causes of failure.

13.3 Computational Errors

Contrary to what the layperson may believe, errors in calculations seldom cause deficient design. If flagrant, they are easily caught; if minor, they may be unimportant. The advent of the computer has immensely refined structural calculations (see Section 4.4) at the cost of drowning the designer in a sea of numbers. The experienced professional, aware of the common occurrence of input errors, will never accept a computer result without checking it long hand with a simplified formula or against personal experience and past results.

It must not be construed from the above that errors in calculations are never responsible for failures. In certain cases, the mistakes in design are so flagrant that their most probable cause may be shown or inferred to be an unacceptably numerical mistake. According to the investigation by the National Bureau of Standards of the failure of the pedestrian bridges at the Hyatt Regency Hotel, Figure 13.1 in Kansas City, Missouri, in 1982, in which 114 people died and 216 were injured, the collapse was due to obvious errors in design (Figure 13.1), compounded by alterations proposed by the fabricators that were not reviewed by the engineers. The reader of the NBS report may be justified in surmising that a numerical mistake, accompanied by a lack of judgment on the part of an inexperienced designer, may have been the source of such a catastrophic error.

13.3.3 Building Codes

A thorough knowledge of building codes (see Chapter 2) requires years of patient study, careful interpretation, and frequent updating. Codes are modified every few years to take into account the accumulation of new knowledge and to clarify the meaning of their requirements. The design professional is confronted not only with a variety of codes applicable to different types of structures and materials, but also with the proliferation of codes by states, counties, and cities.

Most of the structural requirements in local codes are derived from or refer to a few widely accepted national codes. The multiple regional codes of the past are now largely consolidated into the International Building Code (somewhat misnamed as it is a document mainly used in the United States), but there is still sufficient local code variation to demand careful scrutiny, particularly because of their legal implications (see Section 13.6). The situation differs from that existing in other countries, where unique national codes govern at all levels. In any case, it must be remembered that the building codes make minimum recommendations and are only a starting point. Any departure from their requirements does not exempt the designer from technical responsibilities.

13.3.4 Dynamic Loads: Wind and Seismic Forces

Fields in rapid evolution, like those of dynamic design for wind and earthquake loads, have requirements seldom tightly updated in building codes because they demand time-consuming debates before being approved. Moreover, the practice of dealing with dynamic conditions is not common in architecture because most engineers who do not deal specifically with statics. Hence, the designer practitioner must not only be aware of the possibility of dynamic loads, but must keep abreast of the current literature so as to make safe and economic use of the latest information on these dangerous conditions. For example, in the design of the Toronto City Hall Building (Figure 13.2), wind forces that were widely determined by wind tunnel tests indicated an unexpected wind load due to the channeling of the wind through the gap between the two buildings. Similarly, the heavy steel girders of one of our first missile silos were torn off their hinges and collapsed when first used, for lack of consideration on the part of the designers of the dynamic forces due to their accelerated motion.

The wind-bracing systems of a building must meet conditions not only of strength but also of stiffness to minimize discomfort to its occupants and avoid damage to its curtain wall (see Section 2.4). In the Hancock Tower of Boston, Massachusetts, the excessive flexibility of the frame was one of the main causes of the dislodgement of a high percentage of the curtain wall glass panels, and required the damping of its wind-induced motion by means of two dynamic dampers (see Section 2.6 and Figure 2.27) at a high cost and with long construction delays.

The knowledge of earthquake motions and their influence on building is a recent phenomenon. Detailed maps of earthquake intensities in the United States (Figure 2.25) are available today and must be consulted by all members of the construction team, although the technicalities of earthquake design are left to the specialist. It must be realized that data on earthquakes are gathered unceasingly all over the world and that if a result of these investigations earthquake requirements become continuously more demanding.

With each new seismic event, more is learned about the nature of earthquakes and how buildings respond to them. Buildings are being built with a higher degree of seismic resistance, and areas of the United States—thought until recently to be free of higher-intensity earthquakes—have proven to be subjected to them. An unexpected earthquake in Virginia in 2011, for instance, caused structural damage to the Washington Monument and National Cathedral in Washington, D.C. Even in areas of minor earthquake activity, the codes now take seismic design into account, under other considerations, in the detailing requirements of reinforced-concrete frames.

It is comforting to realize that our improved techniques of earthquake design allow high-rise buildings, like the
Torre Latino-Americana in Mexico City, to survive undamaged in earthquakes that caused the failure of low buildings of older vintage. High-rise buildings of contemporary construction that are designed for earthquake forces can in fact fare well in seismic events. The current highest building in Mexico, the 55-story Torre Mayor, is designed with hydraulic dampers in diamond-shaped frames to absorb seismic energy. It survived a large 7.6 magnitude quake in 2003 without damage. Because steel and aluminum are flexible materials that are elastic under smaller loads and deform plastically under larger ones (see Chapter 3), structures with metal frames may suffer large deformations under earthquake conditions but seldom break.

Rigid structures, though, built of brick and mortar, as well as some poorly reinforced-concrete buildings and bridges, are more susceptible to earthquake damage. California’s Loma Prieta 6.9 magnitude earthquake destroyed the double-decked Cypress Street Viaduct (formerly the Nimitz Freeway) in West Oakland in 1989 (Figure 13.3). A lack of sufficient confining reinforcement to prevent shear failure at the junction of the beams and columns contributed to the collapse. The structure was designed and constructed to codes and standards of its day some 35 years earlier, but little was truly understood about seismic action in that era. In a more recent example, during the sixth most powerful earthquake on record, the 2010 Chilean event, a 15-story apartment building toppled over and broke in two. Concrete failure between the basement and first floor has been implicated in the collapse (Figure 13.4).

13.3.5 Thermal Movements and Stress

The same caution must be used in dealing with thermal differences, the main source of locked-in stresses (see Section 2.5). Because of their dependence on a large number of parameters, the investigation of thermal stresses is advised by most codes without specific recommendations on temperature differences or methods of analysis. As a consequence, structural failures may often be attributed to neglected thermal conditions. In a concrete dome for one of the first nuclear reactors built in the United States, the stresses due to a "thermal explosion" (a sudden, large temperature rise on the interior of the reactor), originally ignored, required a modification of the design. Most of the cracks in the brick masonry veneers of air-conditioned buildings, particularly in veneers of prefabricated panels, are due to the rigid connections between the exposed masonry and the interior structure that do not allow for the thermal expansions and contractions of the masonry due to daily and seasonal temperature variations of the outdoor air (Figure 13.5a). Similarly, cracking in the partitions of high-rise buildings occurs when the climate produces large differences in temperature between southern and northern facades. The thermal bending of such buildings (Figure 13.5b) is the source of high shears in the partitions, acting as webs of vertical cantilevered beams, and of cracks due to the tensile component of these shears (see Sections 5.3 and 7.1).

Unbalanced forces due to thermal differential significantly contributed to the collapse of Terminal 2E at Charles de Gaulle Airport in Paris in 2004 (Figure 13.6), just eleven months after its inauguration. The unique cylindrical structure was designed as a perforated "squashed" cylindrical shell of concrete, with an exterior glazing held away from the concrete by posts and tensioning cables. On a morning with an unusually rapid drop in temperature, shrinkage of the surrounding cables caused the posts to put pressure on the concrete. The shell was designed for these pressures; however,
CHAPTER THIRTEEN Structural Failures

at one location a ramp penetrated the shell, thus creating a discontinuity. The imbalance of forces on the shell, coupled with construction deficiencies, precipitated its complete collapse in this area.

Fires are always dangerous to the occupants of a building, but they may also cause structural failures even for non-combustible materials. While steel and aluminum can withstand moderate heat, at a 1000 degrees Fahrenheit (550°C) Aluminum can melt and steel will begin to lose its elasticity and become severely deformed (Figure 13.12). For the same reason, "rebars" in reinforced concrete must be protected with a sufficient layer of cement to provide fire resistance. Building codes (see Chapter 2) pay particular attention to this requirement.

13.3.6 Foundations

The mechanics of soils has become a reputable field of science through the investigations of the last one hundred years: It is one of the most sensitive facets of foundation design. No structural engineer will assume the responsibility of such design without a soil investigation performed by specialized laboratories that specify the bearing capacity of the soil, the maximum differential settlements to be expected, and the type of foundation most appropriate for a chosen location (see Section 4.3). Settlement differentials are of particular interest to the designer since they may cause large cracks in curtain walls and partitions and weaken the structure (13.7). A substantial number of failures in slabs on grade and curtain walls are due to foundation deficiencies related to changes in underground water levels and new construction. The National Theatre in Mexico City, founded on a soil consisting of a mixture of sand and water, subsided many feet at first when its weight squeezed the water out of the sand, but then was pushed integrally upward when a number of high-rise buildings were erected around it. In this case, the evenness of the displacements and the monolithic reinforced-concrete structure prevented damage to the building.

A dramatic foundation failure occurred in China when rain-soaked soil was excavated beside the pilings of a multi-story apartment structure (Figure 13.8). The combination of removing soil that was holding back lateral pressure from the building foundation, plus additional lateral pressure from the excavated soil piled on top of earth behind the building, led to the collapse. With the extra load on earth that was already weakened from rain saturation, lateral displacement caused the entire building to slide forward and snap the foundation pilings, collapsing the entire building into the excavated area.

13.3.7 Structural Redundancy

Some of the most spectacular failures take place in structures lacking redundancy. Static indeterminacy is a necessary but not a sufficient condition of redundancy (see Section 4.2). For example, if a uniformly loaded fixed beam (Figure 13.9a) were to fail at one of its support sections, indeterminacy might theoretically allow it to carry the load as a cantilever (Figure 13.9b). But if it was originally designed to resist the largest bending moments in a fixed beam, which usually occur at its ends (see Section 7.5), the stresses at the cantilever support would increase by a factor of 3, and collapse the beam. If the misjudgment of the same beam failed and it carried the load as two cantilevers half its length (Figure 13.9c), the stresses at the supports of these cantilevers would be 50 percent above the maximum original stresses and dangerously near collapse, since, on an average, safety factors take into consideration a load increase of the order of 67 percent.

Redundancy thus requires providing additional structural resistance in case of failure, particularly when the loads on a structure are supported by a large number of identical elements, as in space frames (see Section 10.9). The roof of the Kansas City (Missouri) Arena was hung by means of 48 steel hangers from three external pipe frames (Figure 13.10). When one of the hanger connections failed during an exceptional rainfall, the load supported by that hanger was transferred to adhering ones that should have been able to carry the additional load. Since they were unable to do so, a chain reaction developed that collapsed a large portion of the roof, luckily without loss of life because the arena was unoccupied at the time. A similar chain reaction of failing hangers was said to cause the collapse of a suspended ceiling at the Jersey City terminal of the PATH railroad, causing two deaths. It must be noted that tensile hangers work in simple tension, distributing the load uniformly over their cross section (see Section 5.1); hence, they do not have the reserve of strength derived from stress redistribution at a section, typical of bending action (see Section 9.3).

Ultimately, a chain is only as strong as its weakest link. The load hanging from a chain is channelled from its support along a single line as a tension member. In the case of a hanging weight, this "single-load path" (SLP) is a straight line. If a link fails, the weight falls. A chain hanging form two supports may support a distributed load along a curved line (Figure 6.6a and 6.7a). The curved chain is also an SLP structure.

The Silver Bridge connecting Point Pleasant, West Virginia, and Gallipolis, Ohio, was such a chain suspension bridge (Figure 13.11). The links were made of eyebars pinned to each other at their ends (Figure 13.11b). Stress corrosion combined with metal fatigue broke one of these links, causing an unstoppable progressive collapse of the bridge (Figure 13.11c).

Modern suspension bridges are supported by multistrand cables, the strands sharing the load in this case as a "multiple-load path" (MLP) or "redundant" structure. If some strands fail, their share of the load is redistributed to other strands. As a consequence, the structure survives until all strands are broken.

A simple frame consisting of four bars forms an unstable parallelogram and moves under the application of a load (Figure 13.12a). With the addition of a diagonal bar, the truss is stabilized and becomes an SLP. The loads are channelled from one support along the bars to the other support (Figure 13.12b). Should any one of the bars fail, however, the truss becomes unstable or will collapse.

By adding one more diagonal, the truss will be an MLP or a redundant structure (Figure 13.12c). In this case, the
13.3.8 Buckling and Redundancy

Redundancy is particularly needed whenever a chain reaction may be started by buckling. It was noted in Section 5.2 that buckling is a sudden phenomenon that occurs without warning and is usually followed by failure. The particular case of buckling involving torsion, called lateral buckling, presents the same dangerous characteristics. The published results of the structural investigations on the collapse of the space frame roof of the Hartford Civic Center in Hartford, Connecticut (spanning 300 by 270 feet [91 x 82 m] and supported on four pylons (Figure 13.13)), details the failure mechanism. The collapse of the roof during a heavy snowstorm was started by the buckling failures of an embossed compression bar near the boundary of the frame. When the load supported by this bar was transferred to the adjoining bars, these in turn failed in buckling, precipitating a chain reaction collapse. In a matter of minutes, the 1,500 tons of steel of the entire roof dropped to the floor of the empty hockey rink—mercifully, just hours after an event had finished, thus avoiding what could have been a tragic loss of hundreds or even thousands of lives. It is not clear whether axial or torsional buckling was responsible for this chain reaction collapse.

In another example, a reticulated dome of light pipe bars (see Section 12.13), over 300 feet (91 m) in diameter, collapsed in Bucharest (Romania) in the 1960s under an exceptional snowfall. The collapse was not due so much to the heavy snow load as to the weak buckling capacity of the dome, one of whose sections "snapped through," that is, caved in. The dome was rebuilt, American style, with rolled section bars, giving it greater buckling capacity.

13.3.9 Connections

As shown in Sections 9.1 and 9.2, the states of stress in the connections of structural elements are particularly complex. Hence, it will come as no surprise to the reader that they are often the source of failures. Codes and manuals give criteria for the design of connections based not only on refined mathematical stress analyses but also on series of tests on models or full-size connections. Since, because of their complexity, failures of connections occur more frequently than those of structural members, connections are designed with high coefficients of safety. This is particularly true of space frames whose connectors may have complex shapes and stress distributions (see Figure 10.48).

13.3.10 Unexpected Loads

Failures have often been attributed to unexpected natural or human-made phenomena, such as hurricanes, snowstorms, tsunamis, and fires or explosions. These do indeed bring havoc to all kinds of small and large structures, but only when the structures were not designed to resist them, when forces reach new highs (a common occurrence), or when engineers forget the lessons of the past. The collapse of the Tacoma Narrows Bridge in Washington State in 1940 (see Section 2.6) was identical with that of the Wheeling suspension bridge over the Ohio River in 1854 (at the time the world's longest suspension bridge), but the memory of this disaster, vividly described in the local press, had vanished. In these cases, wind-induced aerodynamic oscillations of gradually increasing amplitudes caused the collapse. In the case of the Tacoma Narrows Bridge, the great flexibility of the structure and the lack of appropriate wind-bracing contributed to the failure. Descriptions of similar collapses of wooden suspension bridges had appeared in the British press early in the nineteenth century, but had also been forgotten, allowing the repetition of past disasters.

13.3.11 Ponding

In more recent collapses of large structures, the loads due to natural forces did not reach the design values recommended by the codes and must be attributed to faulty design. For example, the collapse of large flat roofs subjected to rainstorms is often due to a chain reaction phenomenon called "ponding." The flat roof caves under the weight of the rain, acquiring a concave shape that does not allow the water to reach the level of the drains. The increasing weight of rain accelerates the caving of the roof, which is transformed into a pond containing an increasing load of water that eventually may collapse the roof (Figure 13.14). Appropriate roof slope to promote runoff and appropriate distribution of drains at correct levels prevent ponding. Flat-roofed buildings are also typically designed with overflow scuppers at their roof perimeter as another safeguard against ponding.

13.3.12 Other Causes of Failure

The human-made causes of failures, such as fires and explosions, can be as damaging as those due to natural phenomena. A high-rise industrial building in New York City had two
13.4 FAULTS IN COORDINATION AND SUPERVISION

The construction team of a large modern structure consists of a large number of specialists: the architect in charge and his/ her team of designers; the project architect; landscape architect; and interior designers; the structural engineer and team; the mechanical engineer and team; the electrical engineer and team; the plumbing engineers and team; the soils engineer; the environmental designer; the contractor and team of specialty subcontractors; the owners’ representatives; and a variety of other experts on certain walls, roofing, costing, scheduling, and materials. Last but not least is the construction manager, who takes responsibility for the feasibility of the design and the coordination of all trades, acts as the owner’s agent, and directs the execution of the job with the help of the scheduling experts, responsible for the sequence of operations and the delivery of materials. At the frequent meetings on the progress of construction, especially on larger projects, it is not unusual to have thirty or more experts around the table, fighting their personal battles in the interest of the expediency of erection of the building.

The smooth coordination between all the facets of construction requires cooperative communication between the members of the team and employers. The architect’s involvement is crucial in the basic requirements of the other trades on the part of the representatives of each trade. The architect, uniquely positioned as team leader, must blend the needs of all the trades and capable of settling the disputes arising from the mutually conflicting requirements of the consultants. In this role he or she is helped by the construction manager, whose job demands practical experience and a capability for conciliation and compromise within narrow limits of acceptability. The construction documents (drawings, specifications, and contracts) are of the utmost importance in avoiding misunderstandings and mistaken interpretations that only too often lead to delays and failures. Whether a building contractor is chosen at the start of the design or by bid on contract documents, he or she must be given documents that leave little doubt as to the meaning of what is to be delivered. The time and effort spent on these documents avoids failures and costly litigation (see Section 13.6).

In spite of the interest and goodwill of the members of the team, unavoidable conflicts of opinion and clarifications or change orders will be issued. In this phase of construction, competent supervision becomes essential on the part of the job’s superintendent, the coordinator of the contractor’s crews, and of the architect’s and/or owner’s representative. It is their duty to ensure that the building is erected in strict accordance with the construction documents, even if minor inaccuracies from the design may be the cause of catastrophic failures. Besides any tendency on the part of the contractor to cut corners in order to maintain the schedule or to enhance profit, involuntary human error may be responsible for such mistakes. In this context, it suffices to mention the consequences of the misplacement of the reinforcement in concrete structures, the deviation from a prescribed sequence of operations in the case of steel connections, the tightening of connections that should allow thermal movements, or the improper curing of poured concrete. These are a few among the many examples that must be carefully monitored to eliminate possible failures.

Faulty rebars placement was to blame for halting construction in 2008 of what was to have been the 49-story Harborside at the Venetian Las Vegas, Nevada. Work on the structure was stopped at the 28th floor when it was discovered that 14 floors had been constructed with reinforcing steel improperly installed in the columns. Weidlinger Engineers determined that the building (now with structurally deficient columns) would be unsafe during a major earthquake, even at a reduced height of 28 stories. After years of little progress, the owner and contractor, an agreement was reached and the building was completely torn down in 2015.

Many of the structural elements, such as steel beams and columns or prefabricated beams, columns, and slabs of concrete (often prestressed), come to the site from a fabrication shop. A check of these elements for dimensional compliance with the design is necessary to make sure that any discrepancies are within prescribed tolerances, lest the structural engineer has to work with the support of the site in the dangerous locked-in stresses (see Section 2.5).

The architect and the engineers must realize that there is a wide range of materials, specifications, and its execution in the field, and that the job of the contractor may appear less creative than theirs but is certainly as complex and demanding. A harmonious collaboration between the “legislative” and “executive” branches of the construction team is a necessity if failures are to be avoided.

13.5 FAULTS IN MATERIALS

13.5.1 Steel

The most essential properties of structural steel are strength and ductility, i.e., the capacity of plastic flow, which guarantees a reduction of stress concentrations and a reserve of strength. These properties depend on the type of steel, the chemical composition, the thermal treatment, and the rolling of steel shapes during manufacture, and should be checked for each batch of steel used by obtaining the corresponding data from the manufacturer: They are essential not only for beams and columns but also for nuts, bolts, and other connection components. Since the 1960s, engineers have been aware of the dangerous phenomenon of delamination due to the welding of thick-steel sections: Unless their steel is adequately treated thermally, any parts of sections thicker than 2 or 3 inches (51 to 76 mm) have a tendency to separate into thin laminae, greatly reducing their strength. Research stemming from these disasters emphasized the importance of checking the treatment of steel used in large structures.

No operation involves greater risks in steel construction than the welding of connections. The welding materials, the type of flame, the temperature, and the speed used in this operation influence dramatically the strength of the connections. Since a number of failures of steel structures are attributable to welds, most codes require welders to be duly trained and certified.

13.5.2 Concrete

Concrete properties depend on its composition, that is, the ratio of cement, sand, stone, and water used in the mixture. Besides the type of cement and the strength of the stone, one must estimate the granulometry of sand and stone, i.e., the size distribution, so as to guarantee that the voids between the stones are filled by the sand grains and that those sand grains by the cement. Both stone and sand must be carefully washed to eliminate impurities. The water–cement ratio is the most important factor in determining the concrete strength: A low ratio increases the strength but makes more difficult the pouring and vibrating of the concrete; a high ratio weakens the concrete. In all projects of any importance, the concrete mixture is designed by a concrete laboratory and its compressive strength is checked after 7 and 28 days by testing cylinders of concrete taken from daily concrete batches. The following unusual episode illustrates the need of vigilance in concrete supervision. During the construction of one of the most famous air terminals in the United States, it was noticed that concrete batches gave strength results at all (some except those reachable on the site in the early afternoon. When all investigations failed to discover the reason for this anomaly, the design engineer decided to follow the track of the concrete mix at a lunchtime, which usually reached the site an hour later. He thus discovered that the track drivers, before stopping for lunch, added a resolving dummy of the trucks so as to prevent the setting of the concrete during their leisurely meal. The concrete reaching the site had the right consistency but a reduced strength due to the higher water–cement ratio resulting from the addition of the dummy.

Particular care must be exercised in locating the steel reinforcing bars in the concrete, but, moreover, the steel bars must be covered with zinc or epoxy whenever severely corrosive salts may percolate through the concrete, as often happens in the slabs of garages when snow-melting salts are used on roads. The architect should be aware of the dangers due to faults in the two most commonly used structural materials, but only the metallurgist and the concrete specialist can assist in the determination of their properties and thus prevent the corresponding failures.