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Special Structures: Past, Present, and Future

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Abstract: *Special structures* are landmarks and testimonials to the achievements of the structural engineering profession. They are true three-dimensional representations of our equilibrium equations and affirmations of our analytical techniques, design standards and construction practices. They include many types of structures, such as: space frames or grids; cable-and-strut and tensegrity; air-supported or air-inflated; self-erecting and deployable; cable net; tension membrane; lightweight geodesic domes; folded plates; and thin shells. This work celebrates the ASCE's sesquicentennial by providing a historical perspective on how *special structures* have evolved, their state-of-practice in the dawn of the 21st century, and a projection of their potential trends and evolution into the future.

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Introduction

The sesquicentennial of ASCE provides a great opportunity to assess the role of *special structures* in the structural engineering profession with a historical perspective on how these types of structures have evolved, their state-of-practice in the dawn of the 21st century, and a projection of their potential trends and evolution into the future.

From the Georgia Dome in Atlanta, the Livestock Pavilion in Raleigh, and Madison Square Garden in New York to the Olympic Stadium in Munich, and from the Pontiac Silverdome in Michigan to the Sydney Opera House in Australia and the Haj terminal in Saudi Arabia, *special structures* are landmarks and testimonials to the achievements of the structural engineering profession (Fig. 1). They are what makes us most interested in and proud of our profession and what binds us together with the architects and architectural and construction engineers in appreciation of the art of structural design and construction. They are true 3D representations of our static and dynamic equilibrium equations, and affirmations of our analytical techniques, design standards, and construction practices. Each special structure is a prototype by itself, rather than a duplicate produced on an assembly line.

Yet, it is not easy to qualify the term *special structure*, as perhaps loosely used in this paper. For the purpose of this paper, "special structure" refers to innovative long-span structural systems, primarily roofs and enclosures to house human activities. More specifically, they include many types of structures, such as: space frames or grids; cable-and-strut and tensegrity; air-supported or air-inflated; self-erecting and deployable; cable net; tension membrane; geodesic domes; folded plates; and thin shells. We exclude tall buildings and long-span bridges, both of which are addressed separately.

Thin shells and tension membranes are considered form-resistant structures, as they resist loads by virtue of their shape. Neither will function if flat, and both carry loads predominantly through in-plane stresses rather than by bending, granted that thin shells bend as well as compress. Other special structures resist loads mostly in flexure. The typical flat space frame or grid supported by columns or walls acts primarily in flexure even though its individual members behave axially (Fig. 2). Depending on the loads, top and bottom chords will be in tension or compression, similar to the flanges of an I-beam, and the diagonals (acting in tension or compression) carry the shear, much like the web of an I-beam (Cuoco 1997).

Structures that resist loads by bending may be categorized using span-to-depth ratio. For example, this ratio is about 20 for a typical wood joist; whereas, a timber beam with larger loads will have a ratio around 12. Steel bar joists usually run about 24, as do many wide-flange beams and reinforced concrete waffle slabs. Space frame is remarkable with as high a ratio as 35–40. On the other hand, span-to-depth ratio has no significance for form-resistant structures. A more useful measure for an arch is the span-to-rise or the span-to-thickness ratio (Fig. 3). Efficient arches have span-to-rise ratios of 2–3 and span-to-thickness ratios of about 40. Larger span-to-rise ratios generally result in larger axial forces and require a smaller span-to-thickness ratio.

Tensile structures are more efficient than arches because they do not buckle. The Verrazano Narrows Bridge in New York, for example, has a span-to-sag ratio of 10 and a span-to-thickness

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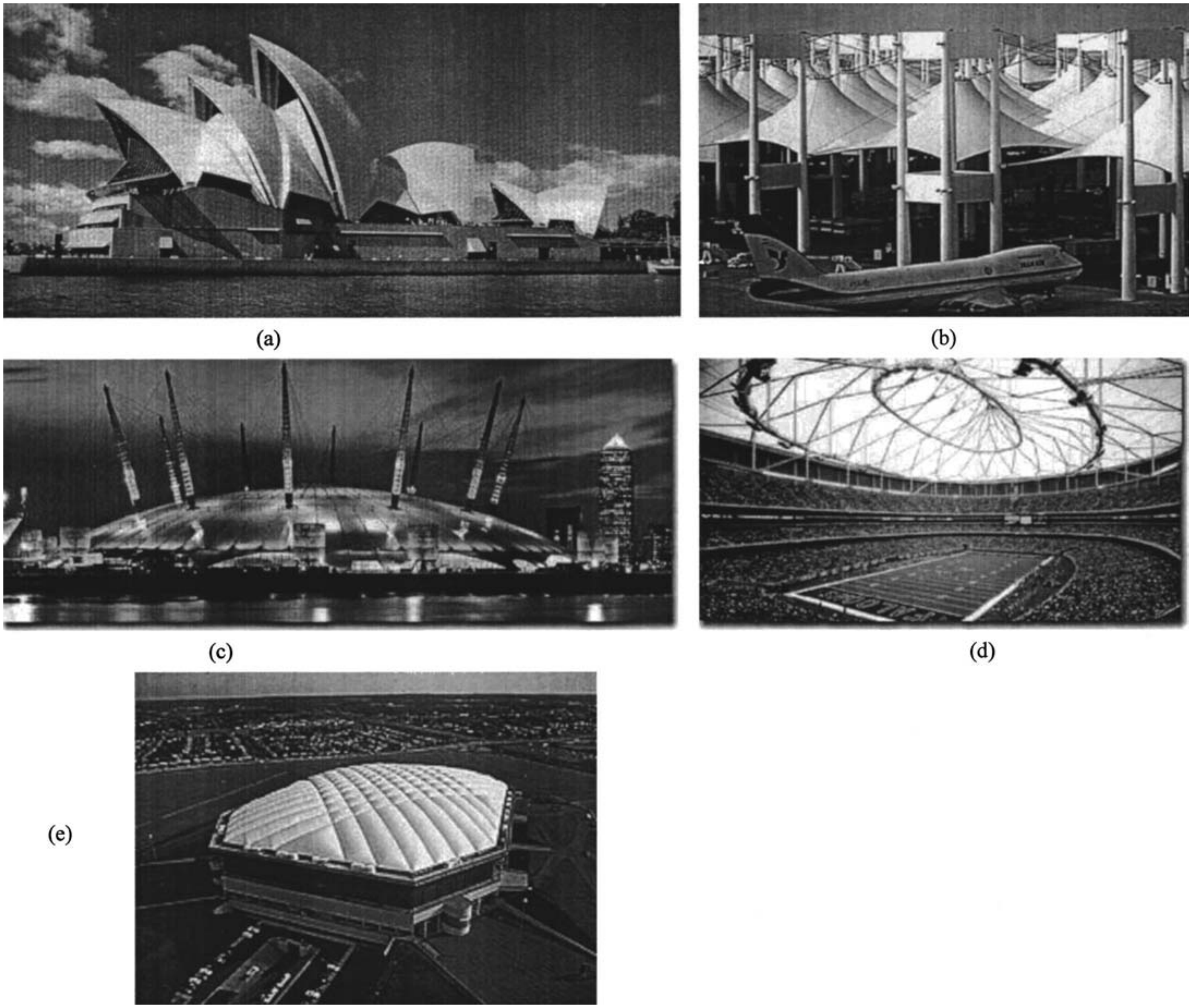

Fig. 1. (a) Sydney Opera House; (b) Haj Terminal (Geiger Engineers); (c) Millenium Dome (Birdair); (d) Georgia Dome (Geiger Engineers); (e) Pontiac Silver Dome (Geiger Engineers)

ratio of about 400 for its cables (Madugula 2002). With span-to-thickness ratios near 300,000, large air-supported membranes are undoubtedly the most efficient structures, although one may argue that they have a zero span length continuously supported on columns of air.

Although efficient in material use, tensile structures generate large pull forces at their base. For example, the concrete compression ring encircling the Georgia Dome is 7.9 m (26 ft) across to take such large forces. While large horizontal thrusts are also present in low-rise arches, they can be more easily resisted than the large ''pulls'' developed by tension membranes and cable domes.

In this work, special structures are categorized into three groups based on the method by which they resist the loads: com-

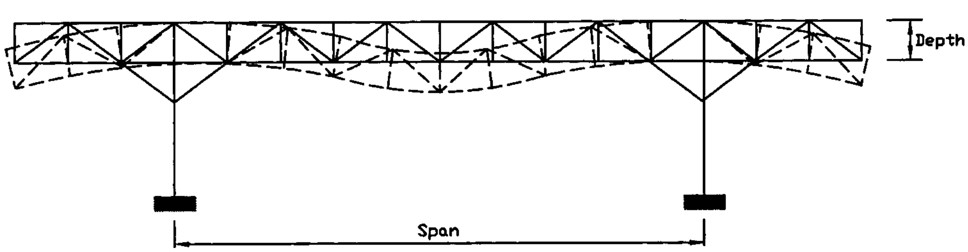

Fig. 2. Space frame acting in bending

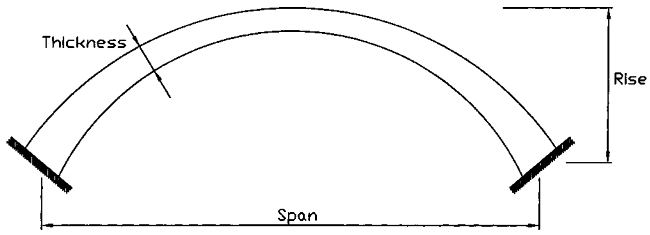

Fig. 3. Ratios in a simple arch

pression structures (shells); tension structures (tension fabric, air-supported, air-inflated, tensegrity, and cable-net structures); and tension/compression reticulated structures (space grids or frames and geodesic domes). The emphasis is placed on the history of special structures (how they came about and why they are unique), their structural behavior (how they withstand the loads), their advantages and disadvantages, their methods of analysis and design, and the noteworthy structures of each type around the world.

Compression Structures: Shells

Brief History of Thin Shells

Architectural thin shells discussed in this work are a modern development. The domes and cylindrical shaped structures of antiquity and the Middle Ages were thick and could only resist compressive loads. The first modern architectural shell is generally credited to that built by the Zeiss optical company in Austria in the 1920s. In the United States, shells were extensively studied by the aircraft industry in the 1930s. In 1933, Donnell, an aeronautical engineer, formulated the general equations for cylindrical shells, including both bending and membrane actions.

While Eduardo Torroja of Spain is credited for the systematic engineering study of *architectural* shells in the 1930s, it was the work of Felix Candela in Mexico that ignited the sudden surge of popularity of shells in the 1950s. His shells were spectacular both for appearance and for bold engineering. At a time when a 75-mm (3 in.) thick shell was considered daring, Candela built a hyperbolic paraboloid shell with less than 16 mm (5/8 in.) thickness for the Cosmic Ray Pavilion at Ciudad Univ. in Mexico City. Fig. 4 and 5 show examples of how Candela skillfully created different shells from the same hyperbolic paraboloid geometry.

It was an article in *Progressive Architecture* (1955) on the shells of Candela that launched the modern shell era by attracting the attention of architects. Figs. 6–12 show some of the remarkable early shells for the air terminal in St. Louis, MIT auditorium in Boston, TWA terminal in New York, Sports Palace in Italy, and Exhibition Hall in Paris.

The latter, designed by Esquillan, is one of the engineering marvels of the 20th century, whose statistics define its uniqueness. In plan, it is an equilateral triangle; 218 m (715 ft) long on a side

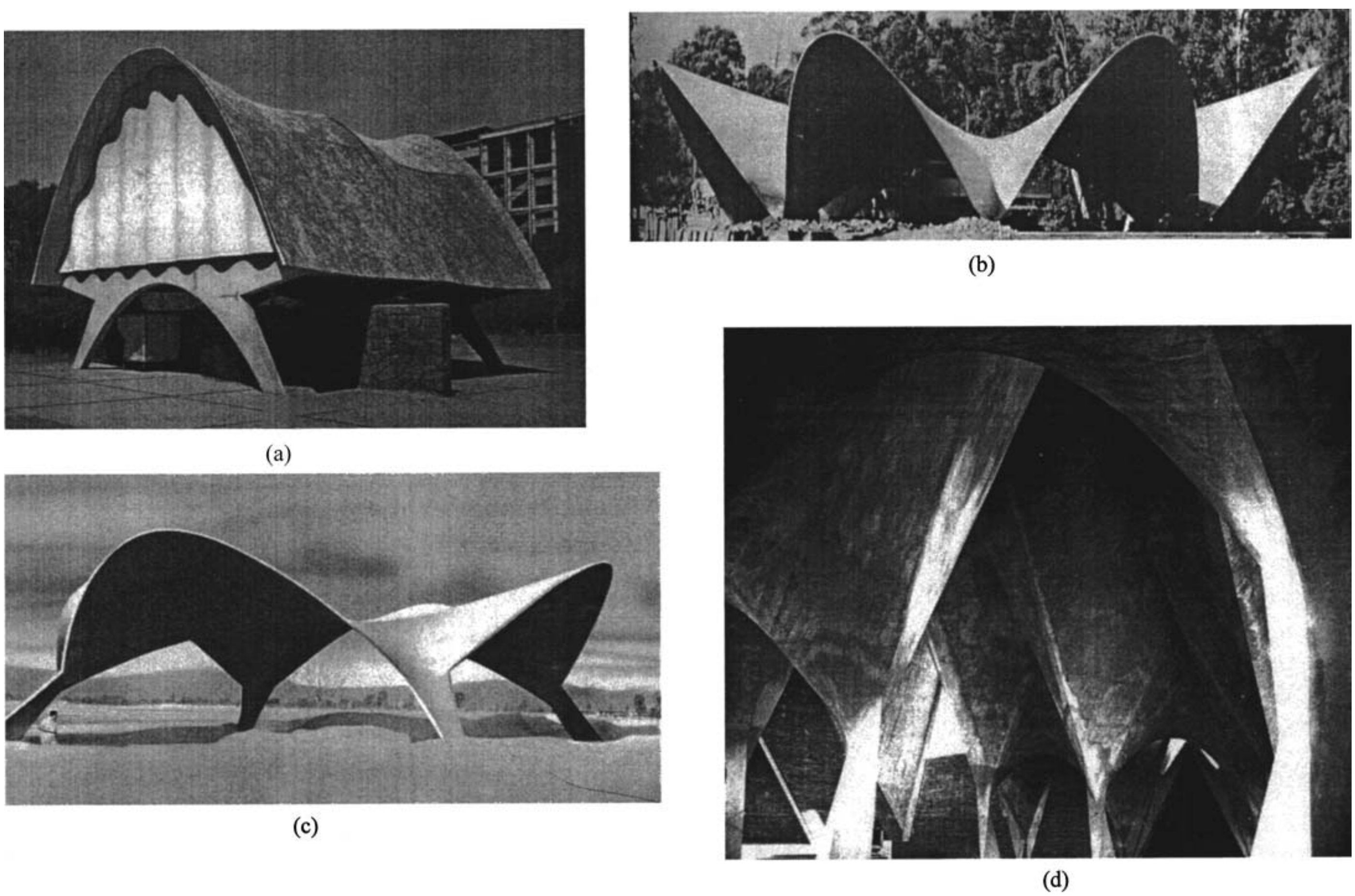

Fig. 4. (a) 16-mm (5/8 in.) thick hyperbolic paraboloid shell of Cosmic Ray Pavilion, Mexico City, (Faber 1963); (b) 61-mm (2.4 in.) thick, 30 m (100 ft) span hyperbolic paraboloid shell of a restaurant, Mexico City, (Faber 1963); (c) 61-mm (2.4 in.) thick, 20 m (64 ft) span hyperbolic paraboloid shell of a sales office, Guadalajara, Mexico (Faber 1963); (d) 38-mm (1.5 in.) thick hyperbolic paraboloid shell of a church tilted on edge, Narvarte, Mexico (Faber 1963).

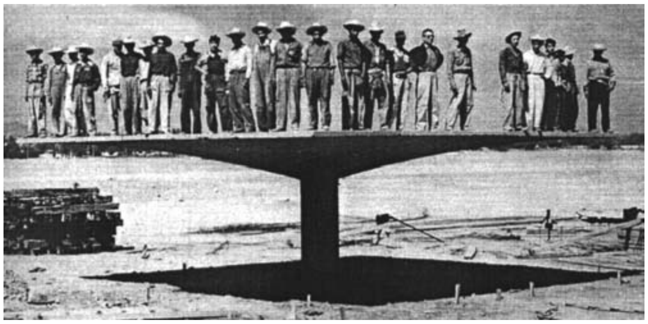

Fig. 5. 83 mm (3.25 in.) thick, 8 m (26 ft) square, 0.6 m (2 ft) rise hyperbolic paraboloid shell, Vallejo, Mexico (Faber 1963)

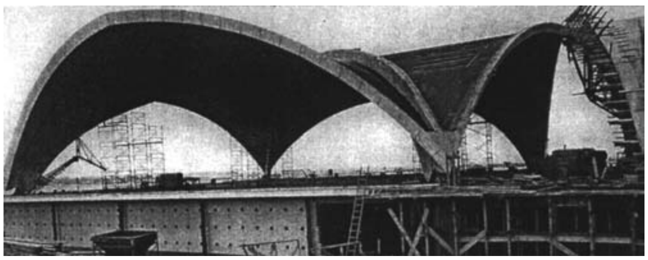

Fig. 6. 114–216 mm (4.5–8.5 in.) thick, 37 m (120 ft) span intersecting cylindrical shells with ribs at edges and at groin, air terminal, St. Louis, Robert and Schaefer Engineers (Joedicke 1963)

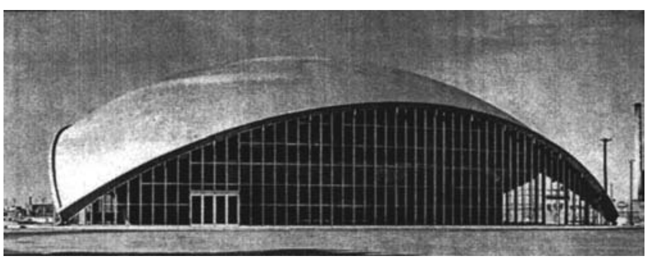

Fig. 7. 89 mm (3.5 in.) thick, 49 m (160 ft) span spherical shell with hinges at abutments, MIT Auditorium, Boston, Amman and Whitney Engineers (Joedicke 1963)

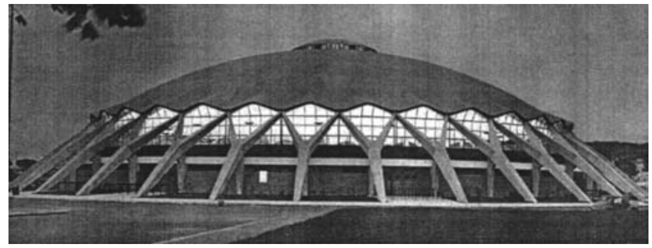

Fig. 9. 59 m (192 ft) span Sports Palace, Rome, Nervi Engineer (Joedicke 1963)

with a rise of 48.8 m (160 ft). If a circular dome were circumscribed about the equilateral triangle, it would be 251 m (825 ft) in diameter, beyond the span of any building today. However, it would be simpler to design the full dome than a triangular piece cut out of it due to the instability of the free edge, which creates a potential for buckling. This problem was prevented using a two-layer shell spread apart by vertical walls. The overall depth of the system is 1.9 m (6.25 ft) at the crown and 2.7 m (9 ft) at the spring line. The thickness of each layer is 60 mm (2.38 in.) at the crown and 120 mm (4.75 in.) at the spring line. The interior precast cross walls are 59 mm (2.33 in.) thick. Thus, no part of this immense shell, the largest ever built, is thicker than 120 mm (4.75 in.). Remarkably, this was all in 1957, before the use of computers.

The history of civil engineering has repeatedly shown that new types of structures have been built before their behavior was fully understood. This is as true of modern shells as it was of the cathedrals of the Middle Ages; that rational explanation for their success was found only after the persistence of their existence forced their recognition. The early practitioners had to rely on intuition and courage rather than on written knowledge. It can be certain that a great deal of anxiety took place before Candela built his 16 mm (5/8 in.) thick Cosmic Ray Pavilion. One could only imagine the fortitude it took to remove the forms from under the 218 m (715 ft) span of Esquillan's Exhibition Hall.

The structures of the skilled practitioners of the art, such as Candela, Esquillan, Torroja, and Nervi are distinguished by their elegance in minimizing the thickness, eliminating the ribs, and avoiding the hinges at the abutments. It suffices to note that the span-to-thickness ratio of a well-designed shell is considerably larger than that of an eggshell. Ribs are used to carry the shear forces from the shell to the abutments and to prevent buckling of

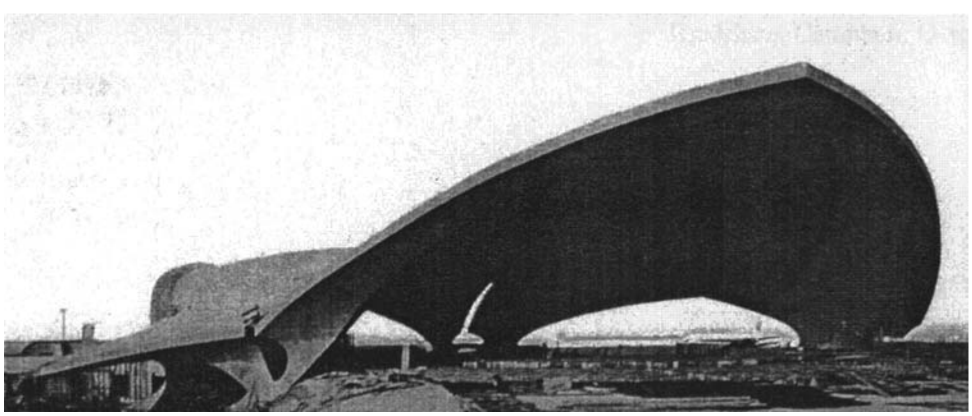

Fig. 8. 178–610 mm (7–24 in.) thick, 37 m (120 ft) span arbitrary shape shell with ribs at edges and in interior, MIT Auditorium, Boston, Amman and Whitney Engineers (Joedicke 1963)

Fig. 10. Inside view of Sports Palace, Rome (coffered ceiling made by pouring a thin layer of concrete over precast concrete boxes which then become part of the structure) (Joedicke 1963)

the edges. However, it is possible to eliminate many ribs by making the shell itself act as the rib. This requires skilled analysis, which test the knowledge and nerve of the designer. Hinges between the shell and the abutments reduce the capacity of the structure and serve only to simplify the design.

Shells and Geometry

There is no type of structure that has so intimate a relationship with space geometry as a shell. There are two important yet simple geometrical observations in shells: all constructed shells are only fragments of a more complete geometrical shape; and all geometric surfaces would either continue to infinity or intersect with themselves.

The shell in Fig. 13 is derived from two intersecting tori or ''doughnut'' shapes, as shown in Fig. 14. In this example, the doughnut has a pinhole-sized hole. The shell of Fig. 13 is shown

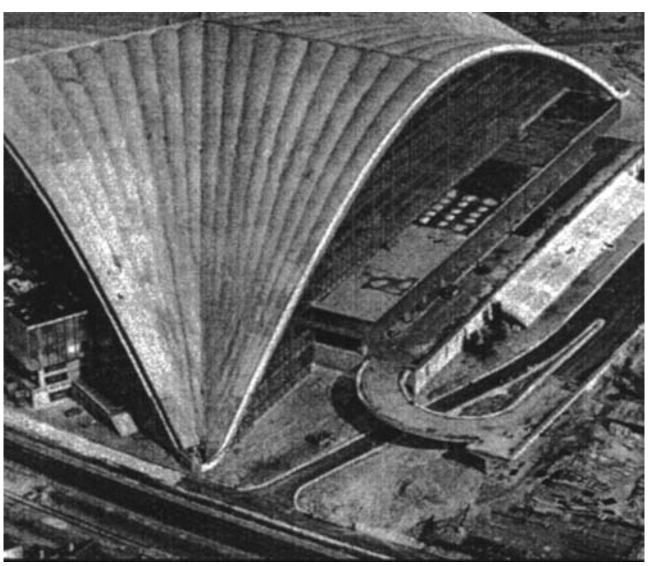

Fig. 11. 218 m (715 ft) side span exhibition hall, Paris, Esquillan Engineer (note size of other buildings in background for scale) (Joedicke 1963)

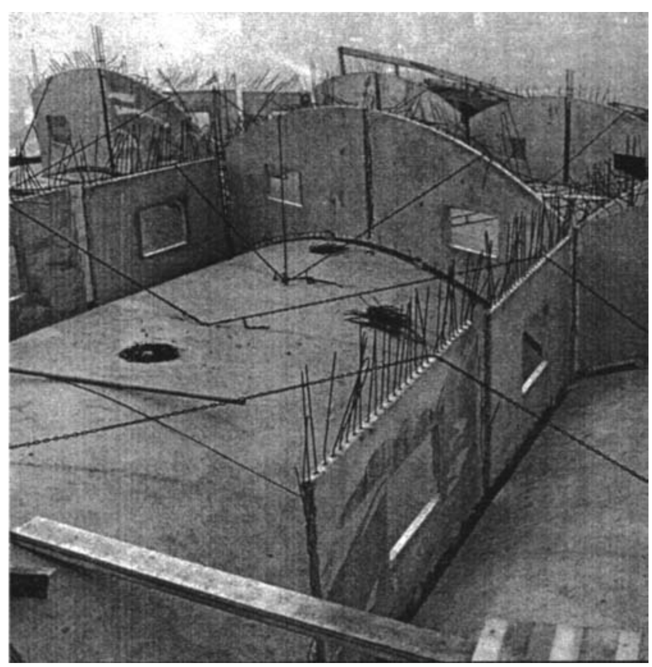

Fig. 12. Interior precast partition walls inside of exhibition hall, Paris (Joedicke 1963)

Fig. 13. 64–127 mm (2.5–5 in.) thick, 37 m (120 ft) span intersecting tori with no ribs or hinges (Richard Bradshaw)

using the heavy lines in the figures. By adjusting the parameters of the tori, any desired rise could be obtained. There are advantages in using mathematically defined geometrical shapes as opposed to arbitrary forms. The input into the finite-element (FE) model will require guesswork, unless the surfaces are described mathematically. Also, the formwork of arbitrary shapes is more expensive. Yet, there are famous shells that are not mathematically defined, such as the Sydney opera house and the TWA building in St. Louis.

Shells can be singly curved (e.g., cylinders and cones) or doubly curved (e.g., sphere or hyperbolic paraboloid). Parabaloid is a shell of revolution made by revolving a parabola about its axis. A hyperbola produces a hyperboloid of two sheets when rotated about its axis of symmetry, and a hyperboloid of one sheet when rotated about the common axis between its two parts. The latter is often used for cooling towers, because it can be formed of straight lines (Fig. 15). Another doubly curved shape formed of straight lines is the conoid (Fig. 16), for which a straight line travels along

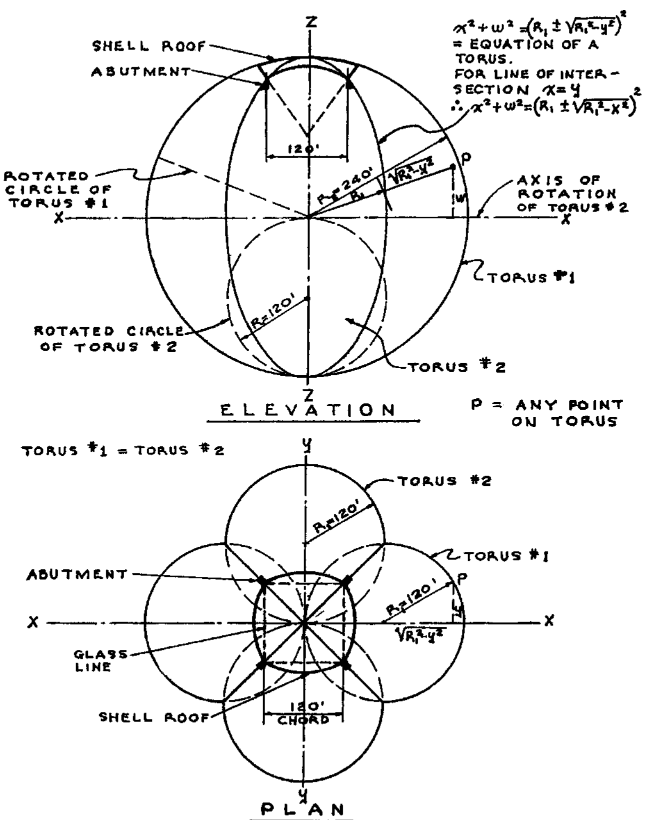

Fig. 14. Shell composed of intersection of two tori

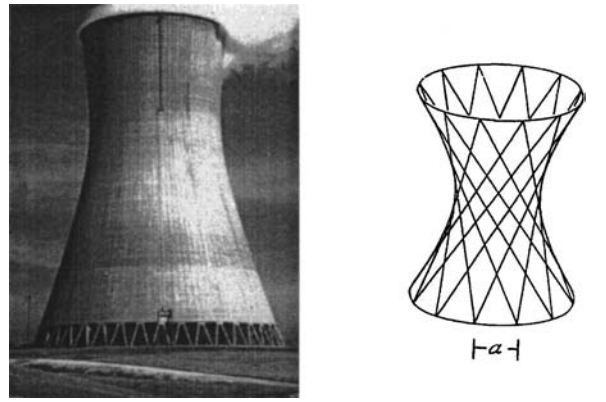

Fig. 15. Cooling tower, generated by straight lines (Gould 1988)

Fig. 16. Conoid, generated by straight line traveling along another straight line at one end and curved line at other end (Joedicke 1963)

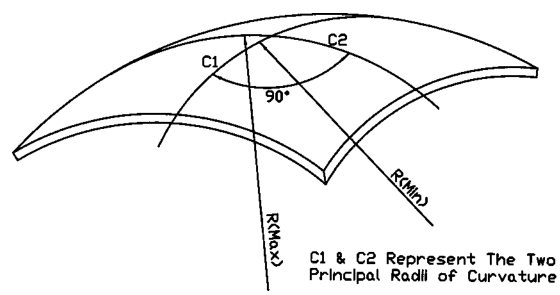

Fig. 17. General doubly curved element

another straight line at one end and a curve at the other end. Shells of translation are generated by translating one curve along another. A circle-translated tangent to a straight line generates a cylinder, and if translated along another circle produces a torus.

Fig. 17 shows a doubly curved surface with two different curvatures. At any point on a surface, there are two principal radii of curvature that uniquely define the surface. Of all the curves on the surface that can be drawn through the point, the two principal radii of curvature will be the maximum and minimum that can exist at the point. The maximum radius of curvature for a cylinder is infinity, while the minimum is the radius of the circle (Fig. 18). Figs. 19(a and b) shows two pieces taken from the outside and inside of a torus, respectively. In the former, both radii of curvature lie on the same side of the surface, and the curvature is considered positive, while in the latter they are on opposite sides of the surface, and the curvature is negative.

All shells have either positive (bowl-shape) or negative (saddle-shape) curvature. The behavior of these two types of

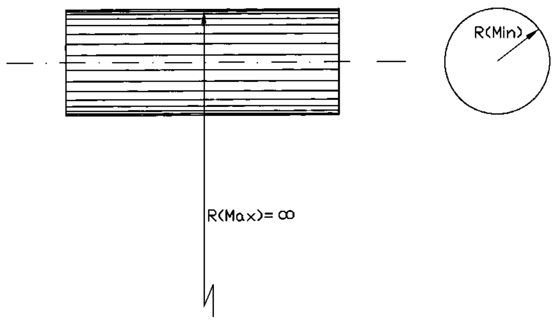

Fig. 18. Cylinder

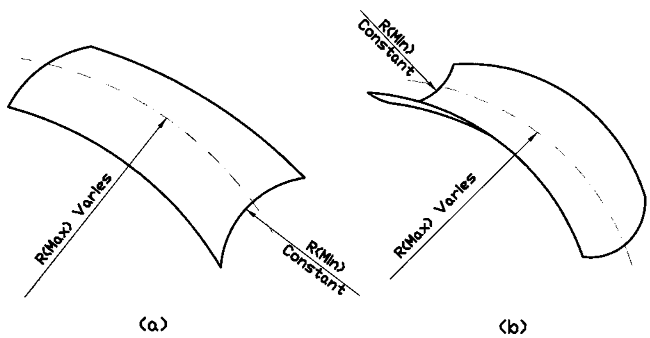

Fig. 19. (a) Cut from outside torus; (b) Cut from inside torus

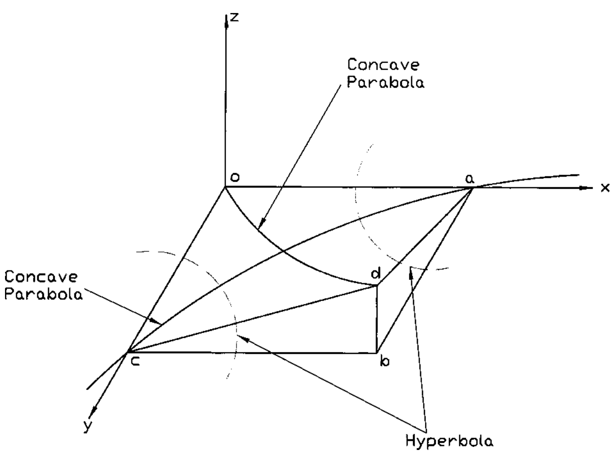

Fig. 20. Generating hyperbolic paraboloid by lifting one corner of plane

shells is very different. Positive curvature shells are subject to buckling, as the entire shell is subject to compression forces. In contrast, material failure is more common in negative curvature shells with brittle materials such as concrete.

Hyperbolic paraboloids (HP) are doubly curved surfaces with negative curvature. An HP can be generated by lifting one corner of a square shape as shown in Fig. 20. Lines parallel to the x- and y-axes remain straight lines. This is very important because the surfaces can be formed with straight forms, which are much more economical than curved forms. An HP can also be generated by translating a convex parabola along a concave one as shown in Fig. 20. Fig. 21 shows an HP in its more usual orientation and a structure built from it. If the convex parabola of Fig. 20 had been translated along another convex parabola instead of the concave parabola, it would have produced an elliptic parabola with a positive curvature (Fig. 22). The figures show that a simple change in the geometric parameters can result in a very different shape with greatly different structural behavior.

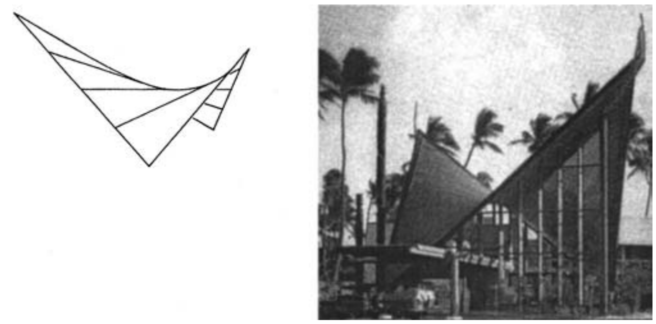

Fig. 21. Another view of HP and structure built from it (Richard Bradshaw)

Analysis of Shells

The structural analysis of shells has had a long and difficult history. Shells were developed and reached their peak popularity just before the ready availability of computers and the FE method. This was unfortunate for the designers of these complex structures because in lieu of rigorous methods they went to considerable effort to verify their designs. Model analysis was one such technique, where plexiglass models, or rarely cementitious models, were strain gauged and loaded with weights. However, model analysis was laborious, expensive, and impractical for testing various trial shapes as easily as in the FE method.

Many cylindrical shells were analyzed using approximate methods, in that when extended in the long direction they approach beams in behavior, and when shortened in the same direction approach arches in behavior. Hence, they fall between the limiting cases of beams and arches. Corrugated iron, which is a collection of cylindrical shells side-by-side, may be analyzed as a beam of corrugated cross section. For short shells such as aircraft hangars, where spacing of the arches is small compared to their span, loads are mostly carried by the arches, not the shell itself.

Another method was to get the funicular or nonbending shape of the shell using hanging weights from a mesh. The Swiss engineer, Isler, froze suspended wet cloth to get the funicular. The dimensions of the prototype were then taken from measurements made on the model. A certain amount of error was thus introduced in the prototype. Also, the funicular shape for dead load is not the same as for partial span loads, which can occur with wind and

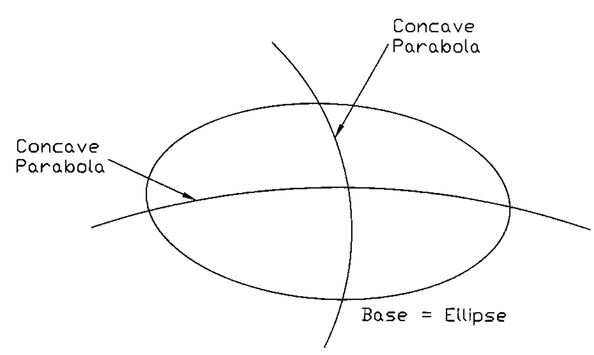

Fig. 22. Elliptic paraboloid shell

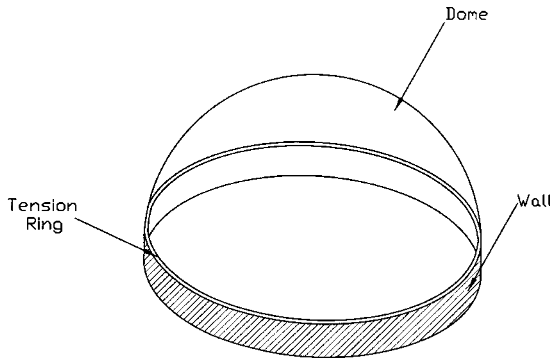

Fig. 23. Spherical shell on top of circular wall

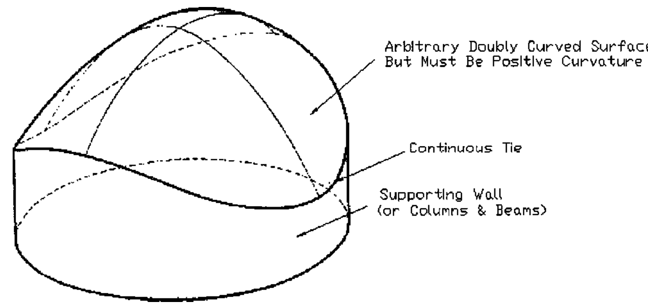

Fig. 24. Arbitrary doubly curved surface

snow loads, where partial span loading frequently governs. Other approximate methods used were to figuratively cut out pieces of the shell and analyze them for static equilibrium. One advantage of these approximate methods was that they forced the designer to develop an intuitive feel for the structural behavior of the shell, which is sometimes missing with the uncritical use of computers.

Of the more rigorous analytical methods, one can refer to the FE and the finite difference methods. In the FE method, the shell is cut into small pieces or finite elements and then "reassembled" using equilibrium and compatibility. In contrast, the finite difference method breaks down the governing equations of shells and solves them to an approximate solution. Prior to the availability of FE, some shells were analyzed by finite difference methods, which required a tedious convergence process without the use of computers. Today FE methods prevail for shell analysis. The shell element must consider anisotropy, creep, and the plastic flow of concrete. There have been failures of concrete shells after erection and initial loading, where creep and plastic flow have played a major role (Beles and Soare 1966).

Shells are usually modeled using triangular or quadrilateral plane elements. The former could be used to approximate any singly or doubly curved surface with positive or negative curvature, as the three corners of the element could always be made to fall on the shell. Although there are certain advantages to the use of quadrilateral elements, it is not always possible to approximate surfaces with them. Fig. 23 represents a doubly curved shell with no axis of symmetry. This surface could be approximated to any degree of accuracy with triangular elements. However, the same may not be true for a quadrangular element, as they are given a twist that is not included in their derivation. The amount of twist depends on the element size and the surface curvature. One characteristic of the hyperbolic paraboloid is that its twist is constant over the entire surface. Therefore, a flat quadrilateral cannot exactly fit its surface. The designer must estimate the consequences of this effect and verify the permissible twist in the FE program.

The analysis of shells requires that the three conditions of equilibrium, compatibility, and constitutive laws be satisfied simultaneously. The latter are the stress-strain properties for the materials of the shell. Most shells are designed with isotropic materials. With the development of advanced composites, their orthotropic and anisotropic behavior must be considered. However, composites have not been used for architectural shells to date. Roof structures are seldom designed for dynamic loads. Earthquake and wind loads may be treated as equivalent static loads.

The above three conditions result in three partial differential equations, two of the 2nd order and one of the 4th order, for the most general case with two different radii of curvature and with combined bending and membrane actions. An early representation of these equations for cylindrical shells may be found in Donnell (1933). Bradshaw (1961) extended those equations to the general case of double curvature, which can describe any 1D member (beam), 2D member (plate), or 3D singly or doubly curved member (shell). If the 4th order bending-related terms are left out, the equations will represent only the membrane action, which is usually sufficient for part of the shell away from the abutments because flexural resistance of thin shells contributes little in this region. Equal radii of curvature result in equations for a sphere. Equations of a cylindrical shell are derived when one radius of curvature is set to infinity; when both are set to infinity, it will result in bending of a flat plate. Finally, the ordinary differential equation for bending of a beam is derived when plate width is set to unity.

Stress analysis of complete shells, such as pressure vessels, is much simpler than for architectural roofs because of the boundary conditions. When the shell is a portion of the sphere, it tends to spread outward at the discontinuous edge. To counteract this a ring is added, but the ring and the shell distort by different amounts, which results in bending stresses in the shell. These incompatible strains must be reconciled analytically, which is not too difficult a task for simple spherical shells. However, when the shell has isolated supports and few (if any) planes of symmetry, it is a severe problem at the discontinuous edges.

If we imagine the architectural shell to be cut from the complete shape, profound perturbations are introduced at the discontinuous edges. The resulting disturbance at the edge may be thought of as causing stress redistributions to flow across the entire shell with diminished effect as they move away from the edge. In many cases of shell analysis, the stresses resulting from the discontinuous edges will dominate the design. Physically, the shell boundaries are treated in various ways. It is sometimes possible to simply leave them as free discontinuous edges. Ribs are frequently added at the edges, though visually disruptive. One of the graceful aspects of Candela's shells is their lack of ribs. It is also possible to design the shell with the rib integrated within the shell itself. Compare, for example, Figs. 4(b and c) and 13 with Fig. 5.

There is a remarkable property of shells supported vertically at their edges. Fig. 23 shows a spherical dome supported on a wall. A tension tie is required around the perimeter at the intersection of the dome and the wall. This tie will be funicular, i.e., it will only carry axial tension forces. This principle has been known since antiquity for circular domes and ties. However, it is important to note that the tie will be funicular for *any shape of either the plan or elevation* (Csonka 1962) provided that the shell has positive curvature and continuous vertical support (Fig. 24). The support may be a continuous wall or stiff beams between ad-

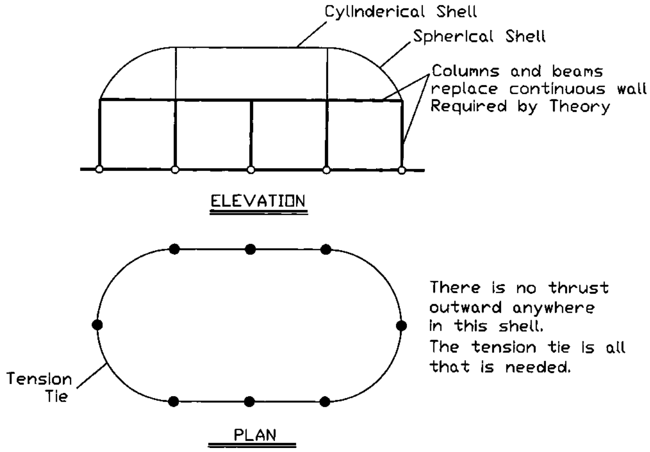

Fig. 25. Cylindrical shell combined with spherical shell

equately spaced columns. It is interesting that the straight parts of the tie in Fig. 25 do not require ties across the building. The thrusts are taken by shear forces through the width of the shell, and only tension forces exist in the tie.

Buckling of Shells

Beles and Soare (1966) have reported buckling failure of shells. Unlike shells of positive curvature that are subject to buckling, in shells of negative curvature, such as hyperbolic paraboloids, buckling is prevented through the tension curvature in the other direction. Virtually all studies on shell buckling have focused on cylindrical, conical, and spherical shells made of metals, and usually on full 360° models rather than the much more complex architectural shells. Buckling of spherical shells has been studied for radial compressive loads. Cylinders and cones have been studied for radial compressive loads, axial, and shear (twisting) loads. Shallow spherical shells have been extensively tested for snap-through or "oil-canning" buckling. Applicability of these tests to large-scale concrete shells, however, is questionable. Initial imperfections in shells can result in their buckling at loads far below their theoretical capacity. Once a shell buckles, its collapse tends to be complete, contrary to plates, which have high post-buckling capacity.

Construction of Shells

Formwork has always been a major expense in shell construction. Several methods such as precasting or shotcreting over balloons and over reinforcing steel cages have been utilized to minimize this drawback. Double layer shells, spread apart, have been used to reduce weight. Full or partial use of straight forms for hyperbolic paraboloids, hyperboloids, cylinders, cones, and conoids also makes the formwork less costly. Rolling forms can be used for cylindrical shells, but the designer must pay attention to the cold joints (Fig. 26). Circumferentially moving forms may be used for shells of revolution, where pie-shaped pieces can act as temporary arches until the entire shell is in place. Joints must be made in places of low stress (Fig. 27).

Cylindrical shells have been cast on top of each other and then assembled with a crane. However, the pieces must be of the geometry shown in Fig. 28 to prevent dimensional creep. This makes the pieces slightly thicker in the middle than at the edges, but the difference is undetectable. Fig. 28 shows a doubly curved

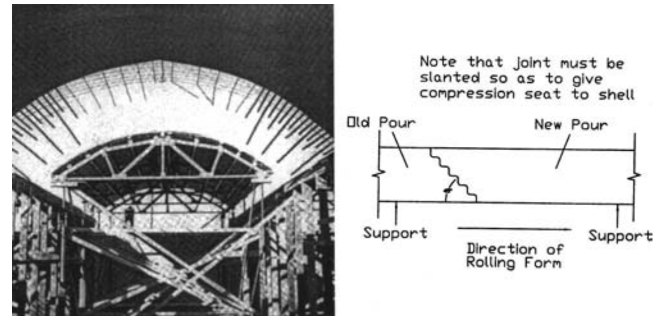

Fig. 26. Showing reuse of forms by rolling scaffolding (Richard Bradshaw)

toroidal shell, which uses the system of casting one piece on top of another. In this case, it is necessary to allow for the dimensional creep in both directions. The bottom shell was built over an earth form with the other shells cast on top of the first one. For precast shells, the cost of the cranes must be compared with the cost of forms.

Shells shotcreted over balloons have been used, particularly where high precision of dimensions is not important. When shotcrete is placed on a balloon, the weight distorts the balloon. This means the shell will not be exactly the initial shape of the balloon. This is not important for a small span shell. For a long span shell, however, this deviation from the spherical shape could be serious as shells are sensitive to buckling due to the initial roughness effect.

Future of Shells

At present, shells have lost their popularity compared to their heyday in the 1950s and 1960s, when architects eagerly adopted them as a new means for artistic expression. They were perhaps so eagerly adopted that they became a fad, and when a backlash inevitably set in, they were abandoned as quickly as they were first embraced. Shells were seldom the most economical way of covering a large space, especially when compared to lightweight tension membranes. Also, their formwork has always been a major cost factor.

There are signs, however, that shells are attracting interest among the new generation of architects and engineers. They will never become the vogue they once were, but they will regain some of their former popularity when used appropriately. There are also new materials such as fibercrete concrete and fiber reinforced polymer (FRP) composites that may be used in shells. At present, they may be too expensive for use in architectural shells, but with time that may change. Composite shells will require ortho- or anisotropic modeling, as well as careful buckling analysis, because they tend to be much thinner than concrete shells.

The future shells will take their place alongside other forms of architecture in structural engineering. When designed properly,

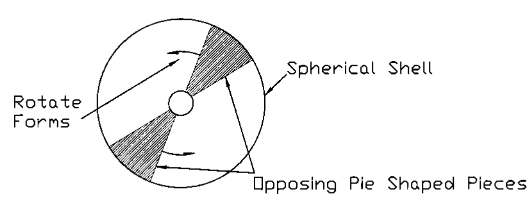

Fig. 27. Use of pie-shaped opposing forms

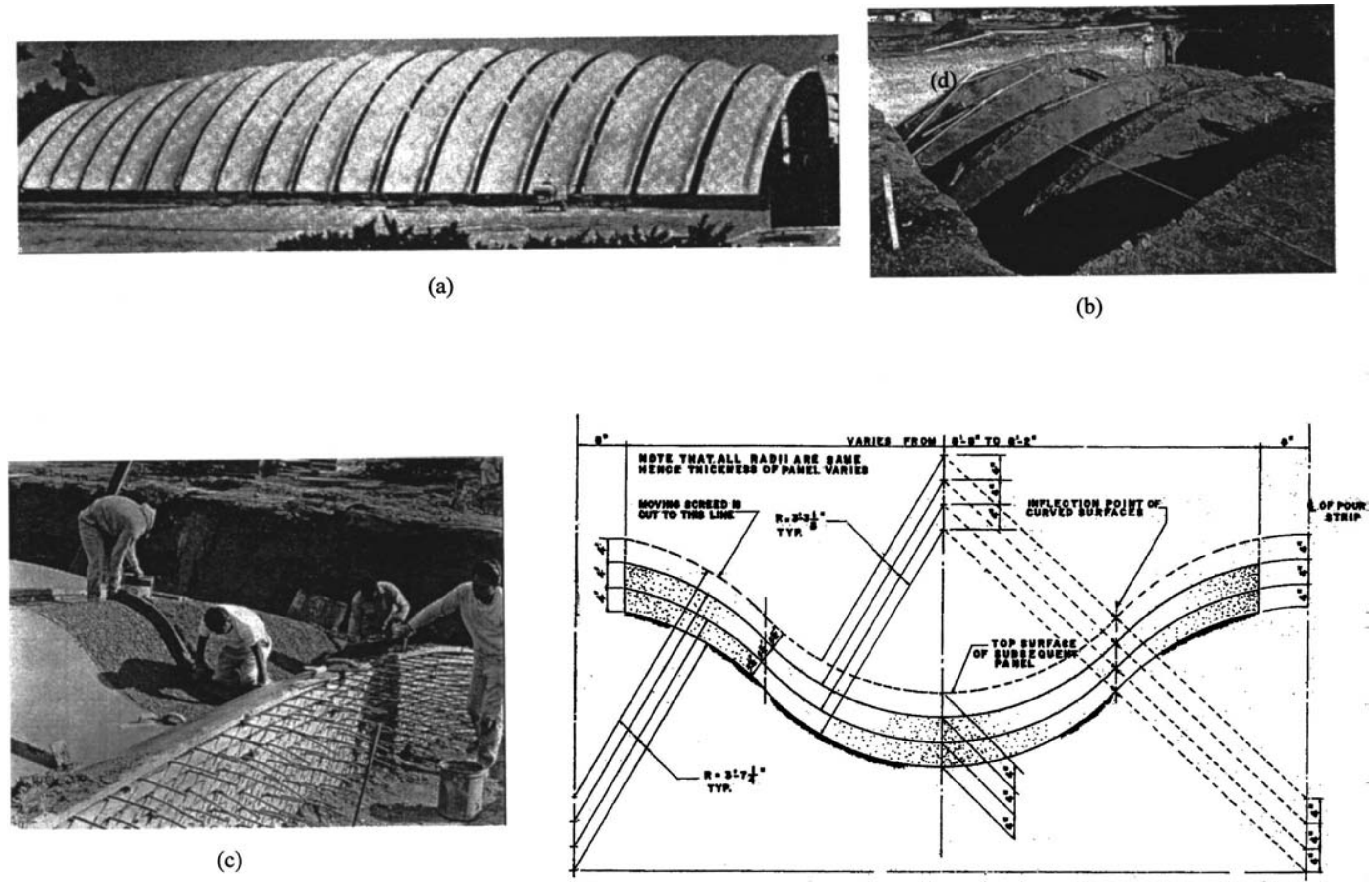

Fig. 28. Corrugated torus shell geometry: (a) Overall view of shell; (b) Earth form for bottom shell; (c) shells cast on each other; (d) geometry of precasting showing the elimination of dimensional creep (Richard Bradshaw)

they are among the most beautiful and efficient of architectural structures. Even those who are not professionals can sense the flow of forces through them. They will present both problems to be solved and opportunities to create for those who take the time to understand them.

Tension Membrane Structures

History of Tension Structures

Tension structures include a wide variety of systems that are distinguished by their reliance upon tensile only members to support load. They have been employed throughout recorded history as in rope bridges and tents. However, large permanent tension structures were generally a 19th century development in bridges and a 20th century development in buildings. The design of large tension membranes has been fully dependent upon the use of computers. Many of the developments in membranes have occurred in the last 30 years, precisely because of the accessibility of powerful computers. The pioneering work of Frei Otto was accomplished using physical models, which, while they well illustrate the desired form of a membrane, are not conducive to the precise determination of the membrane's structural characteristics in a manner necessary for the construction of large complex systems.

Large deployable membrane structures were used to cover touring public assembly events such as circuses and religious revival meetings in the 19th century. These were constructed of canvas and ropes with wood poles, as were contemporary tents. It was the fate of the "Big Top," the Barnum and Bailey Circus' main auditorium tent that burned (Martin and Wilmeth 1988), which created the most significant hurdle for membrane architecture in the United States: the issue of noncombustibility. Prior to the introduction of noncombustible structural fabrics, membrane structures were nomadic, and subsequent to the "*Big Top*" fire, they were relatively small. There were exceptions: Frank Lloyd Wright employed a tension membrane roof of canvas on his school and home, Taliesin West in Scottsdale, Ariz. in 1938. However, permanent tension membrane architecture began in North America in the 1970s.

Structural economy rather than aesthetics or architectural expression initially drove the modern use of membrane structures in North America. Thus, it is not surprising that the development of modern membrane structures was, with some exceptions (most notably John Shaver, the first American architect to develop permanent membrane architecture) primarily the work of engineers. However, these structures, like all spatial structures, are by their nature uniquely expressive, creating architecture that was in some instances a result rather than a goal.

Development of modern membrane structures began in the later half of the 20th century, and communications in the field were such that worldwide experience was quickly disseminated. The work of Frei Otto in Germany was particularly influential. As has been the case with other building technologies, World Expositions, particularly EXPO '70 in Osaka, Japan, were of great

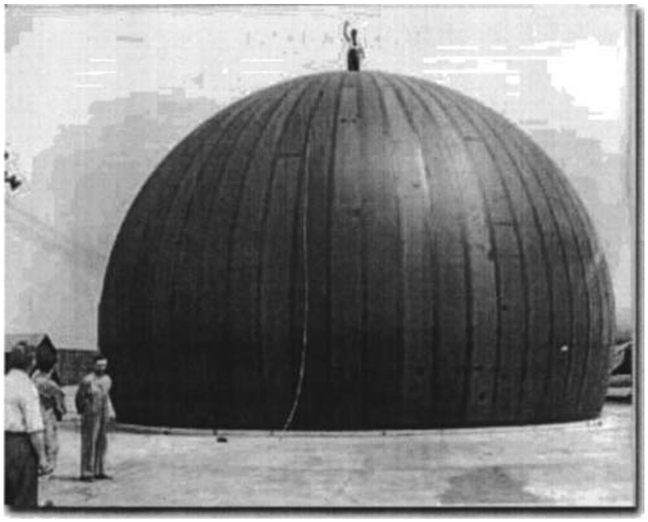

Fig. 29. Walter Birdair atop a Birdair radome in 1956 (Birdair)

significance in the development of membrane structures around the globe.

Simply considered, membrane structures were initially pursued in the United States for their cost-effectiveness. The first permanent membrane structures were the air-supported radar enclosures designed and built by Walter Bird (Fig. 29), as early as 1946. Walter Bird's successes with these pneumatic structures led to his founding of Birdair Structures in 1956.

Architectural membrane systems were a natural extension of tension structures of the 1950s and 1960s for long-span buildings. Fred Severud demonstrated the long-span potential of tension structures in benchmark projects such as the North Carolina State Fair Live Stock Pavilion in Raleigh, N.C., the Yale Univ. hockey rink, New Haven, Conn., and Madison Square Garden, New York. Severud's engineering practice was the incubator of membrane structure design in the United States. Designers of subsequent tension membranes included David Geiger, Horst Berger, Paul Gossen, and for brief periods, Edmund Happold and Frei Otto. Another engineering pioneer of prestressed tension-based structures was Lev Zetlin, who designed the Utica Memorial Auditorium roof and the New York State and Travelers Insurance Pavilions at the 1964 New York World Fair. Architects such as Eero Saarenen successfully exploited the unique architecture of these structures to create new forms for buildings.

While tension structures of Severud and others demonstrated the potential of such architecture in the United States, architectural membrane structures really began with EXPO '70 in Osaka, Japan, when David Geiger was commissioned to engineer the enclosure for the United States Pavilion. The initial pavilion design was abandoned when the project was unable to secure a sufficient appropriation from Congress. The pavilion program was maintained, but the project had to be realized for one-tenth of the original budget. In response to this challenge, Geiger invented the low profile cable-restrained, air-supported roof employing a superelliptical perimeter compression ring. This proved to be an exceedingly economical means of covering large clear span spaces, and quite interestingly, within 15 years of its completion, this structural system was employed to cover more than half the domed stadia in the world (Fig. 30).

Following the success of the United States Pavilion project, David Geiger considered applying it to permanent structures. However, such applications required a strong, noncombustible, durable material. An existing coated fiberglass fabric product, primarily used for conveyor belts in commercial ovens, seemed to have the desirable characteristics. David Geiger brought DuPont De Nemours Company, Owens-Corning Fiberglass Corporation and Chemical Fabrics Corporation together to develop this material for architectural applications. Marking a new era of permanent tension membrane architecture, the resulting product, *teflon* coated *fiberglass*, has since been employed around the world, as early as 1973 and 1974 for the Student Center at La Verne College, Calif. and the Steve Lacey Field House, Milligan College, Johnson City, Tenn. Architect John Shaver designed both of these buildings.

(a)

(b)

Fig. 30. Taoyuan Sports Arena, Taiwan, (a) outside view; (b) interior of Geiger Cabledome (Geiger Engineers)

Horst Berger and David Geiger worked together between 1969 and 1984. While Geiger's interest in membrane structures was for their structural efficiency and economy, Berger did much to demonstrate the aesthetic potential of tension membrane forms in architecture. Together, they developed analysis and design tools and techniques indispensable in the design, documentation, and construction of complex tension structures.

Architects such as Paul Kennon of Caudill Rawlett and Scott, later known as CRS Sirrine, were quick to embrace membrane architecture in the early 1970s and designed a number of landmark projects. They explored the forms and the spaces created by the unique translucent envelope of tension membranes. Raul de Armas and his colleagues at Skidmore, Owings & Merrill were instrumental in bringing the new membrane architecture to the attention of the world with the Haj Terminal at the Jeddah International Airport, Saudi Arabia. His design solution to the unprecedented challenge of sheltering the Haj pilgrims while changing transport modes to Mecca was simple and brilliant. Drawn to tension membrane by the desire to create a translucent canopy, the modular tent forms, which evolved in the final design, were modern and at the same time reminiscent of the tents used for centuries by Haj pilgrims.

Canadian architect Eberhard Zeidler designed two landmark tension structures for EXPO '86 in Vancouver, B.C.: the Canada Pavilion, now the convention hall at Canada Harbour Place, and the Ontario Pavilion. Beginning with the Canada Place project, Eberhard Zeidler continued to explore the sculptural potential of tension membrane architecture beyond its structural origins.

The enclosure of large clear span space created great architectural opportunity for membrane structures. Early successes were a result of the application of this building technology to the recently emerged American building type, the ''domed'' stadium. Uniquely North American until the late 1980s, covered stadia require roof spans without precedence. Membrane structures have been employed more often in covering sports stadia than any other structural system. There is no other building type for which this is the case, due primarily to the economy of membrane structures. Initially, the Geiger low profile, air-supported roof system was used. Later, shortcomings of air-supported roofs led Geiger to invent a new system, the Cable Dome to cover a baseball stadium in St. Petersburg, FL, combining his experience in membrane structures with Buckminster Fuller's ideas of ''tensegrity'' and ''aspension.'' These systems, their variants, and other tension membrane structures continue to be significant in covering long span spaces.

Architecture

Not long after its development, the light translucency of coated fiberglass became an obvious virtue, availing it as a cost-effective substitute to glazing in many commercial projects across the continent. Recently, Zeidler's work has successfully combined translucent tension membrane with glazing to enhance the day lighting as well as architectural composition. With noted exceptions, however, the architecture of membrane structures has been little explored in the United States beyond that driven by the economics.

Most architectural forms have developed from the nature of traditional building materials. Building forms that developed from a material, say masonry, are quite often built of other materials. These forms create an architectural vocabulary well understood by the general public. However, an architectural vernacular of membrane structures has yet to be established due to their recent development. As familiarity with tension membrane architecture increases, they will be employed more often for their architectural forms.

Applications

As with all spatial systems, tension membrane structures exploit 3D forms to support load. However, tension structures are unique in that they noticeably change form in response to loading. This attribute, which anyone who has walked on a trampoline has experienced, is why tension membrane systems are almost exclusively employed as building enclosures, particularly roofs, rather than platforms or floors.

Innovative and structurally efficient tension membrane systems have been mostly employed for roofs. In some applications, the roof is employed as the entire building envelope, but applications of tension membranes as enclosure material for walls alone remain a rare exception. This is not to say that the tension membranes are not suited to this application, only that such possibilities have not been explored. A list of notable tension membrane structures is provided in Table 1.

Table 1. Notable Tension Membrane Structures

Structure	Location
Raleigh Livestock Pavilion	Raleigh, N.C.
Memorial Auditorium	Utica, N.Y.
Museum of Automobiles	Win-Rock Farms, Ark.
Yale Hockey Rink	New Haven, Conn.
Madison Square Garden	New York
Pavilion of the Federal Republic of Germany at EXPO '67	Montréal
U.S. Pavilion at EXPO '70	Osaka, Japan
Munich Olympic Stadium	Munich, Germany
La Verne College Campus Center	La Verne, Calif.
Pontiac Silverdome	Pontiac, Mich.
Haj Terminal	Jeddah International Airport, Saudi Arabia
Stephen O'Connell Center	Gainesville, Fla.
Lindsay Park Sports Center	Calgary, Alberta
Olympic gymnastics and fencing arenas	Seoul, Korea
Schlumberger Cambridge Research Center	Cambridge, England
Redbird Arena at Illinois State University	Normal, Ill.
Martha Mitchell Pavilion	Woodlands, Tex.
Canada Harbour Place	Vancouver, BC
Tropicana Field	St. Petersburg, Fla.
San Diego Convention Center	San Diego
Stadio Olympico	Rome
Georgia Dome	Atlanta
Inland Revenue Center Amenity Building	Nottingham, U.K.
Hong Kong Stadium	Hong Kong
Denver International Airport	Denver
Akita Skydome	Akita, Japan
Gottlieb Daimler Stadium	Stuttgart, Germany
Millennium Dome	Greenwich, U.K.
Oita World Cup Stadium	Oita, Japan
Seoul World Cup Stadium	Seoul, Korea

Fabric Membranes

Almost all existing tension membrane structures use textiles in lieu of a true membrane material. Essentially, practical combinations of tensile strength, tear resistance, ductility, dimensional stability, and flexibility are currently only available in coated fabrics. In almost all cases, the curved surface of a membrane structure is fabricated from flat pieces of coated woven fabric cut from rolls. Seaming is most commonly accomplished by lapped heat seals but can also be done by mechanical means such as sewing or intermittent fasteners.

Fabrics are quite different from membranes as engineering materials. Commonly used coated fabrics are composite materials whose strength is primarily provided by the woven textile and yarn: weather protection, finish, and the jointing ability are provided by the coating. This results in materials that have very low shear stiffness in relation to their tensile stiffness and are also highly nonlinear. The orthotropic behavior of coated fabrics is complex, dependent on its stress history, and is dictated by micromechanics of the weave. Fortunately, most tension structures are not particularly sensitive to the stiffness of the fabric membrane. It is because the ''geometric stiffness'' of the membrane arising from change in geometry and membrane prestress is more significant than the extensional stiffness of the material. However, as a consequence, tension membrane structures exhibit first-order nonlinear behavior, complicating the analysis.

The primary ''stress'' directions of a fabric membrane are the weave directions of the textile: the warp and fill. The warp is the direction of the yarns, which are spooled out lengthwise in the loom or weaving machine. The fill or weft is the yarn in the cross machine direction, which ''fills'' in the weave. As a curved membrane surface is fabricated from flat pieces of fabric, the warp is generally parallel to the seams. Different weaves have different mechanical qualities, which are primarily governed by the convolutions of the yarn and the initial state of crimp in the weave. Generally, the initial crimp in the warp is different from that of the fill. This results in different initial elongation properties in the warp and fill direction. In general, elongation at service level stresses is dominated by weave crimp, rather than strain of the yarn fibers. Consequently, elongation behavior in service has more to do with the weave than yarn characteristics. The coatings employed in most structural fabric tend to attenuate this behavior, especially for transient changes in strain. This results in response to transient loads similar to membranes. However, as almost all currently employed coatings are polymeric in nature, the effects of the coating diminish with load duration as creep of the coating allows the yarn with its weave-dominated behavior to resist the loads.

All textiles have common attributes that are significant in structural applications. Tensile strength of fabrics is greater in uniaxial than in biaxial loading, and failure is almost always a result of tear propagation rather than tensile rupture. This belies the fact that current design practice establishes membrane resistance solely on uniaxial strength. Tear propagation in textiles can be roughly analogous to crack propagation in metals in direct tension. Tears are initiated at cuts, abrasions, or other discontinuities and propagate when the force at the head of the tear reaches a critical value. Tear resistance is dependent on both yarn and weave properties.

Structural Forms

The surface forms of tension membrane structures are architecturally unique. Because the load applied to the surface must be resisted by tensile stresses in the membrane, local curvature of the surface is required. In order to ensure that stresses remain within acceptable limits, it is usually desirable to establish initial curvature in the membrane. This requires that the membrane be placed in state of internal stress or ''prestress.'' The problem of finding prestress equilibrium forms for membrane structures is of great interest, making it important to understand the nature of the forms that can be readily employed in tension structure architecture. At the risk of being somewhat simplistic, we may categorize some of the most common forms as follows:

- Conical ''tent'' shape, such as in the Haj Terminal, is a prestressed anticlastic surface;
- Ridge ''tent'' shape is an anticlastic form characterized by a catenary ridge line supporting the membranes between two point supports (masts) nominally at the edge of the structure. The same concept can be developed in a circular configuration;
- Pleated surface shape, where the membrane surface appears folded or pleated, to form an undulating surface of ridges and valleys. This differs from the previous category in that the surface is only slightly, if at all, anticlastic, and becomes synclastic when subjected to loads. Load is carried in one direction;
- Saddle form is characterized by a single anticlastic surface;
- Vault form is anticlastic, and is usually supported by parallel or crossed arches; and
- Pneumatic forms are all synclastic, and the prestress is established by internal pressure on the membrane.

These basic forms or their combinations have been used to create a myriad of large structures. Tension membrane structures have been successfully combined with tensile net, truss, and dome systems to create lightweight long-span structures. The use of membranes for these cable structures has the advantage over more conventional building materials in that the membrane can well accommodate the relatively soft structures without a need for special jointing or releases.

Some of these systems such as the Tensegrity domes were first realized as tension membranes. These ''domes'' combine Buckminster Fuller's ideas of tensegrity and aspension and are comprised of a network of continuous cables and ''flying'' struts prestressed within the confines of a perimeter compression ring. Similar in some respects to a spatial lenticular cable truss, except that these structures rely upon nested tension rings or hoops rather than continuous bottom chords. The first tensegrity-type dome of any scale was Geiger's Cabledome for the Olympic Gymnastics Arena in Seoul, Korea. He developed the Cabledome system in order to achieve the virtues of his air-supported roof structures without the disadvantages of mechanical support. Two variants of tensegrity type domes have been realized to date, Geiger Cabledome structures and the spatially triangulated dome variant proposed by Fuller and realized by Levy. Both of these have been employed in dome stadia with spans in excess of 200 m (656 ft). These roof structures are unprecedented in their low mass. The Cabledome covering Tropicana Field in St. Petersburg, FL spans 210 m (688 ft) with a unit weight of only 0.24 kN/m^2 (5 lb/ft^2), while it has allowable capacity to carry applied gravity loads of 0.67 kN/m^2 (14 lb/ft^2) or 2.8 times its unit weight.

Design

Design of large and complex tension membrane structures is more reliant upon computers than most structural systems, as they defy classical analysis. The nonlinear behavior of these structures coupled with the need to determine prestressed forms to meet specific design and boundary conditions as well as the loading analysis, necessitates a true ''computer-aided'' design and modeling technique. The procedures for prestressing the system are determined in similar fashion. Finally, the templates used to cut and fabricate the fabric membrane surface are typically computer generated. As with most design methodologies the process is iterative, such that anticipation of the results in the conception of a structure will reduce the general effort involved in the design and engineering of the system.

Tension membrane structures exhibit both geometric and material nonlinearities. The nature of tension membrane structures is such that much of their stiffness is achieved by virtue of initial prestress in the membrane and its supporting components. This prestress is an internal stress condition usually prescribed by the designer to achieve the desired performance of the structure and must be induced into the system in its construction.

Fabric membranes are selected for a given structure based upon their strength, durability, fire performance, optical properties, and finish. Standard practice is to establish the minimum required strength in the warp and fill based upon the uniaxial dry strip tensile strength of the material in the warp and fill. Minimum strip tensile strengths during the expected life of the membrane are established as 5 times the maximum service stress due to the worst service combination of prestress, dead load, live load, and snow load or 4 times the maximum stress of the worst combination of prestress, dead load, and wind load. Suitable tear resistance relies upon careful detailing, installation and inspection to eliminate stress concentrations and discontinuities, as well as to identify and repair minor cuts and damage from installation and handling. Cables are commonly employed in tension membrane structures.

Form Finding

In the simple case of air-supported structures, the prestress is achieved by loading a synclastic shaped membrane with a differential air pressure. The simplest form of air-supported structure for which the prestress can be easily determined is a spherical dome. Assuming that the unit weight of the membrane is small with respect to the internal operating pressure, the membrane stress at a given pressure is proportional to the radius of curvature of the sphere. While analysis of such a structure under wind loads is nontrivial, both membrane patterning and determination of prestress are easily accomplished without the aid of computing. Hence, it is not surprising that the first widely used air-supported membranes were the spherical air-domes.

Prestressed anticlastic tensile structures present a more difficult problem. A wide variety of complex forms can be determined from physical models. As demonstrated by Frei Otto, minimal surfaces can be created using soap films. However, none of these techniques can precisely communicate to the fabricator the prestress and surface geometry information required to fabricate and stress the membrane shape. This became a pressing issue as desirable materials suitable for permanent structures, such as teflon-coated fiberglass fabric became available. Coated fiberglass fabrics have desirable attributes such as their noncombustibility. However, they are significantly stiffer than other materials commonly used in tension membrane structures and consequently require greater precision in patterning. The development of algorithms for defining the surface form or shape of a general class of prestressed networks was the key to the general exploitation of tension membranes in structures of significant scale. There is a number of form-finding algorithms currently in use. All are iterative procedures, as follows:

- The force density approach is a matrix method that solves directly for the geometry of a general network of prestressed tensile components. Iterative techniques allow the designer to prescribe desired prestress conditions for cable and membrane elements,
- Alternatively, a matrix analysis algorithm can be employed for form finding. Basically, elements are assigned very low mechanical stiffness and a prescribed prestress. Equilibrium geometry is determined in an iterative analysis of the structure, and
- Another method of form finding in common use is the method of dynamic relaxation with kinetic damping.

While physical models can be utilized to study membrane forms, the geometric and stress conditions of the membrane surface are almost exclusively determined by using computers. Data from form finding, typically comprised of connectivity, nodal geometry, and element prestress represent a complete model description of the membrane structure and the element properties. Consequently, shape results with the addition of element properties can be employed directly in the analysis. Often, additional elements, such as struts or beams, are added to create an analysis model of a complete structural system.

In order to fabricate the surface established in the form-finding process, it is necessary to establish cutting patterns. The problem is to determine the pattern of flat strips of fabric, which when seamed together will approximate the form's surface. As the shape geometry is determined for a prestressed condition, patterns must be compensated for strain in the fabric. Compensated strip patterns are then used for cutting.

Structural Analysis

General analysis of all tension-based, specifically tension membrane structures requires geometric nonlinear techniques. It is necessary to account for the change in the geometry of the structural network. It has been demonstrated that deflection terms are of first-order significance in structural networks with initial prestress.

Typical matrix methods employ an iterative procedure using the Newton-Raphson method or a variant, often with a damped solution strategy. Common tensile structural systems initially go through strain softening but then exhibit strain hardening once sufficient load is applied. Consequently, nonlinear solution strategies that anticipate strain hardening have been used with success to speed convergence in most common problems. The dynamic relaxation method is also used with success for the general analysis of geometrically nonlinear problems. Most importantly, the principles of superposition do not apply to nonlinear systems. Therefore, all critical load combinations must be analyzed individually.

Material nonlinearity is rarely modeled, although it is inherent to most fabric materials. This is just as well, because material properties are often affected by stress history. While fabric material nonlinearity is typically not modeled, it will likely prove to be useful when the mechanics of fabric failures are better understood and utilized quantitatively in a limit states design approach. The fabric is commonly modeled utilizing linear strain or constant strain triangle membrane finite elements or a network of string elements. These approaches have been widely used with success; each has attendant limitations that the analyst must consider.

Construction and Stressing Analysis

The ability to visualize, analyze, design, and fabricate complex membrane forms can create difficult construction problems. Prestress is as much a property of tensile structures as element properties and geometry. A prestressed state for a structural system can be created in a computer model without regard for the manner in which the prestress would be developed in the structure. Consequently, with redundant structures, techniques are required to establish the sequence of stressing to ensure that the structure will in fact realize the design prestressed state. Moreover, in many complex tensile systems, analysis of the stressing sequence is necessary to assure that various components of the system are not over stressed during stressing.

A technique now commonly employed is the analytical disassembly of a prestressed structural system in reverse order of stressing. The erection and stressing sequence of many complex prestressed structural systems can be determined in this manner. Generally, the accurate construction of many complex structural systems is only possible using appropriate software and suitable techniques for the determination of the stressing sequence.

Space Grid Structures

A space grid structure (SGS) is a 3D system assembled of linear elements (Engel 1968), so arranged that forces are transferred in a 3D manner. The system is also called vector-active, which is made up of two-force members whose primary internal forces are axial tension or compression. A force applied on the space grid system, typically at a node, is distributed among the axial members. When SGSs have depth or thickness, they are commonly referred to as space frames, double-layer grids, or space trusses. Single layer semi-spherical space grids are commonly known as geodesic domes.

The characteristics that make SGSs popular include: the ability to create multipurpose column-free large architectural spaces; light weight reduces their susceptibility to seismic forces; use of small elements facilitates their mass production, transportation, and handling; ease of assembly without highly skilled labor and with limited access; aesthetic appeal, visual elegance, and interesting geometric patterns; and an open form that allows easy installation of mechanical and electrical services. Since the 1940s, SGSs have been developed for the construction market, and have been used for exhibition halls, gymnasia, auditoria, swimming pools, aircraft hangars, world's fair pavilions, and mostly anywhere that a large unobstructed space is required.

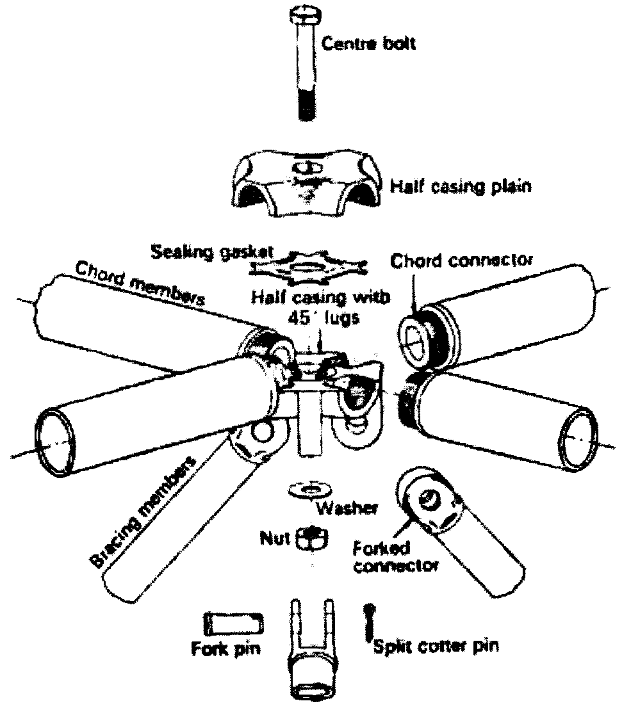

Fig. 31. Connection of struts to the node on a Nodus space frame system (Chilton 2000)

History of Space Grid Structures

Space frames or grids originated with railroad truss bridges in the 19th century (Condit 1961), although the truss system dates back much earlier. Railroad expansion not only brought the development of many common truss shapes, but also led to the development of modern truss analysis. Truss development led to an understanding of how vector-based structures functioned, and to an understanding of the importance of the nodes.

Even though Alexander Graham Bell is recognized for the invention of the space frame structures in the early 1900s (Wachsmann 1961), it was August Föppl who published the first treatise, *Theorie des Fachwerks*, on space frame structures in 1880 (Schueller 1983). This treatise aided Gustave Eiffel with the analysis of his tower in 1889. Bell's obsession with the development of the first flying machines led him to investigate light structural systems. He developed a series of kites that used a tetrahedral structure, and then built architectural objects such as a windbreak wall and an observation tower using the tetrahedral structure (Mainstone 1975).

The next step in the evolution of space frame structures was the development of the lamella structural system, invented in 1908 by Zollinger in Germany and refined by Keiwitt in the United States (Schueller 1983). The roof system is distinctive for its diamond-patterned vaulting, with the sides made of short members of equal length referred to as lamellas. The nodal principles learned from joining large numbers of lamellas particularly benefited the nodal development of space frame structures. One of the most notable lamella buildings was Nervi's precast concrete airplane hangar, which was constructed in 1938.

The first major commercial development of space frame structures began in the late 1930s. In 1939 Attwood received a patent for his space frame system (Condit 1961), which later became known as the Unistrut system. In 1940 Mengeringhausen developed a space frame system in Berlin (Schueller 1983), which later developed into the MERO system. In 1945 Wachsmann and Weidlinger received a patent for their Mobilar system, which differed significantly from the MERO and Unistrut systems in that the nodes were not separated from the strut, and the geometry of the connection mechanism was not as rigid as in earlier systems. Throughout the 1940s and 1950s, these systems continued to be refined as others were being introduced, including the Triodetic system in Canada (Schueller 1983).

Geodesic domes were developed in the 1940s and 1950s by Fuller. The term *geodesic* refers to the shortest arc on a surface joining two points and was first studied by Bernoulli in 1697. Fuller studied the surfaces of a sphere or semisphere divided into large circles. Fuller's motivation in pursuing these structures was to develop an economical shape that could be used in all parts of the world. Geodesic domes have been developed from many materials including wood, steel, aluminum, concrete, and bamboo. The geodesic domes that are considered a part of space grid structures, such as the U.S. Pavilion at the 1967 Montreal World's Fair, are those whose structure is along the arc joining two points. Geodesic domes whose structure is along the surface of the polygons defined by the arcs, such as the Kaiser Dome, are considered shell structures.

The next major development for SGS came about with high-speed computers simplifying the FE analysis of complex structures and computer aided manufacturing.

Systems

A SGS acts as a network of struts and nodes. The connection methodology of the node determines all possible polyhedra within the system. The joint module determines the position of every point off direct connection from the chosen system (Wachsmann 1961). Each node must be connected with at least three noncoplanar struts to maintain stability and to prevent translation. The more axial members that can be accommodated at any given node the greater the number of morphological possibilities for the system (Gerrits 1994).

Fig. 32. Skew-chord Takenaka space truss for a school auditorium, Ferndale, Wash., Geiger Engineers

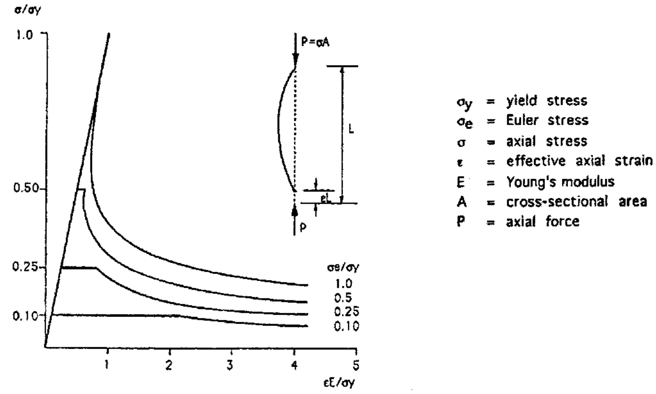

Fig. 33. Mean axial stress versus effective axial strain for axially loaded struts (Gargari 1993)

The vast majority of buildings with space grids are designed using one of many proprietary systems, such as A-Deck, Mero, Moduspan, Nodus, Ocatube, and Unistrut. This list by no means covers all of them. What makes each system unique (Fig. 31) is the geometry of the node, how the struts are connected to the nodes, the method of manufacturing the nodes and struts, and the polyhedral units possible with each system.

Very large space grids are commonly made from nonproprietary systems because of the economics of manufacturing. When designing such a system, the engineer needs to pay particular attention to the connection of the members. The system must be able to handle the rotation of the node caused by nonconcentric axial loads. It must also be able to handle the lack of fit of the members that can lead to residual stresses within the system.

Materials

Most buildings with SGS are made of high strength or mild steel tubes with circular or square shapes as well as channels and special forms, either hot rolled or cold formed. Aluminum, wood, and composites have also been used in different cross sections. The nodes for steel and aluminum grids have been designed in several shapes and forms based on their strength and aesthetics, as discussed earlier. Timber is used in the form of round poles, sawn square sections, and glued laminated elements. Members of timber grids are connected by metal pieces at their ends to each other or to metal nodes. Fig. 32 shows the interior of a skew-chord Takenaka space truss for the roof of an auditorium. The members are "pealer cores," a by-product of the plywood industry, and the nodes are cast steel. Large reinforced concrete space truss struc-

Table 2. Notable Space Grid Structures

Structure	Location	Designer	Year
Biosphere 2	Tucson, Ariz.	Margaret Augustin, Phil Hawes, John Allen with Pearce Systems International	1990
British Air 747 hanger at Heathrow Airport	London	Z.S. Makowski	1974
Climatron	St Louis	R. Buckminster Fuller	1960
Crystal Cathedral	Garden Grove, Calif.	Johnson/Burgee Architects and Severud, Peronne. Szegezdy and Strum	1980
Exhibition hall for PORTOPIA '81	Port Island, Japan	Masao Saitoh and Nikken Sekkei, Ltd	1980
Ford Rotunda Building	Dearborn, Mich.	R. Buckminster Fuller	1953
Grandstand Roof	Split, Croatia	Mero Systems	1978
Javits Center	New York	James Freed of Pei Cobb Freed and Matthys Levy	1988
Kansai Airport	Osaka, Japan	Renzo Piano Building Workshop with Ove Arup and Partners	1996
Louvre Pyramid	Paris	I.M. Pei of Pei Cobb Freed with Peter Rice	1989
McCormick Place Convention Center	Chicago	C. F. Murphy Associates	1970
Meishusama Hall	Shiga, Japan	Minoru Yamasaki and Associates and Yoshikatsu Tsuboi	1983
Palafolls Sports Hall	Barcelona, Spain	Arata Isozaki, J. Marínez-Calzón	1991
Sainsbury Visual Arts Center	Univ. of East Anglia, U.K.	Norman Foster Associates and Anthony Hunt Associates	1978
Sant Jordi Sports Palace	Barcelona, Spain	Arata Isozaki and Mamoru Kawaguchi	1990
Skydome	Toronto	Roderick Robbie and Adjeleian, Allen, Rubeli Limited	1989
Union Tank Car Company	Baton Rouge, La.	R. Buckminster Fuller	1958
United States Pavilion	Montréal	R. Buckminster Fuller	1967
World Expo Building	Osaka, Japan	Kenzo Tange, Tomoo Fukuda and Koji Kamiya, and Yoshikatsu Tsuboi	1969
World Memorial Hall	Kobe, Japan	Mamoru Kawaguchi	1984

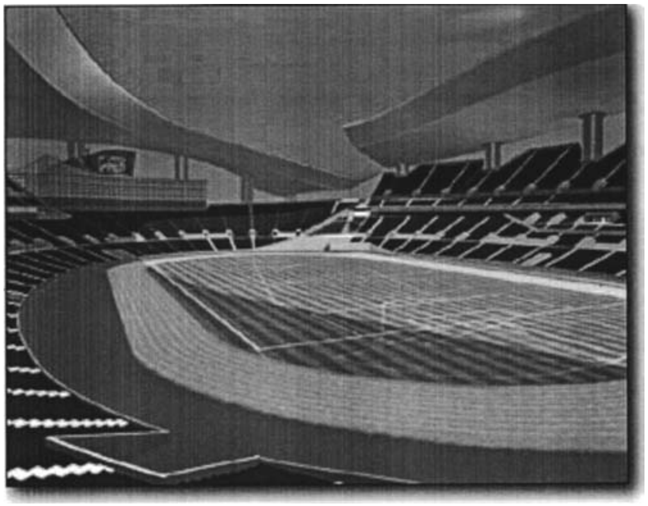

Fig. 34. Guangdong Olympic Stadium in Guangzhou, China

tures, although heavy, have been built, for example, for pavilions at a trade fair site in New Delhi, India.

Analysis and Design

SGSs are currently analyzed using linear elastic theory. The load-carrying capacity of a SGS is usually limited by the first member or set of members to fail. Connections are either made of readily available standard shapes or proprietary prefabricated pieces. Connection pieces are designed for structural efficiency or appearance. It is assumed that the connections will be strong enough so that any failure will take place in the struts or ties.

The struts and ties are treated as straight, axially loaded pin-ended members, for which the load-deformation relationship is linear up to buckling in compression or yielding in tension. Tension members ideally would yield, but may rupture in a brittle manner at the net section or at the connection. For slender compression members, there is a plateau at the maximum load in the load-deformation curve. However, when buckling stress is greater than one-half of the yield stress, failure of such members is sudden. Fig. 33 is a theoretical load-deflection graph based on the assumption that yielding in the extreme compression fiber in a straight pin-ended column limits its capacity. Although idealistic, the graph serves the purpose of showing how the plateau changes

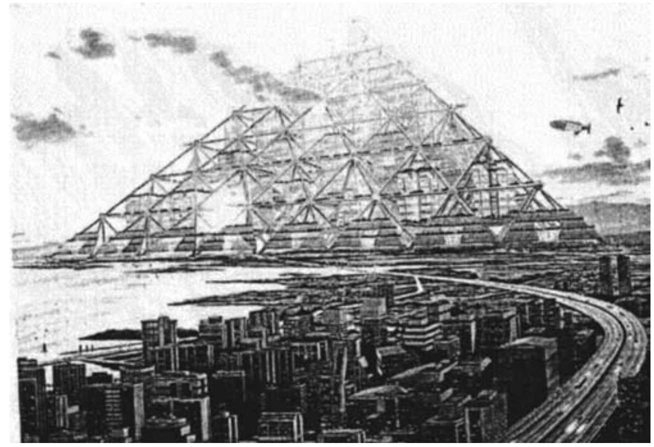

Fig. 36. Futuristic City-in-the-Air (Chilton 2000)

with slenderness. In the practical range of slenderness ratios, behavior of the strut is brittle, and the collapse of the system can be initiated by the buckling of a few members (Schmidt et al. 1982).

There is a serious misconception in the behavior of a SGS. Although the members may be over-designed because of redundancy and sizing constraints, the structure may not be able to reach its full capacity predicted by elastic analysis. This occurs when compression members with practical slenderness ratios buckle before their postbuckling reserve capacity could be developed (Fig. 33). Many attempts have been made to modify the brittle behavior of a SGS, including (1) over-design of compression members to ensure that tension members yield first; (2) relying on nonlinear behavior of eccentrically loaded diagonals to redistribute forces in the chords; and (3) stress redistribution by means of force-limiting devices.

Future Direction of Space Grid Structures

Table 2 shows a list of notable SGSs in existence. The SGS will continue to develop with the extensive use of computers in both manufacturing and design. Computer aided manufacturing allows the cutting and drilling of elements with great precision, while computer aided design can help explore unprecedented complex configurations and geometries. Recent computer design programs have allowed the design of nonplanar forms. Once the form is set,

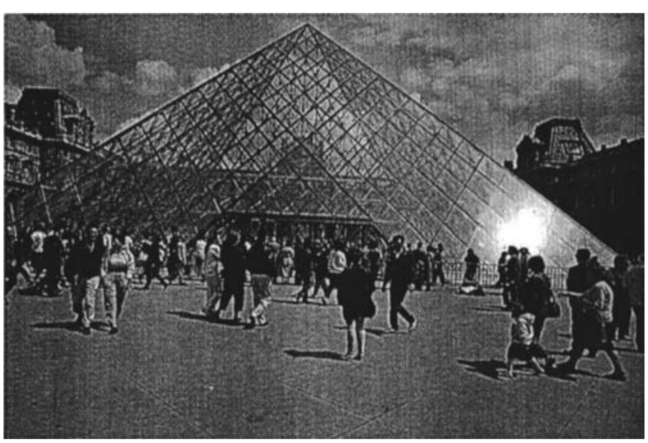

Fig. 35. Louvre Pyramid by Pei and Rice (Patrick Tripeny)

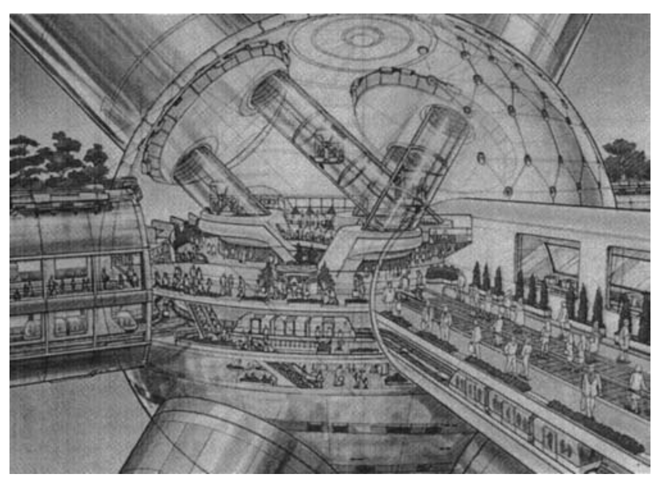

Fig. 37. Futuristic transportation system (Chilton 2000)

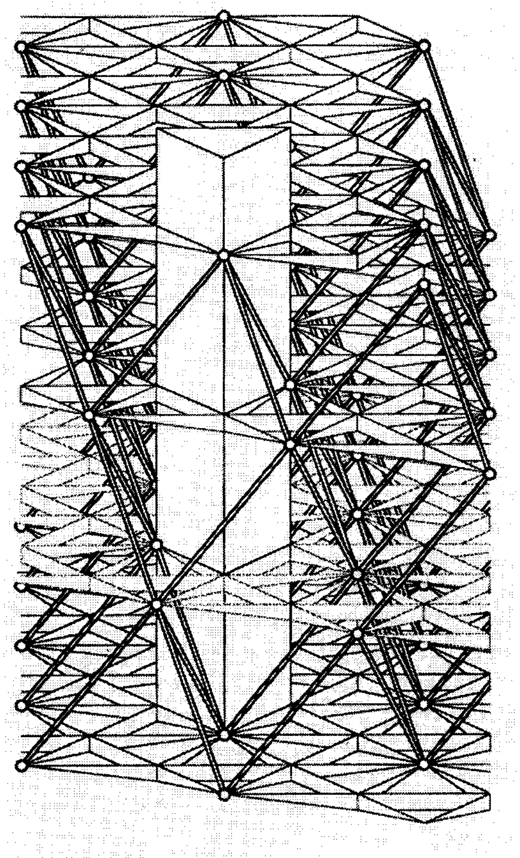

Fig. 38. Multilayer three-way grid hexmod cells (Chilton 2000)

computer programs translate the design into a space frame structure. A good example of this methodology is the roof for the Guangdong Olympic stadium in Guangzhou, China by Ellerbe Becket (Fig. 34).

Advanced work is already taking place in the area of prestressed space-frame structures, where certain members only carry tension forces in different loading conditions. The advantage of these systems is that tension members can be made relatively small in cross section, hence making the structure more transparent. These systems are commonly used in glass wall assemblies, such as the Louvre pyramid by Pei and Rice (Fig. 35).

Another advantage of using tension and compression members is that when assembled correctly they can become deployable structures that can go from a folded state to an expanded structure. While Piñero pioneered these types of structures in Spain in the 1960s, modern examples of deployable structures can be found in the work of Escrig and Hoberman (Chilton 2000).

The Shimizu Corporation in Japan has proposed the building of a pyramidal "city-in-the-air," as a 2,000-m (6,562 ft) high multi-layer grid (Fig. 36), to accommodate over 1 million people during working hours (Chilton 2000). The concept, titled "TRY2004," consists of a square-based, pyramidal, multilayer, space truss mega structure. The concept demonstrates the eminent suitability of multilayered space trusses for the construction of such large-scale projects, using tubular elements where the internal void may be used for transportation. Although this is still a dream, it represents one possible future for the use of space grids (Fig. 37).

The use of polyhedra in the design and construction of buildings of all sizes has been studied by Francois Gabriel, in particular, the architecture of high-rise buildings constructed using six-directional, multilayer, space-filling, space grids, composed of tetrahedral and octahedral (Fig. 38). With this type of space-filling lattice, it is possible to generate continuous horizontal plane grids by orienting the octahedral in two ways: with their long axis set vertically; and with one triangular face in the horizontal plane.

Concluding Remarks

Special structures are what our profession is most proud of analyzing, designing, and constructing. They are the symbols of our civilizations that brighten the horizons of our neighborhoods, and magnify the skylines of our downtowns and uptowns. History, state-of-practice, and potential future of three types of special structures were discussed: shells, tension membranes, and space grids. Considering the lack of standards and codes and direct training for special structures in most engineering curriculums, their design and construction would have not been feasible without the bravery of a few maverick engineers who used their fundamental engineering knowledge to create such landmarks around the globe. It is necessary, however, for academic programs to recognize the unique features of special structures and devote parts of the curriculum to discuss their analysis, design, and construction. If nothing else, these landmarks may serve as great motivation for the next generation of young structural engineers.

Acknowledgments

The writers would like to thank all members of the ASCE Special Structures Committee for their valuable comments, especially Professor Ronald Shaeffer of Florida A&M University and Professor George Blandford of University of Kentucky. The Committee has been very active over the last 10 years, producing a number of publications including those referenced earlier on lattice towers and guyed masts, tension fabric structures, and double-layer grids.

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