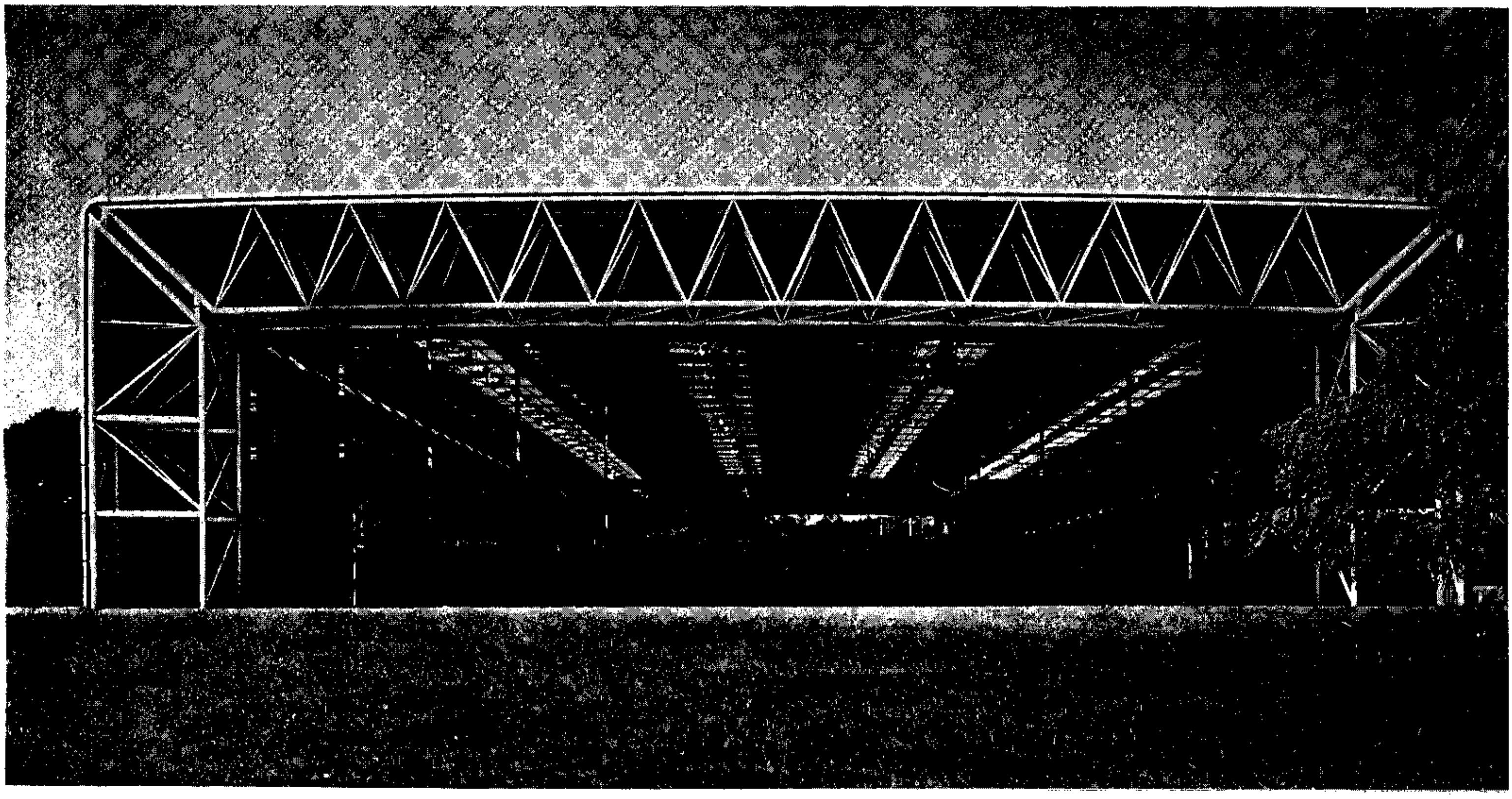
bulletin

of the international association for shell and spatial structures

Prof. D. h-c. Eng. E. TORROJA, founder



Sainsbury Center for Visual Arts

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Volume

Contributions to the Bulletins of the IASS

The Bulletin of the IASS welcomes contributions pertaining to the design, analysis, construction and other aspects of the technology of all types of shells and spatial structures. Papers describing realizations of projects are particularly solicited. All material submitted for publication shall be evaluated as to editorial and technical content. The Editorial Committee reserves the right to accept or reject any manuscript.

Manuscripts shall be submitted in triplicate. The text shall be typed double spaced on one side of standard size sheets. Submit the originals and two copies of all figures, graphs and photographs. Originals of all graphs and figures shall be india ink drawings. Originals of photographs shall be glossy prints of a size suitable for direct reproduction.

Maximum length of manuscripts shall be the equivalent of 20 typewritten double-spaced pages. Brevity is most strenuously encouraged. The organization of all manuscripts shall be as follows.

Title
Author (name, academic degree, professional affiliation)
Summary (not more than 200 words)
Notation (where applicable)
Introduction (including scope of the paper, and statement of the problem)
Text
Conclusions
Acknowledgements (where appropriate)
References
Appendices (where applicable)

In addition, each manuscript shall be accompanied by a list of figures and tables. Except where mathematical derivations constitute the essence of the paper, they should be relegated to appendices, with only the final formulae or results presented in the body of the text. References shall be listed as per recognized international conventions. Authors are requested to cite specific page when referring to a book. A book shall be listed among the references only once. If it is cited a number of times in the text, cite as follows: (cf. Ref. N. p. nn).

Contributions must contain material previously not published, or not readily available to the members of the Association. Submittal of a manuscript shall be interpreted as constituting a grant of publication rights to the IASS. Authors of accepted papers will receive twenty-five reprints free of charge after the publication of the article in the bulletin.

Papers should be sent to:

I.A.S.S. Secretariat
Alfonso XII, 3
Madrid-7
Spain

bulletin

of the International Association for Shell and Spatial Structures

n. 73

August 1980

Volume XXI - 2

index

secretariat: Alfonso XII, 3 - Madrid - 7 (Spain)

ISSN: 0304-3622 Depósito legal: M. 1444-1960

Rafael López Palanco, new President of the I.A.S.S.

During the last E.C. meeting held in Oulu, Rafael López-Palanco, Secretary of the IASS from 1962 to 1974 and member of the E.C. from 1974 to 1980 was unanimously elected New President of the IASS. The next issue of our Bulletin will include a letter of the New President to the IASS members and to the Bulletin readers.

In the same meeting, Prof. Paduart was also unanimously nominated as «Past President» for a period of two years not only as recognition of his great labour in the IASS but so as to make use too of his great experience in the future development of the W. Bureau.

Prof. Haas, that expressed his desire of not being candidate for the E.C. renewal, was elected member of the Advisory Board.

The continuing development of design and construction techniques of shell structures is resulting in an increasing fund of information of practical interest to Architects, Engineers and Contractors. The aim of furthering all branches of this progress has inspired the formation of the **international association for shell and spatial structures,** whose purpose is to organize meetings and congresses for the interchange of ideas and their dissemination by means of periodical publications.

Everyone interested in the various branches of shell techniques and their architectonic possibilities or realizations is invited to join this International Association.

To become a member or to obtain more detailed information, please write to the Secretariat of the International Association for Shell Spatial Structures, Alfonso XII, 3, Madrid-7 (Spain).

Cover:

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University of East Anglia, Gt. Britain

Architect: Denys Lasdun

How have concrete shell structures performed?

An engineer looks back at years of experience with shells

ANTON TEDESKO [1]

The life span of a structure in some ways is comparable to that of human beings. Some show symptoms of decline at an early age and others grow old still performing well. Design, construction, and the environment determine the life span of a structure.

The old adage: «Anything worth doing is worth doing well» applies in great measure to concrete shell structures, as seen from the more than 45-year history of shell construction in America. Doing a job well saves over-all cost and results in less trouble over the years than does a poor job, i.e., one that is not well designed or well built and where the total life costs may very well exceed the first cost by a substantial margin. The «short-cutting of corners», the holding down of initial expense, often results in cost for corrections, repair, and increased maintenance requirements. Corrective measures invariably are expensive and often difficult to achieve. Good design and careful planning of construction is the inexpensive insurance against future troubles; even more so today when maintenance costs are so high owing to energy and labor, to financing practices and to tax laws. Moreover, doing a job well from the very beginning results in desirable side effects: satisfaction and justifiable pride.

Dr. Hermann Rühle, Editor-in-Chief of IASS's Bulletin, asked me to report on experiences, good and bad, with American shell structures that have been in service for a long time. In answer, may I first point out that the United States, as seen from a distance, is often erroneously perceived to be a country with uniform customs, rules, and requirements. The United States, with its contrasting climatic conditions, with its different political subdivisions (states, cities, counties, etc.) and its variety of laws, is bound to have great variety in engineering practice. This variety ranges from the absence of any regulations for building design and construction supervision in the open country to the strict observance and enforcement of good practice in certain of its cities and in a number of the states. Sometimes existing code restrictions, formulated before shells were introduced, hindered the execution of the best shell design. In some areas of the United States a designer, without any interference can make every imaginable error, and is held accountable for negligence only when there is a threat of failure or there is an actual injury. The quality of a shell structure, therefore, depends somewhat on who designed it, and on where, by whom, for whom, how it is built, and how maintained. Guarantees for satisfactory performance in design and construction seldom extend more than 2 years beyond the completion of the structure.

Until about a decade ago, it was not customary to have the calculations of a designing en-

Editor's Note: The author, having gained shell experience by working for Dyckerhoff & Widmann in Germany in the early nineteen-thirties, was sent to the United States to help Roberts & Schaefer Company of Chicago in the introduction of shell structures in America. He has designed and supervised the construction of many major shell structures and was the chairman of the committee which prepared the American Concrete Institute Report on the Practice of Concrete Shell Structures.

[1] Dr. sc. techn., Dr. Eng. (hon.), Consulting Engineer, Bronxville, N.Y., USA. Hon. Member IASS.

gineer reviewed by another engineer. Recently, such reviews have become more and more frequent, especially in connection with major and bold structures.

Where shortcomings were found and where failures have occurred it has often been difficult or impossible to get data on the incident, as many owners will not give information to avoid unfavorable publicity. Failures often result in years of litigation, and the influencing of people by the publication of details is forbidden as long as a case is active in a court of law.

Some years ago, a few failures or near-failures of shells were due in part to the promotional efforts of organizations interested in the manufacture, the selling and in the increased use of cement. Engineers were encouraged to use shells, but some of these engineers were not fully qualified to design them. Also, there were those who misunderstood the seductive simplicity of Candela's work and made errors in trying to imitate him without having his experience and construction background.

There is also a history of trouble when designers of some experience in striving for greater slenderness or elegance, were not aware of the risks they were taking. They did not provide sufficient stiffening, strong enough ribs or unyielding supports to prevent movements, a flattening or creep of a shell, resulting in instability. Where these designers detected unforeseen movements in proper time, emergency measures were taken, usually providing supports in addition to those contemplated before the start of the construction.

Thin shell concrete should not only have strength but also weather-resistance, unless fully protected from atmospheric influences. Air entraining, as well as good concrete placing and finishing practices are primary factors in obtaining durable structures requiring a minimum of maintenance. With time, a limited degree of change in the concrete may be expected and is acceptable. However, concrete need not deteriorate. When concrete defects appear, investigation always shows that the specifications were either defective or not followed, and that for some reason the owner did not get what he ordered.

Before the days when creative designers began using shell structures as a form of visual expression for aesthetically pleasing space enclosures, it was quite common to find concrete shells being considered in competition with structural steel as structural supports, often hidden, within old-style institutional and monumental buildings. The shells of that period usually were covered with copper or tile roofs and they seldom were visible from beneath because of hung ceilings. Like building skeletons of reinforced concrete or structural steel, they will continue to serve unseen until removed to make room for new buildings filling new needs. For many years these shells have served the purpose for which they were intended; no unfavorable history is known.

CASE HISTORIES

The *Sports Arena in Hershey, Pennsylvania*, was my first major structure [1] (Figure 1); I was on my own in deciding what rules to follow. It was the first long-span thin shell structure in the United States where the stability of the shell was an important consideration. The collapse of an airplane hangar in Germany due to creep buckling was very much on my mind at that time *. This 1934 incident was hushed up, as it involved the destruction of numerous training planes of Germany's growing air force. I was concerned with the buckling safety of the 3 ½ in. (9 cm) Hershey shell which cantilevered 19 ft (5.8 m) beyond the supporting arches of 222 ft (68 m) span (see Figure 3).

Not satisfied, during those days of plain reinforcing bars, with the usual lap splices of hooked bars in the zones of high tension of the supporting arches, I provided for the joining of bars by threaded sleeves. The decentering of the thin shell a few days after concrete placing I permitted whenever the concrete had reached a specified E-modulus as established by the deflection of test beams cured on the job and tested at the age of 2, 3, and 4 days. To achieve early decentering and quick reuse of form work, an early-high E-modulus and

* Figure 2 shows the cross section and supporting frame of this 2-span hangar, 25 m deep; door openings were 40 m each.

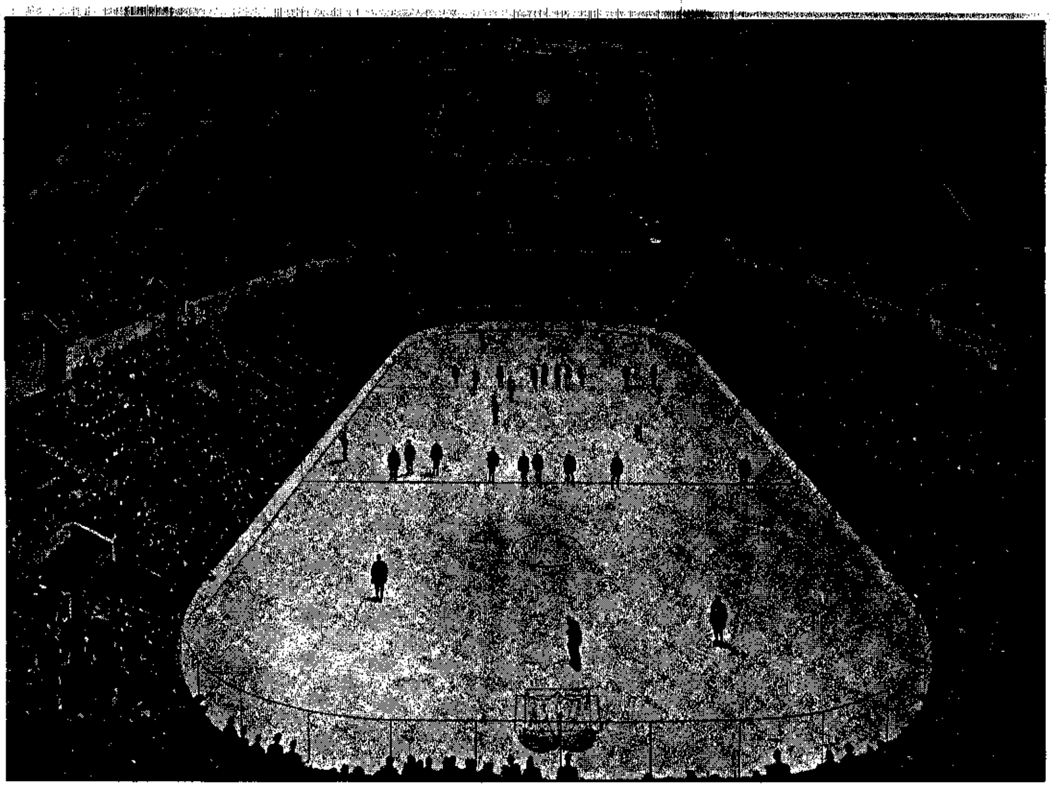


Fig. 1. Sports Arena Hershey interior view.

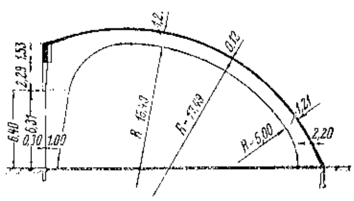


Fig. 2. German Aircraft Hangar collapsed May 1934.

early high strength were required. I placed no emphasis on the usual compressive strength tests because the E-requirements at 3 to 7 days were more demanding. I took precautions *

* I established central vertical control boards at ground level beneath the shell unit to be decentered. Scale drawings of the curve of the shell were pinned to these boards. Vertical wires from various points of the shell were led over pulleys to the control panel; the wires, held stretched by weights, indicated, in full scale at the right location of the drawing, the increasing deflections of the shell as the form centering was slowly lowered away from the shell in a planned cycle. The behavior of this shell and of subsequent shells increased my confidence, leading to much simplified procedures and observations on structures which followed.

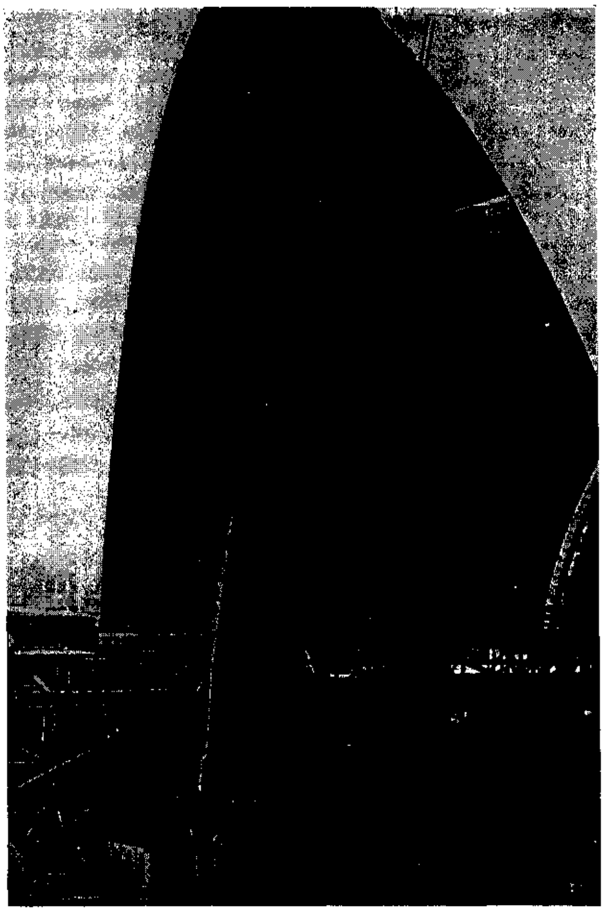


Fig. 3. Sports Arena Hershey - roof unit under construction.

during the decentering so there would be no deviations from a smooth shell curve; in retrospect today this looks as though I was over cautious.

This sports arena was unusual because there was no contractor involved; local labor and students built the structure, working for the construction arm of the Hershey Chocolate factory. Construction was done slowly and in a very conscientious manner. The roof was protected by 4 layers of roofing felt laid in asphalt. I was unable to find out the actual cost of the structure because the bookkeeping was such that the cost for constructing the arena was lost in the cost of Hershey's chocolate bars.

Recently, almost 40 years after completion of the structure, the owners considered supporting a 24-ton concentrated load from the crown of the roof. They asked what provision could be made for the support of an air-conditioning system and of a sophisticated score board at the center of the arch shell structure. I examined the structure and found it in excellent condition; no cracks could be observed in the tension zones of the concrete. Concrete cores were taken and these exhibited a concrete compressive strength in excess of 7000 psi (50 MPa). Approval of the proposed installation of the additional load could be granted without any strengthening of the structure.

At about the same time I was asked to inspect another somewhat similar but smaller concrete shell structure of 121 ft (37 m) maximum span, built 30 years ago, an industrial *Facility for the Repair of Fire Department Vehicles.* This building, however, was constructed without the strict supervision existing for the sports arena just described; the only concrete requirement was that the 28-day test cylinders meet the design strength. Here a concrete was used which barely met the strength specifications and was not weather-resistant. Maintenance was lacking and rain water seeped through roofing and flashing. The concrete cracked badly and disintegrated due to freezing and thawing. The monolithic connection between shell and supporting ribs was gone in many places. Reinforcing bars were either missing, or had rusted through during more than 25 years of exposure. In one place, a 30 ft (9 m) long crack separated rib and shell, wide enough to stick one's hand in. For safety, the structure now requires major restoration, including structural steel members along the ribs to connect and support the shell.

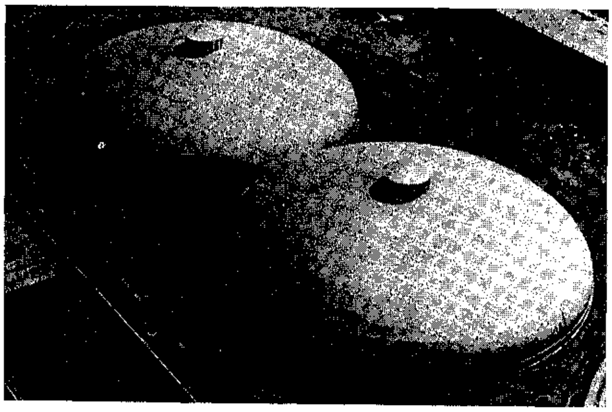


Fig. 4. Domes in Minnesota.

Domes of 150 ft (46 m) diameter were built in *Hibbing (Minnesota)* forty years ago. They are of elliptical cross section and have a rise of 33 1/2 ft (10 m) at the center; the shell thickness is 3 1/2 in. (9 cm), except that it increases to 6 in. (15 cm) at the springing line (Figure 4) [2]. These domes serve as covers for trickling filters at a waste water treatment plant. Roof cover consisted of only an asphalt coating with an aluminum finish.

These domes should have been cast in a continuous operation but, since it was a construction operation designed to keep unemployed people busy, it proved difficult to enforce procedures which would have been quite natural for a competitively built construction project. Consequently, cold joints were left around horizontal rings.

The highly humid operating conditions inside the domes saturated the concrete shell during its early life. Furthermore, the shell was subjected to freezing and thawing during the severe winters of Minnesota. Some efflorescence appeared along horizontal rings where construction joints had been located and where the concrete did not have the desired monolithic character; the roof covering blistered and spalled off along these lines. After inspecting the domes, I recommended cleaning of the shell ceiling and applying coats of linseed oil. For the exterior I recommended repair of the asphalt coating and the application of a roofing membrane. The concrete ceiling was soaked with linseed oil; a roofing membrane, however, was not applied, but the exterior received occasional patching with asphalt.

Recently, 35 years later, I inquired about the condition of the domes, and was told that 8 years ago the structures were rehabilitated. Defective areas of the exterior were repaired with shotcrete and the outside was then painted; freezing and thawing now causes only minor defects. The interior ceilings were sandblasted and received an epoxy coating which prevents the saturation of the concrete by the existing vapor. The domes are now considered to be in fair condition.

Well-insulated, *Flat Segmental Spherical Domes* were used by architects for different kinds of buildings and more prevalently to cover trickling filters and reservoirs. The domes were made in two steps. First, a continuous spiral of a 4 in. (10 cm) thick preformed strip of polystyrene foam, an extruded material consisting of closed cells, was shaped and fused with the use of heat into a dome which became the self-supporting form work. Second, the thin shell concrete dome was cast out of a portland cement mix attaining special properties by the addition of a plastic emulsion. This durable concrete of high flexural strength does not shrink or crack; the thin shell when matured carries the foam dome on which it has been placed.

Failure of such early domes of 180 ft (55 m) diameter under snow load occurred. I was called in and found that the shell was made too thin because of an error in the use of the buckling formula. Subsequent domes were therefore made a little thicker and are thereby successful installations. The concrete used has desirable properties, does not require a roof covering; the interior is insulated by an effective inorganic material.

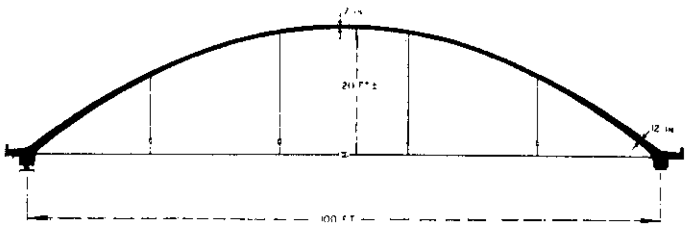
A materials storage hall for a *Cement Plant at Hudson, New York,* built 40 years ago, has a shell roof with barrels spaning 105 ft (32 m) providing a column-free area of 105 ft by 560 ft (32 m by 170 m). Edge members and stiffening end diaphragms of the 15 barrel shells were cast in place but the contractor chose shotcreting for the segmental shell. It was concluded, thereafter, that this was not a practical method for placing concrete of a large flat shell structure and I do not know of any similar shell shotcreted thereafter. The structure is in excellent condition.

A *Plant for the Manufacture of Portland Cement* is located in an area of *Pennsylvania* where compliance with building regulations was not required. Close to 20 years ago, several industrial buildings were built there using arch shell roofs of 100 ft (30 m) and 110 ft (33 m) span. Each of these barrel roofs is of considerable length and rests on high columns. The arch thrust is taken by tension rods spaced 20 ft (6 m) apart, located above the level of bridge cranes travelling the length of the structure.

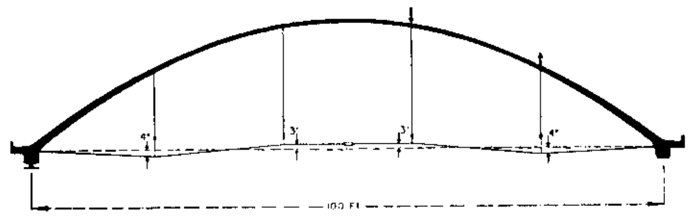
Thirteen years after completion, the structures were in trouble; in fact, the section of the roof near the end of a materials storage building collapsed. I was asked to investigate and the following case history unfolded:

Someone had made design calculations for a barrel shell roof, thinking that the structure would carry like an arch. The designer assumed that the arch would follow the shape of a catenary and that moments due to dead load would be zero. Drawings were supplied to the builder of a form centering assumed to result in the construction of arches of the desired shape. No evidence was found of the involvement of a responsible engineer during the design and construction stages. The structures were built by a contractor who was furnished cement by the owner and who placed good concrete and reinforcing steel as indicated on available sketches.

My investigation indicated that the drawing of the form centering contained dimensional errors and that anyone following this drawing would end up with an irregularly shaped roof. This apparently was discovered by the builder who, making his own adjustments, substituted what he considered a non-objectionable, smooth shape. Unfortunately, this shape was not one that an engineer would have chosen and the structure was thus shaped with built-in dead-load moments which were much greater than the anticipated maximum live load



a. Arrangement of tension tie rods.



b. Arrangement after prestressing.

Fig. 5. Barrel Shell in Pennsylvania.

moments. The concrete, however, was strong, so no immediate trouble developed. The structure deformed through the years as creep continued and as live loads became much greater than anticipated. This live load came from cement dust which fell on the roof and turned into successive layers of caked-on deposits. The sizeable bending moments created deflections, which in turn created additional moments, leading to additional deflections until the geometry of the structure changed to such an extent that the roof became unstable under its own weight. The collapse took place where the cement dust collected on the roof had provided an extra heavy live load.

My rehabilitation of the structure was based on the fact that the loads on the arch were too small near the crown and too large near the quarter point. After the undesirable live load was chipped from the roof, its exact curvature was established as was the additional loading necessary to keep bending stresses within safe limits. I borrowed from the existing force in the tension tie to obtain vertical prestressing forces which would improve the stress condition in the roof shell (See Figure 5). A downward force for the shell at hanger locations near the crown was obtained by shortening these hangers 3 in. An upward force for the shell at other hanger locations was obtained by inserting a pipe strut with a built-in screwjack. This jack was turned so that the strut lengthened until the tension tie showed a downward deviation of 4 in. With a hammer I hit individual tension rods and recorded the sound on a tape recorder. Tuning the tension ties to a uniform pitch, I was satisfied that they would be stressed to take the same force.

Roof measurements, made every 3 months during the past 5 years, not only indicate that the dangerous increase of creep deformations was stopped but also that there was a continuing tendency towards reversal of deformations. Observations continue at regular intervals, but seem to indicate that the structure is now stable.

Many years ago, one of a series of 160 ft (49 m) span *Hangars at an Air Force Base in Ohio* was subjected to a severe unscheduled fire test [3]. An airplane crashing into the door of the hangar set fire to planes and caused explosions of fuel resulting in great heat and upward pressure. The 4 in. (10 cm) roof shell, being equipped with orthogonal reinforcing nets both top and bottom, could take the explosion loads like a pressure vessel. The upward pressure and the temperature differentials resulted in cracks, and calculations indicated that these could have been caused by an upward explosion load thirteen times the downward design live load. On the basis of an examination of the structure, the ceiling was sandblasted and patched with shotcrete to fill the cracks. The hangar roof was then successfully test-loaded with double the design live load after which it received a new roof covering. Figure 6 shows the interior of the hangar before and after rehabilitation. The cost of the repair amounted to only 5 % of the original construction cost of the hangar. If the hangar had been of steel or wood construction the accident would have resulted in a complete loss of the building.

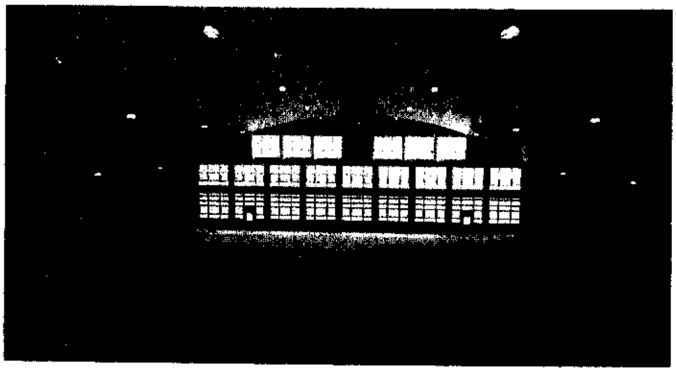
During my earlier years, still influenced by the German buckling failure of 1934, I designed the cylindrical shells of large span structures with an increasing curvature along the shell edges. This placed a penalty on the supporting arch ribs because their shape did not fit the pressure line. The resulting sizeable bending moments controlled the dimensions of the prominent ribs in hangars of the early 1940's of which *Naval Hangars at San Diego* (Figure 7) [4] are examples. These hangars encountered earthquakes without damage, leaving us with the feeling that we had provided more than ample stiffness. A curious accident occurred some years ago: a missile was shot by accident from within one of these hangars. It passed through the rof and left as its only evidence a clean-cut round hole in the roof shell of the hangar.

As my confidence in the stability of our shells increased, we began to favor the supporting arches by giving them a more desirable shape. This change necessitated increasing the shell thickness since the better arch shape caused the shells to become flatter. At the same time, span-rise ratio, arch-column stiffness ratio and arch spacing were systematically varied to study their effect on economy. The overall result was the more pleasing appearance of long-span structures of the late 1940's, shaped to follow more closely the pressure line of the arch; the shells as usual were connected to the bottom of the arch ribs for optimum stiffness, smooth ceilings and simplest movements of the form centering. The *Air Force Hangars* of Figure 8 [5] are examples of this type and show how much better aesthetically they were than their predecessors; their design proportions resulted from thorough investigations, including strain gauge measurements on full scale structures. These structures had ample stiffness and now, after 30 years, are still in excellent condition.

Here again I wish to tell of an unforeseen incident upon completion of one of the roof units: The huge arch form centering, supported on wheels travelling on rails, had been moved forward late one afternoon into the position in which it was to receive the reinforcing and the concrete for the next roof unit. That evening the contractor's workmen neglected to apply brakes and to anchor the form traveller by guying cables as required. An unexpected windstorm during the night caught the 304 ft (104 m) wide wood-trussed centering structure and rolled it back under the completed part of the hangar where it jammed against the concrete on one side, twisted, tilted, and collapsed: Figure 9 shows the damage that resulted in an unexpected delay in the construction.

It was a stimulating experience to work with outstanding architects. For the *Lambert Field Air Terminal Building at St. Louis*, architect Minoru Yamasaki wanted three units of intersecting barrel vaults (Figure 10) [6]. There were

a. Interior View after fire.



b. Interior View after repair.

Fig. 6. Aircraft Hangar in Ohio.

Fig. 7. Sea-Plane Hangars at San Diego.

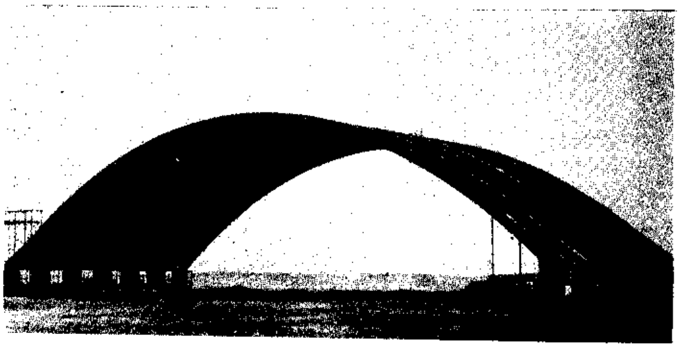


Fig. 8. Air Force Hangar - General View.

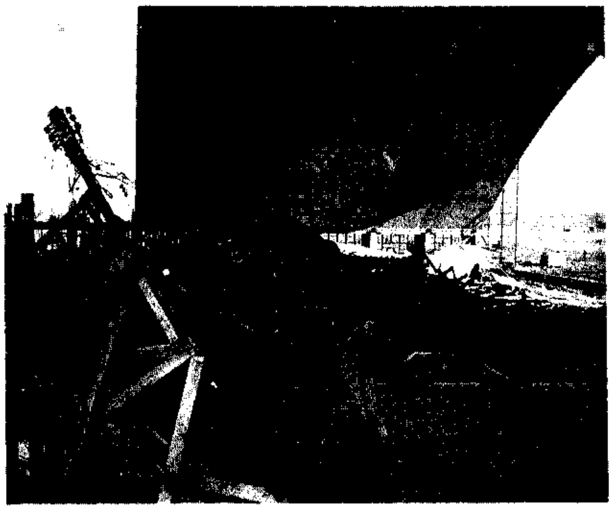


Fig. 9. Form Centering Collapse - Air Force Hangar.

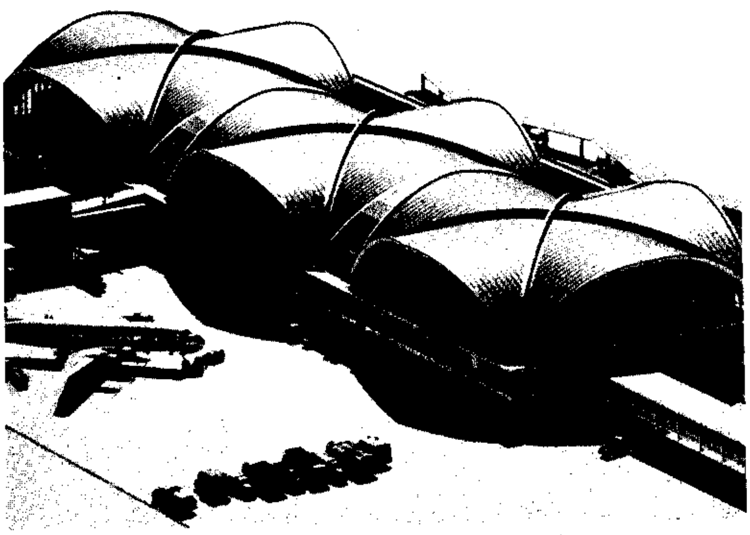


Fig. 10. Air Terminal - St. Louis.

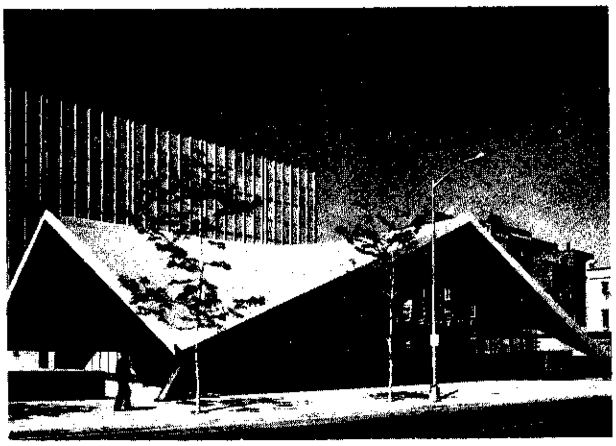


Fig. 11. Shell at Denver.

lengthy discussions as to the most pleasing and effective cross sectional shape for these shells: catenaries, ellipses, parabolas, and circles were considered. During the design stage these curves were displayed in large scale on the walls of our office; it was revealing to find that few people when confronted with these shapes could tell the difference.

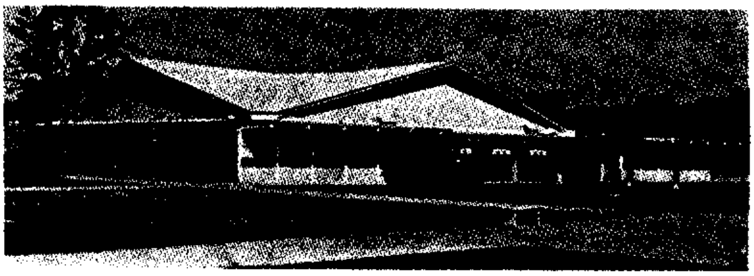
Where the shells intersect, forming the cross vaults or groins, we placed diagonal ribs, the size of which was sometimes criticized later. Unfortunately, the lower stories of the terminal building had already been designed by local engineers before my firm entered discussions with the architect as to the roof design. The framing for the lower stories thus fixed the locations at which we could support the roof structure. These fixed points and the vertical clearance requirements for the terminal's concourse, therefore, resulted in ribs burdened by flexural action. Had it not been too late to move the supporting points beyond the concourse area, it would have been possible to carry the structure by more elegant ribs.

In view of the fact that the ribs had to take sizeable bending moments and could not be kept as slender as members subjected primarily to compression, the choice of the cross sectional curve of the shell became a decision based on aesthetics and construction economy rather than on structural considerations. The use of two-radli-curvature barrel shells simplified the form centering, and resulted in savings by making it easier to keep thickness of concrete and acoustical plaster to required tolerances. Somehow, the architect felt later that a parabolic shape would have been more satisfying. An additional cross vault was recently added, enlarging this attractive terminal to 4 units, 20 years after the basic units were completed. Each unit is square with an overall length of 146 ft (44½ m).

The *Hyperbolic Paraboloid Shell at Denver* (130 ft by 130 ft - 40 m span, Figure 11)[7] built more than 20 years ago, was part of a large development by an owner who was also the construction contractor. This delicate shell is in vivid contrast with the balance of the project which included some major reinforced concrete structures; the shell therefore received special attention. Again, it was enjoyable to work with an outstanding architect, I. M. Pei, who wanted for a prestigious Denver location, a shell surface uninterrupted by protruding ribs. The crown of the structure is a critical area for buckling on account of great compression and the lack of curvature; thus stiffening ribs were necessary for the proposed flat shell structure. We compromised on wide rib bands along the ridges with heavy reinforcing as insurance against creep deformation. The 2 in. layer of glass fiber insulation concealed under the roof covering was interrupted along the ridges of the roof to make space for extra depth of the invisible concrete rib bands.

Numerous studies were made as to be best visual appearance and positioning of marginal ribs of the gables. The structural action of these was confirmed later by fine hair cracks which developed after decentering at the top of these ribs at their peaks; these, however, did not present a problem. Here, as so often, we were reminded that the principal part of our learning process comes from observing our structures in action. Shell design is not like designing a beam; designers need to have experience with full scale structures. Observations at this Denver Shell are a good indication of the fact that the oversimplified membrane theory gives an incomplete picture of the structural action of such a shell. The elegance of this Denver structure is due largely to the architect's careful attention to proportion and handling of detail and to the fullest cooperation between architect and engineer.

Concrete placing operations for the shell were scheduled for a dry summer day and had to be completed on that day because the next was a national holiday. The temperature was expected to be 102° F (39° C) in the shade, but there was no shade, and very low humidity, typical of Denver. I arranged for concreting to start at 6 o'clock in the morning to minimize water loss due to the sun, and I had ordered a special mix, designed for the hot-weather concreting of a thin slab. Early in the morning I was confronted with a fleet of mixing trucks filled with standard concrete as contracted for the entire project by the owner-contractor, who refused to pay more than the unit concrete cost agreed upon between him and the concrete supplier. I knew that the extra cost for hot-weather admixtures was justified and I faced the decision as to whether or not to reject the concrete and to close down the job, or to accept an end product less satisfactory than desirable. Under the given circumstances I concluded that non-acceptance of conditions forced upon me would



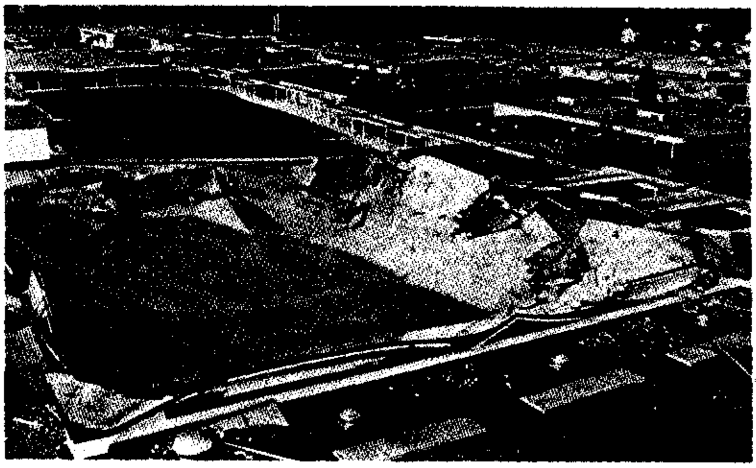


Fig. 12. Collapsed Shell in Virginia.

lead to repercussions and disagreements with non-technical people, and I permitted concreting to proceed even though the mix did not meet the justified requirements I had specified. The placed concrete, in spite of continuing wetting, lost some of its water and the exposed edge members of the shell soon thereafter began to show fine hair cracks. Acutely aware of these fine cracks which could have been avoided, I have inspected them whenever stopping in Denver in later years. In spite of freezing and thawing exposure, their appearance has not changed through the past 22 years; indeed most people may not be aware of them. The roof, with the exception of the exposed concrete of these edge members, is covered by insulation, several layers of roofing felt and pebbles embedded in the asphaltic material covering the felt. The structure is well protected from environmental influences and is in good condition.

A similar but flatter *Hyperbolic Paraboloid Shell* of only 100 ft (30 m) span built in *Virginia* 17 years ago, collapsed 8 years after construction (Figure 12)[8]. Crown deflections due to

Fig. 13. Fire-damaged Unit of Ohio Warehouse.

yielding supports, creep, shrinkage, and the use of light-weight concrete contributed to the stability failure of this shell. The designer remembers asking for my comments on his drawings when they were made, and he remembers my telling him that I considered his shell too flat.

Large industrial areas were covered during World War II with *Cylindrical Shell Roofs* spanning 40 to 60 ft (12 to 18 m). These structures were built in mass-production-procedures with the use of trussed timber forms that were rolled ahead as the work progressed. Early re-use of form-work was important, and heat had to be applied during cold weather periods to obtain the desired early high strength of the concrete. On some of these projects the heating was accomplished primitively by open coke fires placed in metal baskets under the concrete forms. Two incidents are known where form-work over-heated and the supporting timber

Fig. 14. Fire at Virginia Installation.

truss caught fire and burned away. Partly cured concrete shells thus were subjected to great heat and sudden load changes.

A fire at a *Military Installation in Ohio* destroyed the formwork 10 to 20 hours after high-early-strength concrete had been placed. It was decided to remove the concrete unit considered to be fire-damaged, but breaking down the structure was no easy task. The concrete shell and its framework proved flexible and strong; the heated concrete had obtained amazing early strength. In retrospect, the unit could have been test-loaded and might have been saved; however, the owner paid for and had a right to obtain a perfect structure. Replacement at the contractor's expense was therefore justified (Figure 13).

Another fire at a *Virginia Installation* burned away the supporting timber centering at a time when the concrete could have gained only a part of its normally anticipated strength (Figure 14)[9]. Minor spalling and cracking occurred in the concrete shell section which had previously been placed in the area adjacent to where forms where still in place. No visible damage could be observed in the ceiling of the section where the fire had occurred, the plywood forming surface having insulated the concrete against the intense heat. The top of the shells showed a typical pattern of fine cracks indicating that the shells had bulged upward from the expansion caused by the rise in temperature. Roof reinforcing was exposed in only a few places. Where spalling occurred, it was quite shallow. Damaged sections were repaired by sandblasting and by removal of all unsound concrete. Some of the cracks were chiseled out to permit proper grouting. Surface spalls were filled with grout. The underside of the roof was covered with a thin layer of shotcrete to restore appearance. One month after the fire a full-scale load test resulted in the anticipated deflections and strain, with the structure recovering fully after removal of the load. Fire damage repair costs were 2½% of the amount spent for the structure up to the time the fire occurred.

Many millions of square feet of covered roof area of *Industrial Shell Structures* stood up well through the years without distress (Figure 15)[10]. These shells usually were roofed by composite tar-felt coverings; two layers of felt required maintenance or replacement after about 10 years; a 4-ply tar-felt roof was assumed to have a 20-year life. Where these roof coverings and flashings were kept in good repair or were renewed, the condition of concrete shell roofs after a service life of 40 years appeared as good as at the very beginning of their lives. Thin shell installations without roof coverings I do not recommend, not even for warm and dry climates. Examples exist, however, of a few well-executed structures where the shell was left unprotected and the concrete does not show signs of deterioration.

I have mentioned examples of unfortunate experiences because it is easier to draw lessons from examples of poor performance than from good performance. Trouble or failures, however, were seldom encountered on significant American projects. Such failures comprise only a small fraction of the executed total volume of structures, usually involving inexperienced engineers or owners who were unaware of good and accepted practice of design, construction, maintenance, or the use of materials.

Notwithstanding the examples with unfortunate experiences which have been singled out, it is gratifying to report that poor performance is relatively rare, and that the overwhelming number of shell roof installations in America, about 30 million sq ft or 3 million square meters, after many years of service, are impressive to the observer and found to be in good condition [11].

As a result of my years of experience I would stress these three important points on which depends the success of shells: 1. Designers learn from having experience with full-scale structures. 2. Engineers should visualize the actual construction during the design stage; the economy of shells depends on the close collaboration of designer and builder. 3. Even in working with architects of prestige the engineer must stand firm, making it clear as to what can be done and what should not be done.

The adherence to these ideas is essential for the creation of structures which will be sound, practical, more economical, and safe.

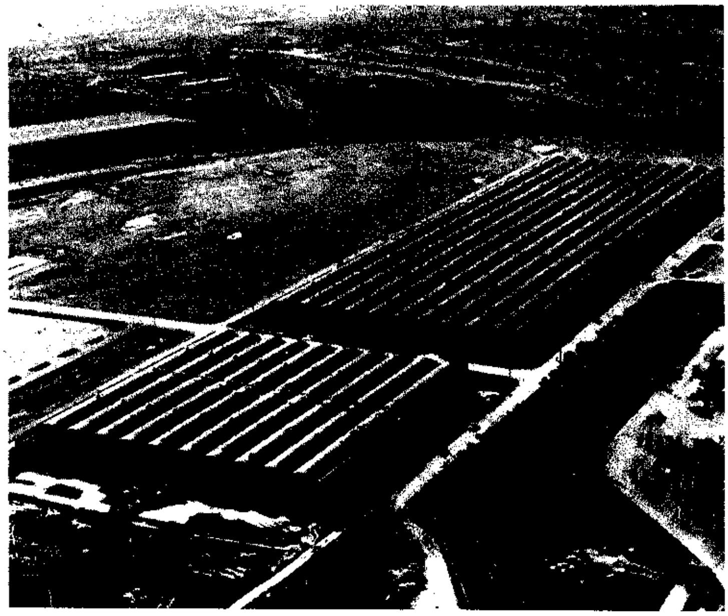


Fig. 15. Industrial Installation in Pennsylvania.

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