GIANT PRESTRESSED HP SHELL FOR PONCE COLISEUM

T. Y. Lin*
Professor of Civil Engineering
University of California
Berkeley, California

Felix Kulka
President
T. Y. Lin International Consulting Engineers
San Francisco, California

Kam Lo
Vice President
T. Y. Lin International Consulting Engineers
San Francisco, California

The recently completed hyperbolic paraboloid concrete shell roof for the Ponce Coliseum in Puerto Rico represents a significant thrust forward in the design of shell structures.

The shell roof cantilevers 138 ft (measured along the edge beams) making it the world’s longest cantilever shell of the HP type.

Post-tensioning was used both in the shell membrane and also along the edge beams to control deflections and stresses. In addition, the piers on opposite sides of the shell were post-tensioned beneath the ground to resist the horizontal thrust.

The finite element method was used to analyze the structural behavior and to calculate the moments, stresses, and deflections in the shell and edge beams.

This article describes the design and construction techniques used in building this shell roof.

*Also, Board Chairman, T. Y. Lin International, Consulting Engineers, San Francisco, California.
One of the world's largest hyperbolic paraboloid shell roofs (see Fig. 1) was completed in late 1971 for the Ponce Coliseum in Puerto Rico. The roof structure covers an area of over 60,000 sq ft and provides shelter for 10,000 seated spectators. Working with a limited budget of 2.4 million dollars, it was decided to use only natural ventilation. This decision was logical since one could take advantage of the local prevailing winds.

A hyperbolic paraboloid roof was chosen because it conforms to the seating pattern and allows unobstructed air flow across the coliseum. The roof is supported on only four supporting piers, thus minimizing air flow obstruction and allowing an open view. By using post-tensioning in both the shell and the edge beams, stresses and deflections could be controlled to obtain an optimum design.

**DESCRIPTION OF SHELL**

The shell structure (Figs. 2-5) has overall plan dimensions of 276 x 232 ft. The complete roof is made up of four similar 4-in. thick saddle-type shells, connected to interior and edge beams to form a structure supported by four piers at the low points. The piers are located at the centers of the four exterior edges. Thus, the entire structure is symmetrical about both axes, which run through opposite piers.

The high points of the shell, rising 40 ft above the low points, are at the four corner tips and also at the center of the entire roof. The clear spans, between opposite piers in the two directions, are 271 and 227 ft.

The normal 4-in. thick shell is gradually thickened over a 5-ft wide strip adjacent to each beam to a 6-in. thickness where it joins the beams (see Figs. 4 and 5). The shell has a cantilever extension beyond the exterior face of the edge beam which varies from 10 ft 6 in. at the piers to zero at the corner tips of the structure (see Fig. 5).

The cantilever edge beams are supported only at the piers and have a constant width of 30 in. throughout their entire length. The depth of the 138-ft cantilever edge beams (see Fig. 6) varies linearly from 18 in. at the
corner tips to 53 in. at a distance of 17 ft from the abutment. From there it increases more rapidly to a maximum depth of 94 in. at the abutment. Similar depths for the 116-ft cantilever edge beams are 18, 44, and 87 in.

The interior beams are 60 in. wide. The depth of the 138-ft long interior beams (see Fig. 7) varies linearly from 18 in. at the center of the shell to 47 in. at a distance 10 ft from the abutment. From there it increases more rapidly to a maximum depth of 72 in. at the abutment. Similar depths for the 116 ft long interior beams are 18, 40, and 60 in.

Fig. 8 is a cross section of the shell slab showing the location of the tendons at interior and exterior edge beams.

The thrusts from the beams are delivered to large pier type abutments (see Figs. 6 and 7), resting on pile foundations. The resultant of the thrusts lies closely along the 45-deg inclined pier. Prestressed tie beams at the foundation level are used to connect opposite piers and carry the unbalanced horizontal thrust coming from the interior beams.

Smaller tie beams running diagonally between abutments are also used to insure that no relative motion between the abutments will occur in case of ground motion due to earthquakes. This assembly of four similar shells into one structural unit provides inherent stability.

**COMPUTER ANALYSIS**

The structure was designed to insure that it has adequate strength, stiffness, and stability under all possible load
conditions. The construction sequence was also carefully considered in the design process.

It was apparent from the beginning that the control of the deflections and the stresses in the cantilever edge beams was critical to the success of the design. A concept of load balancing by properly prestressing the edge beams suggested itself as only a preliminary approach. It was recognized that an accurate analysis was needed to determine the interaction of the shell and edge beam under the dead load of the structure and prestress in the edge beams.

During the final design period in 1968, a detailed computer analysis using the finite element method was performed under various loading and boundary conditions. Because of structural symmetry only one quarter of the total structure had to be considered. To simulate the shell, a quadrilateral mesh, with the lines running parallel to the beams, was selected over the structure. Two triangular plane stress finite elements are placed in each quadrilateral and beam members form a two-way grid along the two approximate normal lines of the nodal points.

The plane stress triangular finite elements are used to represent the membrane stiffnesses of the shell, while the two-way grid of beam members, which are assigned only a flexural stiffness equivalent to the shell thickness, are used to represent the bending stiffness of the shell. The interior and edge beams are represented by beam type members having axial, bending, and torsional stiffnesses.

An IBM 360/65 computer was used. Computer output included the nodal point displacements; principal membrane stresses and bending moments in the shell; and axial forces, torques, and bending moments in the beams. Fig. 9 shows an example of how the principal stress contours were plotted from the computer output.
STRUCTURAL DESIGN

Stresses and deflections were checked under the following conditions:
1. Dead load plus prestress.
2. Dead load plus prestress plus live load of 30 psf.
3. Dead load plus prestress plus wind.

For Case 3 a 33\% percent increase in allowable stresses was permitted.

Several possible wind loading conditions were considered based on available data. It was decided that the following two cases would be included for study:

1. A wind pressure of 56 psf acting either upward or downward on the triangular cantilever portion of the shell.
2. A wind pressure of 56 psf acting upward on the cantilever portion plus the same pressure acting downward on the internal triangular portion of the shell.

Based on the conventional membrane theory for hyperbolic paraboloid shells, only axial forces would exist in the edge beams under dead load of the shell alone. The computer results showed that part of the gravity load of the shell actually produces bending moments in the edge beam even though a great portion of the load will be carried to the abutment by axial forces in the edge beam. Due to the weight of the beams themselves, a large cantilever moment would be expected in the edge beam if it acted alone.

However, since the edge beams are integrated with the shell, the cantilever moment in the edge beam is greatly reduced. About 80 percent of the maximum beam cantilever moment is carried by the interaction of the shell with the edge beam. Thus, a high tensile zone is developed a short distance away and along the edge beams. High shell bending moments also occur adjacent to the edge beam near the tip.

Under dead load alone, the maximum
compression in the shell was 20 kips per ft (or 400 psi) while the minimum stress was 15 kips per ft (or 300 psi) (see Fig. 9).

Under dead load alone, the maximum tensile membrane stress in the shell is about 300 psi. Including the effect of wind or live load, the tensile stress is higher, amounting to 400 psi. These tensile membrane forces in the shell can be resisted by providing normal steel reinforcement. However, to minimize cracks in the shell, post-tensioning shell tendons were used in combination with normal steel reinforcement.

Straight tendons running parallel to the generators of the shell and parabolic tendons running from the tip to the...
Fig. 8. Cross section of shell slab showing location of tendons at interior and exterior edge beams.

center of the shell were both investigated. It was found that straight tendons in two directions gave more satisfactory results in reducing the tension in the concrete shell.

Finally, the 4-in. roof slab was prestressed from 10,000 to 20,000 lb per linear ft. Reinforcing bars of No. 3 at 12 in. on center are placed above and below the tendons, thus increasing the ultimate strength of the shell and limiting local cracking. Additional reinforcing steel is added in the zones of high shell bending adjacent to the beams.

The dead load of all the beams, which is about five-eighths of the total weight of the entire 4-in. shell, gives a more severe condition for beam design compared to the dead load of the shell. This is because it produces greater deflections and bending moments but smaller axial forces.

The possibility of balancing the dead load of the beams by properly prestressing the edge beams was carefully studied. However, total balancing of this dead load was impossible because part of the prestressing force in the edge beam is dissipated into the shell.

After several trials to determine the optimum prestressing, two tendons were added to each edge beam. One tendon, \( F_1 \), ran from the tip of the beam to the abutment pier and the other, \( F_2 \), from approximately four-tenths of the span to the abutment pier (see Fig. 6). \( F_1 \) was stressed from the tip while \( F_2 \) was stressed from the abutment. With the help of prestressing in the edge beams, the tensile zone in the shell decreases both in size and in magnitude but the bending moments in the shell have no significant changes.

The addition of prestress in the edge beams reduces the moments as well as the deflections and increases the axial forces and results in a set of stresses in the edge beam. These stresses were found to be acceptable for design purposes. Actually, there is not a large saving in compression, but tension is considerably reduced. Normal reinforcing steel is used to resist the excessive tension. Without post-tensioning in the edge beams, the size would have to be increased.

Buckling of the edge beams was checked using a conservative approach which neglected entirely the participation of the shell and the beam. The assumed beam proportions were found to be adequate regarding buckling.

The interior beams form two gable frames perpendicular to each other and
their arch action provides strength. Structurally, they act differently from the cantilever edge beams, the deflections and bending moments being relatively small. It was determined that only normal reinforcing steel was needed as shown in Fig. 7.

The edge and interior beams meet at the low points of the shell structure where the four supporting piers are located. Each interior and two edge beams gather loads from the shells and transmit them toward one of the four piers.

Due to structural symmetry, the horizontal thrusts from the two edge beams cancel out each other. The horizontal thrust from the interior beam and the total vertical load form a resultant force having an angle of approximately 45-deg toward the ground.

The inclined pier, as shown in Fig. 7, was chosen so that it would resist the force resultant mainly by direct compressive stresses. Thus, the deformations at the pier top were very small.

The piers rest on vertical piles which have little resistance to horizontal forces. To resist the horizontal thrust coming from the shell above, tie beams connecting opposite piers beneath the ground were post-tensioned with a 2100-kip force.

In addition to an elastic analysis us-
Fig. 10. Because of symmetry the shell roof was constructed quadrant by quadrant. Picture above shows the tendons and steel reinforcement being laid out after the plywood formwork is in place.

Fig. 11. Close-up of tendon and mild reinforcing steel layout.
ing the finite element method, the approximate ultimate moment capacity of the entire structure was examined by taking a diagonal section across the shell just outside the two adjacent piers. It was found that with the tension force along the crown of the shell acting against compressive forces at the edge beams, this ultimate moment exceeds the normal load factor requirements.

CONSTRUCTION

Construction of this structure proceeded according to carefully planned steps. First, the pile-supported foundations were built including all concrete work of the foundation tie beams. Then the piers were constructed. Steel tubing falsework to support one quadrant of the roof and its four surrounding beams was first erected (see Fig. 10). Next, 4 x 8-ft plywood was installed to form the soffit of the shell. Then the tendons and mild reinforcing steel was laid out (see Fig. 11).

To compensate partially for the anticipated deflection at the cantilever tip due to creep, the tip was cambered 3 1/2 in. to balance the calculated deflection for dead load plus prestress. After the concrete reached the required strength, shell tendons were stressed and the cantilever slab was cast.

Because the shell is symmetrical, it was originally intended that the formwork of one slab quadrant be moved three times to complete the four quadrants. For that purpose, a temporary diagonal and horizontal post-tensioning tie were installed between two adjacent

Fig. 12. Temporary tendons under shell soffit stressed from opposite piers.
Fig. 13. Temporary tendon stressing ends (at top of piers).

Fig. 14. Shell roof at late stage of construction.
piers to carry the horizontal thrust of each quadrant (see Figs. 12 and 13).

However, to simplify the operations, the contractor decided to erect formwork for all four quadrants even though this operation increased forming costs. Fig. 14 shows the complicated steel tubular falsework supporting all four quadrants of the shell.

The foundation tie beams were post-tensioned after all the four quadrants of the roof were completed, one by one. A closeup of the foundation tie beam at the stressing end is shown in Fig. 15. Then the supports for the forms under the 4-in. shells were gradually released.

At each stage, at least two tendons were stressed at the perimeter of the shell at one time, such that the forces were symmetrical about each pier at all times. Finally, supports of the edge and interior beams were gradually removed. For the edge beams stressing started symmetrically from all four tips, and for the interior beams from the center towards the piers (see Fig. 16).

On completion of this step, the average deflection of the four tips was measured to be 3 in., which compared favorably with the calculated values. Deflections of the shell were checked periodically for a few months after completion and they were found to be stable and satisfactory.

**CONCLUSION**

The satisfactory construction and proper behavior of this shell roof indicates the desirability of using prestressing. By post-tensioning both the shell and the edge beams, stresses and deflections can be controlled and kept within limits. The usual assumption that edge beams carry their own weight is far from being correct for such large shells. Similarly, the stresses resulting from prestressing in the edge beams are also appreciably absorbed by the shell slabs.
Since stresses in the shells are low for this structure, longer spans can be realized with relative ease. However, the design of the long cantilever edge beams becomes more difficult to handle and may require a combination of prestressing and reinforcing steel to attain an optimum solution.

Shrinkage stresses in the shells will often result in local cracking, particularly near the corners of hyperbolic paraboloid shells. Such cracking has actually occurred in this structure. However, by using tendons and mild steel reinforcement in these areas, the stability, strength, and deflections of the shells are not adversely affected.

It is our opinion that hyperbolic paraboloid shells with post-tensioning can be extended to longer spans. With today's high speed computers, it is possible to make a reliable analysis of complex shells and to predict their behavior accurately. However, at the same time, it is highly desirable to use sound judgment especially when we extend our structures into complex forms, new materials, and longer spans.

**CREDITS**

Computer Consultant: Prof. A. C. Scordelis, University of California, Berkeley, California.
Design Engineering Firm: Sanchez, Davila & Suarez (formerly Raymond Watson Structural Engineers), San Juan, Puerto Rico.
Contractor: Gabriel Alvarez and Associates, Ponce, Puerto Rico.

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