Cylindrical shed construction: the shell roof on the Jamin factory at Oosterhout, Netherlands

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ABSTRACT: The paper provides an overview of cylindrical shed reinforced concrete shells, a type of construction used primarily in industrial buildings. Like other types of shells, most cylindrical sheds were built between the end of World War II and the early nineteen sixties. The article reviews their characteristics and construction parameters based on contemporary studies and briefly documents some of the most prominent structures. The final chapter contains a detailed analysis of the design and construction of what, to the authors' knowledge, is the largest such shell ever erected. Built for the Jamin factory at Oosterhout, Netherlands, this shed was the object of an ingenious destructive study on a scale model conducted in 1955 at Madrid's Central Construction Materials Laboratory.

INTRODUCTION

Cylindrical sheds can be considered as a continuous series of ordinary cylindrical reinforced concrete shells in which one of the parallel sides of each shell is set at a higher elevation than the other. The result is a shed with a characteristic curved section (Fig. 1).Large windows are placed in the free area between shells, which is usually oriented toward the north. Although a small part of this area is occupied by an edge beam in the same plane whose depth is generally defined to be 1/18 of its span (Haas 1950, p 501), a substantial amount of window space is still available. In addition to the benefits deriving from their monolithic nature and the fire resistance inherent in concrete, the primary advantages of these shells, which were normally used in industrial buildings, was the natural lighting afforded. Firstly, the layout ensured a high and regularly distributed daylight factor (ratio of outside illuminance on the glazing to the inside illuminance on the horizontal working area) (Fig. 2). But also because the "smooth, white curved indoor surface of the shell spread northern sunlight evenly throughout" (Havenwerken, undated, p 3), thereby avoiding undesired shadows. Another clear advantage lay in the fact that "with this type of construction, incoming light is not obstructed by the web of diagonals and shapes nearly always present in steel structures" (Haas, 1959, p 500), whose absence also favoured "the struggle against dust" (op cit. p. 502).

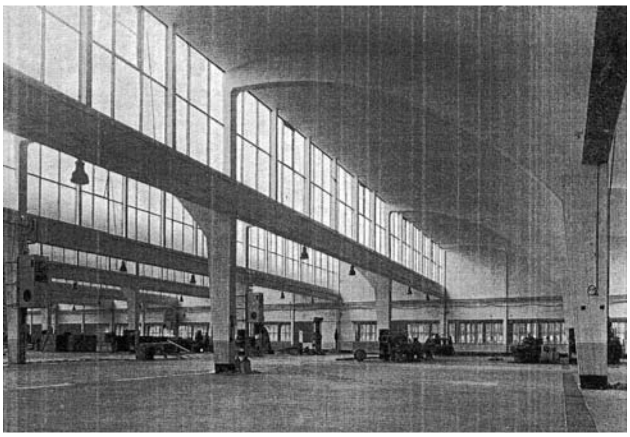


Fig.. 1: Cylindrical shed with vertical windows (Havenwerken, p. 3)

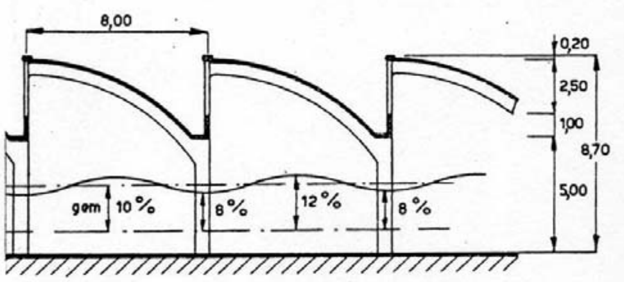


Fig. 2: Daylight factor in cylindrical sheds (Bloem 1954, p. 210)

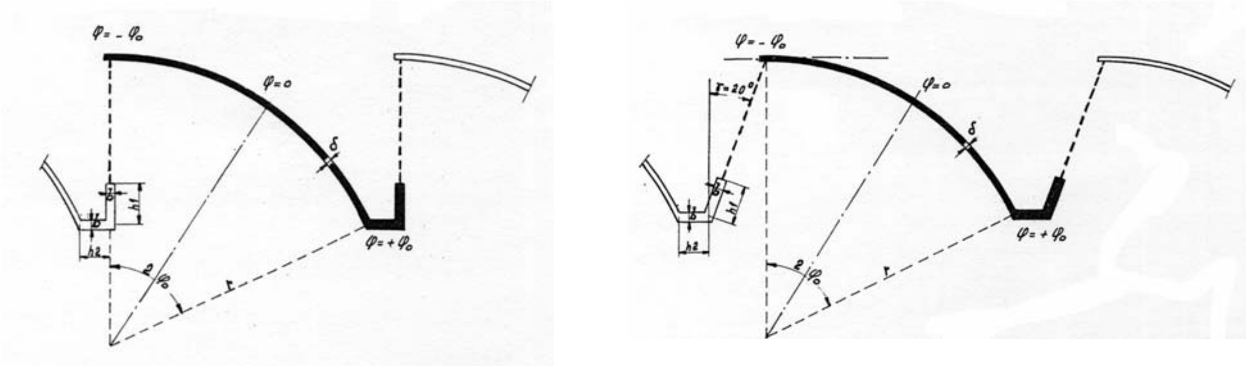


Fig. 3: Parameters in cylindrical sheds. Vertical (left) and inclined (right) glazings (CUR 8B, p. 5)

Nonetheless, these shells were more expensive to build than normal cylindrical shells and their design involved a much larger number of geometric parameters. This, in turn, called for high precision adjustments not only for the module roofed by each shell, but also for its slant and radius of curvature and the angle of the windows (Fig. 3). Increasing the rise, for instance, to guarantee proper draining necessitated raising the slope and concomitantly the area to be glazed and total bay volume. In his adaptation of a study by H. Rühle, A. Volbeda noted, moreover, that “the clearance to the edge beam should be no less than 5 m to ensure proper lighting” (1958, p. 227)(Fig. 2). One last advantage consisted in the good acoustics of the resulting bays, for the noise concentration present in symmetrical barrel vaults and domes was absent thanks to the slant on the concave surface of these cylindrical sheds. According to Haas (Fig. 4), noise concentration was not to be feared because “the shell radii generally prevent the formation of a focal point where sound rays concentrate (which actually exists at around 10 m below floor level). And besides… the edge beams break up the sound waves” (1950, p. 501).

The shells typically rested at the ends on portal frames whose beams were curved and on the gutter beam, which supported the straight lower edge of the shell. The upright bars positioned between the edge beam and the edge of the shell, whose primary purpose was to hold the windows in place, provided point support. This upper edge was seldom built without these intermediate supports. While the structural engineering for these shells derived from symmetrical type calculations, issues such as the usually lesser rise obtained due to the slant (f in Fig. 5) and especially shell asymmetry occasioned significant complications. In the simplest cases of very long shells, however, the structural engineering system used could be simplified by adopting the Lundgren method, in which the shell was treated like a slanted beam subjected to longitudinal bending (Haas, pp 37-38). Despite such greater complexity, however, due to the advantages of their use, asymmetrical shells were studied with the same intensity as their symmetrical counterparts, resulting in a fairly detailed tabulation of their main parameters.

> The earliest attempts were undertaken around ten years ago. Dyckerhoff and Widmann are known to have been working with standard measurements for cylindrical sheds for some time now.
> Shells have been developing gradually in recent years in England as well. The relevant work underway in Eastern Europe has been catalogued, with information not only on shape and standard dimensions, but also on construction characteristics. This idea was re-introduced in Germany in 1953. And a similar attempt at standardization is being studied in Poland. (Volbeda 1958, p. 229)

In The Netherlands the Comissie voor Uitvoering van Research (C.U.R.) reports for design and calculations of cylindrical shells for roofs published in its bulletins 8a, 8b and 12 was regarded by contemporaries to constitute

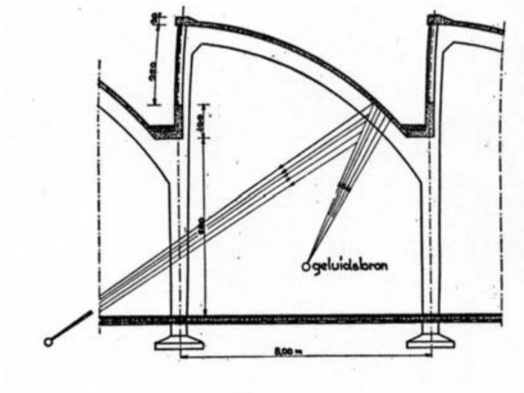


Fig. 4: Noise concentration under floor level (Haas 1950, p. 500)

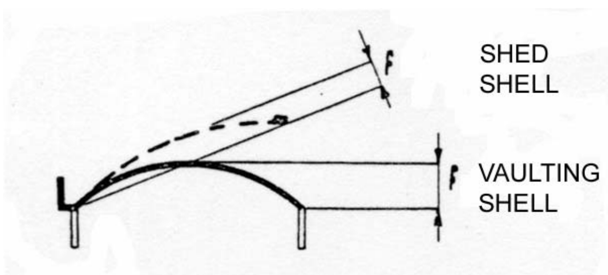


Fig. 5: Deep in cylindrical sheds (Bloem 1954, p. 210)

a relevant standard. The foreword to bulletin 8a illustrates the engineering difficulties posed by and the importance attached to these new structures:

> The static engineering of reinforced concrete roof shells is one of the most difficult problems to be faced by a civil engineer. It entails a command of the highest degree of mathematics, or at least of the part of mathematics needed to deduce the formulas for shell theory.
> "Consequently, on the one hand, engineering is limited to only a few professionals; and on the other, it constitutes an obstacle to the use of roof shells...
> "From the outset, the Commission has aspired to a dual objective in this regard. In addition to developing a simplified calculation method, simple design rules must be established. These latter may serve as a guideline for the architect and in general for all involved in design and interested in information on their economical application. (C.U.R. 8A 1955, p. V)

The range of appropriate dimensions to keep these shells within economically feasible limits (Volbeda 1958, p. 227) ran from 7.5 to 10 m wide and from 10 to 20 m long (exceptionally 25 m), generating vault heights of from 3.5 to 4 m, windows 2.5 m high and an gutter beam 1 m deep. And this, in fact, was the approximate size of many of the first non-pretensioned shells built after the war, in which the windows were generally positioned vertically. That arrangement was likewise economically favourable, for "the non-reinforced glazing that could be used afforded a higher percentage of light", while "if the window surface is slanted, the diffuse light factor is greater, but that entails certain other drawbacks (greater soiling, snow loads, need to use reinforced glass)" (Haas 1950, p 500).

BRIEF OVERVIEW OF PROJECTS IMPLEMENTED

Cylindrical sheds had been tested in Germany during the war years, if not before. One distinctive example, a rope factory, was erected by Schaffhausen no later than 1942 (Fig. 6). Among its particularities was the fact that the shells rested on large triangulated concrete beams of the same height as the vaults, allowing for a transverse span between supports four times the shell width. Another interesting feature was the existence of a cantilevered vault module that rested on the respective cantilevered section of the beam. Each 5.5 cm thick shell covered an area of 6.75 x 16 m, for a transverse clearance between columns of 6.75 x 4 = 27 m, and a 16-m span in the other direction. This same support system, but with only two spans, was also applied in the bays built for the Kon. Ned. Katoen Spinerij (KNKS) thread factory at Hengelo, Netherlands, around 1948 (Fig. 7). These singular examples aside, most of the sheds erected in the first ten years after the war seem to have been built with portal frames, in keeping with the standard model discussed earlier. That system did away with the unsightly presence of indoor horizontal bars running across the bay at gutter beam height.

In light of the extensive use made of these shells, improvements and perfections were quickly introduced, including the use of slanted windows which soon prevailed. Another predominant practice was to position the portal frames on the inside, thereby clearly dividing the shells into distinct sections. One interesting exception is found in the bays built for the French textile company Société industrielle pour la Schappe at Tenay, whose portal frames are positioned above the shells. The smooth interiors resulting from that arrangement generate an effect worth illustrating (Fig. 8). In more standard sheds, the roof over the N.V. Spanjaard textile plant at Borne, Netherlands, stands out for its position within the structure of the building, for this is one of the few examples of a shed built over the second storey of a factory (Figure 9). The factory was designed by the Dutch architectural firm Postma & Postma, which also authored a substantial number of the earliest concrete sheds built in that country.

In later stages the main improvements were geared to facilitating and economizing construction with systems that ranged from making the most of reusable formwork to precasting certain elements. Re-use of wooden forms up to five times was standard practice, and steel slipforms were used in the USSR. The German firm Ed.

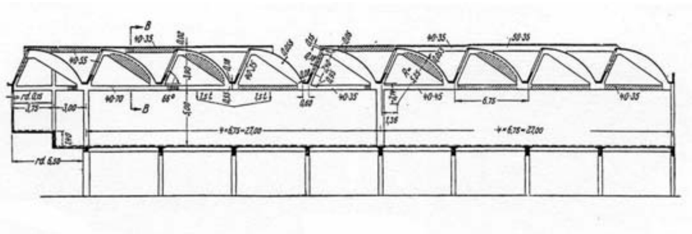


Fig. 6: Rope factory, Shaffhausen (Havenwerken p. 5)

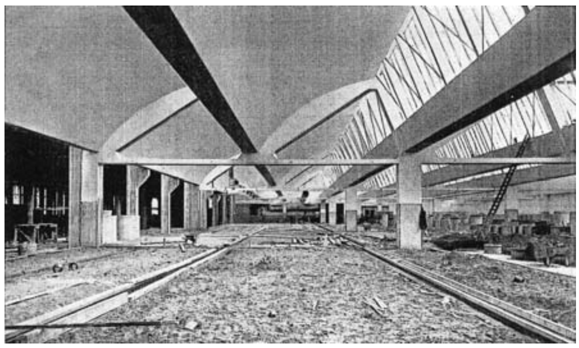


Fig. 7: KNKS, Hengelo c. 1948, (Havenwerken p. 14)

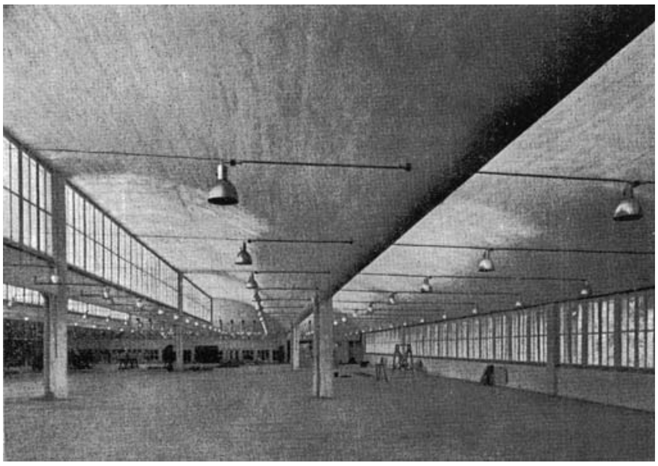


Fig. 8: Textil company Société Industrielle pour la Schappe Tenay, France (Haas 1950, p. 501)

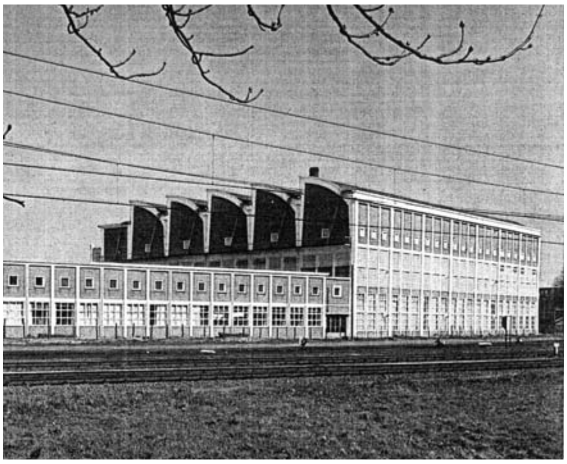


Fig. 9: Textil factory N.V. Spanjaard, Borne, Netherlands (Potsma 1960, p. 29)

Züblin AG, in turn, developed precasting systems for building shells from standardized curved elements. This standard system consisted in forming 7.5 x 15 m modules with elements 85 cm wide, although this width could be doubled in special cases. The Geislingen steel goods plant at Württemberg was erected with that system.
Nonetheless, as in the case of symmetrical shells, the development that led to the greatest progress was the introduction of pretensioning in the early nineteen fifties. According to H. Maaskant, the architect who designed the Cincinatti factory (Fig. 10) at Vlaardingen, Netherlands, completed in 1953 “as far as we know, this is the first time that pretensioned shells have been used in this type of (shed) roofs” (1954, p. 370). Maaskant himself noted that the advantages included, among others, greater cost-effectiveness than in non-pretensioned shells, readier placement of the reinforcement, near elimination of cracks in the pretensioned areas and that “indisputably, since pretensioning leads to wider spans, promising developments may be expected in this regard” (op. cit. p. 371). Indeed, although the Cincinatti shells only covered 8 x 20 m modules, these dimensions were soon amply exceeded, reaching up to 40 m in the roof over the Jamin factory at Oosterhout, Netherlands, whose first sections were completed in May 1955. This last structure, the largest of this type known to the authors of this article, it is discussed in greater detail in the final section below. By way of reference, the largest structure erected with non-pretensioned reinforced concrete, the Bowater paper plant between Chester and Manchester, had shells 30 m long. Another relevant example is to be found in the shells over the main bay to the Brown, Boveri & Cie. electric generator plant at Birr / Argau, Switzerland, (Fig. 11). The plan view area of each shed is 18 x 36 m and the lower edge rises 23.60 m off the floor.

In another vein, Swiss engineer Hossdorff devised an ingenious system for combining precasting and pretensioning, in which the problem of housing the sheathes for the cables in the slender shell was solved by placing them over the extrados. In this case the pretensioning cables served essentially to enhance the cohesion and soundness of the separate precast elements of which the surface is made (Fig. 12). This system was specifically developed to build a warehouse at Wangen, Switzerland, in the early nineteen sixties, in which 64 identical 25.20 x 8,40 m modules were used. Each precast roof member was 1.40 m wide and reinforced around the entire perimeter. The minimum thickness was 4.5 cm, in an attempt to reduce weight for shipping purposes. The most sensitive problem was the choice of the type of joint; after considering several options, a 10 mm mortar-filled joint was adopted. In light of the dimensional precision required to guarantee that joint width, the shop-made forms were built of concrete with a maximum tolerance of 3 mm.

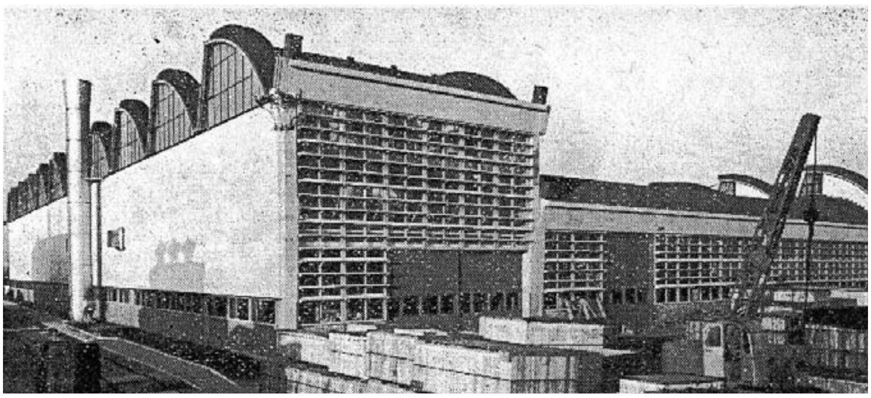


Fig. 10: Cincinatti factory, Vlissingen, Netherlands (Maaskant 1954, p. 374)

Fig. 11: Brown, Boveri & Cie. electric generator plant at Birr / Argau, Switzerland

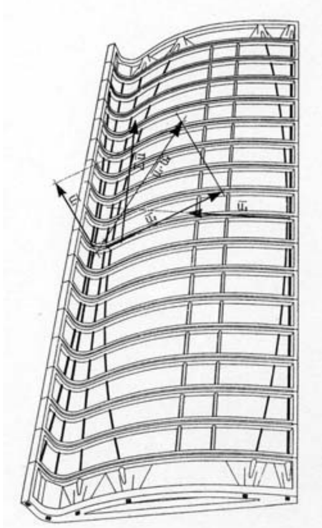


Fig. 12: Hossdorf system, Wangen, Switzerland (Hossdorf 1963, p. 555)

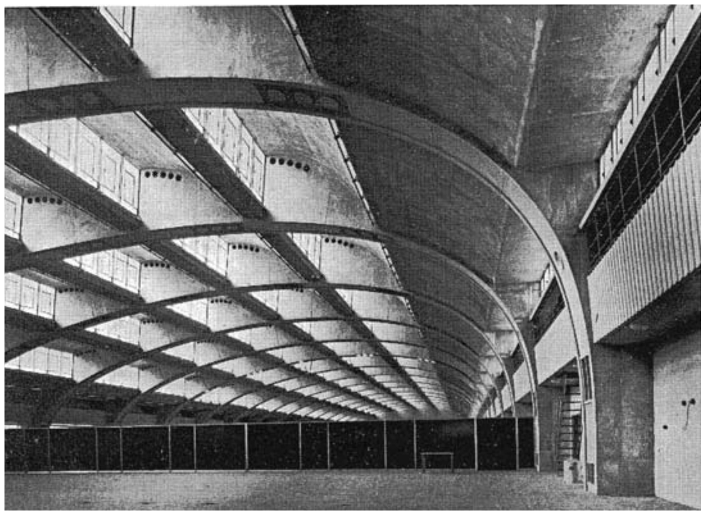


Fig. 13: Bank of England press, Essex (Easton 1957, p. 120)

The common denominator of a further group of solutions was, essentially, the use of large transverse girders rather than portal frames as supports. This idea was discussed, in fact, in the first two examples mentioned in this section, with their large triangulated concrete beams. Yet another example of this arrangement can be found in the Weber & Ott A.G. textile plant at Forchheim, Germany. The option referred to here, however, is the use of large, deep, solid web beams, such as in the case of a hangar at Algiers with 50 m girders that support groups of five parallel sheds (Haas 1950, p. 503). In some cases these beams were actually box girders, whose interiors were also used as air extraction ducts. The Zoeppritz A.G. textile plant at Heidenheim, Germany, and the cigarette factory at Lisnafillan, Northern Ireland, with beam spans of 34.50 and 31.70 m, respectively, are examples; in the latter case, moreover, the beams are arched. The most spectacular example, however, is the Bank of England press at Essex, authored by architects Easton and Robertson, in which an extraordinarily long bay measuring 240 x 37.5 m was roofed with a series of six parallel shells springing from the large transverse arched portal frames comprising its skeleton (Fig. 13). Taken individually, however, the shells in this group were obviously smaller than the standard size.

JAMIN OOSTERHOUT FACTORY

Built in 1955 to a design by architects Masselink, Bruins and van der Zoo, the Jamin garment factory had a 260 x 80 m footprint and was roofed with 12 x 40 m sheds authored and built by engineer A. M. Haas. These cylindrical sheds were actually designed to be 22 sections of continuous 80 m wide roofs, with each section resting on one central and two end portal frames (Fig. 14). The main dimensions were 12.85 m high measured from floor to crown, with a 7 m clearance from the floor and a 9 m radius of curvature. They were 7 cm thick, except at some points on the edges, where they thickened to 14 cm. The bottom and sides of the gutter beam were 20 cm thick. Given the dimensions involved, and to keep thickness and reinforcement to a minimum and counterbalance the distortions on the free edge, the the shells were post-tensioned along their entire length. The system devised, whose effects and details were described in conferences delivered by Professors Haas (1956) and Baas (1956) on 13 October 1955 at the Betonvereniging held at Breda, consisted in

Fig. 14: Jamin. Areal view (Benito 1956, unpaged)

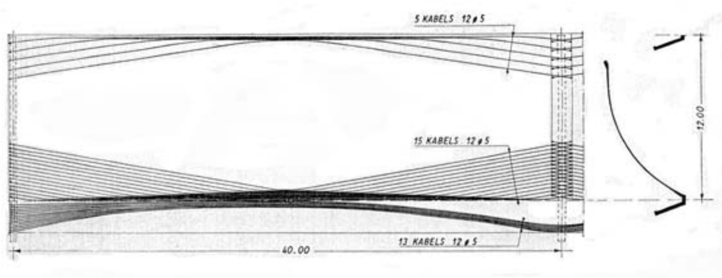


Fig. 15: Post-tensioned cables system (Haas 1956, p.44)

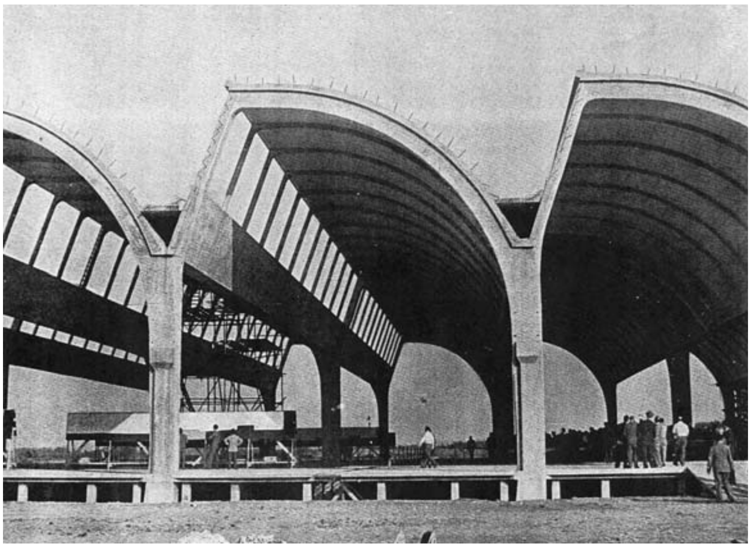


Fig. 16: Bays alter removing formwork (Benito 1956, unpaged)

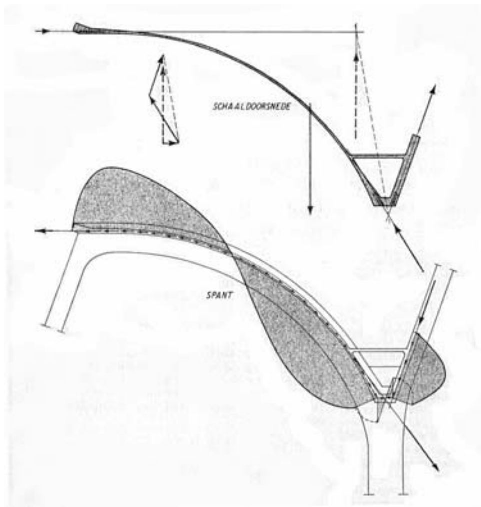
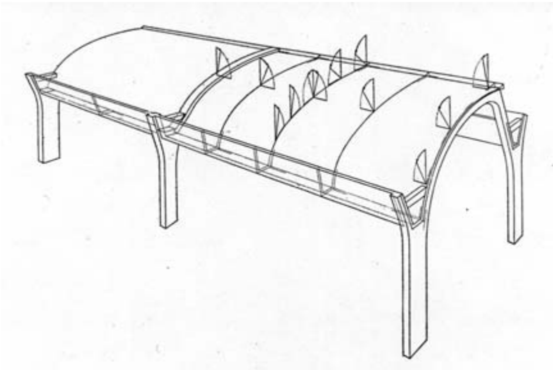


Fig. 17: Post-tensioning actions in the shell (Baas 1956, p.48)

two systems of curved cables in the shell and a third in the edge beam: each cable was made of twelve 5 mm strands. In the shell, the top system comprised five cables and the bottom system fifteen. The beam was fitted with a total of 13 cables that ran uninterruptedly along the 80 m width of the bay. By contrast "in the shells the cables run along the entire 40 m length and are anchored at the end and central portal frames (Fig. 15). This caused a number of complications in the cable layout in the intermediate support... solved by using special precast blocks as final anchors" (Haas 1956, pp 43, 44). The portal frames were likewise post-tensioned.

In light of the singularity of the structure and to check the resulting visual effect in advance, two of the sections were first built as a wooden scale model. Nonetheless, as the photographs show "the sheer size of the building can be deduced by comparison to the human beings standing in it" (Haas; Baas 1955, pp 145-150) (Fig. 16). A photographic competition organized by the Dutch journal *Cement* in 1957 awarded the main prizes to photographs of this structure. Moreover, in addition to the structural engineering conducted according to the Van der Erb method set out in the C.U.R. bulletins cited above (Fig. 17), two scale load models were built to confirm the assumptions adopted and determine the failure mode. A first model, made of cardboard by Nedrlandsche Organizatie voor toegapast-natuurwetenschappelijk onderzoek (T.N.O.), an institution that collaborated with the studio, was built to obtain rough data on the local stability of the structure. That model was made to a scale of 1:35 with 2 mm sheets of cardboard reinforced on the free edges and the edge beam to simulate the actual thicknesses. The material was chosen because the model had to ensure "that resistance to denting was considerably lower than bending strength" (Haas 1956, p. 41). The deduction drawn from the model was that the buckling safety factor, based on concrete resilience, would be approximately 3 ¾, and indeed, that was the first time that this figure was applied to a shell of this type and size. Nonetheless, "the cross-section should maintain its shape as far as possible under the effect of the loads... This can be achieved with ribs ... laid out in keeping with the cross-sections" (op.cit.). These reinforcements were, therefore, included in the final design and erected underneath the shell to "maintain the shape and retard buckling" (Haas; Baas 1955, p. 144), bearing in mind, in addition that "for a 40 m span the useful depth of the shell (calculate as if it were a beam) is only 2 m" (Benito 1956, unpaged). The ribs had a cross section of 14 x 20 cm and were spaced at 3.66 m intervals.

With the final design concluded, a second large scale model, consisting in two complete portal frames, was commissioned from Madrid's Central Construction Materials Laboratory headed by Carlos de Benito. For their relevance and interest, the construction of the scale model and the procedure used to load it to failure are described below, drawing from a contemporary article published in the journal *Informes de la Construcción*. For reasons of dimensional analysis, the modulus of elasticity for the real structure was maintained in the test. This entailed building the model with a reinforced cement and sand mortar, appropriately dosed so that the ultimate tensile and compression loads would be the same as in the real shell. Seeking the proper balance between economy and degree of precision and detail, a scale of 1/10 was used: i.e., the model was 8 m long, 1.20 m wide and around 10 mm thick at its thinnest. Model construction imitated the actual structure in all respects, and high strength steel wire reinforcement sheathed in plastic was used for the post-tensioning. The mortar was cured in a moist atmosphere for 20 days. Tensioning was performed with a Barredo anchorage system and specially designed wedge jacks fitted with a dynamometer and a retention bolt. After the scheduled load was reached the latter "kept the wires tensioned, to be able to remove the respective jack; but if the stress on the wires had to be checked or the wires had to be re-tensioned, the jacks could be re-fitted" (Benito 1956, unpaged). Since the cost of placing jacks on all the wires simultaneously was prohibitive,



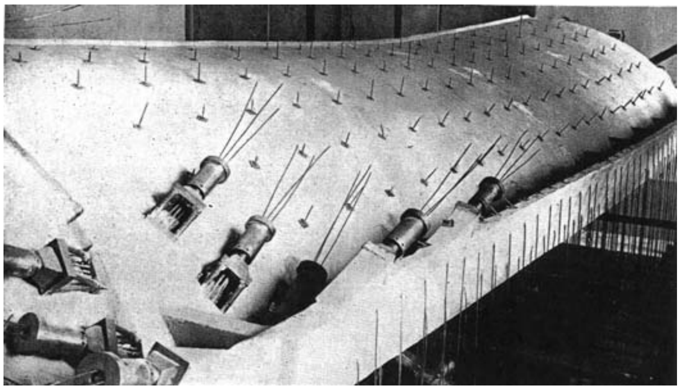


Fig. 18: Rotations measuring (Benito 1956, unpaged)

Fig. 19: Longitudinal bending effect (Benito op. cit.)

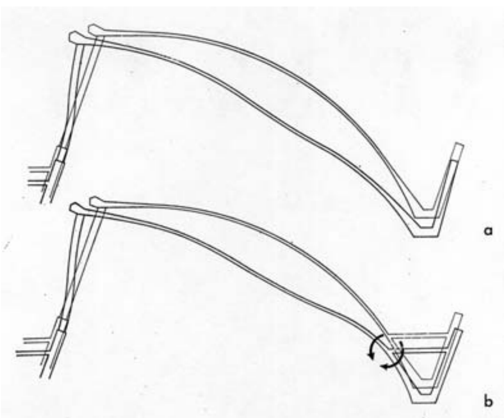
the structure was tensioned in two phases, the first to half the load. A check was run to ensure the absence of friction-mediated loss. The load tests were begun after post-tensioning, 28 days after the model was built.

The highly original loading method merits separate discussion. The weights were hung from the model because that arrangement facilitated strain and crack monitoring from above as well as instrument positioning. In addition, the loads had to be applied simultaneously to simulate real-life situations. To that end the system ordinarily used by that laboratory at the time was followed, as described below:

> The model to be tested is built inside a 2 m deep deposit covering an area somewhat larger than the model; its interior houses 9-kg cylindrical receptacles 9 cm in diameter and 1.60 m long which float when the water deposit is filled. These receptacles are duly hung from the structure to be loaded; as the deposit is drained, the structure is gradually and uniformly loaded across its entire surface" (Benito, 1956, unpaged).

A wide range of load intensities can be obtained by merely varying the spacing or filling the receptacles with gravel. Given that the receptacles do not float when loaded to more than 9 kg, two load increments can be established, one with the receptacles submerged and the other in an empty deposit, which was the one used for the ultimate failure loads. Deposits could be made of brick or bolted steel plate apt to allow for re-use.

A series of loading and unloading tests were first conducted, measuring rotation and creep in a series of sections to a precision of 0.0001 radians and 0.01 millimetres, respectively (Fig. 18). These values were used to verify or rule out Hookean behaviour in the structure, and check the design assumptions and theories. Given the reversibility of the procedure, the flaws appearing in all stages could be recorded until the failure test was conducted. Two clearly distinct movements could be observed overall, "one as a result of longitudinal bending, which generated deformations as if on a continuous two-section beam; consequently, the maximum deflection was exerted in the middle areas of the spans" (op.cit.). This caused cracking at the bottom of the beam and in the shell, wich disappeard when the loads were removed (Fig. 19). The other movement, which occurred at the same time as the above and was especially striking "generated horizontal creep at the top of the roof as a result of transverse bending in the shell" (op.cit.). The cracks, moreover, were observed to be



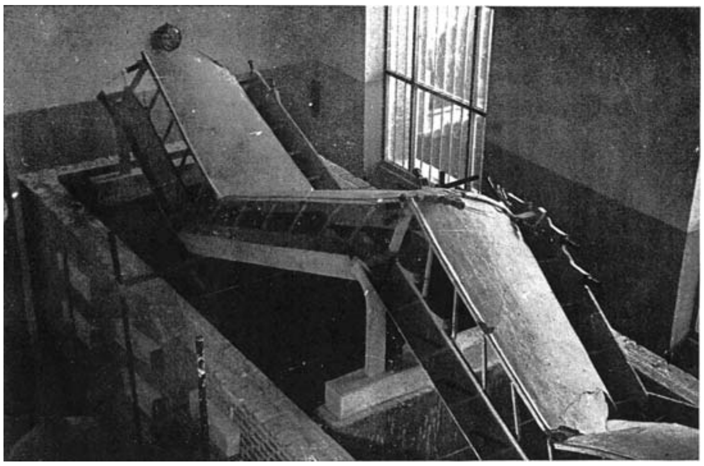


Fig. 20: Central section movements (Benito op.cit.)

Fig. 21: Model collapse (Benito op.cit.)

perceptibly normal to the tensioning cables, while the shell curvature declined considerably due to such bending. Very briefly, the central section could be said to descend as a result of longitudinal bending and shift horizontally due to transverse bending. Moreover, the existence of bracing joists on the portal frame side of the gutter beam introduced a disturbing bending moment that cause the bottom of the shell to crack (Fig. 20).

One last effect observed was the much greater horizontal shift on the top edge of the shell, that occasioned "considerable bending on the uprights supporting the skylights, which eventually cracked at the ends" (Benito op.cit.). The portal frames, in turn, were clearly impacted by the shell movements, with the outer frames bending inward, i.e., off-plane, and cracking due to bendings in-plane. The shell finally collapsed due to tensile failure in the sections positioned over the central support, which was followed by failure in the mid-span sections (Fig. 21). As the failure load was 100% higher than predicted in the design, a safety factor of 3.5 was regarded to be satisfactory.

The most prominent feature of the work begun after the trials were completed was the formwork structure, which consisted in wood scaffolding that rested on a concrete working slab built over an accessible utilities chamber with a 1.5-m clearance. Sufficient formwork was built to erect three sections at a time, and all three sets were re-used a total of seven times to build the twenty two sections. The formwork, which was also made of wood to which three-ply cladding panels were bolted, had a double wall on both sides and at the bottom of the gutter beam. Concrete placement took two days per span. That notwithstanding, post-tensioning delayed work somewhat with respect to conventional construction times, for "the longer time required for hardening, tensioning and injecting the sheaths meant that post-tensioning took nearly 12 days more than reinforced concrete construction" (Baas 1956, p. 49).

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