Concrete Folded Plates in the Netherlands

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INTRODUCTION

In the nineteen sixties, the study and construction of reinforced concrete shells reached what was very likely their acme. Within this category of structures, so-called folded plates merit special consideration for, while fairly thin, their surfaces are flat and therefore differ from other thin shells in that they neither benefit from the properties of curvature nor exhibit full membrane behaviour. To quote Professor Cassinello, initially,

> The earliest folded plate solutions […] drew from their close resemblance to corrugated plate and cylindrical shells. The underlying idea is quite simple: longer spans can be accommodated with relatively small increases in weight by enlarging the lever arm of the structure; the top and bottom chords of each slanted slab house the main reinforcements while the shear stresses are absorbed across the sloping sides.
>
> (1974, p. 537)

F. Candela distinguishes prismatic structures and folded slabs from other thin shells in that they are "subjected to a combination of membrane and bending forces" (Faber 1970, p. 23) and American engineer Milo Ketchum, specializing in the design and construction of such structures, wrote with respect to their advantages that:

> The analysis was straightforward, used methods with which I was familiar, and the structural elements were those we used for other concrete structures. […] It is possible to analyze folded plates with more precision than barrel shells.
>
> (c. 1990)

Ketchum also contributed to christening this type of structures:

> I always disliked the name "hipped plate", and when I was chairman of an ASCE committee, I was at least partially responsible for changing the name to "folded plate".
>
> (c. 1990)

Nonetheless, despite Ketchum's remarks to the contrary, except for the simplest – which are admittedly the most common – cases, i.e., forms with parallel horizontal folds, a general theory applicable to the structural analysis of such plates is anything but straightforward. C.B. Wilby summarized the key milestones in the history of the analytical study of such structures in the following terms:

> The principle was first used in Germany by Ehlers in 1924, not for roofs but for large coal bunkers and he published a paper on the structural analysis in 1930. Then in 1932, Gruber published an analysis in German. In the next few years many Europeans – Craemer, Ohlig, Girkman and Vlasov (1939) amongst them – made contributions to this subject. The Europeans' theories were generally complex and arduous for designer use. Since 1945 simplified methods have been developed in the USA by Winter & Pei (1947), Gaafar (1953), Simpson (1958), by Whitney (1959) adapting the method by Girkman, by Traum (1959), by Parme (1960) and by Goble (1964).
>
> (1998, pp. 2, 5)

Candela found Kazinsky's (1948) contributions to be highly relevant as well (Faber 1970, p. 73).

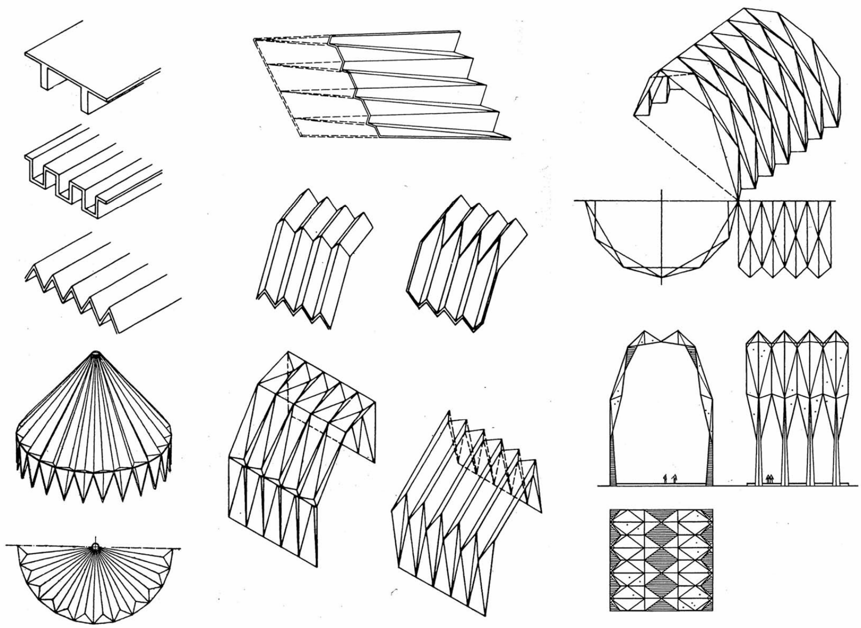

Figure 1. Examples of folded structures (Angerer 1974, pp. 75-6; Engel 2001, p. 227).

The enormous variety of imaginable forms for such structures has been depicted in a number of texts **(fig.1)**. The ones actually erected, however, generally adopt the simplest forms, although certain very complex combinations have on occasion been used (St Josef Church at Neuß-Weckhoven, Germany, 1966-67, *Bauwelt* 1967, p. 912; Gadet Chapel, USAF Academy, 1963). The layouts devised to build domed forms also merit mention, although since such designs generally include curved elements, they cannot be regarded as pure folded plates. Polyhedral forms can naturally also be considered to be folded structures consisting of polygonal facets totally or partially comprising such a surface. Yet another beneficial property is: “their advantage, in comparison to shells, is that formwork for flat surfaces entails fewer difficulties” (Angerer 1972, p. 51). It should nonetheless be borne in mind in this respect that such: “Simplified formwork […] intensifies the risk of buckling, inasmuch as non-curvature makes such shells more sensitive to this effect”. For this reason, they are “more limited in terms of amplitude and loading” (Cassinello 1960, p. 542) than curved forms.

THIN SHELLS IN HOLLAND

The interest with which developments around this new structural approach were followed in The Netherlands led to the country’s hosting of the Third International Symposium on Shell Structures from 30 August to 2 September 1961 at the Delft Polytechnic School under the auspices of RILEM (Réunion Internationale des Laboratoires et Experts des Materiaux, Systèmes de Constructions et ouvrages) and IAAS (International Asociation for Shell and Spatial Structures) and the leadership of Dutch engineer A. M. Haas, a renowned expert in the field at the time. Dutch contributions to thin shell theory include the publication by the Delft T.H. (“Technische Hoogschool”) and the T.N.O. (“Nederlands Organizatie voor Toegepast Natuurwetenshappelijk Onderzoek”) of “a simple manual, C.U.R. [Commissie voor

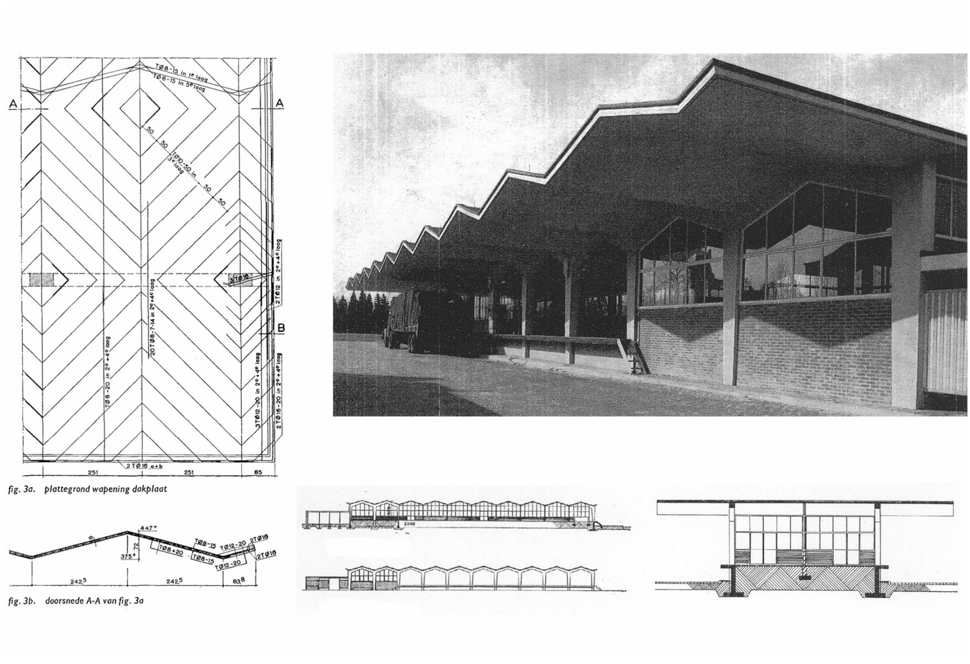

Figure 2. Glanerbrug customs building (Beltman and Spit, 1962, pp. 228-30).

uitvoering van research] No. 12, with guidelines on the design and calculation of cylindrical thin shells", which "serves a good purpose and is also used abroad" (Haas 1961, p. 423). Although not cited in the above passage, issues 8a and 8b of the same series addressed related subjects. Haas, who acted as consultant for the folded roof over the wholesale fish market at Scheveningen (c. 1964), one of the largest in the country, is cited by Wilby in connection with European preferences for analysis: "In 1974 Professor Haas of the Netherlands told the author that in mainland Europe 'we go on Grikmann'" (1998, p. 5).

According to a report carried out by the Dutch magazine *Cement* the year of the symposium, the 131 shell structures that had already been built in The Netherlands by that time (*Cement* 1961, pp. 451-3) could be classified in the following categories: 14 domes; 35 cylindrical shells; 41 shedframes; 7 corrugated plates; 1 butterfly-type doubly cantilevered shell; 2 conical shells; 3 hyperboloids; 3 conoids; 14 hyperbolic paraboloids and 11 folded plates. One of these folded plates was a hexagonal hipped roof, two were mansard-like roofs (with a horizontal plane flanked by two slanted planes) and the rest were all normal pitched roofs. Surprisingly, the list did not include the folded triangulated roof over the Van de Leer offices at Amstelveen (1959), designed by the German-born American Marcel Breuer, despite its singularity and the author's renown. The most consummate folded plates were built after the symposium, however, culminating in the shell completed in 1965 for the roof over the Delft Polytechnic School, an emblematic structure authored by Dutch architects Bakema and Van der Broek. The following synopsis is based on the details described in the articles published on the most representative of these structures.

EARLY SMALL-SCALE WORKS

The hexagonal shell roofing the water tower at Dubbeldam, dating from 1914, is regarded as the first - and very progressive for its time – folded plate to be erected in The Netherlands (*Cement* 1961, p. 440). The builders, Stulemeyer, pioneered concrete construction in Holland. Opposite sides of the hexagonal base were 9 m apart, while the roof itself was approximately 5.80 m high. The plate was 8 cm thick.

Larger roofs of this type, all with V-shaped folds and built in the late nineteen fifties, are exemplified by the structure covering the customs building at Glanerburg (1959) **(fig.2)**. This roof consists of a continuous folded plate cantilevering 4.5 m on each end and resting on two 10 bays parallel portal frames, separated 9 m from each other. The slanted beams over these bays meet at mid-span where they form a vertex 0.72 m higher than the beam-column joint; in other words, at 29.7%, the slope is fairly shallow. The shells fold upward at the end of the portal frames; this final pleat, measuring 0.838 m and thickened along the free edge, serves to stiffen the structure horizontally.

The standard thickness in this roof is 8 cm. In their article describing the shell, Beltman and Spit noted that while the cantilevers were decisive for the calculations, the greatest stress and strain were concentrated on the free folds, which had to be thickened and more heavily reinforced (1962, pp. 228-30). They also reported that the elasticity equation for the cantilevers was based on C.U.R. report No. 8. The reinforcement was laid in 5 layers, the first and fifth to absorb transverse moments, the third for shear stress and spacing, and the second and fourth to act as the main reinforcement, for which straight longitudinal bars could be used thanks to this somewhat unusual arrangement. Indeed, the National Building Service recommendation in force at the time called for only 3 layers; while this would have allowed for a thinner shell, the use of arched bars as the main reinforcement that this solution entailed was not cost-effective in this case.

The roof over the new Den Helder station designed by architect G.J. van der Grinten **(fig.3)**, which has spans and cantilevers of roughly the same dimensions, was also completed in 1959 (*Cement* 1961, p. 441). The two structures differ, however, in the shape of the building to be roofed and the arrangement of the folds. The station's elongated hexagonal floor plan, with total measurements of approximately 35 x 30 m, determined the use of two symmetrical shells with non-parallel, outwardly convergent folds. Consequently, the size of the Vs is not constant, but declines from the ridge outward (with the distance between valleys ranging from approximately 2.65 to 1.35 m). Since each shell slants slightly downward towards the eaves, the resulting overall shape can be likened to a folded pitched roof. Here the shell rests on horizontal beams, component parts of four parallel orthogonal portal frames. The V sections are on a 45º slant, with three units per bay, supported under the valleys only. The free edges on the sides are horizontally stiffened with 50 cm wide strips and the reinforcement in the valleys is thicker than in the rest of the structure. The steep incline contrasts sharply with more typical solutions using shallower slants that call for less costly centring, such as in the preceding case. The three 50 cm ϕ skylights in the middle section are of particular architectural interest (Schelling 1959, pp. 15-8).

EXAMPLES OF MEDIUM-SCALE SPANS

The folded roof over the Verenigd Plastic.verkoopkantoor N.V. laboratory building at Zeist (1960) marked another significant step in folded plate construction **(fig.4)**. Erected under the supervision of architect D. Masselink in conjunction with G. Beenker and AKU engineering services (Caspers, 1962), it spans the entire 20 m width of the bay. Unlike the preceding roofs, this one has no cantilevers, resting as it does on end supports spaced at 2.5 m intervals, one under each valley. The Vs are 1 m high and slightly raised – 20 cm – at mid-span. As the incline of the Vs, adopted to comply with building height restrictions, is shallower than normally required in folded structures, the concrete had to be pre-stressed. This was achieved with two Freyssinet series of cables, one consisting of two wires laid practically horizontally along the valleys and the other, also containing two wires, anchored at the vertices and arched across the sides of the plates. The shell is 9 cm thick, although at the valleys and vertices it had to be thickened to 25 and 37.5 cm to cover the wires. As the photographs show, with its clear height of 4.02 m, the resulting shell is a prominent element in the visual delimitation of space in the laboratory. This effect is intensified by the contrasting height of the joists on the portal frame structure at one end of the building.

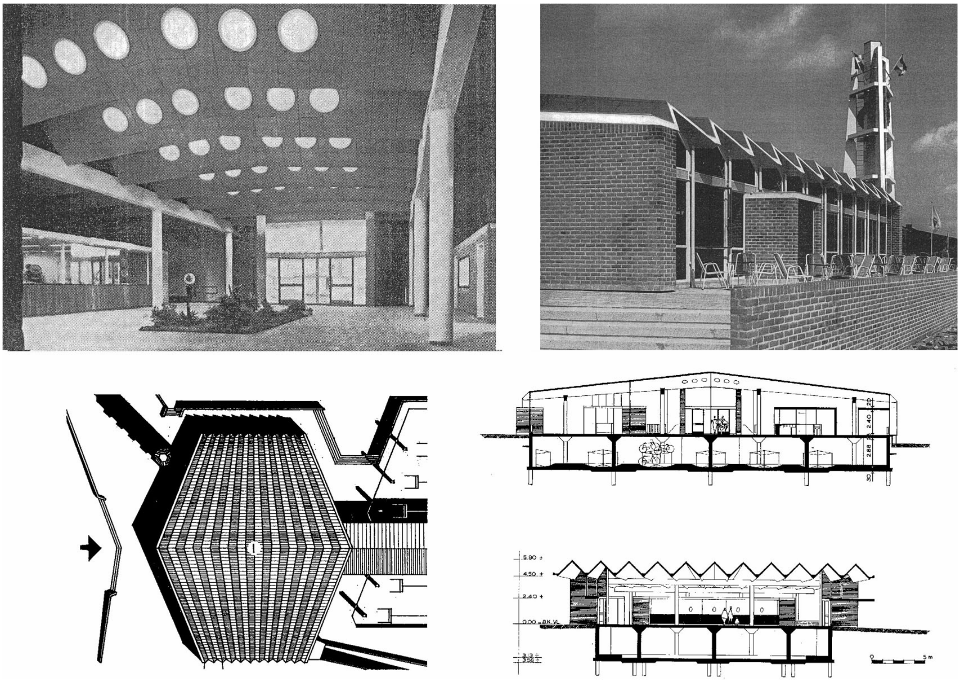

Figure 3. Den Helder station (*Cement* 1961, p. 441; Shelling 1959, pp. 16, 18).

Another fine example of structures of intermediate dimensions is the church at Hoensbroek (c. 1964), whose 25.50 m long, 2.80 m wide roof plates span a distance of 21.40 m **(fig.5)**. This building is of particular interest because - for reasons of construction time demands – the shell was not cast in place in keeping with usual practice. Rather, 13 precast members were used instead. The V-section joists rest on precast facade enclosure panels, likewise V-shaped. All the components were manufactured in a shop at Venray and shipped by train to Hoensbroek in a special car, and from the respective stations to the worksite by lorry (Meischke, 1964). This procedure was actually based on the experience acquired during construction of a prior church whose 20 m hypar plates were manufactured in the same plant and shipped by road (1960). Because the pieces forming the roof are separate entities, buckling problems arose in the wings that had to be addressed in a specific study (Beckelmans 1964). The units are 1 m high, as in the preceding example, and 8 cm thick. The facade panel is 20 cm thick, which includes 2 cm of Friglith insulation, and has a fold angle somewhat smaller than on the roof, as well as vertical windows on the sides.

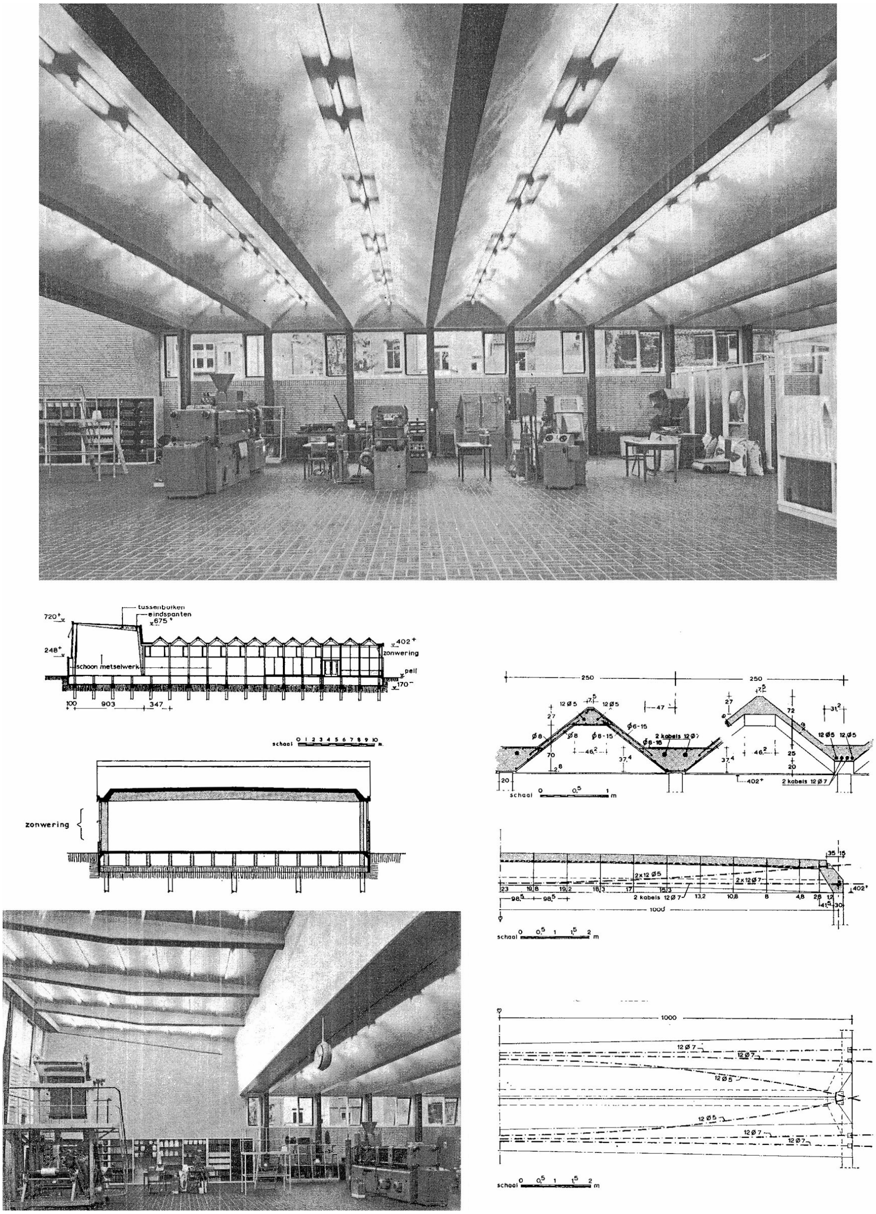

Figure 4. Verenigd Plastic-Verkoopkantoor N.V laboratory building at Zeist (Caspers 1962, pp. 17-9).

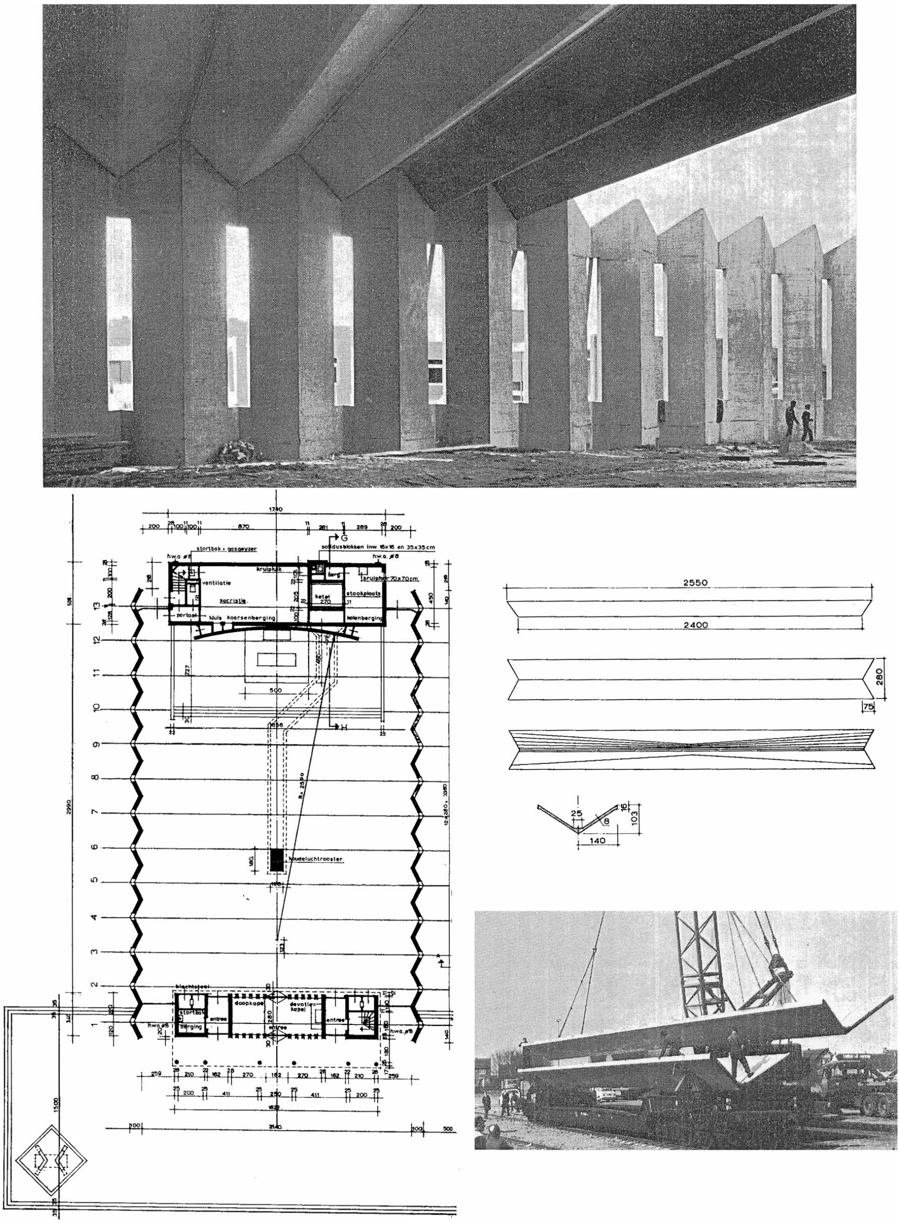

Figure 5. Church at Hoensbroek, Heerlen (Brekelmans 1964, p. 468; Meischke 1964, pp. 352-4).

LARGE-SCALE STRUCTURES

The first of the two large-scale folded roofs identified in Holland covers the new wholesale fish market at Scheveningen (c. 1964) **(fig.6)**. It covers the main lobby and an area housing a canteen and small auditorium, located in between a very long (23 x 15 = 345 m) warehouse with a saw-toothed roof and the office building. The offices, together with the area under the roof, vest the building with a sort of monumental forefront. The roof design is more complex than the structures described above, cantilevering 10.8 m beyond the supports and spanning a distance of 27.3 m. But in addition, it drops vertically at one end to form the enclosure wall on that side. The folds have equilateral triangular sections, but on one side of the free-standing column the size decreases by half to accommodate twice the number of sections. Moreover, all the foregoing, including the shape of the folds (which taper towards the smaller base), is adapted to a trapezoid floor plan. The dimensions are roughly 37 m long by 22.75 and 29.25 m wide on the short sides, with the entire roof sloping slightly but steadily downward from where it begins to cantilever.

The overall shape of the roof is designed to adapt the structure to bending moments:

> The depth of the cantilever was increased to accede to the builder's desire to adapt the height to the moment diagram. The roof is statically determinate, cantilevering outward from free-standing columns on one side of the building and resting - with an articulated joint - on a bearing wall on the other, as shown in the longitudinal section. […] A final fold was built at the columns to better adapt their depth and the depth of the bearing wall to the moment diagram; the height of this fold is at the bearing wall the same as for all the others.
> (Seyn and Hofman 1964, p. 467).

The chief problem, naturally, in light of the slenderness of the columns - nearly 9 m high - was lateral stability in this area, "which is why triangulated stiffeners were placed on both sides of the roof to absorb lateral forces; the roof rests on and is laterally supported by these members" (Seyn and Hofman 1964, p. 467). Once stability was ensured in this area, the issue of structural instability in others was addressed - in particular the risk of cantilever buckling. When Prof. A.M. Haas and the Stevin laboratory were consulted in this regard, they found that "in addition, a stabilizing strip would be required along the cantilevered edge, as well as transversal bracing at the point on the folds subject to the maximum positive moment" (Seyn and Hofman 1964, p. 467). Such stiffening was achieved by additional bracing between the end folds in the area around the columns. It is interesting to note that the same idea of a stabilizing strip, can also be found in the folds spanning the widest bay in the UNESCO headquarters auditorium in Paris, a key structure and one of the most handsome folded roofs in the world, designed by Breuer and Nervi (1958). Be it said, however, that this structure has no cantilevers.

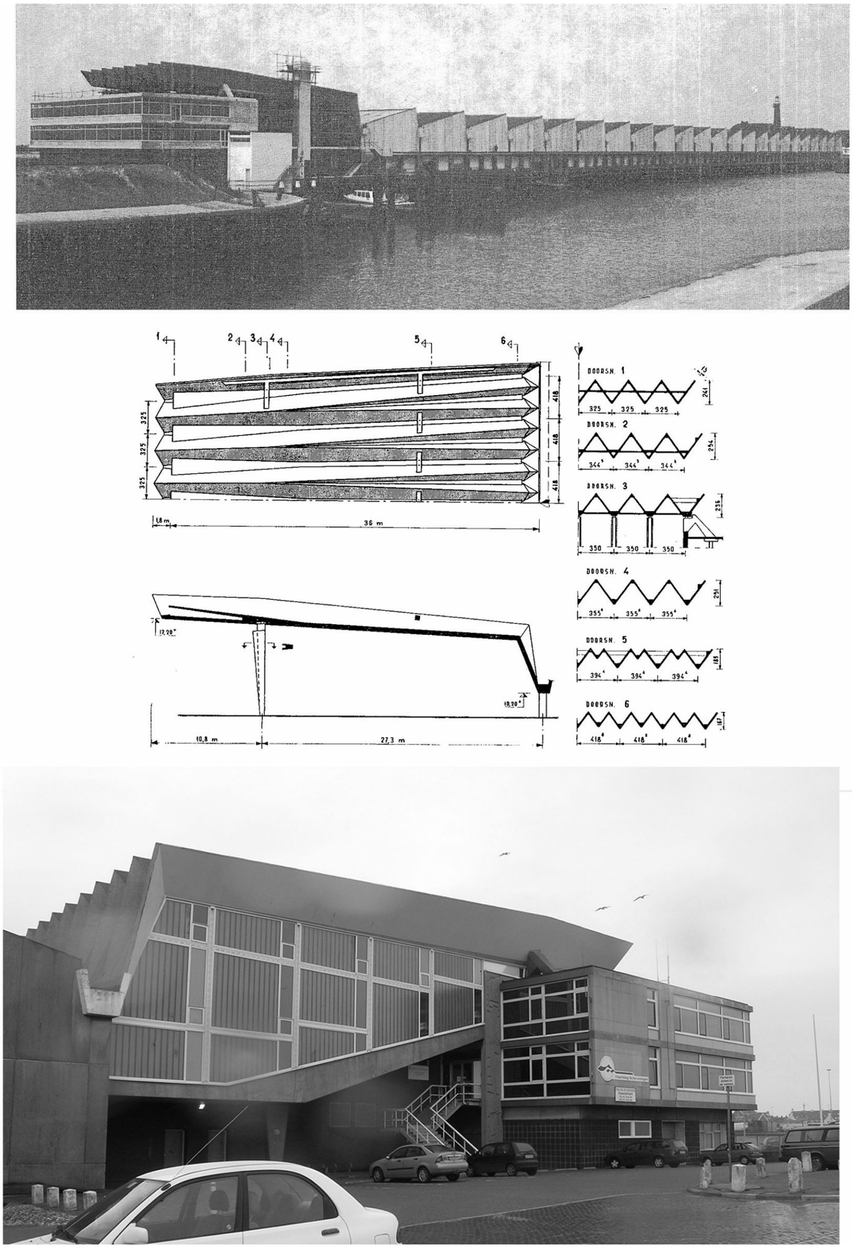

Figure 6. Wholesale fish market. Scheveningen (Seyna and Hofman 1964, pp. 461, 466; TU Delft).

The Scheveningen roof was erected with no prestressing and neither arched bars nor stirrups were used. Instead of the latter, bars slanted at a 45º angle were laid in both walls of the V to absorb the chief tensile stresses. The folds are 2.51 m high at the highest point and 15 cm thick. The steep slope necessitated the deployment of double-wall falsework, although the major difficulty was the stiffener used to stabilize the cantilever, for the continuity grooves that had to be left in the fold surface. After concreting, the falsework had to be removed through the narrow space between the stiffeners and the sides of the folds. For that reason and for the high quality workmanship throughout, the general contractor was publicly congratulated in the above article.

The Delft PS auditorium

The second example of a large-scale folded roof, the most prominent one to be built in The Netherlands, covers the Delft Polytechnic School auditorium. Work on this building was authorized to begin on 1 March 1961 but as a result of all manner of difficulties (bad weather in the winters of 1961-2 and 1962-3, construction-related complications and labour strife during the period) the scheduled completion date had to be postponed to 1965. The composition of this building is much more complex than any of the ones described above, since the roof does not simply cover a large open area, but rather a wide variety of spaces requiring very different support systems **(fig.10)**. Nonetheless, the most striking and interesting element from the standpoint of structural solutions is the large trough-shaped auditorium located above grade at the front of the building. This unusual design was the outcome of the need to meet two conflicting but essential requirements: the building, located at one end of the Mekelweg or main thoroughfare on the Delft PS campus, had to be highly visible from the road without obstructing traffic.

> The solution to this paradox was to build an auditorium on the first storey while leaving the ground storey underneath both visually and physically open for access to the Mekelweg.
>
> (Van den Broek, 1963).

In other words, the raised floor of the auditorium rests on two very wide (polygonal section) columns that are set back from the building perimeter to balance loads **(fig.7).**

Consequently, the floor slab cantilevers 14 m outward of these columns. This arrangement determined the need for an entirely cantilevered roof, since:

> It may be immediately inferred from the description of the trough (the floor under the building amphitheatre), a structure with an area of 1600 m2 resting on two columns in the middle, that it would not be able to support the weight of the roof as well. This rules out the possibility of resting the roof on the trough. Moreover, while the edge of the roof is set back 2 or 3 m from edge of the trough, it is still located 12 m beyond the columns; and since there may be no columns at any intermediate point in the auditorium, the roof must be built to project outward from the centre of the building.
>
> (Dusschoten 1964b, p. 270).

Given that loads could not be supported anywhere within the auditorium proper, the closest structure suitable for this purpose was the wall along the front end enclosing the service shafts. This meant cantilevering the roof 32 m in this part of the building. Together, then, the trough and roof would look much like two semi-open valves of a sea shell. The suspended roof envisaged in one of the preliminary designs was dismissed as unviable both for reasons of acoustics and constructional feasibility (Van den Broek, 1963, drawings; Dusschoten p. 270) **(fig.7)**. It was finally concluded that all the requirements could be met with a folded roof. All the structural members in the auditorium may be said, then, to be based on folded plate systems, since the trough-shaped floor slab may also be classed in this general category.

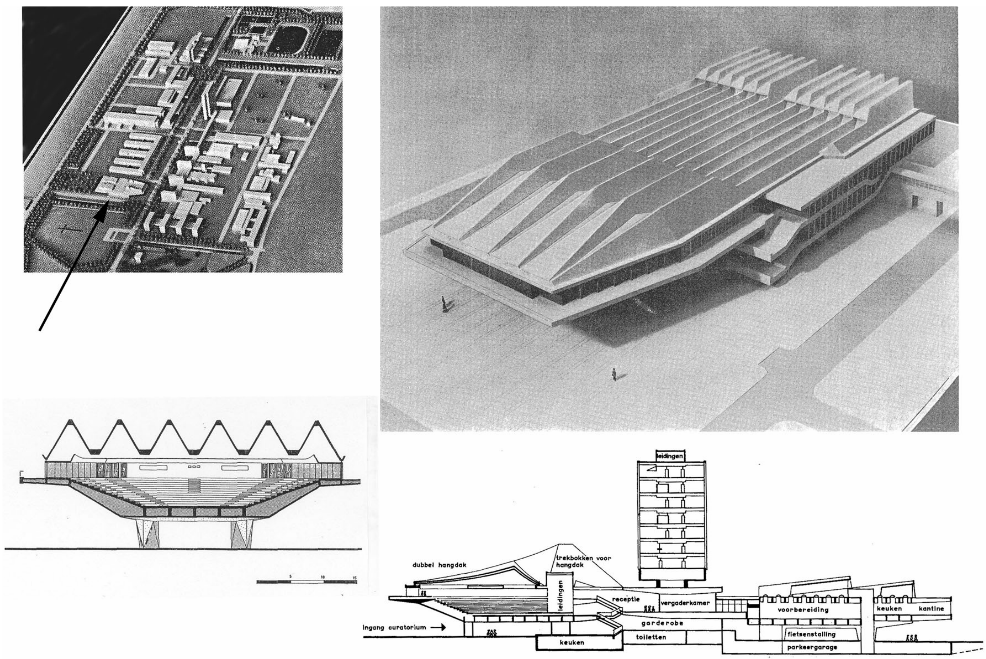

Figure 7. Scale model of campus layout, cross-section and first preliminary design for the Delft PS auditorium. (Broek 1963 pp. 730-1). Scale model of the final version (Dusschoten 1964, p. 270).

Although the study that follows focuses primarily on the roof, certain aspects of the trough are also discussed.

The basic shape of the roof cross-section is a series of six equilateral triangles measuring 7.40 m on each side. The enormous depth (6 m) generated is only necessary at the spring line, however; i.e., axis 13 on the longitudinal section of the building **(fig.8)**. For this reason, a few metres beyond that base and across the rest of the structure, all the unnecessary material is "eliminated" from the roof to form a grid that follows the lines of fold geometry. This lattice-type structure also simplified the installation of skylights. The rear (tensile) support for this grid consists in a huge girder at section number 18, in turn resting on four columns subjected to tensile forces, see number 6 on the axonometric drawing **(fig.8)**. Much of the centre of the roof over the building also rests on and counterbalances this beam. At section 13, the stage is spanned by a large concrete triangulated lattice girder that rests on the service shaft walls. Consequently, it supports the middle of the roof across a width corresponding to two base triangles. As it projects above the vertices of the folds, this beam is externally visible from above.

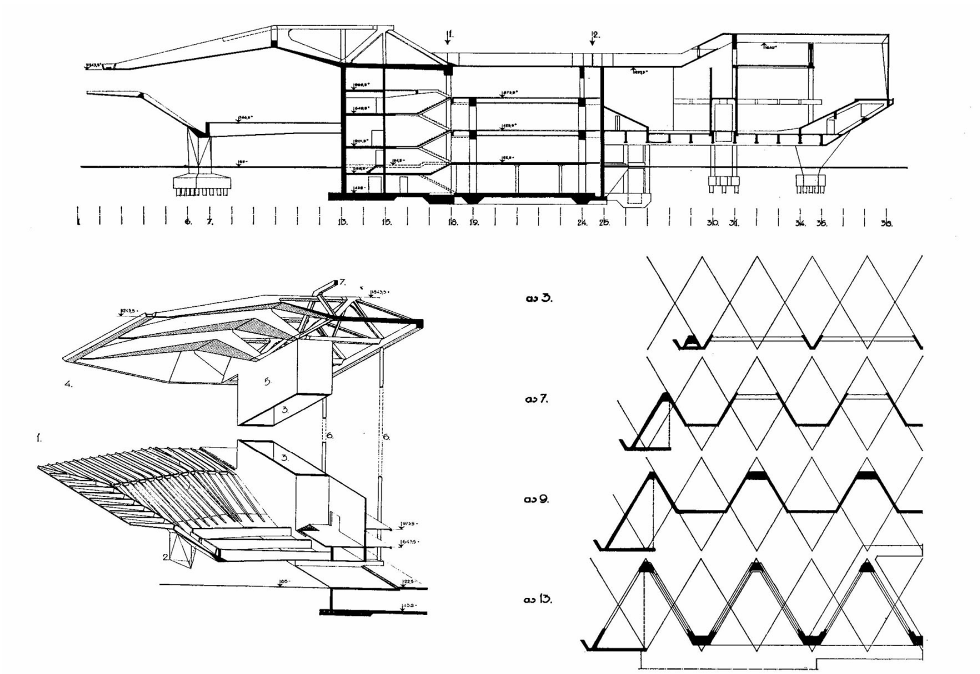

Figure 8. Reference longitudinal-section, axonometric perspective and cross-sections of the auditorium roof (Dusschoten 1964, pp. 271-2).

The cantilever itself is not a standard folded roof element, designed as it was to be as lightweight as possible. It does not in fact comprise a continuous shell, but rather a series of cantilevered beams. Separated on the free ends, these beams are nonetheless interconnected by membranes that form a continuous slab beginning at a point between sections 7 and 8. Their U sections are slightly open, with sides that lean outward from the bottom up. Longitudinally, they tend to converge at a point on the free edge. Their shape can be likened to a canal whose section tapers down to nearly nothing on one end. As a result, the inter-beam space steadily increases towards the outer edge, lightening the overall weight of the structure. What makes the roof look like a continuous folded structure from outside is therefore the arrangement of the lightweight beams and plates that comprise its surface. At section 10, the longitudinal profile tilts abruptly downward. The underside or soffit of the resulting structure determines the shape of the auditorium ceiling, largely calculated to meet acoustic requirements. Another important element is the gutter-shaped tie beam that runs along the entire front of the roof, connecting the tips of all the cantilevered beams **(figs.9, 10)**.

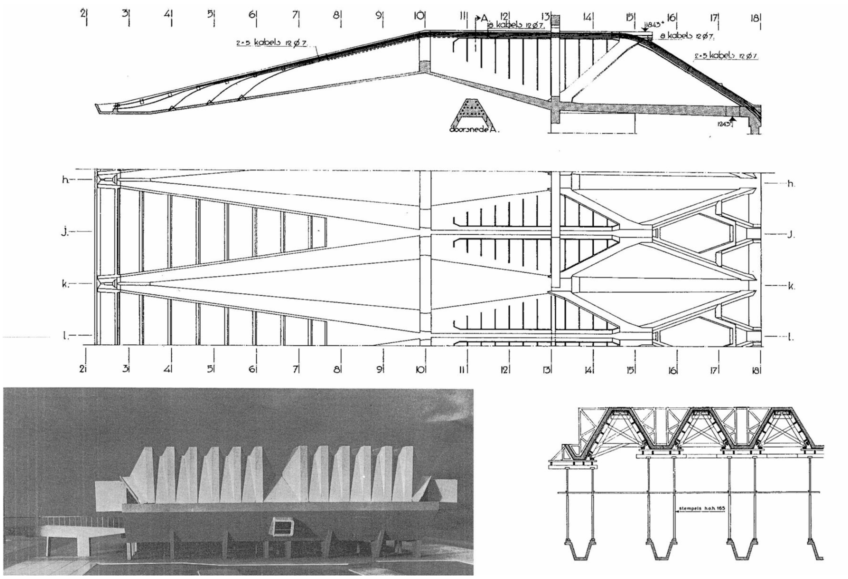

Figure 9. Auditorium roof structure plan and section. Rear of building showing vertical folds (model) (Dusschotten 1964b, p. 274). Section of double-wall formwork in mid part of building (Timmers 1965, p. 230).

Due to the trapezoid shape of the floor plan, the two outer folds are shaped differently from the rest, with a slab slanting downward to a horizontal line at the same elevation as the gutter beam. Since this outer slab is not laterally restrained, it could potentially bend and open outward. This is prevented by a wide horizontal stiffener positioned along the edge at the same height as and attached to the gutter beam, all around the perimeter. Such slab or stiffener might also be viewed as a final end fold, turned inward. Its free inside end is suspended from the vertex directly above it **(fig.9)**.

Tensile stress is absorbed by 18 prestressed cables, each consisting of twelve 7 mm Ø wires, laid along the top of the beams from the spring line to the cantilevered tip, and partly hanging down over the sides **(fig.9)**. For reasons of acoustic insulation, in particular as protection against the overhead jet noise, a very lightweight roof would not have been suitable; a weight of at least 400 km/m2 was deemed to be necessary.

As the scale model shows, the folded roof solution was not used over the main auditorium only, but on the building as a whole. The size and shape of the folds over the auditorium determined the size and shape throughout, although on the rest of the building the number of folds was doubled and their height consequently lowered. Although the arrangement is the same across the entire roof, the elevations and dimensions differ, as shown in the longitudinal section. From sections 18 to 31 (including the slanted abutment), the units are 2 m high and span a maximum of 18 m, whereas in the rest of the building, from sections 31 to 38, the height is 1.10 m and the spans measure 10.50 m. Moreover, this final section drops vertically to form the rear enclosure wall. Although the height is somewhat greater than strictly necessary in both cases, it was maintained for reasons of design consistency.

Initially, this part of the roof was to be precast, but it was finally cast in situ and prestressed. The steep slope of the sides (60º) necessitated the use of double wall falsework **(fig.9)**. As a general rule, much of the structure was prestressed more to prevent cracking than to enhance strength. This precaution was indispensable, for the concrete was to be exposed on exteriors and interiors both, with no further finish, for reasons of economy. And the roof was, naturally, no exception.

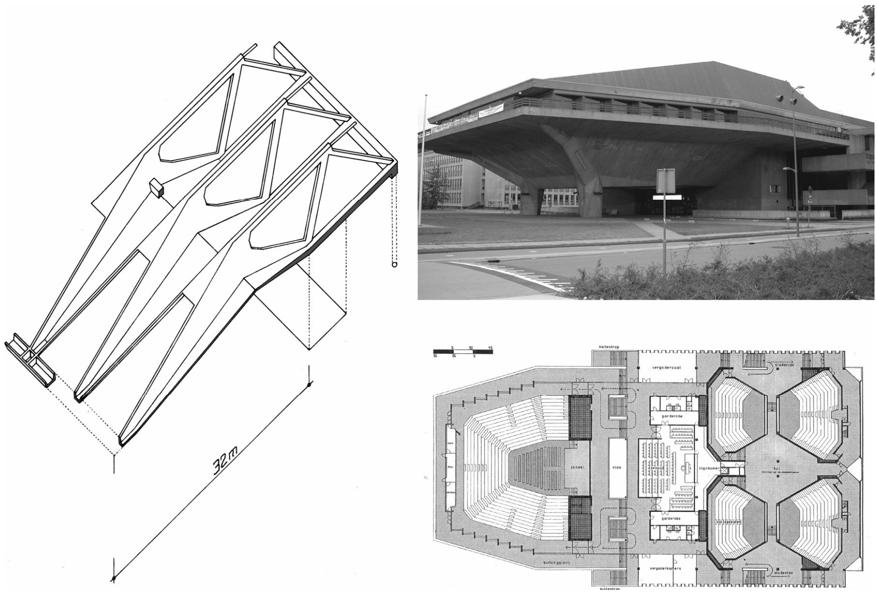

Figure 10. Axonometric diagram of the auditorium roof (author), auditorium at present (author), general floor plan (Broek 1963, p. 731).

Duschotten described the trough or structure supporting the amphitheatre in the following terms:

> The around the service shafts that enclose the open [sixth] side of the auditorium. The stage, 14 m wide, is located between these shafts. The legs are positioned 15 m inward of the edge and spaced 14.50 m apart.
>
> (Dusschotten 1964, No. 3 p. 165).

The structural solution for the base or bottom of the trough – five very deep beams - entailed no particular complexity. The slanted sides, however, were built as though they were folded plates, as described above. Nonetheless, given its vast size, this structure cannot be regarded as a "pure" thin shell. The upper part is reinforced with a series of ribs, both perpendicular to the edges and along the joints **(figs.11, 12)**. The former are required because without them the bending stress on the plates would have necessitated a slab 60 cm thick at the least favourable point. Thanks to these ribs, 25 cm wide throughout with a maximum height of 80 cm and spaced at 2 m intervals, the slab is no more than 16 cm thick. The ribs along the joints, in turn, are needed to support and tie the ribs described in the preceding paragraph, but also to absorb the huge stress converging on the joints themselves. The diagonal ribs at the corners that slope towards the legs are particularly impressive: calculated to weigh 1880 t each, they have a section measuring 2.5 m2 at the bottom (although it tapers).

Spatially, the structure might be viewed as a grid comprising a series of ribs, when in fact it was engineered like a folded plate system, at least as far as the bottom and the first tier of slanted slabs are concerned.

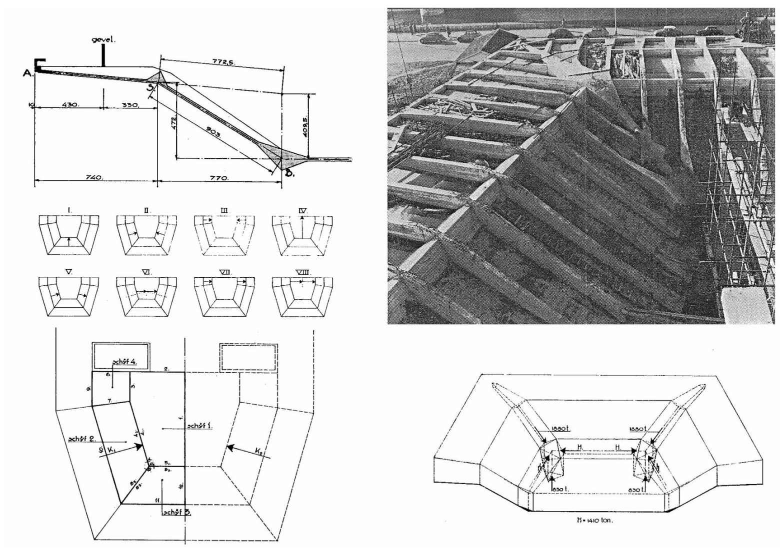

Figure 11. Trough dimensions, structural diagrams and photograph of reinforcing ribs (Dusschotten 1964, pp. 166-8).

By contrast, the more shallowly sloped upper tier was in fact designed as a ribbed slab resting on the lower tier and embedded in the service shaft walls. In the article cited above Dusschotten describes and explains the structural calculations used for the floor and lower tier, in which for reasons of symmetry each slab is statically determinate **(fig.11)**. With respect to the process followed, he writes: "this ingenious folded plate approach – that we owe to engineer F.A. Vreede –

was implemented with the aid of Cremona diagrams for four symmetrical and four asymmetrical loads" (Dusschotten p. 168).

For a number of reasons, all discussed in the article, prestressing was used extensively in this structure. The most important of these reasons include: a) smaller thicknesses and therefore lighter weights, b) the need to impede changes in shape and c) the need to create compressive stress on the plates comparable to shear stress. All this led to a complex system of cables in the beams, reinforcement ribs and plates. Since in the latter the cables had to be laid on a deep curve (up to two radians), the forces acting on the plates are not perpendicular; this in turn reduces friction and leads to a substantial loss of stress **(fig.12)**.

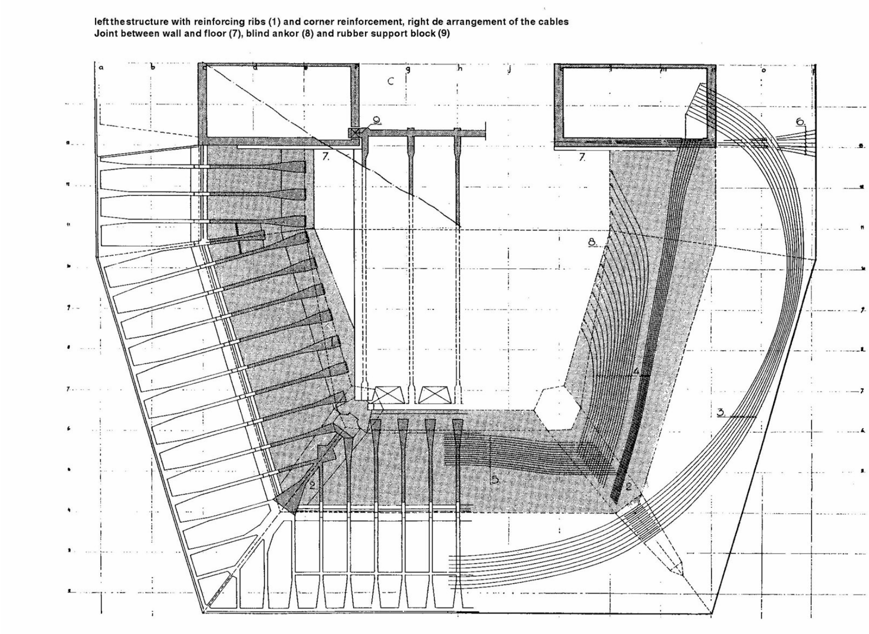

Figure 12. Auditorium floor plan showing reinforcing ribs and pre-stressing cables (Dusschoten 1964, p. 166).

Be it said, finally, that the roof and trough structures are not completely independent of one another. At the (cantilevered) front, the roof beams are attached to the trough below with vertical Dywidag bars to ensure that,

> the ends of the beams were aligned with a slight tensile force because "the beams could not be expected to bend equally". These bars are designed to prevent "the roof from weighing down on the trough while at the same time ensuring that the distance between roof and trough was unaffected by snow, wind or differential shrinkage or deflection.
>
> (Dusschoten, 1964, p. 273).

Horizontal movement, however, needs to be allowed.

The auditorium building was the most complex folded plate structure erected until that time in Holland, although as noted, it was not a pure shell because its sheer size necessitated the adoption of mixed solutions, inasmuch as the structure could not have been built with shells alone at any reasonable cost. That explains why such a large portion of this paper is devoted to this structure, the final example of Dutch folded plates considered in the present study. Nonetheless, there are other examples of interest, at least two of which are worthy of mention. They are not discussed hereunder, however, because they exceed the limits defined, given that they can only be marginally regarded to be Dutch endeavours. One is the Van der Leer offices cited earlier in this paper, built in Amstelveen by the American architect Marcel Breuer. The other is the roof over the splendid town hall at Marl, Germany (1958 -c.1960), designed by the Van den Broek and Bakema studio. Both these structures might be discussed in a follow-up of the survey initiated here.

In any event, perhaps the first conclusion that may be drawn is the very short time (less than seven years) during which folded shell structures were built in Holland, since their use was virtually abandoned soon thereafter (at least, to the best of this author's knowledge). Substantial progress can be said to have been made in this period, with cantilevers increasing from an initial 4.5 m to 32 m in the Delft auditorium. Spans, in turn, at least doubled in size, from 9 to over 20 m. Another characteristic worthy of note is the rational approach taken in most of them, insofar as they represented the most suitable solutions to the problems posed with the resources available at the time. Proof of that may be found in the thoughts and reflections of their authors, summarized here. Lastly, these structures were much less common than other types of shells, particularly the cylindrical variety. The present study was undertaken precisely to remedy the present general lack of awareness of their existence, especially since the attention and detail with which they were described in contemporary literature is a clear indication of their authors' conviction that they were indeed interesting and exceptional structures.

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