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Planning and Execution of a Prestressed Cable Roof

Planification et exécution d'une couverture suspendue en câble précontraints

Planung und Ausführung eines vorgespannten Kabeldaches

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Introduction

The cable roof above the icehockey-stadium at Tampere, Finland erected 1964 comprises a simple structural system as shown at fig. 1; after a brief description of the roof structure proper, of the anchoring arrangements and of the principles of the statical calculations, attention will be given to practical experiences.

In Scandinavia light hanging roofs are introduced in two different structural concepts namely either as Mr. Jawerths system (see introductory report page 106) based on prestressed cable trusses or as cable nets where two sets of cables of opposite curvature are established in a prestressed state. Wind suction necessitates prestressing of these light structures and this prestressing represents at the same time the advantage of increasing the rigidity of the system and the disadvantage of adding to the loading.

The Scandinavian structures usually cover rectangular areas and the edge elements to which the cables are anchored will often be straight; in this case the problem is to bring the large horizontal reactions from the cables down to the foundation in an elegant way and the economical influence from the shape of the anchoring structures should be critically analysed at a very early stage of the concept. A hanging roof will almost automatically be cheap, but the surrounding anchoring system might be rather expensive.

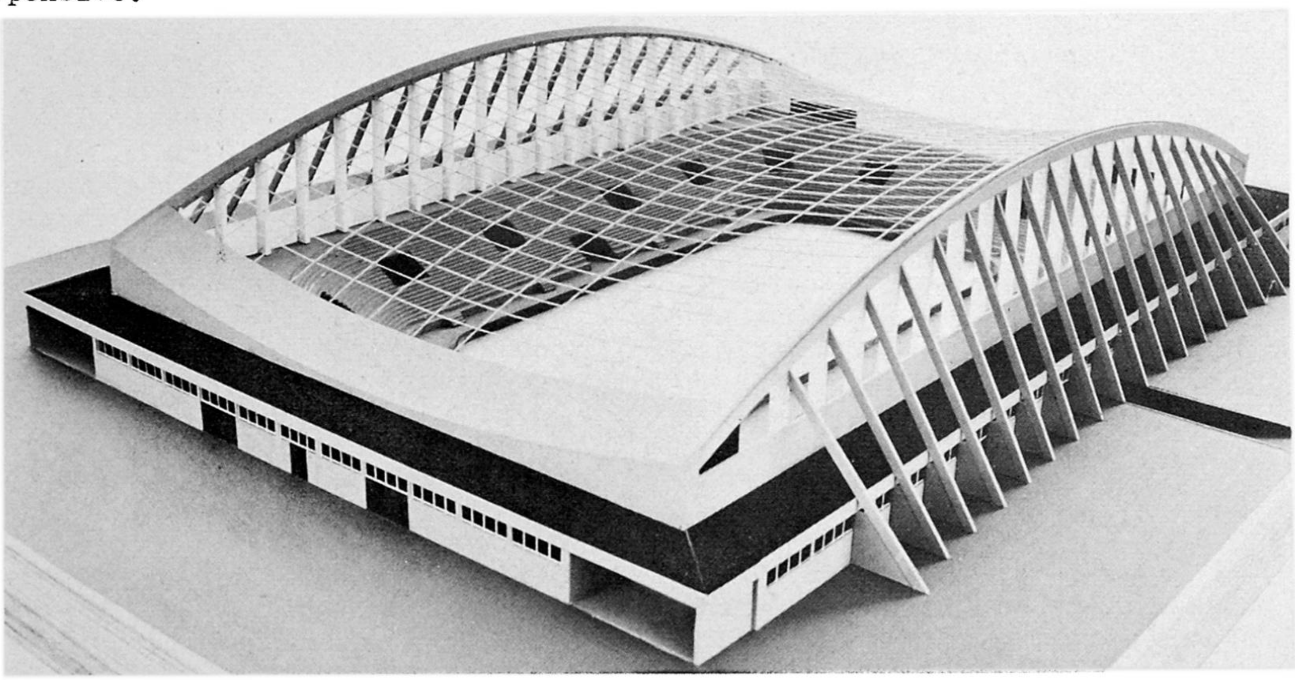

Fig. 1: Model photo

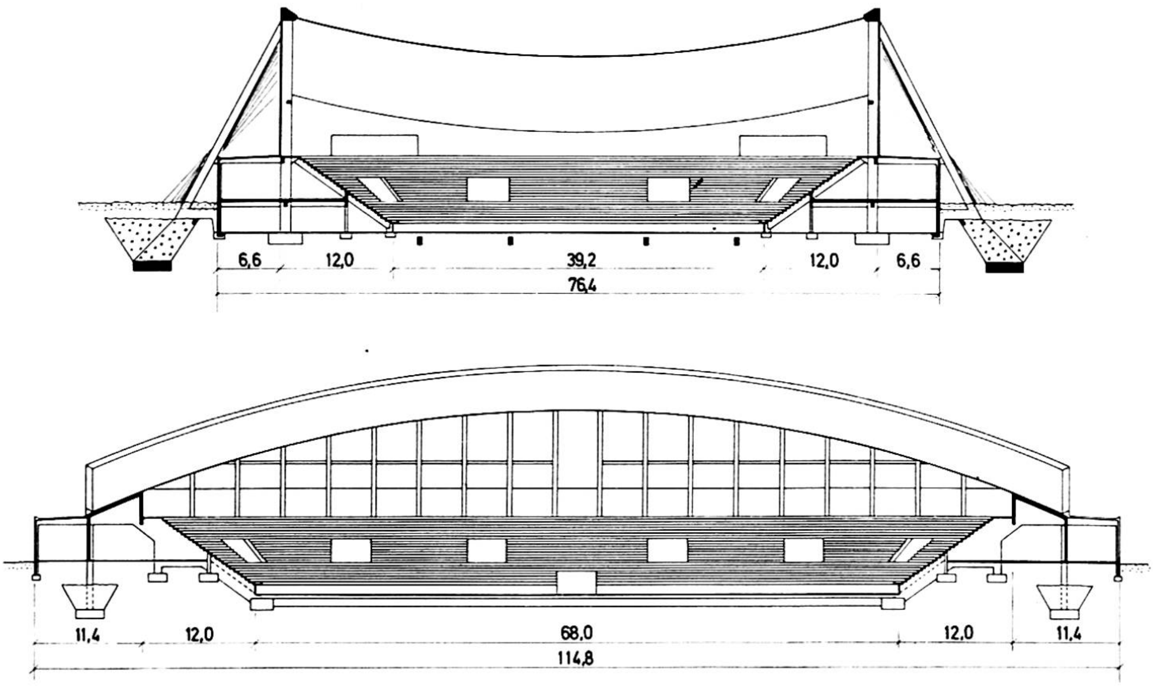

Fig. 2: Sections through icehockey stadium

At Tampere the time left for design and erection was very scarce and the choice of cable type, prestressing technique and anchors took into account that the building process should proceed simultaneously with the detailing.

The Structure

The geometrical proportions of the building are shown at fig. 2. The length of the hall is 92.0 m and the width is 63.2 m. The shape of the roof is a hyperbolic paraboloid with a sag for the suspension cables of 3.9 m and a rise for the prestressing cables of 8.4 m. Spacing of the cables are 80 cm for the suspension cables and 160 cm for the prestressing cables, all cables being solid ø 26 mm S.H. bars with an ultimate tensile strength of 110 kg/mm^2.

The prestressing cables pass above the suspension cables and any two crossing cables are linked together with a friction clamp (see fig. 3) which also serves as support for the wooden rafters.

All the cables are anchored to the R-C edge structures, the suspension cables to the two vertically curved beams above the facade columns and the prestressing cables to the doubly curved roof slabs above the gable buildings. As indicated at fig. 4 all cables pass the edge beam through a pipe to the anchor at the outer side; on account of the cable movements vertically and horizontally caused by variations in prestress and live load all the pipes end up with a "trumpet" sleeve made of polyether, reinforced with glassfibres.

The composition of the roof proper as shown at fig. 4 did permit an execution without any scaffolding.

A central problem of the design was how to anchor the edge beams to take up the horizontal component of the suspension cable forces; preference was given to conventional gravitation anchors because cheap rock boulders could be procured for production of the necessary mass concrete.

The inclined ties from the edge beams to the R-C plinth below the mass concrete counterweight were executed in posttensioned concrete with passive anchors in the plinth. In this manner a very rigid support for the suspension cables was obtained.

The forces from the prestressing cables are introduced in a R-C profile with great torsional stiffness. This profile is composed by the doubly curved slab above the gable building and the two adjoining vertical walls (see fig. 2). The profile is supported horizontally by the facade structures and by four rigid frames in the grand-stand structure.

The Statical Calculations

The live load from snow was assumed to be 150 kg/m^2 in the Tampere area; four different cases of snow distribution were investigated.

For statical wind load only suction developed by wind directions parallel to the two main axes of the hall was considered. As no wind tunnel tests were available and as the suction values for flat roofs indicated in the codes in different countries vary rather much, the loading assumptions were greatly influenced by model tests described by Beutler in ref. (1). Finally the suction at the windward side was estimated to -80 kg/m^2 and at the leeward side estimated to -40 kg/m^2; as the highest in Finland registered velocity pressure was 70 kg/m^2, this assumption should be quite conservative when the great size of the roof surface is considered.

For this structural concept both exterior and interior temperatures influence the actual stresses in the cable net; the temperature variation inside the hall is estimated to 25°C and the outside temperature range is put to 45°C. Creep and shrinkage in the posttensioned ties are evaluated as well as the relaxation of the steel cables.

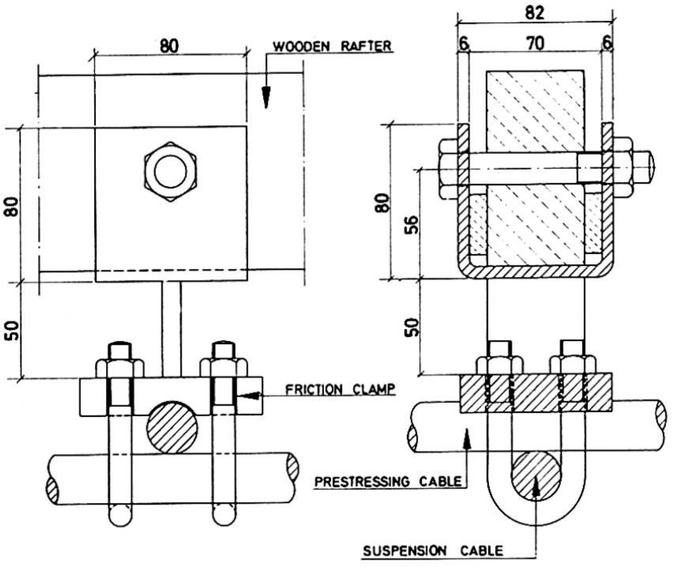

Fig. 3: Cable link

All the calculations for the cable net proper follow the method suggested by F-K Schleyer in ref. (2); information about the mesh sizes selected for the different loading cases and the numerical work in general is presented by the author in ref. (3) and (4) where attention is given to the corrections due to additional loads resulting from the first calculation step with non-linear terms.

The necessary prestressing of the cables is found by trial and error and the final "equivalent" pressure defined as the pressure the two nets perform against each other in the pure prestressed condition is approx. 90 kg/m^2.

The actual factor of safety as the collapse of a cable depends on the ultimate strength of the cable anchors and of the thread of the cable socket sleeves. Preliminary calculation indicated definitely that the dimensioning of the cable net would solely depend on the choice of the permissible stresses at working conditions with due regard to all extra influe-

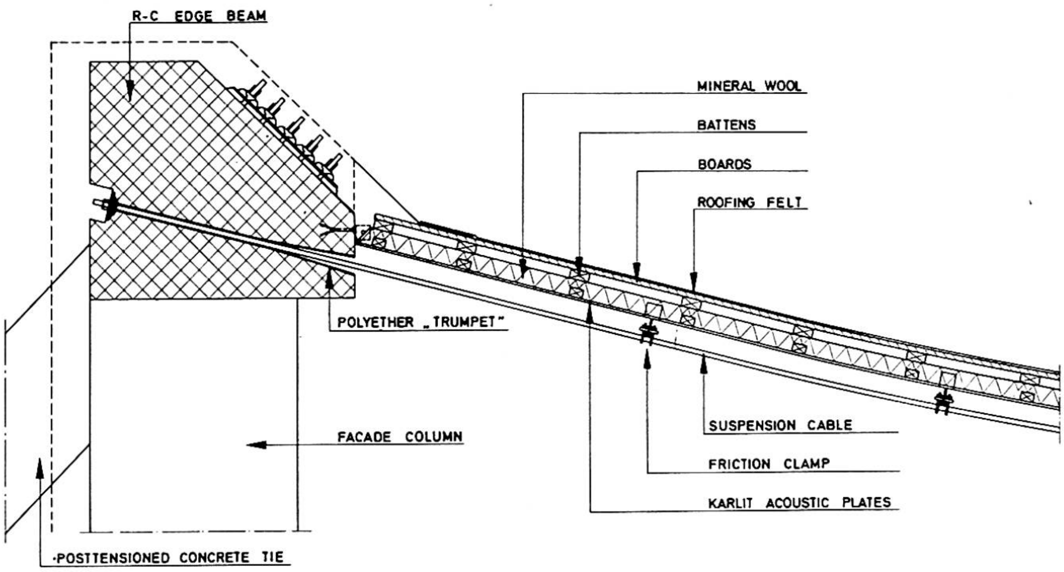

Fig. 4: Section through roof near the edge

nces (creep and shrinkage of ties, temperature changes, relaxation etc.). In this case maximum cable stresses of a magnitude of approx. 6,000 kg per cm^2 were accepted, this being 5-10% less than are normally accepted for the use of the same cables in posttensioned concrete; it should be pointed out that the investigated load combinations are fairly intricate and as well that the stress calculation from the linear theory is proved to be on the safe side. While the above-mentioned extra influences only increase the maximum stresses in the suspension cables approx. 15% the maximum stresses in the prestressing cables increase approx. 60%; this fact has considerable importance for the dimensioning of the stabilising gable buildings.

The dynamic wind effect on a suspension structure of this type is difficult to analyse, the evidence by Vaessen in ref. (5) gave no reason to assume that the wind pressure at a specific point of the roof surface should pulsate into step with the self exciting vibrations of the roof; we found still less reason to assume that all the local wind pressures on the great roof surface should coordinate to a pulsation into step with the said vibrations. As the fatigue strength of the threads in the cables is relatively low it was attempted to estimate the number of stress variations greater than the fatigue strength but of course this is a primitive approach. It should be emphazised that the cable net technique has the advantage that a random local fatigue fracture does not lead to collapse of the complete structure but the notice permits to take countermeasures.

Some Experiences from the Erection Work

Naturally it proved difficult to erect the formwork and to cast in situ the geometrically complicated and large perimeter R.C. structures with the exactitude usually expected for normal R.C. work, but no serious consequences occurred.

A good deal of trouble came up in fixing the "trumpets", through which the cables pass, in the right direction and level; the design asked for a margin of $\pm$ 0.5 cm as to the position in height of the two ends of the pipe, but levelling control found deviations on the finished structure 4 or 5 time this value; fortunately enough deviations of this size never occurred for two adjacent cables, but the deviations were rather evenly built up over quite long distances. As all the hanging cables had to respect the actual levels of the trumpet centers an adjustment analysis of the height of all hanging cables in the erection situation has to be carried out.

Each suspension cable consists of four cable parts interconnected with socket sleeves. According to a proposal put forward by the contractor a preliminary bridge was built along the central axis. The cable parts were lifted by crane and while the central socket sleeves could be mounted from the above-mentioned bridge, the other two sockets sleeves at the quarter points were screwed in position from light movable bridges cantilevered from the previously erected suspension cables. Temperature correction as to the position in height of each suspension cable was compulsary as the actual temperature varied approx. 10°C during the time of erection.

The prestressing cables were put up by hand from light movable bridges on the hanging cables. As the friction clamps at each crossing of two cables had to be active before the prestressing took place, the cable net was built to a geometrical shape worked out in such a way that the net after prestressing would project in two sets of equidistant straight lines. In order to facilitate this procedure all cable parts were cut with a margin of 0.1 cm and all intersection points marked with coloured lines on both suspension cables and prestressing cables before they were brought in position.

An adjustable momentum wrench was applied for the tightening up to the friction bolts in the clamp. Test was carried out to evaluate the correlation between the tightening moment and the resulting friction in order to obtain a reasonable safety against sliding of the connection.

The prestressing of the cable net was only introduced through the longitudinal prestressing cables which were tensioned two by two working simultaneously from the two facades towards the center line, each cable being activated from both ends. The final stresses were built up by repeating this procedure six times. After each stage the elongation and pump pressure at either cable end were registered.

Strain gauges were mounted on certain cables as a further control. For the check of whether the required prestress condition was attained everywhere in the cable net, measurement of the deflections took place at 25

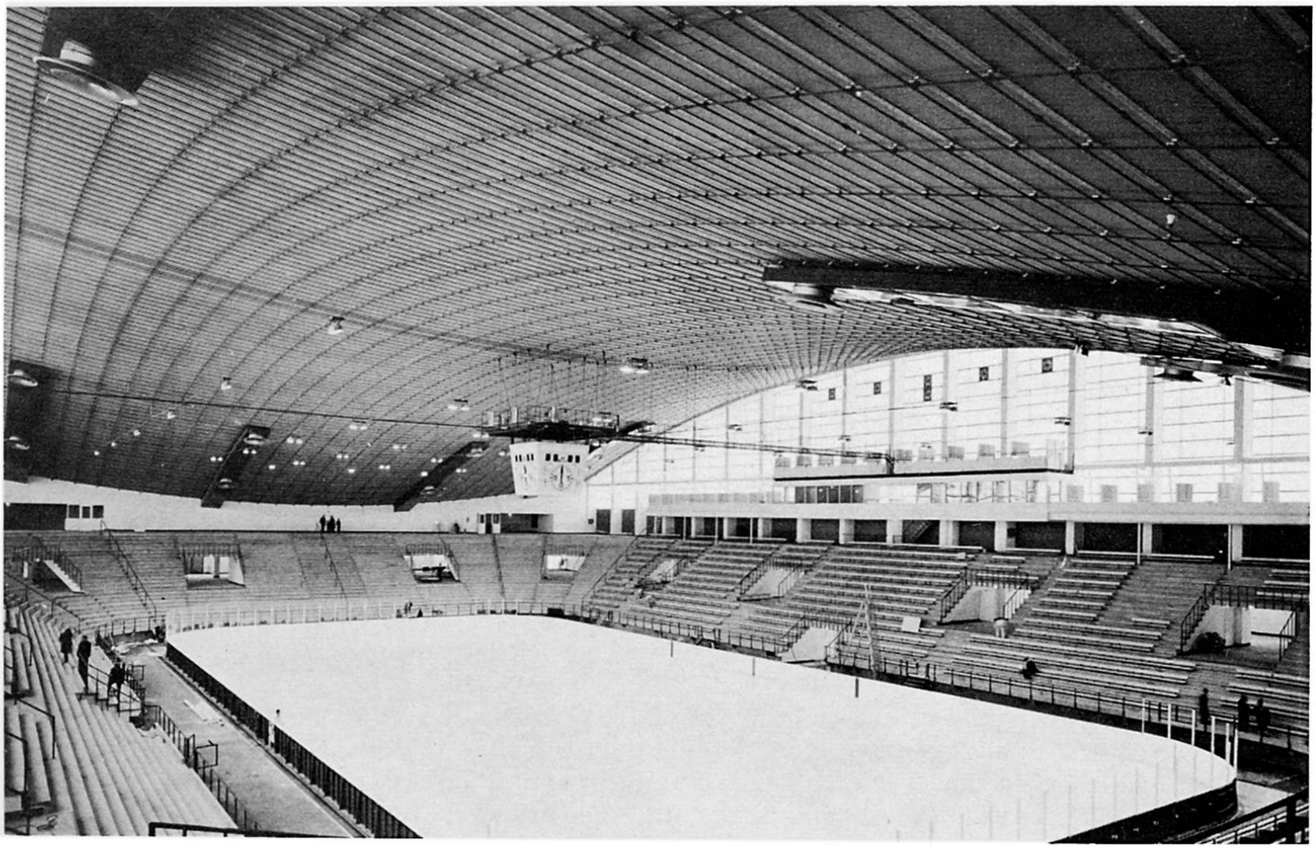

Fig. 5: Interior of the finished stadium

points for comparison with the predicted ones. Interpretation of these results indicated that the cable net had been slightly "overstressed" (5-10%) and it was also observed that one of the sides had obtained a bit higher stresses than the other one, but this fact was deemed of no consequence. The total erection of cables and the prestressing lasted approx. 6 weeks.

More accurate calculation of the cable deflection near the supports is fairly difficult as this involves extensive computation; in the actual case great care was exercised to ensure that the "trumpets" had reasonable variations in curvature in all directions to avoid any tendency to breaks. The distances between the cables and the "trumpets" edges were measured before and after execution of the roof cover.

Observations After Completion

The stadium has now been in use for 7 years and has proved that the choice of this type of suspended roof-structure was suitable.

Observations during the past years indicate that no vibrations of any kind due to the action of the wind have occurred. It can also be mentioned that the snow uses to distribute itself with very high loads near the gable building at the leeward side; this special loading has in no case caused deflections, which exceed what has been calculated.

Visual control now has also confirmed that the shape selected for the "trumpet" is satisfactory because the cables keep reasonable distance from the "trumpet" edge.

As far as the water tightness of the roof is concerned, there has been no problems due to the fact that the total roof-surface is separated by longitudinal and transversal expansion joints, which absorb the movements due to the above-mentioned loads.

From the spectators point of view the solution is very satisfactory because it is possible to see the whole game area from every part of the stadium without any obstacles.

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Summary

The contribution describes the design of a 63.2 x 92 m hanging roof above the new icehockey stadium at Tampere, Finland, built during 1964. The structural system is a rectangular prestressed cable net composed of high tensile steel bars. After comments on the support systems and the statical calculations the article is concluded with some remarks about the practical problems on the site and later observations.