Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 71 (1994)

Rubrik: Session 1: Long-span structures for building and spaces: materials and concepts

SESSION 1

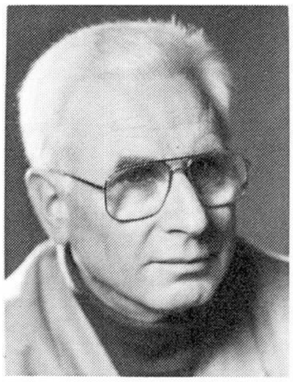
LONG-SPAN STRUCTURES FOR BUILDING AND SPACES: MATERIALS AND CONCEPTS

Conceptual Design of Long-Span Roofs

Concept et projet de toitures de grande portée

Konstruktiver Entwurf weitgespannter Dächer

Jörg SCHLAICH
Prof. Dr
Univ. of Stuttgart
Stuttgart, Germany



Jörg Schlaich, born 1934, studied in Stuttgart, Berlin and Cleveland, USA. Designer of bridges, buildings, long span lightweight structures, cables and solar energy, researcher in structural concrete. Director of Institute for Structural Design, Univ. of Stuttgart, and Partner of Schlaich, Bergermann and Partner, Consulting Eng.

Rudolf BERGERMANN
Dipl.-Ing.
Schlaich, Bergermann and Partner
Stuttgart, Germany

Rudolf Bergermann, born 1941, studied in Stuttgart, designer of bridges, buildings, long span lightweight structures, cables and solar energy.

SUMMARY

For long spans the dead load should be minimised, which simultaneously evokes the problem of stability under varying loads such as wind and snow. Balancing lightness and stiffness is a basic challenge of structural design. The different approaches are discussed and illustrated by two examples.

RÉSUMÉ

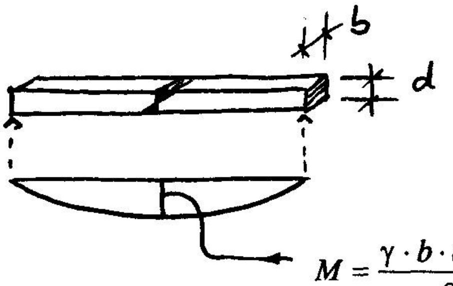
Dans le cas des grandes portées, il faut réduire le poids propre le plus possible tout en assurant la stabilité de la structure sous l'effet des charges variables du vent et de la neige. La recherche du juste équilibre entre la légèreté et la rigidité est l'un des aspects les plus intéressants du projet. Les méthodes adéquates sont présentées et illustrées par deux exemples.

ZUSAMMENFASSUNG

Für grosse Spannweiten muss man die Eigenlasten minimieren, aber gleichzeitig sicherstellen, dass die Struktur unter den zeitlich veränderlichen Wind- und Schneelasten stabil bleibt. Leichtigkeit und Steifheit richtig abzuwägen ist einer der interessantesten Aspekte des konstruktiven Entwurfs. Die dafür geeigneten Möglichkeiten werden diskutiert und mit zwei Beispielen illustriert.

1. DESIGN PRINCIPLES

In designing long-span structures engineers first of all must get rid of their habit to think in fix proportions, forgetting about the dominating influence of scale. It is misleading to give the required thickness d of a beam or girder as a fixed fraction d/L of its span L with d/L = 1/18 for a single span and 1/22 for multiple span or so. No: Simply by designing a beam or slab with rectangular cross-section to carry its dead load only we find:



$$M = \frac{\gamma \cdot b \cdot d \cdot L^2}{8}$$

γ: density of the material
β: strength of top and bottom fibre in tension and compression
L_{lim}: limit span of a beam under its own weight
d/L_{req}: required depth/L_{lim}

$$\frac{\gamma \cdot b \cdot d \cdot L^2}{8} \cdot \frac{6}{b \cdot d^2} \le \beta \quad \rightarrow \quad \frac{d}{L_{req}} \ge \frac{3}{4} \cdot \frac{\gamma}{\beta} \cdot L_{lim} \ne const. \qquad \text{(see fig. 1)}$$

d/L is not a constant figure but increases linearly with the span and depends further on the efficiency of the material γ/β; the lighter the material and the higher its strength, the smaller will be the required thickness d of a beam of a given span. But its proportions will change dramatically with its absolute size, a fact which can be well observed from natural structures (fig. 1).

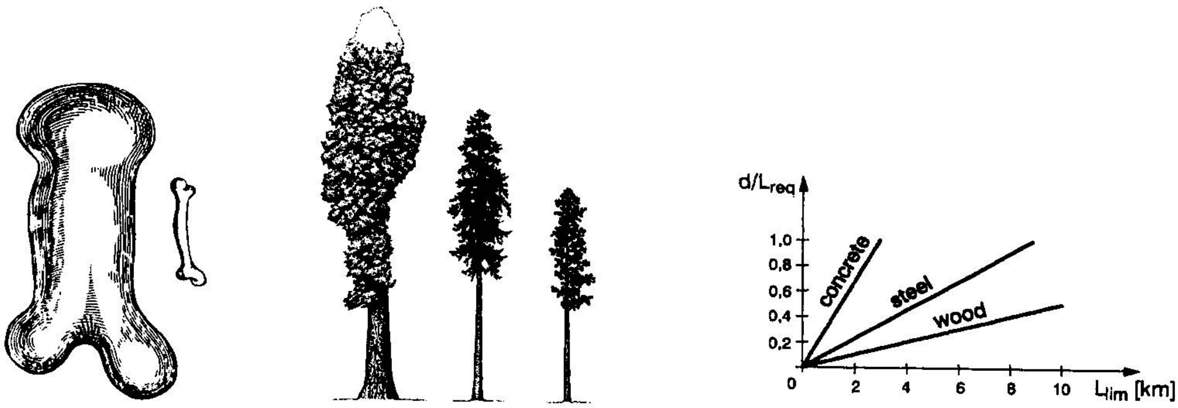


Fig. 1 The role of scale in natural structures [1][2]

With increasing span, solid girders become more and more clumsy, their dead load eats away their strength and for long-span structures something has to be done to by-pass this awkward situation.

The answer is of course well known: We have to leave away all that material of our beam which is not fully used: at mid span, we need to keep only the top and bottom fiber and leave away the web and towards the supports we can reduce the chords but need the web. Thus, via T-, TT-, U-girders and hollow slab- and box-girders, we reach the different types of trusses (fig. 2, top):

Their main feature is that they avoid bending and carry loads only by axial forces, thus making full use of the material's strength and reducing their dead load to a minimum.

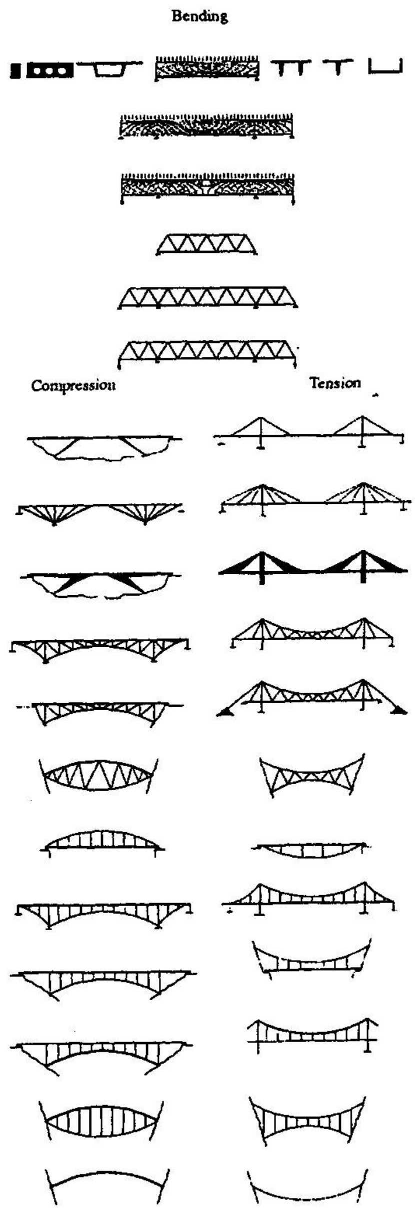


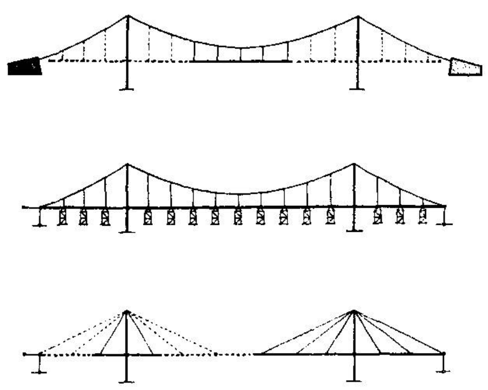
Fig. 2 The development of the girder

The next step towards lightness for increasing spans is to subdivide the girder into a primary structure acting either in compression (fig. 2, bottom left) or in tension (fig. 2, bottom right).

The primary structure in compression (tension) may either attribute its tension- (compression-) partner to the subsoil - in that case we speak of an earth-anchored or a true arch (suspension) bridge - or utilize its secondary structure (usually) for self-anchorage.

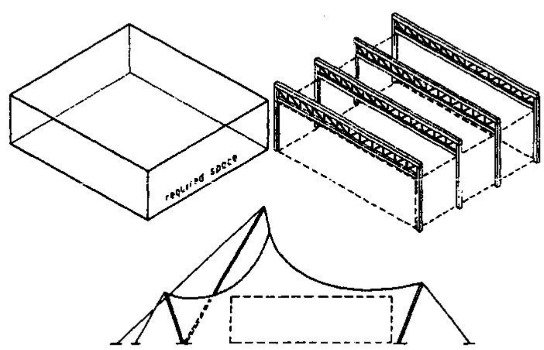
An earth-anchored structure is (usually) easier to erect but costlier, whereas for a self-anchored just the opposite is true. The cable-stayed system combines both advantages: it is self-anchored (resulting in cheaper foundations) and can be constructed free cantilevering without temporary falsework (fig. 3). Beyond its self-balancing function the purpose of the secondary system is to stiffen the primary system and to serve as an envelope in case of a building or as a road/railway in case of a bridge.

In case of a building - the theme of this symposium - the envelope needs not necessarily be straight or horizontal, as it does in case of a bridge, but also there these hybrid structures (M. Saitoh speaks of Beam-String-Structures BSS in case of the primary structure acting in tension) are functionally better adapted to the required space as double-curved space structures, which we shall discuss further below (fig. 4)[3].



The primary structure, especially if it acts in compression, must be stabilized or stiffened by the secondary structure. Making use of all possibilities to stiffen the primary system
- the weight of the structure as a whole
- the bending stiffness of the girder (and the arch)
- the geometry of the primary system (triangular or quadrangular mesh)
- overall prestress (only applicable to cable bridges)
- and even combining them in an intelligent way
- not speaking of combining different materials in the same bridge

Fig. 3 The earth-anchored suspended girder needs abutments but can be built without falsework. The self-anchored girder needs temporary falsework but avoids horizontally loaded foundations. The cable-stayed girder combines both advantages.



stands for the intellectual appeal and the joy of structural design. Some examples of the behaviour of hybrid structures are given in figure 5. The further we proceed down along the list of girder types, as sketched in figure 2, the more flexible they become, but we simultaneously realize that the types further up in the list have to pay for their higher stiffness, resulting from triangular mesh, with improved fatigue strength and ductility of their structural elements, especially their cables.

Fig. 4 Hybrid structures adapt better to the required space than double-curved structures which usually encase too much volume.

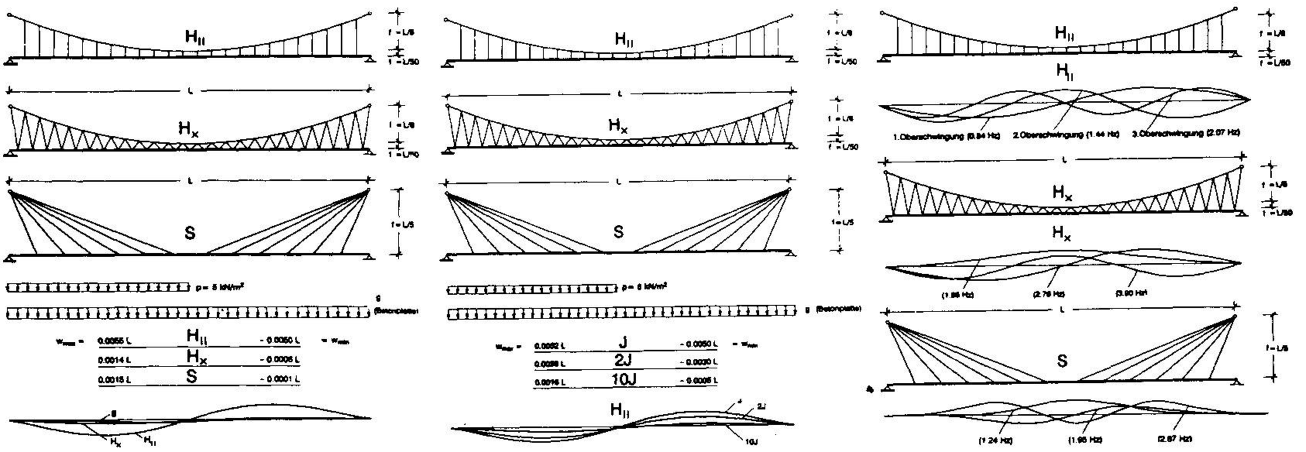
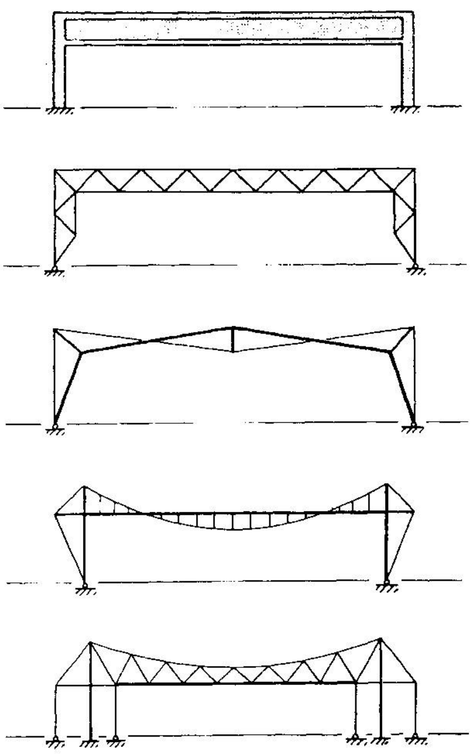


Fig. 5 Comparison of deformations and frequencies of suspended girders with vertical hangers (H"), inclined hangers (Hx) and of cable-stayed girder (S) with varying moment of inertia J of the girder.



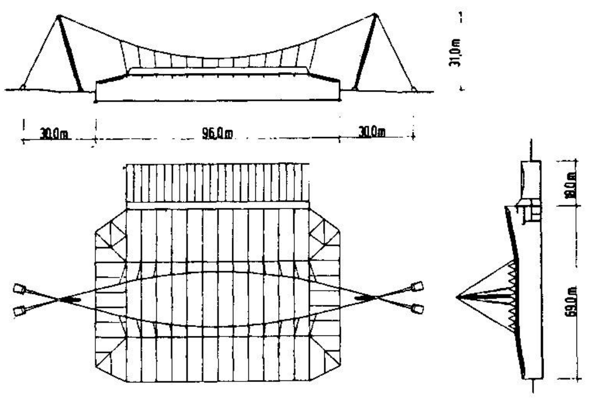


Fig. 6 Hybrid tension structures for an ice-skating hall at Memmingen (left, several alternatives) and for a multi-purpose hall at Karlsruhe [3]

If we are not too timid as far as deformations are concerned, we can make use of this whole catalogue also for long-span building roofs, as many examples recently built demonstrate (fig. 6). We are inclined to call this type of buildings "High Tech Architecture", insinuating that they are an invention of our times. This is not so as a stadium roof proposed as early as 1927 by Heinz and Bodo Rasch from Stuttgart clearly demonstrates (fig. 7).

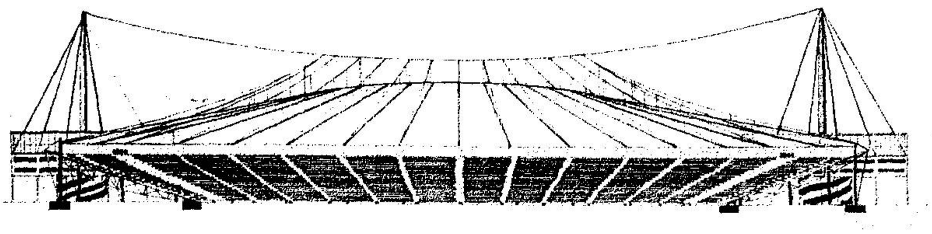


Fig. 7 Stadium project by Heinz and Bodo Rasch

With these hybrid structures the load bearing and the enveloping functions are usually independent or additive. As mentioned, this may be an advantage as far as the encased volume is concerned (fig. 4) but of course if both functions are combined, the overall result must be more efficient. This brings us to the double-curved surface structures (fig. 8).

Similar to the girders (fig. 2), they either combine compression and tension or they work primarily in compression or in tension. And again, proceeding down the list, they become increasingly flexible or deformable, depending on the type of stiffening respectively the topology of the surface
- the continuous surface
- with triangular mesh
- with quadrangular mesh

At their base, both lists (fig. 2 and fig. 8) meet with the same unstiffened arch or catenary cable.

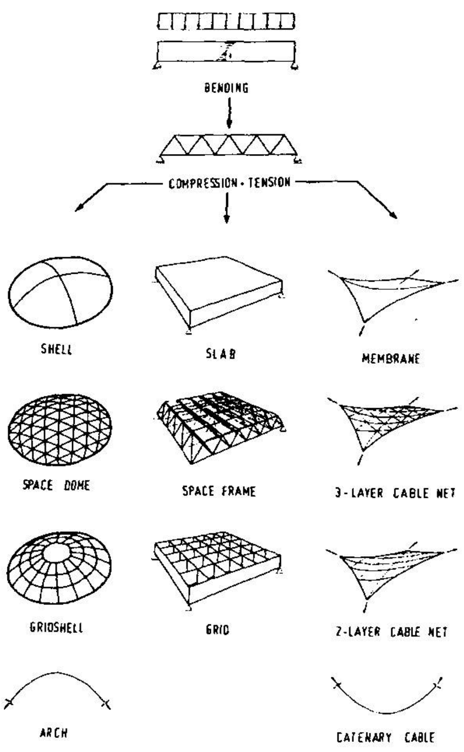


Fig. 8 The development of double-curved surface structures

Of course, beyond these pure surface structures there is again the possibility of combining them with other structural members, e.g. the shells may be stiffened by cables in the shape of spokes wheels or the cable nets by interaction with a roofing which avails of some shell action.

We further recognize again prestress, mechanically applied to surfaces with anticlastic curvature or pneumatically applied to surfaces with synclastic curvature.

And again, there is a close interrelation between the topology of these structures and their construction method, usually prefabrication and erection: The concrete shells, though most efficient structures, and the very concrete structures, suffer from their costly frameworks, since double-curved surfaces are non-developable. Several efforts have been made to revive concrete shells. Heinz Isler is most successful with that, as demonstrated by his beautiful shells. Another approach is the use of pneumatic formwork (fig. 9). More common than concrete shells are today the spherical grid domes in their different topologies, amongst them best known the geodesic dome [5]. The problem is how to cover a double-curved surface with triangular mesh with as many as possible elements, struts and nodes, of equal geometry (see example 1 in section 2 of this paper).

On the tension side of figure 8 we face in principle the same manufactural problem. Some basic answers are compiled in figure 10.

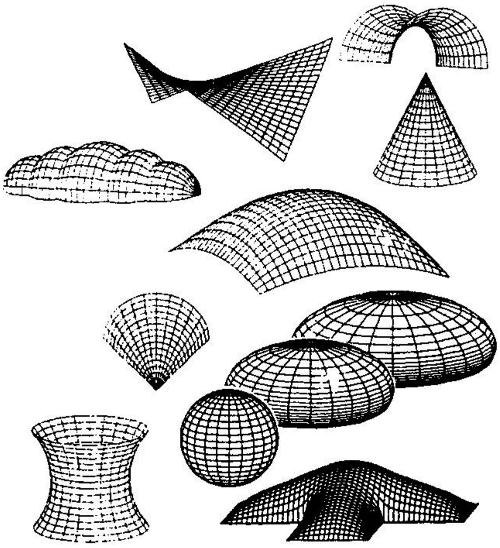


Fig. 9 Pneumatic formwork for concrete shells: suitable shapes [4]

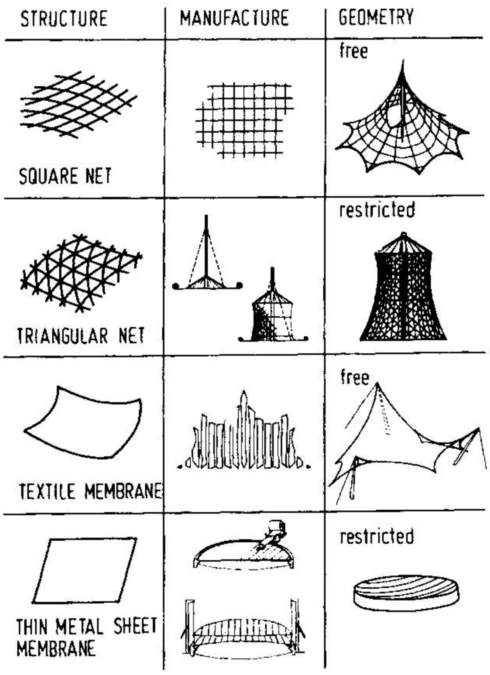


Fig. 10 Double-curved tension structure: The interrelation between type of structure/ manufacture/geometry or load bearing behaviour

The 2-layer cable net with quadrangular mesh is easy to manufacture and permits a variety of forms, unmatched by any other structural type, but it has to pay with large forces, large deformations and finally high costs. The Munich Olympic Roof (1972) is up to date the largest application [6]. The 3-layer cable net has an ideal membrane-shell load bearing behaviour but out of manufactural reasons, its geometry is limited to rotational shapes. The cable-net cooling tower at Schmehausen (1975) was the largest application [7]. Textile membrane structures can be completely prefabricated in the shop using a cutting pattern, are brought to the site, unfolded there and erected without any falsework. They can be combined with primary cable structures (as shown by example 2 of this paper). Convertible roofs are the high art of membrane structures [8]. Finally metal membrane structures, rarely applied up to now, can either be made from strips and welded on site, as done for the Moscow Olympic Structures [9], or thin stainless steel sheets can be pneumatically deformed utilizing their immense plasticity, as demonstrated with the manufacture of solar concentrators [10].

The authors are aware that the above was only a short summary of basic design principles of long-span roofs. But instead of filling all pages allotted for this paper with abstract considerations, a short presentation of two structures, recently designed by the authors, may be more illustrative.

2. TWO EXAMPLES: A GLASS COVERED GRID SHELL AND A HYBRID MEMBRANE STRUCTURE

2.1 The glass roof over a courtyard of a museum in Hamburg

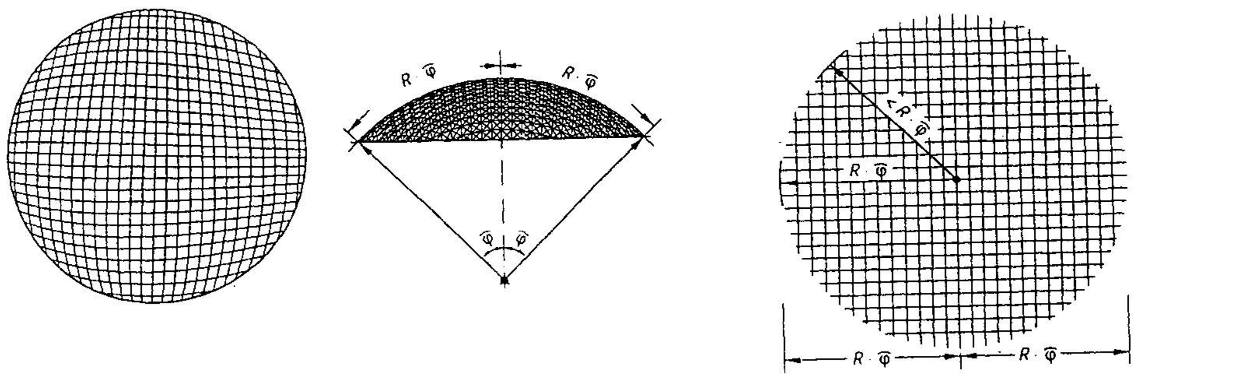
Glass roofs are attractive from an architectural as well as climatical point of view. Having already been the symbol of the new architecture of the Industrial Revolution during the 18th and 19th century, they experienced a revival during the second half of this century through the work of pioneers like Walther Bauersfeld, Konrad Wachsmann, Buckminster Fuller, Max Mengeringhausen, Frei Otto and others.

Obviously the most favourable basis for a translucent roof is the double-curved reticulated spatial structure with triangular mesh. Such structure, however, especially if directly glazed without intermediate glass frames, evokes three basic problems:

- The geometrical problem to cover a double-curved, i.e. non-developable surface with triangles, having for manufactural simplicity as many as possible members and nodes of equal size. (This problem obviously got some relief through recent progress in CNC-manufacturing.)
- Glass panels are preferably produced in quadrangles, of course permitting a variation of their angles. Therefore only two out of three members of the triangle constituting the structure should support the glass.
- Especially for double glazing the quadrangular glass panels must either be produced double-curved to fit the structure's surface, or the geometry of the structure must be chosen so that the four node points of each mesh are in one plane and may be glazed with plane glass panels. For single glazing, however, some warp of the glass panels is acceptable.

It is impossible to discuss this whole issue and all possible solutions here. Therefore, one solution, recently developed by the authors, shall be presented and exemplified [11]. The aim is to arrive at a structure corresponding to what is called "space dome" in figure 8 but suitable to be built for non-mathematical free shapes, too.

The basic grid of the structure when developed into a plane is a square net consisting of flat bars (fig. 11). This plane square net may be turned into almost any type of shape by changing the original 90° mesh angle. The quadrangles become rhomboids. This way any double-curved structure suddenly becomes "developable", and accordingly simple is the assembly of the basic grid. It entirely consists of bars, identical in length, bolted together, pivoting at their intersections. Bars of different length occur only at the outer edge as dictated by the structural geometry. The mesh angles are determined by the intended structural shape.



a) View — b) Elevation (with diagonal cables) — c) The bar grid, developed into a plane (= plane square net)

Fig. 11 The structure when developed into a plane is a right-angled, square grid of bars of equal length.

However, this basic quadrangular mesh pattern does not yet have the favourable structural characteristics of a shell to withstand wind and snow loads. Hence, the square mesh is braced diagonally with thin cables to achieve the required triangles. Diagonal bars would all vary in length, entailing the never ending task of cutting and fitting them. Therefore, instead of bars, cables are used, running beneath the bars from one edge to the other, fixed by clamping plates at the joints. Therefore, there is no need for measuring the constantly changing length of the diagonals. Later the cables are simply fitted, prestressed and clamped over the entire length without any problems. The prestressed cables work both ways, in tension and compression, and very efficiently support the structure. The glazing is clamped directly onto the flat bars. Consequently the glass panels are rhomboids with constantly changing angles (fig. 12).

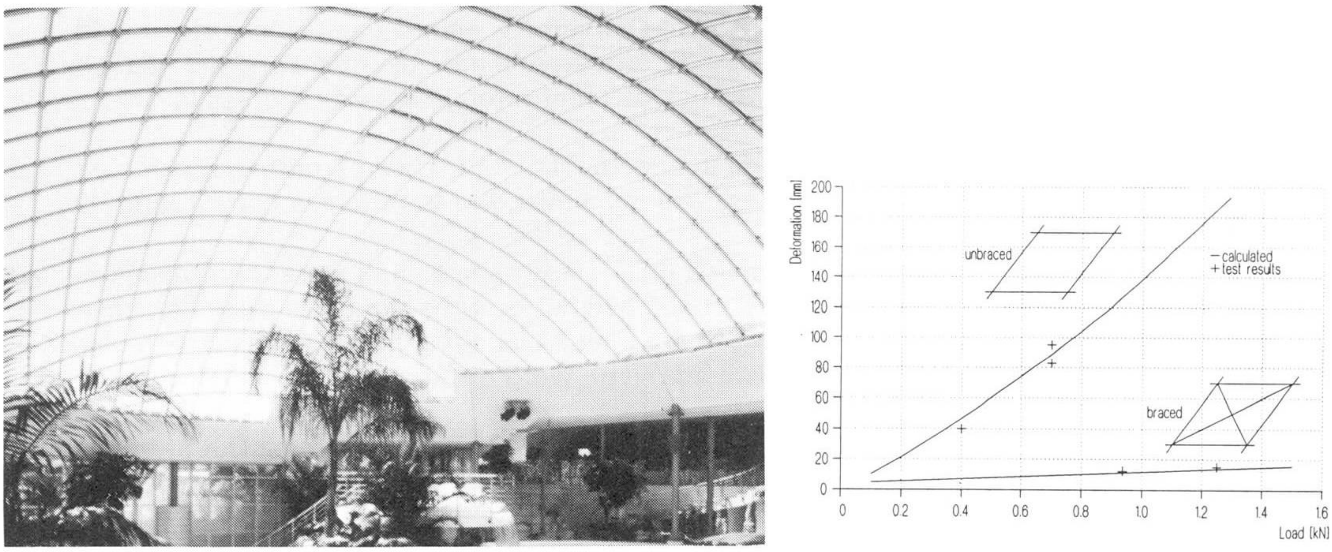


Fig. 12 Grid dome braced with cables and test results for a braced and unbraced dome

For the L-shaped courtyard of the Hamburg museum, the lightest and most transparent structure had to be designed, which would burden this historical building as little as possible - in both senses of the word: it was not to alter the overall appearance, and must only transmit minimal additional loads to the historical building.

The net dome has two barrel-vaulted sections, with spans of 14 m resp. 18 m, with a smooth transition between them. There the geometry is the result of an optimization, transferring the majority of the roof loads via membrane compressive forces and avoiding bending stresses (fig. 13).

The structure consists of 60 x 40 mm flat bars of St. 52.3, galvanized and painted white - in other words, hardly any more than the minimal dimensions of supporting members for a glass cover. These bars form a quadrangular net with a uniform mesh of about 1.17 x 1.17 m. Cables, installed afterwards, prestressed and clamped down at their joints form the varying diagonals.

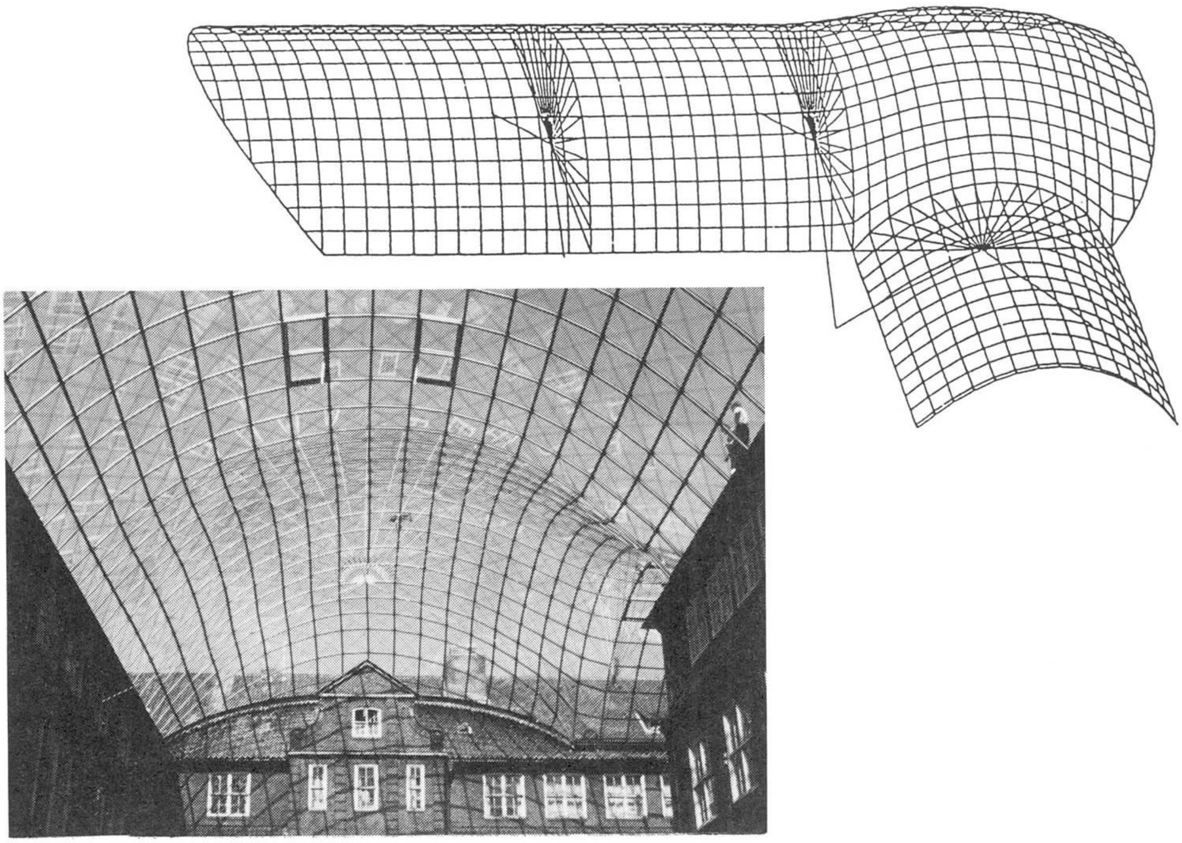


Fig. 13 Three-dimensional graph and interior view of the Hamburg grid shell

These minimal cross-sections enable the grid shell structure to transfer dead load as well as snow and wind loads. Since extremely high snow loads on one side due to drifting or trapping of snow in the roof valley could not be ruled out, the "somewhat softer" areas of the barrel vault were additionally stiffened with spokes wheels consisting of cables radiating from a "hub".

The glazing, 2 x 5 mm laminated safety glass, was placed directly on the flat bars and secured with plates at the joints. A heating wire was inserted between the glass support and the steel bars to prevent condensation.

The roof was designed and built in just 6 months.

2.2 The membrane roof over the grandstands of the Gottlieb-Daimler stadium at Stuttgart

For the 1993 Field and Track Worldchampionship in Stuttgart, within 18 months the largest membrane roof in Europe, covering 34,200 m^2, has been planned and built, covering an existing stadium.

The requirements to protect at least 90 % of all spectators, the poor soil conditions and restricted space were the main parameters which made this light weight structure, designed to be mostly self-anchored, obviously superior to other alternatives, like steel cantilevers or roofs with longitudinal and transverse truss girders (fig. 14).

Fig. 14 Aerial view of the completed roof

The roof length along the main axes comes to 200 and 280 m respectively. The outer edge of the roof in plan is formed by two partly circles, showing a radius of 104 m in the curves and of 248 m behind the main grand stands. The roof width or cantilever length is constanly all around. The main roof structure consists of two rectangular steel box compression rings, supported by 40 tapered steel box columns at 20 m spacing, further of 40 radial cable girders, leading from the compression rings to the inner cable tension ring, which keeps all cables under sufficient prestress (fig. 15). This primary structure, formed by the steel and cable elements, is sufficiently stabilized in itself and needs no further stiffening by the secondary structure. Horizontal wind forces are transferred down to the foundations by bracing cables between some support columns, arranged in the roof quarterpoints.

The 40 cable girders are made by the upper "snow"-cable and the lower "wind"-cable, stressed against each other by the suspender cables at 7.5 m distance. Since the tension ring cable and the compression rings form concentric circles in plan, the steel box elements get merely axial compression forces without any bending under prestress and dead load.

Fig. 15 After the erection of the shell structure the tension ring is assembled on ground

After placing the 40 columns, followed directly by the assembly of the two compression rings, the 8 cables of the inner tension ring were laid out on the ground, connected to one ring and the 40 radial cable trusses also preassembled and attached to the inner ring as well as to 40 lifting devices, which from the upper outer ring pulled the cable structure up and into the final position .

This lifting procedure was predetermined in all intermediate stages and carefully controlled by continuous force- and geometry measurements until the structure reached its final position after 3 weeks only. All radial cables were then attached to the rings by inserting the bolts into the hinges and the primary structure was ready for the membrane installation. Geometrical measurements confirmed that very close tolerances, necessary for the membrane structure, can be reached and confirmed by carefully planned application of 3-D computer software for analysis as well as for the preparation of drawings during shop fabrication.

Tubular steel arches with a tie span the distance between two "wind"-cable/suspender cable nodes in circumferential direction. Seven such arches are needed for each of the 40 panels, producing 6 saddle-shaped membrane roof parts inbetween and two steeper outer elements, running from the last arch to the edge cables (fig. 16).

This secondary structure weighs only about 8 kg/m^2 roof area. The front and rear membrane edges are circular with thin diameter flexible cables in pockets prestressing the membrane. The radial membrane panel edges are fixed and clamped by metal strips, which are anchored to the "wind"-cable from both sides, thus balancing the horizontal tangential forces directly. A secondary membrane, welded to the main panels, and overlapping, makes this radial joint watertight. The welded seams in the membrane run in radial direction, due to fabrication and aesthetic reasons. The PVC-coated polyester membrane with an additional Fluorpolymer-protective layer on the upper surface is of Güwa Type III, specified with a minimum translucency of 8 % and a white colour.

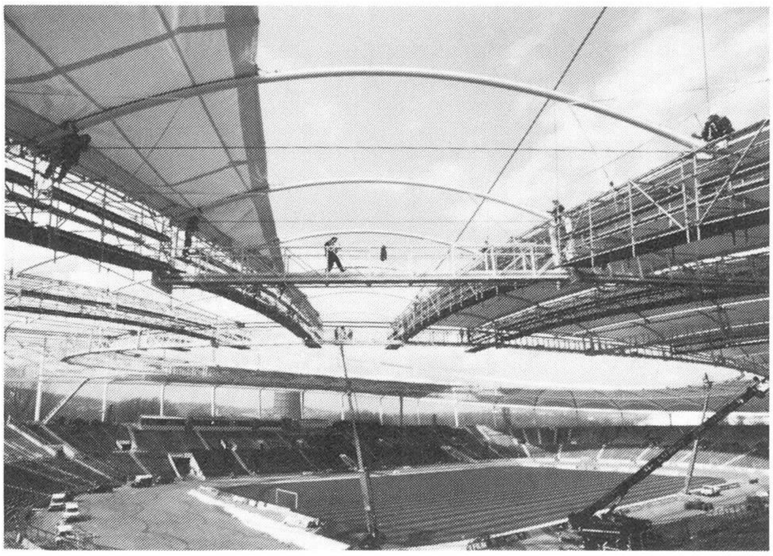


Fig. 16 Installation of membrane panels

Within an extremely short period for design and construction, an elegant and filigree roof structure has been created, providing the adequate light and cheerful ambiente for sportive events (fig. 17).

Fig. 17 Interior view of the completed roof

REFERENCES

1. Goldsmith, M.:Buildings and Concepts, Rizzoli, New York, 1987

2. Schlaich, J.: Über die Grenzen des Baubaren, DAI-Zeitschrift 8/9, 1978

3. Schlaich, J.: Les Structures Légères. Annales de l'Institut Technique du Bâtiment et des Travaux Publiques, Décembre 1989

4. Sobek, W.: Auf pneumatisch gestützten Schalungen hergestellte Betonschalen. Dissertation Universität Stuttgart, 1987
Schlaich, J., Sobek, W.: Suitable shell shapes. Concrete International Jan. 1986
Schlaich, J.: Do concrete shells have a future? IASS-Bulletin Nr. 89, June 1986

5. Nooshin, H., Makowski, Z.S. editors: International Journal of Space Structures. Multi-Science Publishing Co. Ltd.

6. Leonhardt, F., Schlaich, J. et al: Vorgespannte Seilkonstruktionen - Das Olympiadach in München. Der Stahlbau Sept., Oct., Dec. 1972, Febr., March, April, June 1973

7. Schlaich, J., Mayr, G., Weber, P., Jasch, E.: Der Seilnetzkühlturm Schmehausen. Der Bauingenieur, Nov. 1976

8. Bergermann, R., Schlaich, J.: Cable-Membrane Roof for the Arena in Zaragossa, Spain Structural Engineering International 4/92

9. Trofimov, V.I., Eremejev, P.G.: Thin Sheet Metal (Membrane) Roof Structures. IASS - Madrid 1989

10. Schlaich, J.: Direct Use of Solar Energy. Structural Engineering International 2/94

11. Schlaich, J., Schober, H.: Glass-covered Lightweight Spatial Structures. IASS-ASCE International Symposium 1994, Atlanta, Georgia, USA

Technical/Economic Evaluation of Cables for Long-Span Structures

Réflexion technico-économique sur les câbles de structures de grande portée

Technisch-ökonomische Betrachtung von Seilen für weitgespannte Tragwerke

Michael J. COOK
Group Director
Buro Happold
Bath, UK

Edmund HAPPOLD
Chairman
Buro Happold
Bath, UK

W. Ian LIDDELL
Manager
Buro Happold
Bath, UK

John W.S. HEARLE
Chairman
TTI Ltd
London, UK

Roger E. HOBBS
Director
TTI Ltd
London, UK

Michael R. PARSEY
Managing Director
TTI Ltd
London, UK

SUMMARY

Large 'tents' and 'airhouses', which have achieved the status of permanent structures, require ropes or cables to support loads. Collaboration between materials scientists and engineers is essential for advance. Compatibility with cladding membranes and other factors leads to a list of engineering requirements. Modern ropes exist in a large number of materials, particularly high-performance fibres, and constructions, whose quasi-static and long-term properties must be selected to meet the needs.

RÉSUMÉ

Les câbles ou faisceaux de câble sont indispensables au transfert des charges des "tentes" et des "bulles" de construction permanente. Les progrès dépendent d'une étroite collaboration entre ingénieurs et spécialistes de la science des matériaux. Les exigences techniques requises découlent de la compatibilité des câbles avec les membranes de couverture, tout comme d'autres facteurs. Les câbles modernes sont réalisés dans une grande variété de matériaux, surtout les fibres ultra performantes. Le type de câble doit être choisi en fonction des propriétés quasi statiques et de longue durée requises.

ZUSAMMENFASSUNG

'Zelte' und Traglufthallen, die den Rang permanenter Bauten erreicht haben, benötigen Seile oder Drahtbündel zum Lastabtrag. Ein Fortschritt kann nur in enger Zusammenarbeit von Materialwissenschaftlern und Ingenieuren erzielt werden. Die Verträglichkeit mit Verkleidungsmembranen und anderen Faktoren ergeben eine Liste entwurfstechnischer Anforderungen. Moderne Seile gibt es in vielen unterschiedlichen Werkstoffen - insbesondere hochleistungsfähigen Fasern- und Aufbauarten, deren quasi-statische und Langzeit-Eigenschaften den Bedürfnissen entsprechend ausgewählt werden müssen.

1. INTRODUCTION

Since civilisation began, man's built environment has been defined more by the materials available than by his imagination. Where there was nothing but ice, man developed the igloo; where there was mud, man produced bricks; where there were fibres and skins, man developed the "tent". In tents, the interior volume is enclosed by a membrane, usually a textile fabric, whose tension carries its own weight and any applied loads (due to snow, wind, etc). The membrane is held up in the air, supported by rigid elements in compression or, less efficiently, bending, or, in the more recent development of "airhouses", by air pressure.

In the 20th century, polymer engineering led to high strength fibres, weather resistant plastic coatings, and high strength films with life-time performances well in excess of earlier natural products. This has allowed tension structures to develop rapidly in recent years [1], achieving permanent structure status in several countries, including the USA. However, the materials are not yet perfect, and nor are they necessarily being used in the most effective way. Engineers have found design and manufacturing techniques which allow currently available materials to be used safely and reasonably economically. But, as materials science advances, the structural engineer must also move and influence it into directions where there is a perceived need or exploitable opportunity. Collaboration is vital. In this paper, we concentrate on the role of ropes and cables, though this cannot be separated from the developments in membranes.

In principle, in simple tents, such as the classical bell-tent and modern frame tents, and in airhouses, ropes are not needed, except perhaps as guy-lines to hold out walls or to give added stability: the membrane itself fills all the mechanical needs. In other forms of tent, masts have to be stayed by lines. However, in large structures, there is a need to gather the surface tensions in the membrane into linear elements, such as cables, ropes, and webbings, before transferring the tensions to the supporting medium. This requirement has been intensified by the introduction of new light-weight films, which have no fabric reinforcement. To control strain, they can only be used in conjunction with linear elements of stiffer materials acting in tension, compression or bending.

A total structure will consist of a number of fields bounded by the linear elements (cables, beams etc). Each field may be composed of a number of fabric panels sewn or welded together.

In addition to their principal structural functions, linear tension elements have many other roles in places of assembly and long-span structures. These range from demanding purposes, such as the support of movable equipment, to minor applications, such as cords used to pull curtains.

2. REQUIREMENTS FOR LINEAR STRUCTURAL ELEMENTS

The scale of structures using stressed membrane skins to carry the load is limited by the strength of available membranes. To control deflections in a tension structure the membranes must be prestressed. This can be achieved with flat fabric fields, but greater stiffness can be gained by introducing anticlastic curvature (saddle shapes) into the surface to produce immediate geometric stiffness under applied loads. For large structures, the size of a membrane field is usually limited by the strength of the fabric and the degree of curvature built in. To achieve sufficiently high curvatures, and so limit stresses, the surface of a large structure must be broken down into fields each with their own anticlastic curvature, separated by linear tension elements (cables, ropes or belts), which collect load from adjacent fields.

Alternatively, the fabric must be supported by an independent network of cables, which carries the load out of the cladding membrane directly. Elements used as linear restraints at the junctions of membrane fields (scallops, ridges and valleys) are more dependent on compatibility with the membrane performance than complete load carrying networks.

The prime criteria for selection are: (1) Facility to transfer load from membrane to element, allowing for parallel and perpendicular force components; (2) If not free to slide, element strain compatible with membrane strain; (3) Elements flexible enough to follow curves in three dimensions; (4) Elements with sufficient in-line stiffness to ensure load transfer; (5) Consistent, predictable in-line stiffness, unchanging with time, preferably linear; (6) Durability to exposure conditions; (7) Easy termination and economical transfer of tension to other components; (8) Cost, including site handling, compatible with total structural cost; (9) Required strength; (10) Easy transportation; (11) For networks, easy linking at crossing nodes.

In order to understand the nature of the forces on and deformation of typical linear elements in a tension structure, we consider two examples.

(1) A typical structure, spanning 40m and 60m long, consists of two central fields and two end fields. Maximum fabric stresses in the central fields (due to applied loads) will be about 25kN/m, creating boundary cable tension of 200kN and ridge cable tension of 500kN. A common difficulty arises at the field boundaries due to the anisotropy of the material. The highest stress levels occur in the warp or weft yarns, which are oriented in the direction of principal curvatures. Since the yarns are rarely orthogonal to the edges of fields there are both perpendicular *and* tangential forces on the linear elements at the boundary. Usually the tangential forces are transferred to the linear elements through sewn-on edge belts or bolted metal edge clamps, thus transforming the shear forces into rope tensions. The normal forces are directly carried by sideways pressures on the cables or ropes, with the residual force balanced by components of tension in a rope following a curved path. Smooth transfer of these forces is difficult to achieve, especially where fabric, belt and cable each have their own nonlinear elasticity, and where connections are imperfect. This situation would be simplified if one element were suitable for carrying all the load from the fabric regardless of weave orientation.

(2) In a typical airhouse, the connection between fabric and ground perimeter demonstrates the problems created by incompatible straining. The fabric strains under in-plane longitudinal stresses, but the ground will not strain. This results in shear distortion of the membrane towards the ground. An airhouse spanning more than 40m will require cable restraint to limit in-plane fabric stresses, and define distinct areas of membrane to transfer load into the network. The form which they take up must be predicted, so that the fabric can be patterned to match it. Hence predictability of strain is a very important feature. Compatibility of strain becomes an issue when the restraining cables are not free to move across the membrane surfaces. This effect will occur where two-directional cable nets are used to "lock-in" to the deformed membrane surface.

Design loads may be over 500kN for cables in large roofs, but actual loads are highly variable. Usually the prestress levels will be only 10 to 20% of the ultimate design load, which is predicted to occur only once in the building's lifetime. However, the frequency and magnitude of load vary with the patterns of wind and snow forces. Wind loads, having a significant dynamic component, will create short term peaks of stress measured in seconds, whilst snow loads may have durations of several days. Whether wind or snow is the critical load case will clearly depend on the location of the structure. In addition to the dominant in-line tension, any curvature leads to tensile and compressive bending strains and other forces: these are greater at detail points such as eyelets.

For membranes where tear propagation is the dominant failure mode, design stresses must be sufficiently limited to ensure that relatively small tears remain stable. Factors of safety between 8 and 6 on tensile strength are commonplace. For linear elements, factors of between 2 and 3, on ultimate breaking strength, are normally acceptable when used as an integral part of a permanent structure, whilst a factor of 5 would be used for stand-alone lifting equipment where the chance of overload is greater and the handling might cause damage.

In building, construction at least cost is a vital consideration. Investment decisions normally underemphasise "least lifetime cost" in favour of "least initial cost". Membranes currently used are PVC coated polyesters with lifespans of 10 to 15 years and PTFE or silicone coated glass fibre with lifespans of 25 to 35 years. The additional linear elements must have at least a comparable lifespan. If a replacement membrane is planned, longer-life elements might be appropriate, but there is a strong possibility that new materials with better characteristics and different properties will become available, and so make complete replacement more logical. Where the membrane merely clads a cable network, and if strain compatibility is not important (as with a polymer panel cladding on a cable net), independent replacement of the cladding is highly feasible. Therefore there may be sound economic justification for selecting ropes or cables that outlive the membrane.

3. MODERN FIBRE ROPES

From ancient times until 150 years ago, ropes were made from natural fibres; then steel wire ropes displaced fibre ropes from serious uses in structural engineering. The situation changed again 50 years ago with the invention of two generations of strong man-made fibres, the first (nylon, polyester etc) more extensible, and the second (carbon, aramid etc) with high modulus. Since these were continuous filaments, high twist was not essential, and ropemakers have developed a range of new constructions. The choice is enormous: it is estimated that, for any given purpose, 500,000 combinations could be considered. Architects need specialist advice from experts who know the field and have facilities for computing performance. In this paper, we can only give some general information, more related to the main structural need formulated above than to the ancillary uses also mentioned in the introduction.

Table 1 lists materials currently available for ropes, with rough indications of strength and stiffness relative to weight and price; more detailed plots have been given elsewhere [2]. Except for low cost for minor uses, which favours polypropylene, and ancillary uses needing high extensibility and energy absorption, which favour nylon, the choice is between polyester, at lower cost, or the high-modulus high-tenacity (HM-HT) fibres at lower weight. The brittle (in bending) fibres, carbon and glass, are only suitable for use when bonded by a matrix as solid pultruded rods. Such rods may also be used with other fibres, in order to reduce the problems of axial compression.

---

natural fibres with limited performance
hemp, sisal, cotton etc [1/0.1/2/2]

man-made fibres adequate for common uses
regular melt-spun polyethylene, polypropylene [2/0.2/5/1]

intermediate performance fibres
polyamide (nylons) [3/0.5/4/1]; polyester [3/1/4/2]
glass [5/1.5/10/10]

advanced [HM-HT] fibres from liquid crystals
aramid (®Kevlar and others); LCAP (®Vectran); PBO [7/4/1/1]

other high modulus, high strength (HM-HT) fibres
high-modulus polyethylene (HMPE) (®Spectra, ®Dyneema) [10/7/1/1.5];
carbon [8/10/0.4/2]

steel wire [1/1.5/1/5]

Table 1 Materials with approximate ratings for:
[strength÷wt/stiffness÷wt/strength÷price/stiffness÷price]

Ropes and cables may be categorised in five broad groups. *Twisted* constructions comprise the familiar three and four strand ropes. High twist was necessary to hold short fibres together, but the tensile efficiency is low and torque balance is poor. Ease of splicing on site leads to their use as guy lines for tents and marquees. *Plaited* and *braided* constructions, interwoven from equal numbers of clockwise or counterclockwise strands, may be either solid (e.g. 8-strand) or circular. Single circular braids have hollow centres. Double braids are made with a second single braid over an inner braid. *Stranded (wire rope)* constructions are similar to twisted constructions, but have lower twist to improve tensile performance. The strands (or wires) are arranged in concentric rings, and may be designed to minimise torque generation. They should be used when the application requires the rope to be worked over sheaves (pulleys) at high loads. Whereas jacketing is merely desirable in some other constructions to provide wear and light resistance, it is essential in stranded rope to hold the construction together. *Parallel* constructions comprise parallel yarns, strands or sub-ropes held together by an external cover, which may be extruded polymer or a braided jacket. They have high tensile efficiency, but should not be worked over sheaves at high loads. Some types need special terminations. *Pultrusions*, which have excellent strength and fatigue resistance, but are costly to make and difficult to terminate, comprise an assembly of fibres in a rigid or flexible resin. Other linear elements include flat woven or braided webbings, which may be easier to join to membranes or to use as slings. Round slings, consisting of parallel yarns in a textile casing, offer much higher specific strength for lifting applications. Since many variants of the above types, including blended forms, are possible, it is easy to see why the choice is so large. Furthermore, terminations may cause as many problems as the ropes themselves; the available types are variants of grips, splices, resin sockets, and barrel-and-spike.

Computer programs have been developed to predict the load-elongation and torque-twist properties of ropes [3]. These can be used to design ropes with properties which match the membrane properties, and meet the other requirements specified above. By way of example, Table 2 gives properties of a few candidate ropes for the strength requirement of 500kN.

FIBRE	ROPE-TYPE	WEIGHT kg/100m	STIFFNESS kN/1% extn	PRICE £/100m	NOTES
manila	three-strand	500	40	600	GRADE 1
steel	galvanised	300	360	700	6×36IWRC
	plastic coated	"	"	1000	stranded
	stainless	"	"	2500	
poly-propylene	8-strand X-plait	200	20	500	worked & rested
polyester	double braid	220	30	1500	ditto
nylon	double braid	150	15	1200	ditto
polyester	parallel strand	150	40	1000	ditto
carbon	pultrusion	70	600	10,000	1x6 helical
aramid	parallel yarn	50	180	2500	w'rkd & r'std Kevlar 29
HMPE	stranded	40	80	1600	stiffness 0.1Hz

Table 2 Rope properties at 500kN strength (approximate values only), excluding terminations.

In addition to the quasi-static properties, long-term performance is vital. Failure often occurs at terminations. Of the six "fatigue" mechanisms identified in ropes [4], the following are likely to be of most importance in building structures. Creep rupture, determined by "average peak load", is a major weakness of HMPE, and, to a lesser extent, nylon, and is always the default mechanism in the absence of others. Internal abrasion is most serious in nylon. Axial compression fatigue is a potential killer in HM-HT ropes, and, to a much lesser extent in polyester and nylon. Even when the ropes themselves are always under positive tension, axial compression can occur (and has caused embarrassing failures in marine uses), especially with low minimum loads, due to twisting of ropes that are not torque-balanced, differential response of components, or bending. Computer programs have been developed to model these fatigue responses [5] (and hysteresis heating), though the theory is less certain than for the quasi-static behaviour and input data are not always available.

4. THE COMPETITIVE POSITION

At present engineers are generally selecting steel ropes and strands wherever high strength, high stiffness and durability is needed. Steel dominates the field for structural rigging, "cable" nets and boundary scallops. However, steel is not ideal for the following reasons: steel ropes have a nonlinear stress-strain relation at low (working) stresses; it is difficult to transfer forces tangentially into a steel cable; steel cables cannot be sewn or welded onto coated fabrics and require special, costly additional clamps; steel ropes can damage and chafe the fabrics used; normal oils present in cables can harm fabric coatings; for transport, steel cables need to be removed from the fabric panel to allow it to be packaged small and to avoid damage; steel cables and their terminations must have corrosion protection unless stainless steel is used. Against this, steel cables have a cost advantage over any man-made fibres that might be able to perform better. Advanced fibres, such as Kevlar, offer potential advantages of high and reliable stiffness and light weight. They are attractive to the structural engineer for these reasons, but their use can rarely be justified. Their most notable use is as stiffeners to sails for racing yachts, where relatively small improvements in performance and control win the race. In building structures, properties are rarely that critical and price dominates. However, with current advances in the technology, the cost advantage of steel may not continue to be the case for long, especially when the indirect cost savings as well as the direct cost are taken into account.

The clearest advantage of the most recently developed fibre ropes over steel cables is low weight-to-strength ratio. This can be useful for mobile structures to give ease of handling, but becomes especially relevant for extremely large air-supported structures covering many hectares. For a structure spanning 500 meters, the weight of restraining steel cables is likely to be between 8 and 15kg/m^2. With normal inflation pressure of 20 to 30 g/m^2, such a weight is very significant; therefore operating pressures would have to be increased. However, recent research shows that the sensitivity of such structures to aerodynamic instability, as well as buffeting from the wind, depends on the net over-pressure. Using an aramid rope of the same strength, the weight of the restraining cables would reduce to between 2 and 4kg/m^2. In addition to the stability advantage, the saving in inflation equipment, easing of access route pressurisation problems, and reduction in required strength of cladding membrane needed to withstand the pressure could be sufficient to pay for the additional capital cost of the cables. Design studies would be needed to determine the long-term durability of such ropes, particularly in terms of the need to avoid axial compression fatigue.

The above example is based on a technical solution in terms of rope strength; but there is also a strong case for considering an intermediate solution using polyester ropes. The weight would be greater, thus reducing some of the benefits, but the rope cost would be lower and current indications are that polyester is more rugged in use. Rope experts have more confidence in its resistance to fatigue. Another consideration is the axial stiffness of the rope, which needs to be considered in relation to compatibility with membrane properties.

5. CONCLUSION

It has taken some forty years to develop mebrane materials which can adequately carry loads and which have a decent life. Simultaneously, it has been necessary to develop analytical tools for engineers who design with these materials. Now that both are available, architects world-wide are designing tents. In large structures, the membranes are bordered and prestressed by linear tension elements, which not only gather high forces but act as tear stoppers. These elements have their own problems and, again, the role of new materials must be addressed. As well as the intrinsic properties of ropes, cables and webbings, questions of connections, including clamps and terminations, come up.

Force is not the principal issue. It is more important to consider compatibility - of strain and other aspects of physical behaviour, performance in fire, lifetime, and so on. Above all, economy is a major factor: a 10% increase in load-carrying capacity means little, but a 10% reduction in cost is a revolution. This paper encourages architects and engineers to address the problems of linear tension elements with an open mind, taking account of the availability of new materials and structures and the many complex and interacting features of performance and cost. Since the next major building type, once the visual conservatism and reluctance to think of buildings as machines is overcome, is the airhouse, which is the most economic climate moderator one can think of, the opportunities for new linear tension elements will be even more important.

REFERENCES

1. HAPPOLD E. and LIDDELL W.I., Tension Structures, 14th Congress of IABSE, New Delhi, 1992, 137-150.

2. HEARLE J.W.S., BURGOYNE C.J. and HOBBS R.E., Fatigue and Length Effects in Fibre Ropes. Workshop, El Paular, Madrid 1992. IABSE, Zurich, **66**, 307-318, 1993.

3. LEECH C.M., HEARLE J.W.S., OVERINGTON M.S. AND BANFIELD S.J., Modelling Tension and Torque Properties of Fibre Ropes and Splices, Proc. 3rd Int. Symp. Offshore and Polar Engg., Vol. 2, 370-376, 1993.

4. PARSEY M.R., BANFIELD S.J. and HEARLE J.W.S., Life of Polymeric Fibres in the Marine Environment. 2nd Int. Conf. on Polymers in a Marine Environment, Inst. Marine Engineers, London, 1987.

5. HEARLE J.W.S., PARSEY M.R., OVERINGTON M.S. and BANFIELD S.J., Modelling the Long-Term Fatigue Performance of Fibre Ropes. Proc. 3rd Int. Symp. Offshore and Polar Engg., Vol. 2, 377-383, 1993.

The Valeria of Roman Amphitheatres: The Colosseum

Les vélariums des amphithéâtres romains: le Colisée

Die Sonnensegel römischer Amphitheater

Giorgio CROCI
Prof.
Univ. "La Sapienza"
Rome, Italy

Piero D'ASDIA
Assoc. Prof.
Univ. "La Sapienza"
Rome, Italy

Dina D'AYALA
Researcher
Univ. "La Sapienza"
Rome, Italy

Piero MEOGROSSI
Architect
Soprintendenza Archeologica
Rome, Italy

SUMMARY

The paper reviews the current archaeological and structural assessments on the layout of the velarium. The aim is to suggest a methodology to understand the architectonic and structural choices, validating the various hypotheses through the structural analysis. The results obtained in this first step of the research will be used to narrow the field of the possibilities and concentrate further studies of better accuracy, on the models that look more reliable.

RÉSUMÉ

Les auteurs donnent une vue d'ensemble des évaluations archéologiques et structurales sur les tracés des vélaires de l'antiquité. Il s'agit de proposer une méthode pour comprendre les choix architecturaux et structuraux, en vérifiant les diverses hypothèses par des calculs statiques. La première phase de recherche fournit des résultats qui servent d'une part à réduire les variantes possibles et d'autre part à préciser les études supplémentaires, pour que les modèles présentent une meilleure fiabilité.

ZUSAMMENFASSUNG

Der Beitrag gibt einen Ueberblick über die gegenwärtige archäologische und bauliche Rekonstruktion von Sonnensegeln in der Antike. Ziel ist die Erarbeitung einer Methodik zum Verständnis der Entwurfskonzepte, für die unterschiedliche Hypothesen durch statische Berechnungen überprüft werden. Die im ersten Schritt erhaltenen Forschungsergebnisse dienen der Eingrenzung der Möglichkeiten und der Konzentration vertiefender Studien auf vielversprechendere Modelle.

1. The shape of the velarium: topographical and archaeological hypotheses

The architecture of the *Coliseum in specie ovi*, as it is called by Cassiodoro, was highly influenced by the existence on its building site of the older *stagna Neronis*, an artificial lake, in the *Domus Aurea*, placed at the centre of the city in the valley among the Celium, Palatinum, Velia and Oppium Hills.
The geometry of the plan is an oval with 4 centres of curvatures organised in 80 arch-bays repeated three times in the vertical and horizontal planes and connected with radial walls which bear the *cavea's* seats.
The two principal axes, of which the major one is aligned along the *Via Sacra,* measure 188 m and 156 m. Only the outer 50m where actually built over the ground with a degrading section 50 m tall at the top of the actic level of the outer wall. So that the area to be covered by the *velarium colore coeli* (Pliny, 9, 1, 24) measures approximately 23000 m^2.
The centre of the *Coliseum* is based on a Pythagorean triangle superimposed to the two axes and the centre of the valley whose measure is recognised by a specific topographical relation which organise the over all shape of the monument by an urban axis (fig. 1). This axis links the *ara maxima Herculis,* to the *Curiae Veteres,* two important knots of the *Roma quadrata* grid narrated by Tacitus, and it connects a number of significant centres of monuments, from different roman periods, as the *Apollum templum* by Augustus, the Flavi's labyrinth, the Adrian's *Adonea,* Constantinus's Arch, the Nero's octagonal room. This axis, called as the *urbis axis*, measures 19° N-NE (corresponding astronomically to the course of the sun of the XXI April, day assumed to be the Rome's birthday) and it is orthogonal with the ancient *via Lata.*
Such important considerations support the in situ exam over the moulding and the exam of seven architectonic pieces erratically dislocated on the archaeological site. They allow to formulate the hypothesis that these blocks of the same shape and two different sets of dimensions (average measures being 950x600x500 mm and 720x440x500 mm) were utilised as a further prop of the poles which hold the cables. The two different dimensions can suggest the idea that the poles had different functions. In particular it seems acceptable that the bays of the velarium (slack as they look in the Pompeii's fresco), were of the same extent as the lower structure, so to have a tent, in the shape of a Latin sail, covering each arch bay. The brackets on the outer side of the acticum level being three for each bay (fig. 2) there were three poles for each sail. We therefore think that the central pole was the one to which the principal cables, called *rudentes,* 80, one for each bay, were connected, and the only one working during the phase of mounting. Once the velarium had taken shape and place the forces were redistributed on the other two poles by means of secondary ropes.
The length of the poles is estimated to be about 12 m of which 7 to 7.5m above the moulding of travertine showing for each bay three rectangular holes of 400x500 mm in which the poles were slipped through and, resting on travertine brackets spanned 2.25 m., were fixed to the wall by a planking.
The geometry and structure of the velarium therefore seem to follow the same logic as the geometry and structure of the travertine construction below, conveying, through the repartition of the forces among the three poles, the flux of stresses toward the arch-pillars system.
To lay-out the cables net, mounting it and lifting it in the final position took approximately four days and the work of 300 men of the imperial fleet.

2. The shape of the velarium: structural hypotheses

The structural elements which can be still seen today on the monument and the historical and archaeological documents, presented in the previous paragraph, lead to convincing hypotheses about the construction of the velarium and its assemblage and rising in place; however they are not sufficient to define completely the structure and the building technology used. The structural analysis can be usefully applied to enlighten these aspects and therefore to reduce the uncertainty associated to different lay-outs , pointing out among these, only the ones which fulfil the stress requirements according to materials and technology available at the time.
Confronting the different configurations proposed by various authors, it seems possible to define a number of points which form a common core for further discussion:
a) the main cables present a radial lay-out, one for each bay of the outer travertine arched structure, and they were probably made of hemp, this being the material commonly used by sailors. Most likely diameters were

in the range from 40 to 60 mm (the cross section being measured when in tension) and each cable was tensed by a capstan controlled by two to three men. This hypothesis, better than the one with 240 cables, allows to have homogeneous bays and even distribution of loads and stresses (if not for the slight variation connected with the change in curvature along the oval) therefore avoiding asymmetries not liked by Romans and making the structures of the velarium congruent with the lower structure.
b) 80 capstans were placed above the roof of the giant portico in front of the central pole of each bay, which held the pulley of the principal cable; during the mounting phase, the central pole bore all the load of one bay, and it was probably sustained by two other poles put in a shoring; once the cables were tensed and the velarium put in place, before removing the shoring, the load on the principal cable was shared onto the two lateral poles by means of secondary cables (see fig. 3) tensed by winches set into blocks of travertine at purpose shaped. The presence of the boundary-stones, with four holes, placed at the ground level outside the monument, has been interpreted by some authors as an anchorage for the cables. It seems unlikely for a number of reasons: a huge increase in the amount of materials, a difficulty in the coordination of the operations due to lack of visual connection, the hindrance that these outer cables could cause during the flux and defluxion of the audience.
c) the canvases were directly tied to the principal cables in roughly rectangular pieces of width slightly longer than the distance between the two adjacent cables and length of 3 to 4 m., so to have elements of workable dimensions (20 to 30 m^2 was the average dimension of Latin sails) and to allow independent movements of the different pieces under the action of the wind, so to reduce the global pressure;
d) the inner ring to which are connected the low points of the principal cables, repeats, with smaller dimensions, the shape of the oval plan of the structure, sharing the same centres of curvature. The four arcs in which the oval is divided by the change of curvature have the same length , but different angles, the one with smaller radius being 1.84 rad, the other 1.30 rad. Considering the thrust of the cables (20 on each arc equally spanned) like a pression uniformly distributed over these arcs, one obtains an hoop thrust which differs with respect to the mean value (calculated for even angles of $\pi/2$ rad) for a $\pm$ 17% . As it is no possible in such a structure to have variable thrust this means that the mean value is reached by means of a change in sag in the cables: for the cables in the area with smaller radius of curvature the sag will be reduced of 17% and on the other it will be increased of 17% so to have an opposite increase and reduction of the hoop thrust in each area. The inner ring will show a saddle shape similar to many roofs of modern stadiums.
Starting from these points of common agreement a number of questions remains open:
1- the actual configuration of the principal cables, the covered surface, the limitations imposed by the visibility line and the possibility of shadowing the podium related to the position of the sun;
2- the connection of the canvases to the cables: the possibility of moving them along the cable means a further load due to the secondary mesh of rope;
3- the configuration and realisation of the inner ring made of hemp or iron rods connected by small rings;
4- the actual dimension of the poles, the need of struts during the mounting phase and the compatibility of the stresses;
5- the action of wind and rain, at least in their mild summer manifestations, and their compatibility with the global lay-out of the velarium and the dimension of its structural element.

3. Structural analysis

To work out the structural analysis we need to define the mechanical features of the material and the distribution and entity of actions. The nominal values mentioned below have been assumed from old codes or from materials which are today similar to the old ones:

	cross sect.	weight	failure stress	elastic modulus
hemp cables	2000 mm^2	30 N/m	30 - 40 N/mm^2	300 - 400 N/mm^2
oak wood poles	0.12-0.20 m^2	1000 -2400N/m	40 - 60 N/mm^2	12000 N/mm^2
iron bars	900-2500mm^2	70-200N/m	200 N/mm^2	210000 N/mm^2
linen clothes	1. m^2	10 N.		

To verify the first point mentioned above, the lower bound for the sag, not to interfere with the visibility line of the upper seats, is 25 m. (fig. 4). Although some of the archaeological reconstructions show the velarium as practically flat, the analysis shows that the upper bound can be not less than 7 - 11 m for cables spanning from 40 to 60 m. The minimum of the last two measures, 40 m., is conditioned by the extension and duration of the shadowing over the lower seats, while the maximum span is bounded by the increase in load with relation to the strength of the cable. The analysis proves that for the longer span tested (61 m) is not possible to have the overload of two additional cables to movement the canvases and therefore in this way is implicitly solved the second point of the previous paragraph (fig. 5).
The geometry and cross section of the inner ring is related to the length of the cable on one side and to the horizontal component of the trust (T_h) on the other, assuming that the sag in the cables along the ring will be variable as already discussed at the point d) of the previous paragraph. The hoop stress in the ring will be:

$N = \frac{40T_h}{\pi} = 12,7T_h$, and varies , for a sag of 9 m, from 171.5 KN to 381 KN for a span ranging from 40 to 60 m, while for a sag of 15 m the corresponding values are 127 KN to 222.5 KN. For the span of 61 m the case of reduced load due to unmovable canvases has also been considered, producing a hoop force of 222.5 KN and 159 KN, for sags of 9m and 15m respectively. Taking into account the variability of the distance between cables at the lower points as a function of their span, an average weight of the inner ring on each cable of 500 N can be assumed.
According to the span and sag ranges considered, the cable reaches the pole forming an angle with the horizontal variable between 18° and 29°, while dimensional analysis implies that the position of the capstan, placed as close as possible to the pole, will give to the cable coming from the pulley an angle with the vertical line of approximately 20°. This means that the resultant of the thrust will form with the axis of the pole an angle of 40° to 46°. Therefore the horizontal and vertical components will be respectively in the range of 1.18 T to 1.30T and 1.41T to 1.30 T, giving place to the following generalised stresses in the poles:

$M = O \cdot 6.6 + V \cdot 0.3, \quad S = O, \quad N = V$ .The stresses for the most severe case vary from 11.3 to 14.23 N/mm^2 for cross section of 500x400 mm, and from 24. to 31.6 N/mm^2 for cross section of 400x300 mm. Confronting this values with the ultimate strength value it can be seen that the pole is able to resist those loads with a safety factor variable between 3.6 and 1.3 depending on the cross section.

4. Conclusions

The results obtained represent the first iteration of a process of trial and error that should be replicated introducing the variation suggested by new information and their critical analysis. However they are useful to bound the field of the possible solutions. In particular:
- taking into account that the decrease in load (from 25 to 15 m sag) is of about 18 %, while the increase in thrust is far from the limit value, it can be thought that the more likely shapes were the ones with sag variable from 7 to 15 meters, depending on the span. It is worth to mention that, when observed from below at a distance of approximately 50 m from the lowest point, over an overall length of 180 m. entities of the sag from 10 to 15 m could have given the impression of a flat velarium.
- while for the smaller spans and bigger sag is still possible to think that the ring would be made of hemp ropes (requiring a section of approximately 15000 mm^{2}with a weight 265 N/m) , when the longer span are considered, and always for sags close to the upper limit value, the iron rods appear convenient, involving much smaller section and therefore a weight approximately estimated in 180 N/m.
-at least during the mounting phase, for the longer spans of the cables and smallest cross section of the poles a strut was required to lift up the cables net.

It is not possible here to discuss in due depth the reduction of the global pressure of the wind obtained by the cuts in the canvases, further studies and possibly tests on a model in the wind tunnel being necessary to achieve reliable knowledge of the phenomena involved. The effects, however, could have been partially absorbed by relatively small sub vertical stays tied at the lower levels of the seats. The stays would have

been slack o slightly tensed, so not to affect the configuration under dead loads only. The increment of vertical action due to the rain water or the wind flowing among the poles, these do not seem to invalidate the above results. For the rain, the load previously taken into account for the canvases, already included a waterproof treatment by linen oil and a water film 0.5 mm thick. On the other end the cuts in the canvases and their slope allow to assume that, even during a fairly intense rain, the increase on the effects will not be, on average, greater than 35% or 70% according to the different configurations which allow or not to roll up and down the canvases. This simple analysis, therefore, with such an increase significantly reducing the security factors mentioned above, suggests that very likely the span of the principal cables would not have exceeded the 50 to 55 m, and that for such configurations the option of fix canvases was the most likely.
As for the wind, with a simplified equivalent static analysis can be determined that un upper bound for the speed of the wind will have been 14-15 m/s. Such a speed would have caused an increase to the vertical force that in the depression area would equal the dead load effects, while in the pression area would reduced the safety factor to 1.5, which seems not reliable. For the same speed the horizontal component seems to be not of much influence the increase or decrease in the horizontal thrust being of a 10% of the values discussed above.

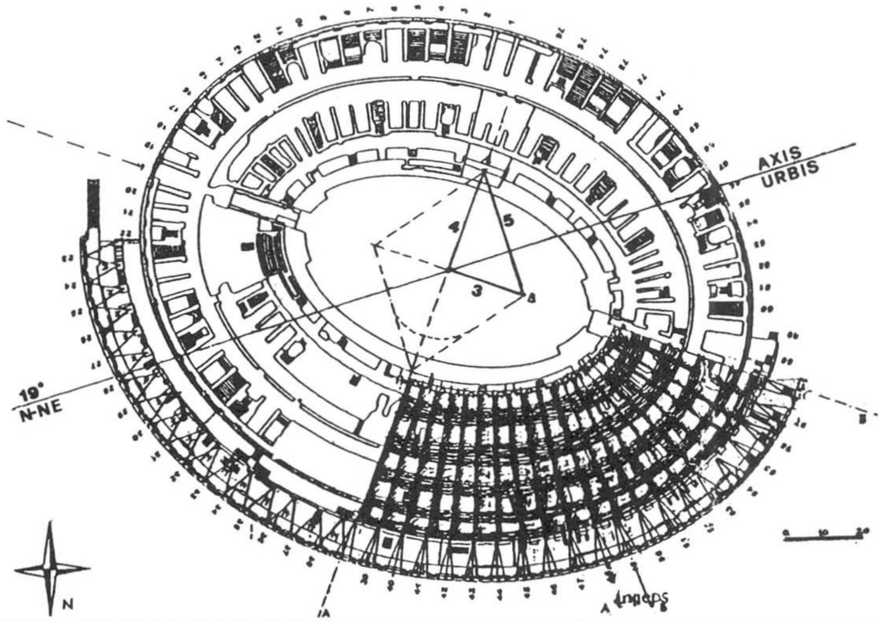


Fig. 1 : Plan of the Coliseum with the lay-out of the velarium

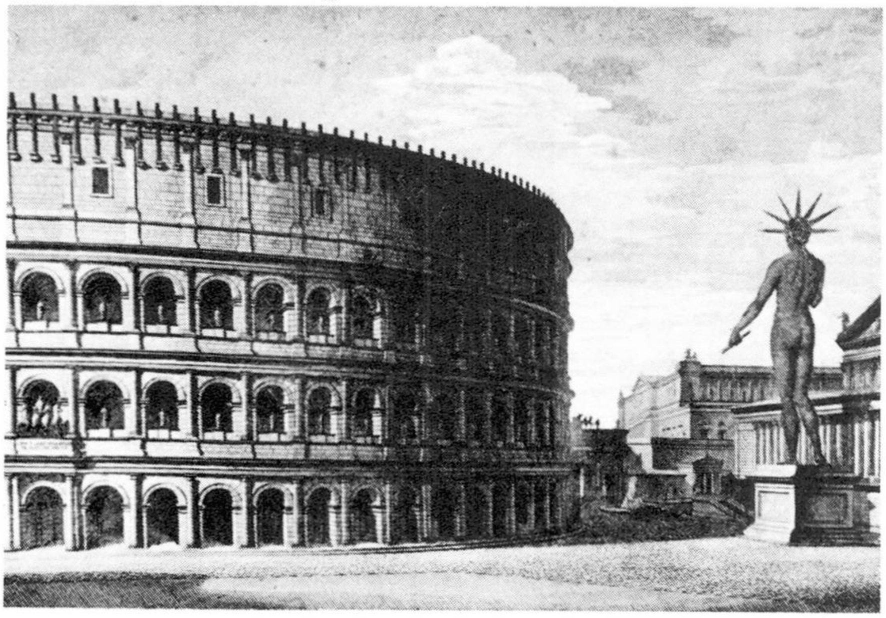


Fig. 2: The historical reconstruction of Canina (1860).

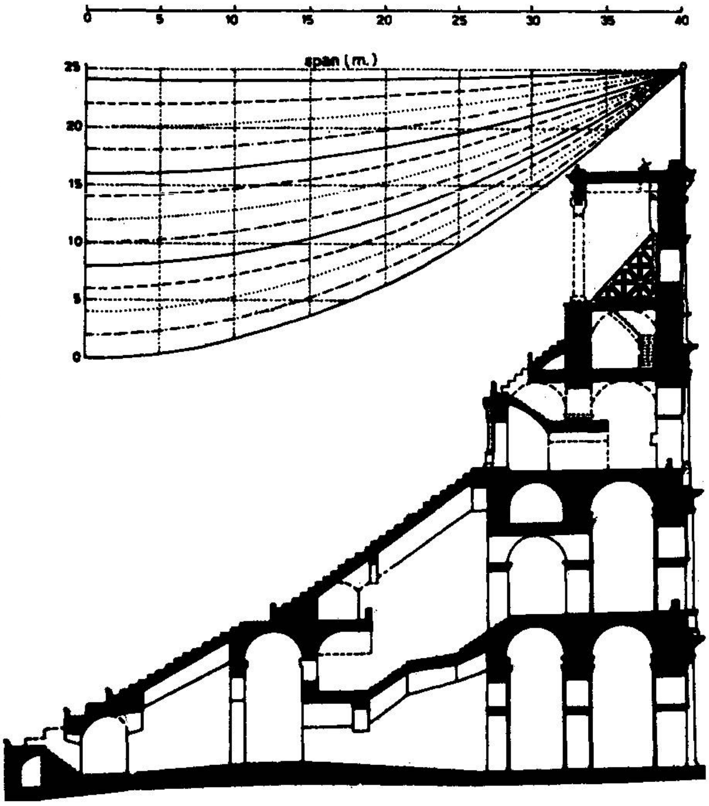
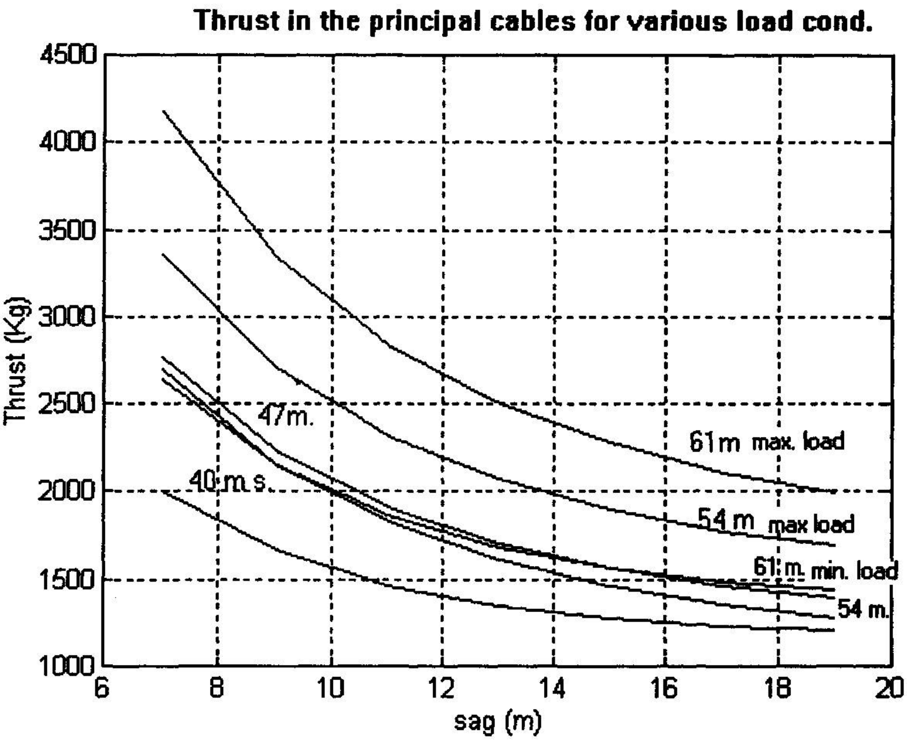


Fig. 3 and 4 : Possible shapes of the velarium



Design and Erection of Long-Span Hypar-Networks

Conception et montage de structures réticulées "Hypar" de grande portée

Entwurf und Montage weitgespannter Hypar-Netzwerke

Valdek KULBACH
Prof. Dr
Tallinn Technical Univ.
Tallinn, Estonia

Valdek Kulbach, born 1927, graduated from Tallinn Technical Univ. as a designer. He got his degree of candidate of sciences at Polytechnic Inst. in St. Petersburg and doctor's degree at Tallinn Technical Univ. Now he is working as professor in the field of steel structures.

SUMMARY

Two types of prestressed networks will be examined: network, encircled by two inclined plane arches and hypar-network within an elliptical contour. Determination of initial form and stress-strain state of the network will be discussed. As examples of cable-networks the acoustic screens for song festival tribunes will be described.

RÉSUMÉ

La communication traite de deux types de surfaces réticulées par câbles de précontrainte: un réseau tendu entre deux surfaces courbes inclinées et un réseau de forme paraboloïde hyperbolique à bordure elliptique. L'auteur discute de la détermination de la forme initiale exempte de contrainte et de l'état contrainte-déformation dans le réseau. Il fournit comme exemple les structures réticulées par câbles prévues pour les écrans acoustiques de tribunes lors de concerts.

ZUSAMMENFASSUNG

Zwei Arten vorgespannter Seilnetze werden betrachtet: Ein Netzwerktyp, der zwischen geneigten Bogenebenen aufgespannt wird, und ein Hypar-Netzwerk mit elliptischer Berandung. Der Beitrag diskutiert die Bestimmung der spannungslosen Grundform und des Spannungs-Dehnungszustand im Netzwerk. Als Beispiele werden die Seilnetzwerke der akustischen Schirme für Tribünen eines Schlagerfestivals beschrieben.

1. PRELIMINARY REMARKS

For many types of long-span structures the round or oval layout may be statically most suitable. On the other side, for many assembly and sports buildings that form of building may be also more functional as common rectangular one. For instance, when we have in the center of building a great sports field with a running track, the round oval plan enables to locate the spectators' seats according to conditions of the best visibility. The same is valid for a number of other assembly buildings. Especially suitable for that kind of conditions may be buildings with the roof, formed as a surface with negative Gaussian curvature or so-called saddle-formed roofs. In the number of saddle-formed roof structures very important position have the cable-networks. In the following we shall investigate the most suitable types of suspended roofs, formed as prestressed cable-networks inside a closed contour beam.

2. INITIAL FORM OF NETWORKS

2.1 General remarks

The form of a prestressed network will be determined by the conditions of equilibrium of its nodes. In most cases the network will be formed by two families of crossing cables. Theoretically we have a plurality of possibilities for choise of configuration of network surface. In every case the Gaussian curvature of the network surface has to have a negative value in all its nodes.The most suitable roof structures for building practice to our mind are orthogonal or approximately orthogonal networks. In the following we shall investigate mainly the orthogonal networks. Approximately orthogonal may be considered the networks, formed by free mutually sliding cables at the time of prestressing the network. In the first case the horizontal component of inner force is constant on its length. In the second case invariable on the length of the cable is its inner force itself. In the first case the contact forces between carrying and stretching cables will be vertical, in the second case they will be applied in the direction of the bisector of the angle between the neighbouring sections of cables.

For simplification of our problem we may image the network as a system of two continous families of virtual cables, uniformly distributed on the length of spacing between the nodes. In this case the cross section areas of cables will be characterizied by the effective thicknesses of the families of carrying and stretching cables. In this case the condition of equilibrium will be presented as differential equations. The other possibility is to analyse the network as a discrete system. For networks with a great number of cables the real network may be replaced by a virtual one, which consists of a reduced number of cables; the cross section areas of substituting cables must be correspondingly increased. The conditions of equilibrium of nodes in that case will be algeabric equations.

2.2 Hypar-networks

The continous surface of a prestressed orthogonal network may be described by the elliptical equation

$$G_0 \frac{\partial^2 z}{\partial x^2} + H_0 \frac{\partial^2 z}{\partial y^2} - 0 \tag{2.1}$$

where G_0 and H_0 - the prestressing forces of the carrying and stretching cables correspondingly, suited to the width unit of the network.

The equation (2.1) will be satisfied for hypar

$$z = f_x \frac{x^2}{a^2} - f_y \frac{y^2}{b^2} \tag{2.2}$$

Corresponding prestressing forces will be determined by uniformly distributed contact loads p_0

$$G_0 = \frac{p_0 a^2}{2 f_x} \tag{2.3}$$

$$H_0 = \frac{p_0 b^2}{2 f_y} \tag{2.4}$$

The most suitable layout for the hypar will be the ellipse (Fig.1)

$$\frac{x^2}{a^2} + \frac{y^2}{b^2} - 1 = 0 \tag{2.5}$$

where a, b - semi-axes of the ellipse,

f_x, f_y - the rises of curvature of carrying and stretching cables correspondingly.

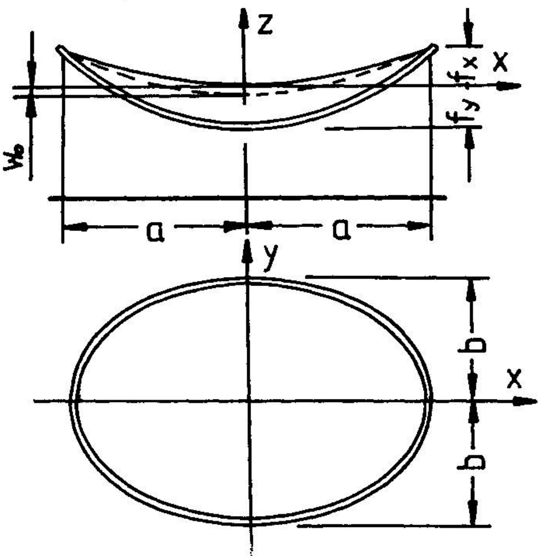


Fig.1 Hypar-network

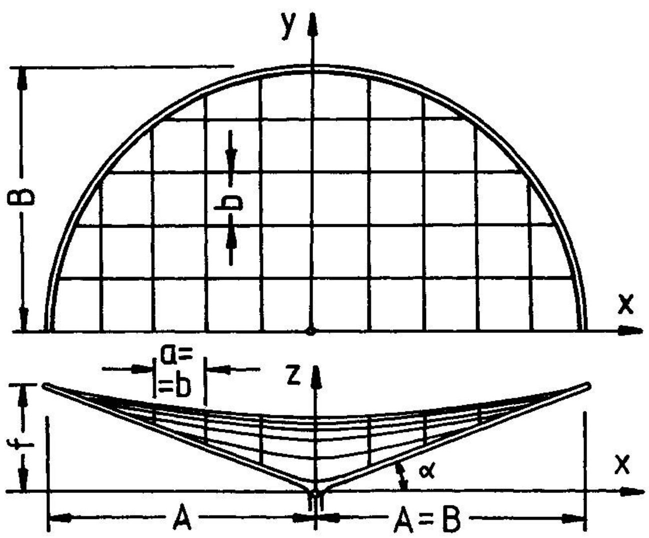


Fig.2 Orthogonal network with round contour

2.3 Orthogonale network with given form of contour

For the node i,k of an orthogonal weightless network the condition of equilibrium may be written in the form

$$G_{0i} \left(\frac{z_{i,k+1} - z_{i,k}}{a_{i,k}} + \frac{z_{i,k-1} - z_{i,k}}{a_{i,k-1}} \right) + H_{0k} \left(\frac{z_{i+1,k} - z_{i,k}}{b_{i,k}} + \frac{z_{i-1,k} - z_{i,k}}{b_{i,k-1}} \right) = 0 \tag{2.6}$$

where G_{0i} and H_{0k} - the prestressing forces for the i-th carrying and the k-th stretching cable correspondingly;

$a_{i,k}$; $b_{i,k}$ - the length projection of the k-th section of the i-th carrying cable and the i-th section of the k-th stretching cable correspondingly.

For the network with constant mesh dimensions the linear equation (2.6) may be written in the form

$$z_{i,k} = \frac{(z_{i,k-1} + z_{i,k+1}) + \lambda\,(z_{i-1,k} + z_{i+1,k})}{2\,(1 + \lambda)} \tag{2.7}$$

where $\lambda = \frac{H_{0k}a}{G_{0i}b}$.

The system (2.6) consists of equations, set up for all internal nodes of the network. The ordinates of contour nodes have to be given as the initial data of the problem. An orthogonal network, prestressed inside the contour, which consists of two inclined semicircle plane arches may be examined as an example (Fig.2).

3. CALCULATION OF DISPLACEMENTS AND INNER FORCES OF HYPAR-NETWORK

The stress-strain state of the network is to be described by means of conditions of equilibrium and equations of deformations compaibility [1]. For the case of action of vertical loads p we may write (Fig.1)

$$G\frac{\partial^2(z+w)}{\partial x^2} + H\frac{\partial^2(z+w)}{\partial y^2} = p \tag{3.1}$$

$$\frac{G-G_0}{E\,t_x}\left[1+\left(\frac{\partial z}{\partial x}\right)^2\right]^{\frac{3}{2}} = \frac{\partial u}{\partial x} + \frac{\partial w}{\partial x}\left(\frac{\partial z}{\partial x} + \frac{1}{2}\frac{\partial w}{\partial x}\right) \tag{3.2}$$

$$\frac{H-H_0}{E\,t_y}\left[1+\left(\frac{\partial z}{\partial y}\right)^2\right]^{\frac{3}{2}} = \frac{\partial v}{\partial y} + \frac{\partial w}{\partial y}\left(\frac{\partial z}{\partial y} + \frac{1}{2}\frac{\partial w}{\partial y}\right) \tag{3.3}$$

where u, v - displacements in directions of axes x and y correspondingly,
G, H - the final horizontal forces of the network,
E - modulus of deformation of cables,
t_x; t_y - effective thicknesses of the carrying and the stretching cables correspondingly.
For elimination horizontal displacements we may equate the network edge displacements to displacements of the contour beam, caused by action of horizontal forces of the network.

After approximation of the deflection function for the network and using Bubnoff-Galjorkin procedures we get for the relative deflection $\zeta_0 = \frac{w_0}{f_x}$ a cubic equation

$$(1+\psi+4\xi)\zeta_0^3 + 3[(1-\alpha\psi)+2(1-\alpha)\xi]\zeta_0^2 + \{2[(1+\alpha^2\psi)+(1-\alpha)^2\xi] +$$
$$+ (1+\alpha^{-1})(1+\chi_x)(1+\vartheta\xi)p_0^*\}\zeta_0 = (1+\chi_x)(1+\vartheta\xi)p^* \tag{3.4}$$

where $\alpha = \frac{f_y}{f_x}$; $\chi_x = \frac{5f_x^2}{3a^2}$; $\chi_y = \frac{5f_y^2}{3b^2}$; $\psi = \frac{a^4 t_y(1+\chi_x)}{b^4 t_x(1+\chi_y)}$;

$\vartheta = 1 + \frac{1}{\psi}$ - geometrical factors;

$\xi = \frac{5E\,t_y a^3\sqrt{\frac{a}{b}}}{72E_cJ_c(1+\chi_x)}$ - parameter of contour rigidity;

E_cJ_c - the bending rigidity of contour beam;

$p_0^* = \frac{9p_0a^4}{10E\,t_xf_x^3}$ - parameter of the network pretension;

$p^* = \frac{9pa^4}{10E\,t_xf_x^3}$ - loading parameter

The horizontal components of the network inner forces may be presented as follows

$$G = G_0 + \frac{5E\, t_x f_x^2 \zeta_0[(2+\zeta_0) + 2(1-\alpha+\zeta_0)\xi]}{9a^2(1+\chi_x)(1+\vartheta\xi)} \tag{3.5}$$

$$H = H_0 - \frac{5E\, t_y f_x^2 \zeta_0[(2\alpha-\zeta_0) - \frac{2}{\psi}(1-\alpha+\zeta_0)\xi]}{9b^2(1+\chi_y)(1+\vartheta\xi)} \tag{3.6}$$

4. ESTIMATION OF BEHAVIOUR OF HYPAR-NETWORK

Hypar-network with elliptical contour, supported by the vertical columns, is characterizied by some specific qualities. The contour without any outer horizontal supports may freely widen or narrow under action of cable forces, caused by loading the network. Therefore the contour stiffness and other system parameters has decisive import for the stress-strain state of the network. In the following will be described some aspects of behaviour of the network and its collaboration with the contour beam:

1) Dependence of relative deflection of the network on load parameter has obviously nonlinear character (Fig.3), similar to analogous relationship for the plane suspended systems;

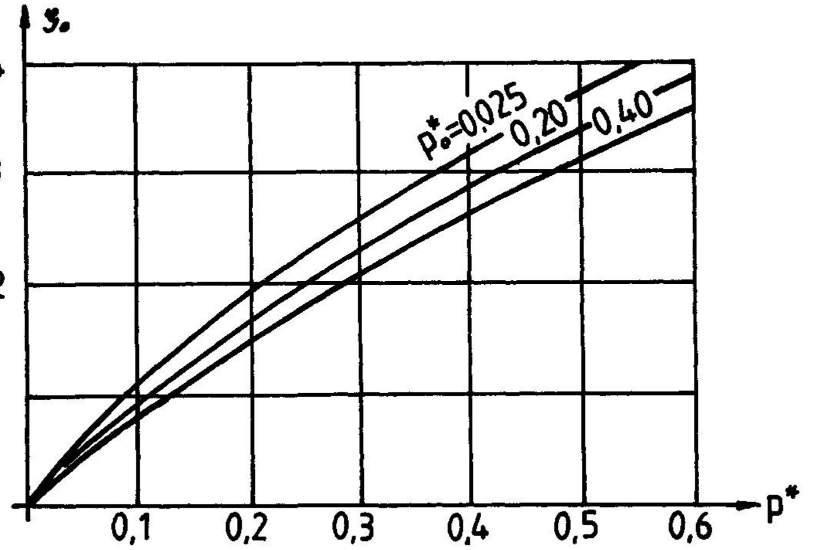


Fig.3 Dependence of displacements on the load parameter

2) Inner forces of carrying cables under action of network loads will increase by all values of bending rigidity of the contour. Inner forces of stretching cables will decrease in the case of great and increase by small bending rigidity of the contour beam. Therefore the bending moments of flexible contour beam are comparatively small.

3) Dependence of network deflection on the parameter of contour rigidity is shown on the Fig.4. It is remarkable, that by very great and very small rigidity of the contour beam, deflection of the network does not change remarkably.

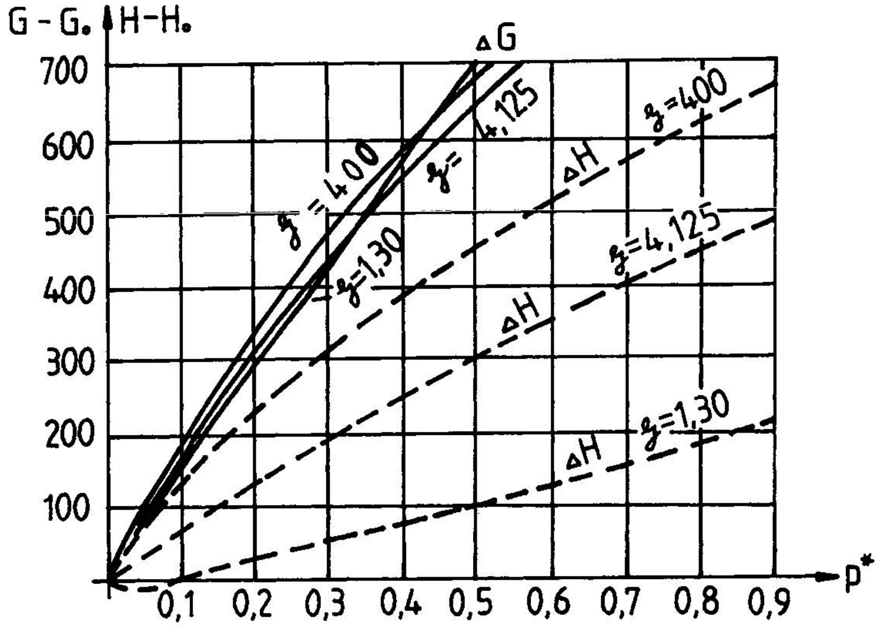


Fig.4 Dependence of cable forces on the load parameter

5. ERECTION EXPERIENCE OF SADDLE-FORMED SUSPENDED ROOFS IN ESTONIA

Two saddle-formed network structures have been erected in Estonia as acoustic screens for song festival tribunes. The first of them was erected in Tallinn in 1960. It is inclined network, prestressed inside the contour, formed by two plane arches and supported by massive counterforts. The bearing structure of the acoustic screen of the tribune in Tartu erected in 1993 is an hypar-formed network within a contour, constructed as a spatial tubular rod with axis, having elliptic and parabolic projections. The contour is supported by three plane supports, connected with the contour and foundation by linear hinges. The supports do not resist symmetrical horizontal displacements of the contour. Therefore the interaction between the network and the contour is of particular importance for balancing outer loads. Because of acoustic requirements the cladding of both screens in Tallinn and in Tartu was made from timber panels. For tribune in Tartu the collaboration between the network and timber shell is taken into account.

REFERENCES

1. KULBACH V., Hanging Roofs as Acoustic Screens for Song Festival Tribunes in Estonia. Transactions of Tallinn Technical University, No 721 /1990.

2. KULBACH V., Behaviour of Tubular Contour of Hypar-formed Hanging Roofs. Tubular Structures. London and New York, Elsevier Aplied Science, 1990, p.437-432.

The Lightest Retractable Roof

Toiture escamotable ultralégère

Das leichteste einziehbare Dach

Matthys LEVY
Principal
Weidlinger Associates
New York, NY, USA

Matthis Levy, a Swiss born civil engineer graduated from Columbia Univ. with MS and CE degrees. He is the designer of the Georgia Dome in Atlanta, GA, and New York's Javits Center and other structures built around the world.

SUMMARY

Tenstar Domes with cable networks rigidized using tensegrity concepts can be adapted to a wide variety of configurations. For the Georgia Dome, completed in 1992, an oval plan was covered with a teflon coated fiberglass membrane stretched over a triangulated Tenstar Dome. Alternate configurations have now been proposed for roofs spanning as much as 360 m and one scheme includes a retractable oculus that greatly enhances the usefulness of a facility by providing all-season, all-weather protection for special events combined with the advantage of natural daylighting for sporting events.

RÉSUMÉ

Les coupoles Tenstar à réseaux de câbles raidis selon les principes Tensegrity sont adaptables à de nombreuses configurations. Ainsi, une membrane de fibres de verre revêtue de téflon et tendue au-dessus d'une coupole Tenstar subdivisée en triangles a permis de couvrir la surface ovale pour le Georgia Dome achevé en 1992. L'auteur propose d'autres formes pour les toitures de portées supérieures à 360 m; un projet de toiture comporte une membrane escamotable offrant une protection permanente par tout temps aux manifestations spéciales, avec éclairage naturel pour les rencontres sportives.

ZUSAMMENFASSUNG

Sogenannte Tenstar-Kuppeln aus Seilnetzen, die nach Tensegrity-Prinzipien versteift sind, können einer Vielzahl von Bauformen angepasst werden. Für den Georgia-Dom (1992 fertiggestellt) wurde eine ovale Grundfläche mit einer Teflon-beschichteten Fieberglas-Membrane überdacht, die über eine in Dreiecke unterteilte Tenstar-Kuppel gespannt ist. Es sind Konfigurationen für Spannweiten bis 360 m in Diskussion. Ein Konzept beinhaltet eine einziehbare Dachhaut, die durch die Kombination eines jahreszeitunabhängigen Witterungsschutzes und natürlichem Tageslicht die Verwendbarkeit einer Anlage für spezielle Veranstaltungen bedeutend erhöht.

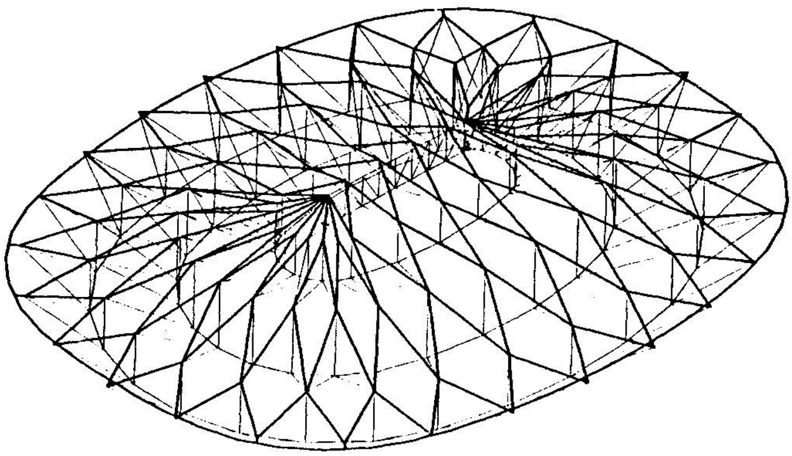
1 Introduction.

The first historical retractable roof was the canopy over the coliseum in Rome and was more like a horizontal curtain hung from a series of parallel ropes strung between the top of the stage house and the back of the stands. It served more as a sun shade rather than a weatherproof cover because of its numerous open joints. Thirty years ago, a modern retractable roof was built in Pittsburgh consisting of a number of orange peel sectors supported along the edge on a circular track and in the center of the arena, from a cantilevered arm. The roof, apart from being very expensive, suffered from mechanical problems and is now permanently shut. When the Toronto Sky dome was completed in 1988, modern technology and sophisticated computer analysis dealt successfully with the mechanical difficulties that had plagued the Pittsburgh structure and its temperature and deformation dependent deformations. However, the cost issue remained as that structure exceeded $400 million dollars. The Toronto Sky dome and recent Japanese retractable domes have been constructed as rigid, heavy, steel structures, either spanning across an arena or cantilevering from the edge.

The Tenstar retractable dome[1] offers another approach based on the principle that a cable structure that remains in place over the opening is virtually invisible and totally transparent. By introducing a structure that supports a retractable roof, a lightness of structure is achieved that is directly proportional to a "lightness" of cost.

1.2 The System.

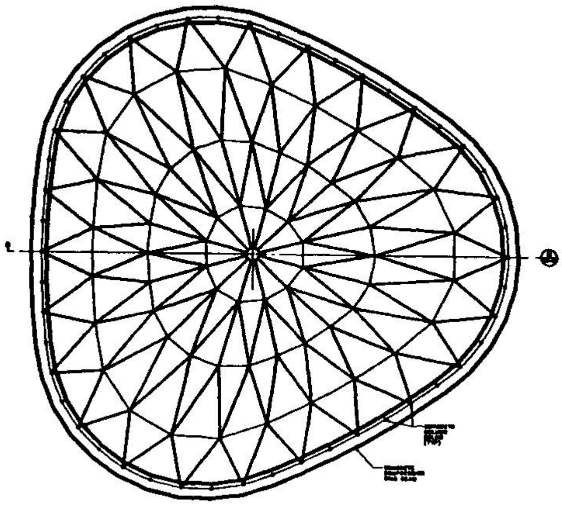
The first Tenstar Dome was completed in Atlanta in 1992 and consists of a 240m by 193m fabric covered oval dome. It was based on the tensegrity concept proposed in 1954 by the American original, Buckminster Fuller. This structure consisted of ever smaller annular rings, rigid in their

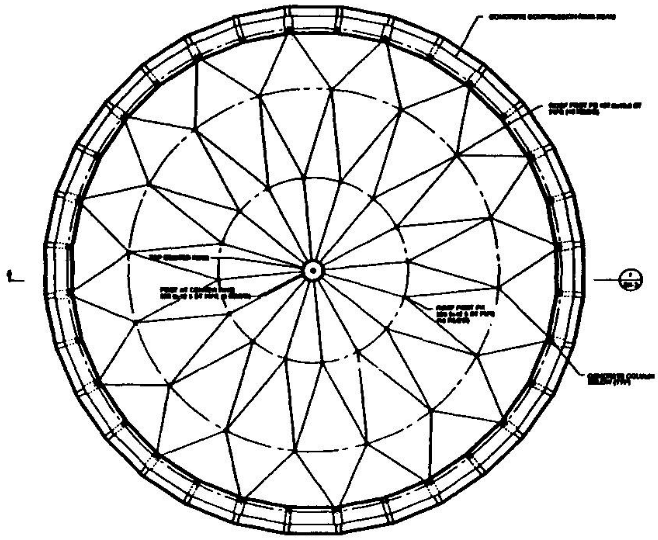


[1] Patent Pending

vertical planes, connected to each other with cables running from the top of the large ring to the bottom of the next smaller ring. He described it as a structure in which islands of compression reside in a sea of tension. Structurally, it can be described as a radially oriented succession of discontinuous trusses in which the bottom chord is a series of hoops tying together all the trusses.

In the Tenstar Dome, each node is braced by triangulated cables forming, on the top surface, a continuous net. The resulting arrangement is an extremely stiff structure in which the stiffness is obtained both from the triangulation and the prestress necessary for a cable net. The total dead weight of the resulting structure, including its fabric cover is an incredibly low 0.3 kN/m^2, less than one half the wind suction load specified by most building codes as well as the required live load.



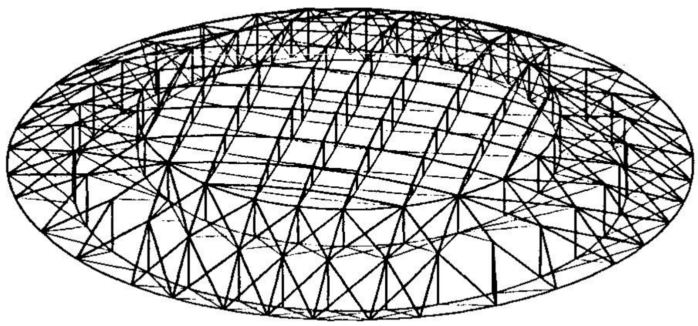


The system allows and incredible variety of alternative configurations: the plan may be a circle, a curved triangle, or an oval; the covering material may be fabric or a rigid metal deck; openings in the roof may be introduced leaving only an annular ring to cover grandstands and most exciting of all, a retractable roof can be devised.

2.1 The Retractable Option.

The retractable option we have studied offers the lightest weight and concurrent lowest cost roof of this type developed to date. The scheme consists essentially of a cable dome with parallel cables on the top surface to which parallel tracks are attached. The spacing of the cables is of the order of 15m which renders this sparsely populated cable grid relatively invisible to the spectator looking up at the sky. Nevertheless, the spacing is small enough to permit a lightweight truss structure to be designed to ride on it. This movable roof is assembled from a series of separated triangular trusses, linked to each other in the direction of travel in the manner of a caterpillar. In the orthogonal direction, the sections are connected with spring loaded rods to allow rotation and some lateral displacement. This concept permits the moving roof to adapt to a deformable cable supporting structure.

By accepting the minimal visual impingement of a cable net, an extremely lightweight movable roof structure is possible compared to the heavy, trussed, and often cantilevered movable roofs that have been built.

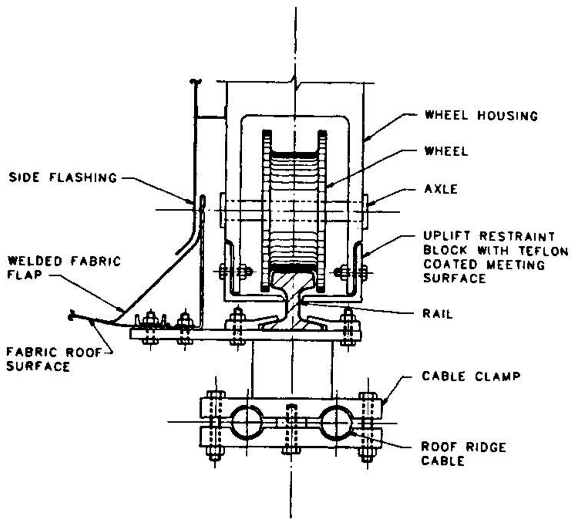


2.2 Alternative Configurations.

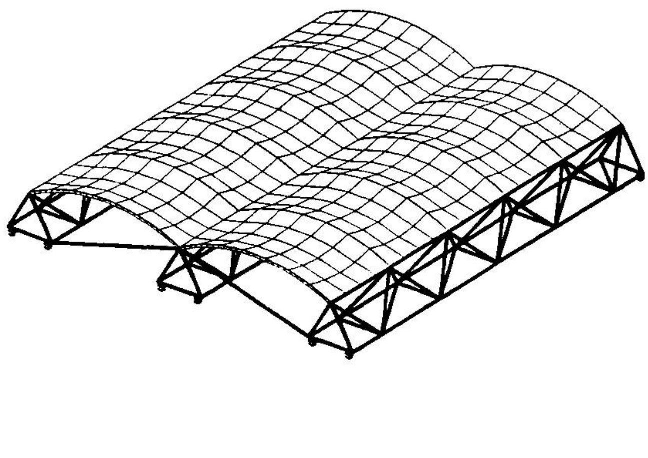
Two types of these retractable roofs have been studied: An oval roof with a rectangular retractable oculus; a round roof with four petal-like retractable sectors. In both cases, the retractable sections are arranged for the greatest simplicity and ease of operation and maintenance.

Because the retractable roof sections are lightweight, the bogies and operating mechanisms including wheels, housings, motors are all scaled accordingly and are lightweight. Two operating systems have been considered: A cable driven system with fixed motors located on the compression ring. A wheel driven system with built-in small horsepower electric motors directly geared to the wheels on the operable section.

At the meeting stiles of the roof sections, a weatherproof interlocking device has been proposed. As the sections approach each other, a cam displaces a spring loaded cap on one section that then overlaps a curb on the other section.

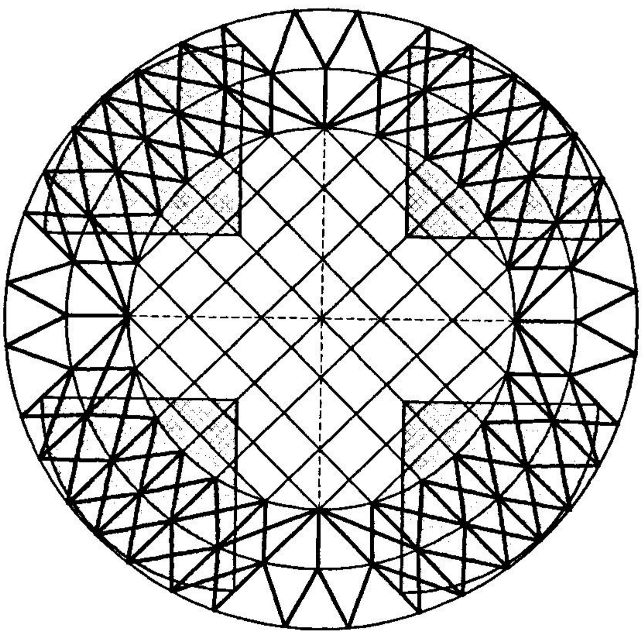


WHEEL HOUSING DETAIL

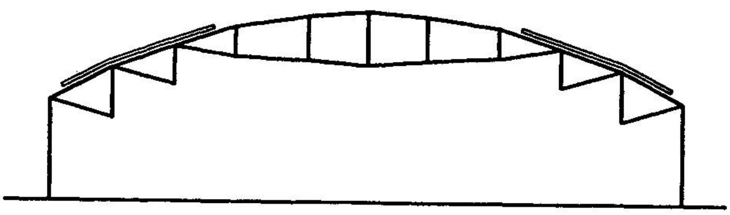


ISOMETRIC VIEW

The fixed portion of the roof can be covered in fabric using the hyperbolic paraboloid panels used on the Georgia Dome or can have a rigid covering of metal panels supported by steel joists. The movable roof panel can similarly be covered in fabric using saddle shaped panels between sprung arches or can have a rigid metal covering. There is an inherent advantage in using fabric as the roof covering because it obviates the need for a separate waterproofing membrane that, because of the flexible nature of the roof, needs flexible joints, all of which are sources of potential leaks. On the other hand, in locations where insulation and the need to support large snow loads are important, a rigid roofing material may be more appropriate.



ROOF CABLE LAYOUT PLAN
OPENED POSITION



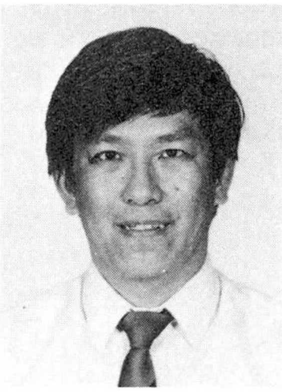
CROSS SECTION

Retractable Roof Using the 'Reciprocal Frame'

"Cadres réciproques" pour les toitures escamotables

"Wechselseitige Rahmen" für einziehbare Dächer

Ban Seng CHOO
Civil Engineer
Univ. of Nottingham
Nottingham, UK



Paula N. COULIETTE
Civil Engineer
Univ. of Nottingham
Nottingham, UK

John C. CHILTON
Civil Engineer
Univ. of Nottingham
Nottingham, UK

SUMMARY

The 'Reciprocal Frame', recently patented, is a three-dimensional beam grillage structural system used primarily in roof construction. The beams in the grillage both support and are supported reciprocally by each other. The plan view of the beams is similar in appearance to the lines forming the iris of a camera shutter. Its versatility in form and consistency in strength make it a competitive design for sports arena and stadia. Structural design, drive device, roof operation and covering are discussed.

RÉSUMÉ

Breveté récemment, le principe des "cadres réciproques" consiste en un système de grilles de poutres tridimensionnelles dont les poutres portent et s'appuient réciproquement. La disposition des poutres ressemble aux lignes d'un diaphragme d'appareil photographique. Par la diversité des formes et la résistance généralement disponible, ce genre de couverture offre une solution concurrentielle pour des salles de sport et des stades. La discussion porte sur le calcul du projet, le mécanisme d'entraînement, le mode d'ouverture et de fermeture, ainsi que le type de toit mobile.

ZUSAMMENFASSUNG

Beim kürzlich patentierten Prinzip der "wechselseitigen Rahmen" für einziehbare Dächer handelt es sich um ein dreidimensionales Trägerrostsystem. Alle Träger stützen sich wechselseitig. Die Trägeranordnung ähnelt den Linien des Zentralverschlusses einer Kamera. Dank Formenvielfalt und der durchgängig vorhandener Festigkeit resultiert eine wettbewerbsfähige Lösung für Ueberdachungen von Hallen und Stadien. Tragwerksentwurf, Antrieb, Arbeitsweise beim Ein- und Ausfahren sowie Abdeckung werden diskutiert.

1. INTRODUCTION

Across the globe architects and engineers are exploiting the versatility of retractable roofs for sports stadia and arenas. In North America, the Toronto Skydome [1,2], the Olympic Stadium in Montreal [2], and the Civic Arena in Pittsburgh [2], are well known examples of structures with retractable roofs. The structural design of each one is unique. In the Skydome, three separate roof sections retract and nest over the top of a fixed panel; each panel is a different size and shape from the others. The Montreal Olympic Stadium has a fabric roof that can be rolled up into a tower but because of difficulties in retraction, it now remains permanently retracted. The Civic Arena in Pittsburgh features a successful retractable dome where six separate sections pivot about a pin and roll along curved rails before coming to rest over two cantilevered fixed sections.

Additionally, there are several structures which are in operation in Japan, such as the Mukogawa High School Swimming Pool, the Ariake Colosseum, and the Fukuoka Dome [3]. Apart from these buildings, which have successfully opening and closing roof systems, there are many other designs being developed for retractable roofs on sports complexes all over the world. One such design being worked on at the University of Nottingham is the Reciprocal Frame Roof [4], see Figure 1. A related reticulated roof in the form of a spherical dome was proposed by Emilio Pérez Piñero in his patent 9102733 of 1961 [5]. The likeness of these two structures to the iris of a camera shutter makes them possible solutions for retractable roof systems.

2. THE RECIPROCAL FRAME ROOF

2.1 Architectural Aspects

As the Reciprocal Frame Roof has the same configuration as a camera shutter, it is easy to visualize its opening and closing. The static configuration has considerable visual impact and appears very dynamic when viewed from the floor. The primary beam structure seems to be rotating about an axis, in empty space, at the centre of the roof. During the retraction process, like the leaves of an iris diaphragm, each beam rotates individually about its external support, opening and closing the structure, see Figure 2 A to D. A major advantage to using the Reciprocal Frame Roof as a retractable roof system is the versatility in floor plan design as this is not limited by the need to provide extensive running rails (either in a straight line or constant curve) that are required by most other systems. Since the outer and inner polygons in the structure need not have the same shape, this roof can cover stadiums with virtually any geometry.

2.2 Structural Components of the Roof

The primary structure of the Reciprocal Frame is a circuit of beams spiralling around an imaginary centre. Each beam in the grillage both supports and in turn is supported by the other beams in the structure, hence the name reciprocal. The beams are placed tangentially around a central closed curve so that they rest upon the preceding beams creating the closed circuit. An enclosed polygon is formed with a set of radiating beams equal to the number of sides of the polygon. The outer ends of the beams in the grillage are supported by columns or walls. These columns or support positioning on the walls also form a polygon but its shape need not be the same as that of the inner polygon. As the beams rest on each other, there is a rise from the outer to the inner polygon which depends on the height between beam centre lines at their intersection, number of beams, etc., creating a three dimensional structure.

2.3 Structural Behaviour

The configuration of the beams in the Reciprocal Frame Roof causes applied vertical loading to be converted into downward thrust acting through the outer supports. In a regular circular or polygonal form carrying a uniformly distributed vertical load, the beam reactions are all equal to the total roof load divided by the number of beams. However, when a point load is applied

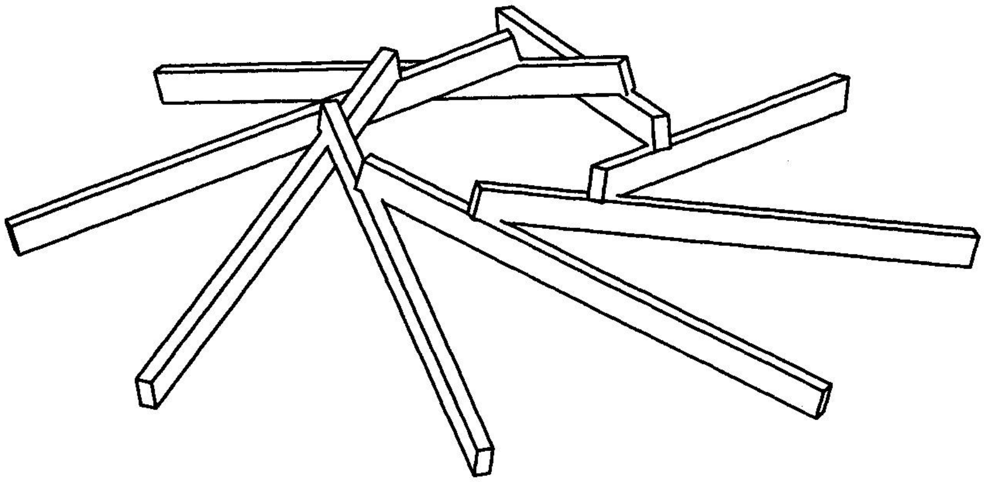


Fig.1 Three-dimensional view of Reciprocal Frame

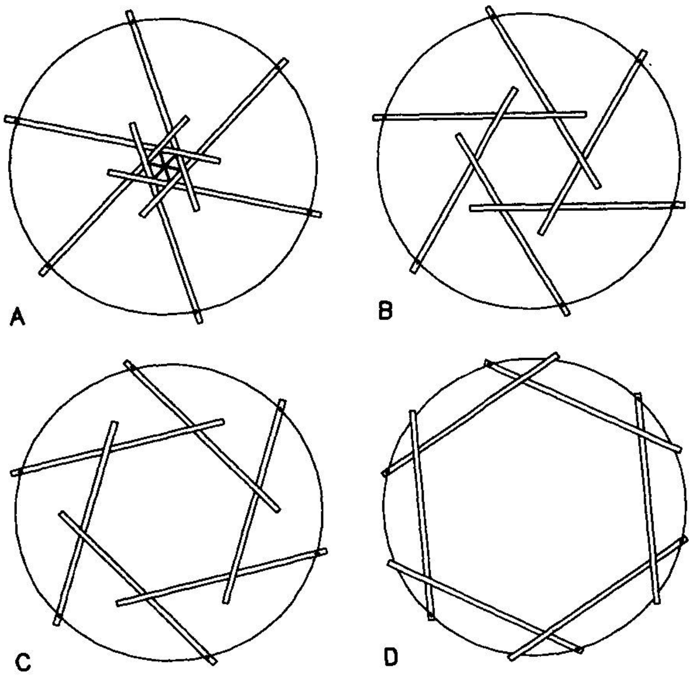


Fig.2 Plan views of retractable Reciprocal Frame structure

to an individual beam it will be partially carried by all the beams in the grillage and individual beam reactions will depend on the position of the load and geometry of the grillage. Any load applied to the structure affects all beams and similarly the deflection of one beam produces displacement in all beams. Since the elements of the Reciprocal Frame are resisting the loads by bending, no central compression ring or external tension ring is required. When the roof retracts, it is simply changing the shape of the inner polygon and therefore similar load distributions and characteristics occur. These features are particularly interesting when the structure is subjected to wind and snow loading. Non-symmetrical loads, for instance, a snow load on one side of the roof, are distributed more evenly throughout the structure.

3. ROOF OPERATION

3.1 Geometry of Opening/Closing System

To open the Reciprocal Frame Roof, each beam rotates about its outer support. The degree to which each rotates depend upon the number of beams in the grillage and the shape of the inner and outer polygons. However, if the beams are of the same lengths, it is important that they all rotate simultaneously and by the same amount about their outer supports. Due to the complex three-dimensional geometry, displacement of one beam greatly affects the shape and structural integrity of the roof. As the roof opens, each beam rotates about its outer support, both vertically in the plane of the beam and horizontally toward the outer polygon. Since the beams are all interdependent, it is important that they are always being supported by the previous beam. Opening the roof moves the inner support of the beams along a curved path towards the outer support, as shown in Figure 3, therefore, the beam must be at least as long as the maximum distance to the support. This distance is only slightly longer than each beam but the exact distance depends on the shape of the polygons and number of beams in the structure.

3.2 Drive Mechanism

The beams of the roof can rotate independently using individual synchronized motors or can be connected by an outer ring which is mechanically rotated, in turn rotating each of the beams through the required distance. However, use of such a ring greatly inhibits the amount of opening in the roof because the radius of rotation of the beam and the radius of the entire roof (radius of ring) are different. To account for this difference, it is possible to insert a connecting link between the ring and the beam at an angle which turns the beam without requiring a large rotation of the ring. Without the ring, the need for massive track structures (as in the Skydome, etc.) disappears.

3.3 Joints and Supports

The beams are connected to the outer supports using a hinge which allows for vertical and horizontal rotations. If tapered beams are used, the vertical rotation of the beam could be greatly reduced or possibly eliminated. The inner joints must allow for both continual support of the preceding beam and the movement of both beams. To accommodate this motion and maintain support, a rolling joint must be used. It is important to note that the joint does not connect to either beam but simply rolls between the two, acting only as a guide for the retracting movement and maintaining the position of support required.

4. DESIGN

4.1 Design Advantages

There are several advantages in using the Reciprocal Frame Roof as a retractable roof structure. First of all, it or equivalent structures, such as a salt storage building of 26 metre span made of 11 tapered, glued laminated beams using an analogous planar grillage as reported by Natterer, have already been successfully used as a static roof structure [6]. Also, because of its unique load distribution there is no need for internal support and, therefore, can be used in large arenas and stadia. Another benefit to the structure is that the strength is in the principal design and not a specific material allowing freedom in choosing the building material. The 'beams' discussed in this article refer to the basic structural members. These members can actually be steel, timber or concrete beams or steel or timber trusses, or even planes of space frame. Another benefit to this design is that it is not limited by shape. Both the outer polygon and inner polygon can be of any shape to suit the need. Because they need not have the same shape, the roof can be used to cover oddly shaped buildings and still maintain its retractable characteristics.

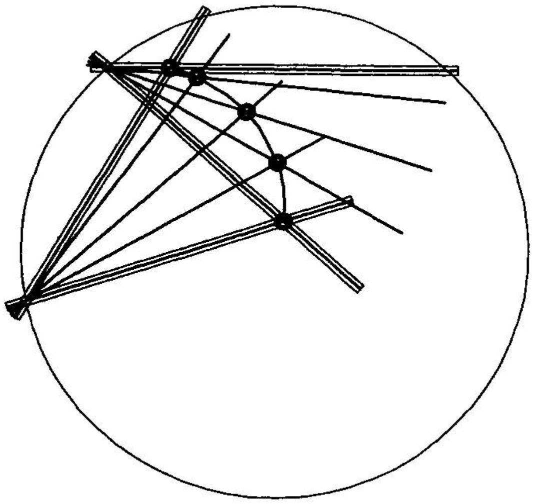


Fig.3 Curved path of intersection point of beams

4.2 Details of Design

Because the retractable Reciprocal Frame is a new concept, which has not yet been considered for use in a "real" project, specific details have not been addressed completely. However, these details have not been completely overlooked. Such topics as roofing material, drainage, and other potential difficulties in the design have been reviewed.

The Reciprocal Frame Roof allows for flexibility in choosing the cladding material. If static panels are used, the beam and panel would act as a plane in retraction. It is also possible to use a fabric membrane which folds when the roof is open and stretches out when the roof is shut. Another design consideration in making this roof structure practical is drainage. However, the roof has a natural slope to it and the solution would be found in directing the flow of water to a strategic point. As with any roof, the use of proper detailing and sealants would eliminate leaking. The most obvious of the obstacles to overcome in the Reciprocal Frame Roof is the hole in the centre which is formed because the beams can only completely close theoretically if they have no thickness. This problem however, can easily be solved by attaching small panels to the sides of the beams, at the central polygon, which come completely together when the beams are in their closed position.

A further consideration is that of disproportionate collapse. As each beam depends on all of the others for its support, the removal of or damage to any one beam may result in collapse of the whole structure unless it is appropriately designed. One way of overcoming this potential weakness is to provide suitable supports and to design the beams to act as cantilevers under the reduced loading required for the accidental limit state whilst utilising the reciprocal action for full ultimate loading. However, if the opening leaves of the roof are constructed from space frames, the structural stability does not rely totally on individual beams and the problem of disproportionate collapse can be avoided.

5. CONCLUSIONS

In conclusion, the Reciprocal Frame Roof has considerable potential for use as a retractable covering for sports arena and stadia. It has a powerful geometry with a dynamic visual impact. The design is very versatile and can be used with a wide variety of floor plans. Some examples of which can be seen in Figure 4.

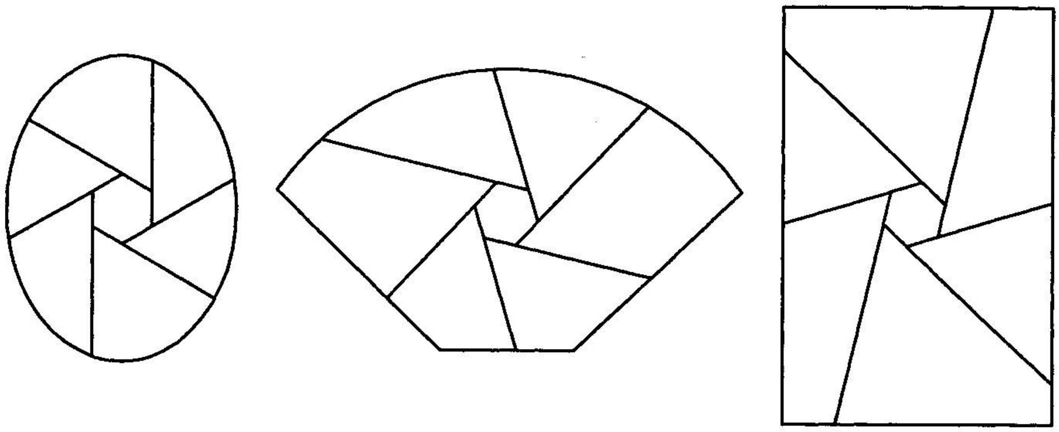


Fig.4 Reciprocal Frames over various plan shapes

REFERENCES:

1. ROORDA, J., SHERBOURNE, A.N., SRIVASTAVA, N.K., "Toronto Skydome" Proceedings from the International Association for Shell and Spatial Structues - Canadian Society of Civil Engineers International Congress on Innovative Large Span Structures, Toronto, Canada, July 1992, Volume 1, pp.51-115.

2. O'CONNER, LEE., "Toronto Skydome: The House that Engineers Built," Mechanical Engineering, October 1992, pp.54-57.

3. NARITA, H., "Examples of Retractable Roofed Domes in Japan" Proceedings of the International Association for Shell and Spatial Structures - Canadian Society of Civil Engineers International Congress on Innovative Large Span Structures, Toronto, Canada, July 1992, Volume 1, pp.498-509.

4. CHILTON, J.C. and CHOO B.S., "Reciprocal Frame Long Span Structures" Proceedings of the International Association for Shell and Spatial Structures - Canadian Society of Civil Engineers International Congress on Innovative Large Span Structures, Toronto, Canada, July, 1992, Volume 2, pp.100-109.

5. ESCRIG, F. "Las Estructuras de Emilio Perez Pinero" in Arquitectura Transformable, Textos de Arquitectura, Escuela Technica Superior de Arquitectura de Sevilla, Spain 1993, pp. 30-32.

6. NATTERER, J., HERZOG, T. and VOLZ, M. Holzbau Atlas Zwei, Institut fur Internationale Architektur, Munich, p. 179.

Developments in Cold Formed Roof Structures

Développements dans les toitures structurales écrouies à froid

Entwicklungen in kaltverformten Dachtragwerken

John C. CHAPMAN
Director
Chapman & Dowling Assoc.
Haywards Heath, UK

John Chapman was successively Reader in Structural Engineering at Imperial College, Director of the Constructional Steel Res. and Dev. Organisation, and Group Techn. Director of George Wimpey plc. He is a Visiting Professor at Imperial College, and a Fellow of the Royal Acad. of Engineering.

Robert PRINGLE
Product Manager
Space Decks Ltd
Chard, UK

Robert Pringle spent fifteen years in site engineering and project management for a variety of major civil engineering and building projects, in four continents. For the last ten years he has been engaged in product development and construction management of cold formed structures.

Patrick DOWLING
Prof.
Imperial College
London, UK

Patrick Dowling is Head of Civil Engineering at Imperial College and is Vice-Chancellor and Chief Executive of Surrey Univ. He is a Director of Chapman & Dowling Assoc. He has been Chairman of the lead committees dealing with EC3 since its inception. He is a Fellow of the Royal Acad. of Engineering.

SUMMARY

The advantages of cold formed sections are discussed, with particular reference to two space frame roof systems. Designs methods which are used for lip buckling, flexural torsional, and lateral torsional buckling are outlined. The transfer of forces and moments at the connections is described. The range of structures in which the system have been applied is indicated.

RÉSUMÉ

La communication présente les avantages des sections profilées et formées à froid, en se référant à deux systèmes de toiture tridimensionnels. Les auteurs exposent les grandes lignes des méthodes de calculs utilisées pour déterminer le voilement de bordure, la torsion par flexion et le flambement par torsion en flexion. Ils examinent en outre le transfert des forces et des moments sur les assemblages. Ils présentent la gamme de structures pour lesquelles ces systèmes ont été appliqués jusqu'ici.

ZUSAMMENFASSUNG

Diskutiert werden die Vorteile kaltverformter Profile im Zusammenhang mit zwei Raumfachwerk-Dachsystemen. Dabei sind die Bemessung für Randbeulen, Biegetorsion und Biegedrillknicken und die Einleitung von Kräften und Momenten an den Anschlüssen wichtige Fragen. Die Breite der bisherigen Anwendung auf Tragwerke wird umrissen.

1. INTRODUCTION

As techniques of structural analysis, powerfully assisted by rapidly developing computer hardware and software, reach maturity, the scope and economy of construction depends increasingly on developments in manufacturing methods, construction equipment, and materials. Cold forming, with its associated processes, is opening new avenues for structural design.

Adjustable cold rolling mills now enable purpose designed sections to be produced from a stockholding of steel strip. Holes are pre-punched on the rolling line, members are cut to length by flying shears, and when required, powder coating is then applied. Dimensional tolerances of ± 1mm, and bolt diameter clearances of 1mm are normal. Sections with a perimeter length of up to 1.2m and up to 8mm thick can now be rolled. Pre-galvanised strip with a yield stress of 390 N/mm^2 is available up to 5mm thick. The cold rolled components for a roof structure covering 5000 sq m can typically be produced in 5 working shifts. Components can usually be man-handled. Costs per unit weight are somewhat greater than for hot rolled sections, but this is offset by weight reduction, corrosion protection and response time.

In addition to the production advantages, the designer has the freedom to exercise ingenuity in devising optimum sections to meet architectural and structural requirements. The structural analysis of cold formed members is however considerably more complex than for hot rolled members, because of the various forms of instability which must be considered. Many more developments in production methods, in design, and in applications, can be expected.

The investigations which provide the background for this paper have centred around two patented systems, but the approaches and methods which have been developed have general application.

2. THE HARLEY SYSTEM 80

The Harley joint for a square on square space frame is shown in Figure 1. The guiding principle of the Harley system is to minimise the cost of the nodal connection. Chord splices are located away from nodes, and occur every three or four modules. Joint economy is achieved at the cost of introducing end moments to the chords. The moments are much greater in the inner chords, and are a maximum in the vicinity of columns. The provision of moment capacity across nodes does however exclude the possibility of toggle failure, and enhances the robustness of the structure. The number of components is minimised by bending the flattened ends of the tubular diagonals to enter between the back to back channels, so forming an architecturally neat connection. The creases of compression tubes are not fully supported; this limits the diameter of the tube which can be used, and causes an end eccentricity. Very formable steel is required, and during assembly the tubes are supported only at the lower ends. The length of the end tab is limited by the width of the section, so the end distance limits the tensile strength of the tube. Nevertheless, the system has proved to be economic, and many structures have been built using the Harley system [1].

3. THE MULTIFRAME SYSTEM

This system also uses back to back channels, but the diagonals are connected through wing plates, and the flattened ends of the tubes are not bent (Figure 2). The tab length is not limited by the width of channel, more than one bolt can be used if required, the end distance is not limited, and the end eccentricity of the diagonals is very small. The wing plate transmits the resultant force of the four diagonals to the nodal connection, and leaves no voids between the back to back channels. The greater eccentricity moment is in the longitudinal direction of the wing plate, which therefore assists the transfer of this moment, by virtue of the bending strength of the trough. The cost of the additional components is offset by increased structural efficiency, and by removing the demand on formability imposed by bending the tabs. Also the tube size is not limited, and much greater spans are possible than with the Harley system. The system can also be applied to planar, triangular, and box trusses, and to portal frames.

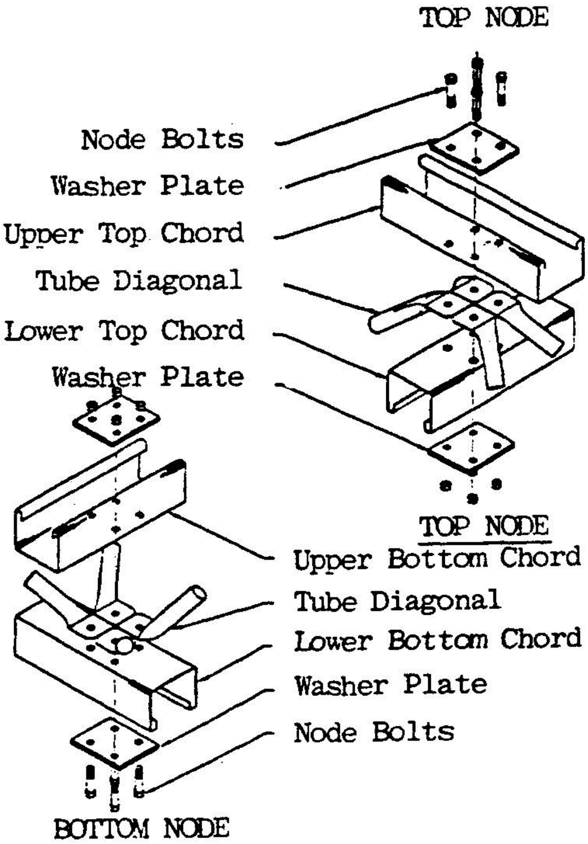


Fig.1. Harley connection

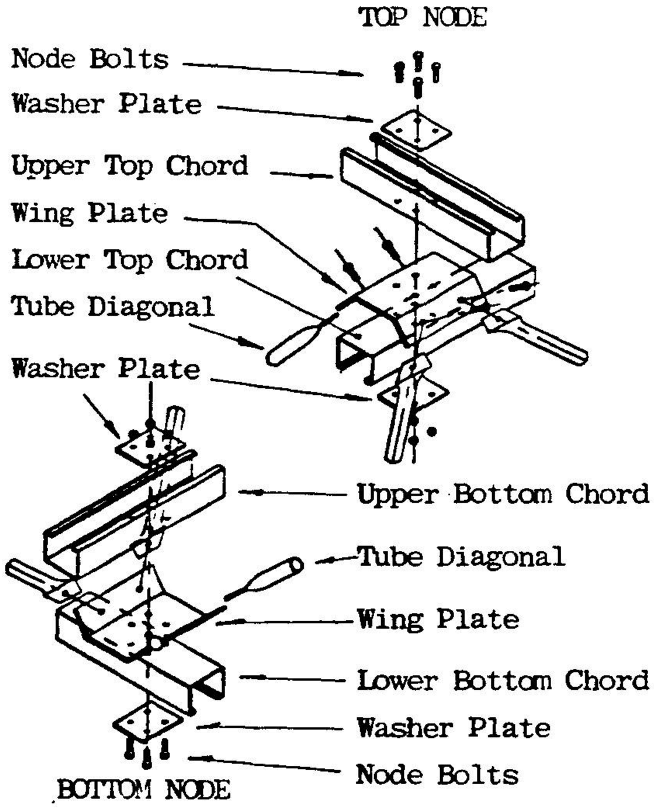


Fig.2. Multiframe connection

4. STRUCTURAL BEHAVIOUR

The following paragraphs outline the phenomena which must be considered in design. A general description of the development is given in [2] and details can be found in [3].

4.1 *Local buckling*

The forces and moments are computed assuming fully effective sections. Section strength checks are then made taking account of loss of effectiveness due to local buckling. The effect of stress level in an element on the effective width may be taken into account. The effect of coupling of the sides of the section may also be considered in calculating the critical stress.

4.2 *Lip buckling*

The lips of the channel can buckle laterally as struts on an elastic foundation, the "foundation" stiffness being provided by the transverse deformational stiffness of the section (Figure 3). An important question to consider was whether local buckling of thin walled sections would significantly reduce the deformational stiffness of the section. Finite element analysis confirmed that for typical lip/local buckling wavelength ratios (about 7:1), the effect of local buckling on lip buckling is insignificant. The lip buckling resistance can be found by assuming an initial deformation (related to manufacturing tolerances) which is affine to the critical lip buckling mode, and using a modified Perry equation [4]. The effect of lip buckling on flexural or torsional flexural buckling can then be taken into account approximately by reducing the yield stress to the lip buckling resistance. There is some evidence however that when torsional buckling develops before lip buckling, so that both lips buckle laterally in the same direction, in a single half wave, lip buckling is thereby inhibited. There are two reasons for this - the lip buckling stress for a single half wave is much greater than for the preferred mode, which typically has three or four half waves, and when the lips buckle in the same direction, the foundation modulus is increased. Systematic studies are now in progress to study, quantify and establish criteria for this interactive behaviour.

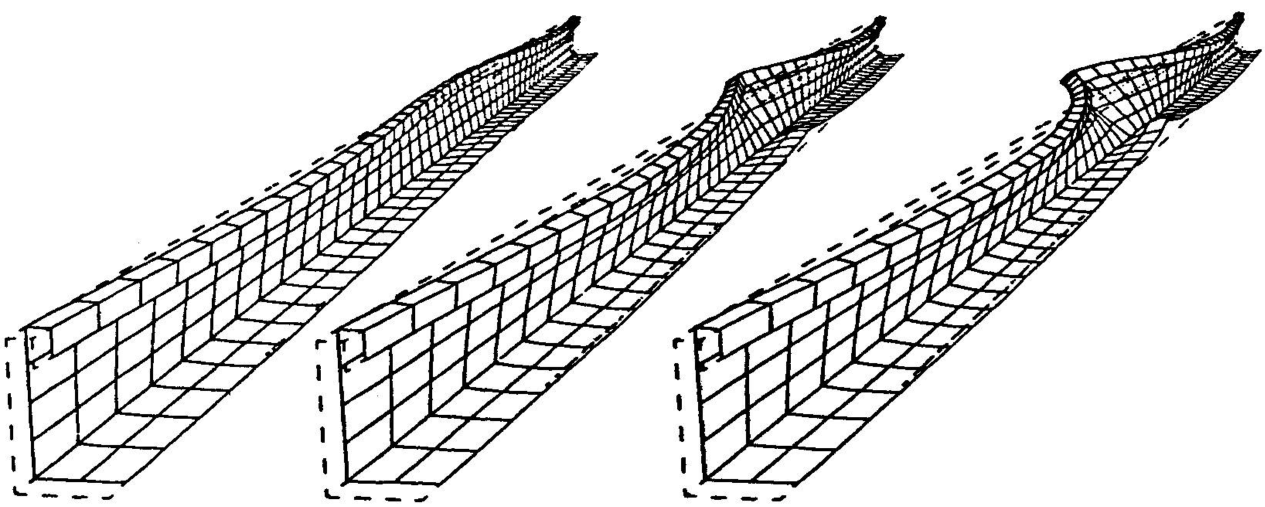


Fig.3. Development of Lip Buckling

4.3 *Torsional buckling*

In current design codes, torsional or torsional flexural buckling is treated by substituting the critical torsional buckling stress for the Euler buckling stress in the Perry equation for imperfect struts. In fact the torsional buckling stress distribution (Figure 4) is quite different from that for flexural buckling, and can be found by applying Young's equation for the deflection of imperfect struts separately to the translation and rotation of a member buckling torsionally[5]. It was also shown that a simple modification to the Perry equation, which then correctly represents torsional buckling of a doubly symmetric section, provides a satisfactory approximation for torsional flexural buckling. The section shown in Figure 3 is prone to lateral torsional buckling, which also can be treated by the method just described.

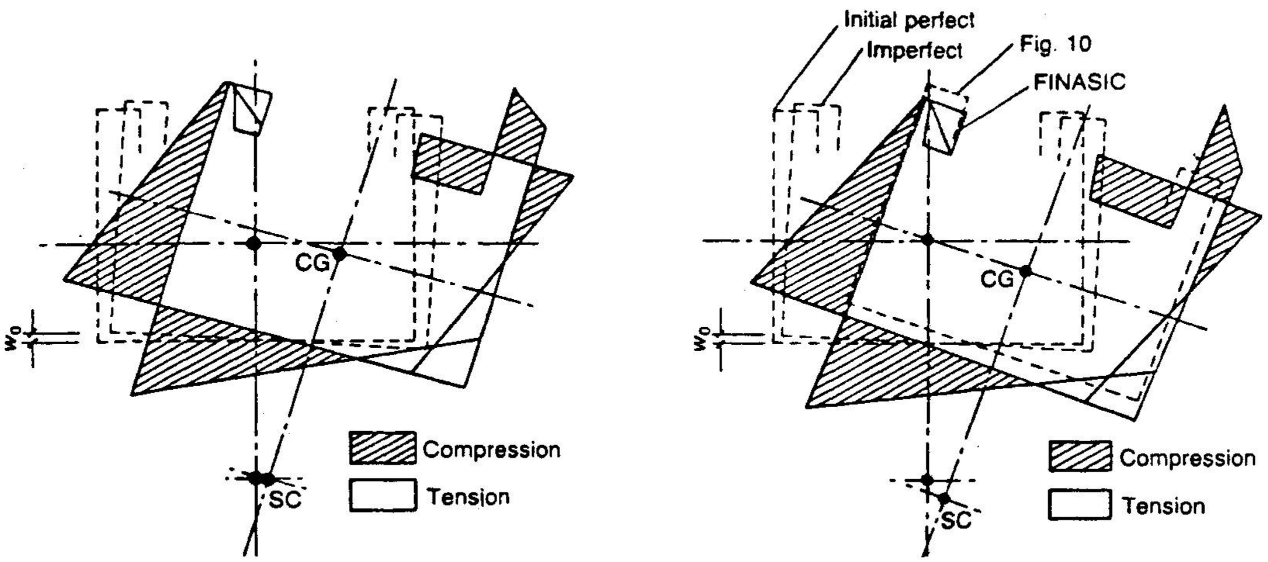


Fig.4. Comparison of Torsional buckling stresses - design model and finite element analysis

Torsional flexural and lateral torsional buckling interact strongly, because both buckling modes consist of rotation and translation of the section, the member buckling in a single half wave. When moment causes lip tension, torsional flexural buckling is inhibited to an extent which depends on the magnitude of the moment. It may conservatively be assumed that the resistance is equal to the torsional flexural buckling resistance, or to the flexural buckling/moment interaction value, whichever is smaller. Similar considerations arise in respect of interaction between axial tension and moment which causes lip compression. The interaction between the buckling modes is an important theme of current research.

4.4 *Nodal forces*

In the Harley system, vertical forces are transferred at the tube creases to the washer plates, which are held together by four bolts, which resist tension and shear. The washer plates transfer components of the horizontal tube forces as axial forces applied to each chord. The washer plates also transfer components of the eccentricity moments to the chord tables and thence to the chord webs, which are therefore subject to vertical forces which vary from tension to compression over the length of the washer plates. The table of the chord is subject to varying vertical shearing forces between the edges of the washer plates and the chord webs. Where necessary, nodal inserts are provided, which prevent web crippling and table shearing. The crease of a compression tube is not supported over its whole length, and the tube size must be limited to prevent crease deformation, and also to limit the eccentricity moment applied to the tube. The crease of a tension tube is fully supported, and the strength is determined by the tab bearing stress, which is limited by the end distance from the bolt hole.

In the Multiframe system, the wing plates and washer plates transfer the resultant forces and moments to the chords. The greater moment is transferred over a length measured from one end of the wing plate to the other end of a washer plate, so the vertical channel web forces are reduced. The flattened ends of the tubes remain straight, which simplifies fabrication and places much less demand on formability. The tubes are axially loaded, a more efficient end distance can be adopted, and more than one bolt can be used if required. The wing plates must have sufficient resistance to bending about the mid-thickness. Bending is caused by eccentricity, and by misalignment, which results from manufacturing tolerances and from joint translations and rotations due to load.

In any system employing flattened tubes, if the strength is not limited by the tensile connection, flattening of the tube can occur before the tensile strength of the tube is attained. In a compression tube which is short enough not to buckle, outward bulging of the end regions can occur, if the tube is sufficiently thin.

5. DESIGN, FABRICATION AND CONSTRUCTION

It will be apparent from the above discussion that many design checks must be made, in addition to the frame analysis. Remembering that many more structures are designed in outline than are actually built, the necessity for computerising the design process is apparent. In fact the development of these systems depends as much on the computer as on the manufacturing process, which itself is computer controlled. The computer executes and couples analysis, design, estimating, scheduling, and the control of manufacture.

The high rate of production necessitates careful pre-checking of component schedules and dimensions. The dimensional accuracy which is inherent in the production system ensures rapid assembly on site.

Roof structures are normally assembled at ground level. In the Multiframe system, the bottom chords are laid out on level stools, and the wing plates are inserted and bolted. The diagonals and top wing plates then provide stable pyramids on which the top chords can be laid and bolted.

Large roof structures are raised by special lifting columns, which ensure that all lifting points are maintained at the same level. This system also has advantages on restricted sites.

6. APPLICATIONS

The Harley system can provide spans with perimeter columns up to about 40m, and internal spans between single columns up to about 26m. The Multiframe system does not have specific limits, but is economically viable for perimeter supported spans up to at least 70m and internal spans up to at least 40m. The systems have been applied to a wide variety of roof structures and canopies, including sports halls, transportation terminals, industrial buildings, and retail facilities.

7. REFERENCES

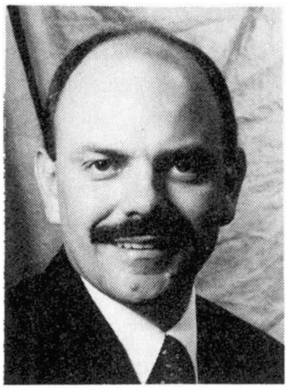
(1) CODD B E, WHITE S H. Eagle Centre Market refurbishment, Derby. The Structural Engineer. Volume 70, No.5, 1992.
(2) CHAPMAN J C, BUHAGIAR D, DOWLING P J. Developments in cold formed space structures. Conference on space structures, Guildford, Surrey, September 1993.
(3) BUHAGIAR D. Behaviour and design of structures using thin-walled cold-formed sections. PhD thesis, Imperial College, London 1993.
(4) BUHAGIAR D, CHAPMAN J C, DOWLING P J. Design of C-sections against deformational lip buckling. Eleventh international conference on cold-formed structures. University of Missouri-Rolla, October 1992.
(5) CHAPMAN J C, BUHAGIAR D. The application of Young's buckling equation to design against torsional buckling. Proc ICE August 1993.

Composite Long-Span Joists

Poutres mixtes de grande portée

Weitgespannte Verbund-Träger

W. Samuel EASTERLING
Assoc. Prof.
Virginia Tech
Blacksburg, VA, USA



W. Samuel Easterling, born 1959, graduated at West Virginia Univ. and Iowa State Univ. He has been involved in research of composite floor systems for the past 10 years. Now at Virginia Polytechnic Inst. and State Univ. his research in composite floor system is ongoing.

Thomas M. MURRAY
Prof. of Structural Steel Design
Virginia Tech
Blacksburg, VA, USA

Thomas M. Murray, born 1940, obtained a Ph.D. from the Univ. of Kansas in 1970. He has held faculty positions at the Univ. of Oklahoma and the United Air Force Academy. He is currently the Montague-Betts Prof. of Structural Steel Design at VPI&SU and an expert on floor serviceability.

SUMMARY

The benefits of open, essentially column free floor spaces in buildings are well recognised. Such configurations provide maximum flexibility in leasable space arrangements and thus give owners the ability to easily accommodate the request of new tenants. Composite long-span joists (trusses) are one structural system that provides these large open areas. Results of a comprehensive research program that focused on the behaviour and strength of composite long-span joists will be reviewed in this paper. Three building projects that utilise this structural system will be described.

RÉSUMÉ

L'avantage de disposer de locaux libres de colonnes est bien connu. De tels agencements offrent la plus grande flexibilité pour mettre d'importantes surfaces en location ou de les adapter aux exigences des locataires. Les poutres mixtes de grande portée (poutres et charpentes en treillis) offrent une bonne solution pour ce genre de locaux. Les auteurs présentent une vue synoptique des résultats obtenus à l'aide d'un programme de recherche sur le comportement de la résistance de poutres mixtes de grande portée. Trois projets de bâtiments illustrent l'application de ce système porteur.

ZUSAMMENFASSUNG

Der Vorteil offener, stützenfreier Räume in Gebäuden ist wohlbekannt. Derartige Anordnungen bieten dem Eigentümer grösstmögliche Flexibilität in der Bereitstellung vermietbarer Geschossfläche und der Anpassung an die Erfordernisse neuer Mieter. Weitgespannte Verbundträger (Fachwerke) sind ein mögliches Tragsystem für solche offenen Räume. Der Beitrag gibt einen Ueberblick über die Ergebnisse eines umfassenden Forschungsprogramms zum Festigkeitsverhalten von Verbundträgern grosser Spannweite. Der Einsatz dieses Tragsystems wird anhand dreier Bauprojekte geschildert.

1. INTRODUCTION

The benefits of open, essentially column free floor spaces in buildings are well recognized. Such a configuration provides maximum flexibility in leasable space arrangements and thus gives an owner the ability to easily accommodate the requests of new tenants. Composite long-span joists (light trusses) are one structural system that provides these large open areas. In addition to the benefits of column free space, the open web configuration of the joists permits easy access for mechanical and service systems, without necessarily increasing the floor-to-floor heights in the building. Both of these characteristics are significant benefits and make composite joists an economical structural system, as was realized in three recently constructed projects in the United States.

The use of composite joist systems in the U.S. is hampered by the lack of a design specification. At present, if a structural engineer wishes to consider a composite joist alternate, there are two scenarios available. One is for the engineer to use the available literature and design specifications, including those of other countries, to design a system using available cross sectional shapes. The other option is for the engineer to request design assistance from a joist manufacturer that has some experience in composite joist design. Due to the optimization inherent in joist manufacturing, the second option will likely be the most economical.

Several buildings have been constructed using composite joist or truss floor systems, most notably the Sears Tower in Chicago and the World Trade Center towers in New York. However, these composite trusses were designed by the structural engineers on the respective jobs and were unique designs. In the United States, joists or light trusses are structural elements that are manufactured in shops that are solely dedicated to this type of fabrication. This is in contrast to a conventional building fabrication shop. Joists are selected from design load tables (performance tables) developed and approved by the Steel Joist Institute (SJI).[1] The joist specifications used by the SJI closely follow the American Institute of Steel Construction (AISC) allowable stress design specification.[2] Because of the efficiency and optimization in the joist design and manufacturing process, it is unusual for joists to be independently designed by the structural engineer and fabricated in a conventional building fabrication shop. It is in the context of the U. S. design and manufacturing process that composite joists are discussed in this paper.

Results of a comprehensive research program that focused on the behavior and strength of composite long-span joists are highlighted in this paper. Also, a brief description of three building projects that utilized the composite joist floor system are presented. Finally, the present situation in the United States, with respect to composite joist design specifications, is presented.

2. RESEARCH PROGRAM

A comprehensive composite joist research program has been in progress at Virginia Polytechnic Institute for the past several years. The project has included destructive tests to evaluate strength of both full-size joists and push-out specimens, non-destructive tests to evaluate human occupant induced vibration characteristics and analytical studies using non-linear finite element analysis.

2.1 Full-Size Joist Tests

A series of 11 full-size composite joist tests have been conducted at Virginia Polytechnic Institute. The joist specimens ranged in span from 12.19 - 17.07 m and in depth from 356 - 915 mm. All joists were either a Warren or Modified Warren configuration. The loading configuration consisted of eight concentrated loads placed equidistance apart, thus approximating a uniform load arrangement. All test specimens were loaded to failure.

Comparisons were made between the predicted and experimental strength and stiffness values. The predicted values were made using procedures similar to those outlined by Chien and Ritchie.[3] Modifications to these procedures were made as outlined by Gibbings, et al. [4] and Nguyen, et al.[5] In general the results were quite acceptable, as indicated in Table 1 (see section 6. NOTATION).

TEST	M_e/M_c	I_{effe}/I_{effc}	$\Sigma Q_n/T_{bc}$	Stud Position
1	0.89	0.89	1.34	unknown
2	0.97	0.89	2.23	unknown
3	0.96	0.99	1.68	unknown
4	0.76	0.81	0.98	weak
5	0.92	0.69	1.27	strong
6	1.10	1.24	1.39	strong
7	1.15	1.07	1.38	strong
8	1.17	1.01	1.40	strong
9	1.19	0.88	1.37	alternating
10	0.93	1.04	1.03	alternating
11	0.93	0.83	0.99	alternating

Table 1 Summary of Composite Joist Tests

Test No. 4 was an exception in that the experimental-to-calculated strength ratio was lower than deemed acceptable. The behavior of this specimen was due to the placement of shear connectors in the weak position, along with having the shear connector ratio less than unity. Detailed presentations of the tests can be found in the project reports.[4,5]

2.2 Push-out Tests

During the course of the joist test program, problems were encountered in which the shear studs appeared to fail at lower than expected loads. These premature failures were primarily attributed to the influence of studs placed in the weak, or unfavorable, position. This problem led to a study of the strong vs. weak position issue,[6] which was previously identified by several other researchers. An additional project is underway at the time of this writing in which the behavior of shear studs placed in metal deck profiles is being further investigated. It is believed that the results of the current study, along with those conducted previously at Virginia Polytechnic Institute and elsewhere, will result in changes to the stud reduction factors presently used in the AISC specifications.[2,7]

2.3 Occupant Induced Vibration Studies

Because of the relatively long span of composite steel joists and corresponding large open floor areas without permanent partitions, the possibility of annoying floor vibrations caused by human activities exists. A number of tolerance criteria are used in North America to determine if a proposed floor design may be annoying to future occupants. However, all of these criteria were calibrated using floor systems with spans in the range of 6.5 - 12 m.

The criterion proposed by Murray[8] was used to evaluate the three floor systems described in the following section where spans are 15 - 36m. All three systems easily satisfied the criterion. In addition, tests of the bare floor of the Nations Bank Building (see Section 3.1 for a description) were conducted. It was determined that the calculation methods in the Murray criterion are applicable to such long-span composite joist floor systems and that the predicted human response was accurate. That is, the floor system was found to be free of annoying vibrations due to human activities. Also, no complaints have been received concerning floor motion in any of the three completed buildings described in Section 3.

3. BUILDING PROJECTS

Three building projects were constructed in the last several years in the United States that utilize composite joist floor systems. The design process for each of the three buildings was similar in that the structural engineer worked closely with the joist manufacturer. Designs according to the requirements set forth by the structural engineer of record (EOR) were carried out by the joist manufacturers' structural engineering staff and subsequently approved by the EOR.

3.1 Nations Bank Building

The Nations Bank Building, originally the Sovran Bank Building, is located in Knoxville, Tennessee. The 11 story building was designed by the office of Stanley D. Lindsey and Associates, Ltd. of Atlanta, Georgia. Composite joists are used on floors 6-11 in the office rental space, with levels 1-5 being a concrete parking structure.

The composite joists have a depth of 1,016 mm and are used for spans of 18.9 m. The joist spacing is 2,540 mm. The composite slab consists of a 76 mm deep x 0.91 mm thick composite steel deck with a structural lightweight concrete fill, yielding a total slab thickness of 160 mm. Joists were fabricated using steel with a nominal yield stress of 345 MPa and have an approximate mass per length of 50 kg/m. A total of 32 welded headed studs (19mm x 132 mm) were used as shear connectors along each joist. A more complete description of the project is given by Swensson[9].

3.2 312 Elm Street Building

The 312 Elm Street Building is located in Cincinnati, Ohio, for which the structural design was done by the office of Stanley D. Lindsey and Associates, Ltd. of Atlanta, Georgia. It is a 26 story tower with the first 10 stories being a concrete parking structure and the upper 16 stories being office rental space.

The composite joists in the building have depths of 813 mm and spans of 14.78 m. The joist spacing is 3,048 mm. The composite slab consists of a 51 mm deep x 1.1 mm thick composite steel deck with a normal weight concrete fill, resulting in a total slab thickness of 115 mm. Joists were fabricated using steel with a nominal yield stress of 345 MPa and have an approximate mass per length of 60 kg/m. A total of 32 welded headed studs (19mm x 90 mm) were used as shear connectors along each joist. A more complete description of the project is given by Corrin and Swensson[10].

3.3 Associated Wholesale Grocers Building

The Associated Wholesale Grocers Building is located in Kansas City, Missouri and was designed by the office of A. Renczarski and Co., Inc. The building is unique in that it provides two stories of office space over an existing single story structure.

A composite joist floor system was selected for the new structure, given the span requirements of 36 m. Joists with a depth ranging from 1,525 - 2,030 mm are used in the project. The joists are spaced from 2,310 - 2,565 mm. A composite floor consisting of 51 mm deep x 0.91 mm thick composite steel deck, topped with a 127 mm deep (total thickness) normal weight concrete slab is used in the building. . Joists were fabricated using steel with a nominal yield stress of 345 MPa and have an approximate mass per length of 125 kg/m. Welded headed studs (19mm x 106 mm) were used as shear connectors along each joist, with the number varying between 46 and 88.

4. DESIGN SPECIFICATIONS AND GUIDELINES

The design of composite joists is hampered in the United States by the lack of an accepted specification. However, a composite joist specification is currently under development by an American Society of Civil Engineers (ASCE) committee. Once the document is complete it will be published in the *ASCE Journal of Structural Engineering* for review and comment. Subsequently, the document may be further developed into an ASCE Standard.

The draft version of the specification is in a Load and Resistance Factor Design (LRFD) format and includes a specification, commentary and example problem. The specification draws heavily on the pioneering work done in Canada, as prescribed in the Canadian steel specification.[11] Research conducted at the University of Minnesota[12,13] and Virginia Polytechnic Institute[4-6] has also been used as support information.

A recently published design aid[14], while not a specification, provides the information necessary to make trial composite joist selections. The manufacturer's catalog was developed using available specifications, [1,2,7,11] of which only one contains information specifically for composite joists.[11] The general design procedure used to develop the design catalog was also used for the three building projects described in section 3 of this paper. The structural engineer can use the design catalog to obtain approximate designs, and thus weight (cost) estimates for a composite joist alternate. Subsequent to the initial design estimates, the structural engineer can work with the joist manufacturer to refine and optimize the design.

5. ACKNOWLEDGMENTS

The authors wish to acknowledge Nucor Research and Development and Vulcraft, both divisions of Nucor Corporation, for sponsoring the research program at Virginia Tech. Additionally, our thanks go to David Samuelson and Wayne Studebaker of Nucor Research and Development for the technical assistance they have provided throughout the project. The authors wish to acknowledge the following graduate assistants that conducted research related to the project during the last several years: Chris Cook, David Gibbings, Son Nguyen, Bruce Queen and Chuck Sublett.

6. NOTATION

I_{effc} = calculated effective moment of inertia
I_{effe} = experimental effective moment of inertia
M_e = experimental moment capacity
M_c = calculated moment capacity
T_{bc} = bottom chord yield force (using measured material properties)
ΣQ_n = summation of shear connector strength (using measured material properties)

7. REFERENCES

1. *Standard Specifications, Load tables and Weight Tables for Steel Joists and Joist Girders*, Steel Joist Institute, Myrtle Beach, SC, 1992.
2. *Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design.* American Institute of Steel Construction. Chicago, Illinois. September, 1989.
3. CHIEN, E. Y. L. and RITCHIE, K. L. *Design and Construction of Composite Floor Systems*, Canadian Institute of Steel Construction, 1984.

4. GIBBINGS, D. R., EASTERLING, W. S. and MURRAY, T. M. "Strength of Long-Span Composite Joists." Report No. CE/VPI-ST 91/02. Virginia Polytechnic Institute and State University, Blacksburg, VA, 1991.

5. NGUYEN, S. N., GIBBINGS, D. R., EASTERLING, W. S. and MURRAY, T. M. "Further Studies on Long Span Composite Joists," Report No. CE/VPI-ST 92/07. Virginia Polytechnic Institute and State University, Blacksburg, VA, 1992.

6. EASTERLING, W. S., GIBBINGS, D. R. and MURRAY, T. M. "Strength of Shear Studs in Steel Deck on Composite Beams and Joists," *Engineering Journal,* AISC, 30(2), 44-55, 1993.

7. *Load and Resistance Factor Design Specification for Structural Steel Buildings*. American Institute of Steel Construction. Chicago, Illinois. September, 1986.

8. MURRAY, Thomas M. "Building Floor Vibrations," *Engineering Journal,* AISC, 28(3), 102-109, 1991.

9. SWENSSON, Kurt D. "Long Span Composite Joists in the Sovran Building." Proceedings of the AISC National Steel Construction Conference, AISC, Kansas City, Missouri, 24.1-24.18, 1990.

10. CORRIN, Michael E. and SWENSSON, Kurt D. "Eccentric Braced Frames: Not Just for Seismic Design." Modern Steel Construction, AISC, 32(8), 33-37, 1992.

11. *Limit States Design of Steel Structures*. CAN/CSA-S16.1-M89, Canadian Standards Association. Rexdale, Ontario, 1989.

12. CURRY, J. H. "Full Scale Tests on Two Long-Span Composite Open-web Steel Joists," M. S. Thesis, University of Minnesota, Minneapolis, Minnesota, 1988.

13. LEON, R. T. and CURRY, J. H. "Behavior of Long Span Composite Joists," Building Structures, Proceedings of the ASCE Structures Congress, ASCE, Orlando, FL, 390-403, 1987.

14. Vulcraft Composite Joists. Vulcraft, A Division of Nucor Corporation, Charlotte, NC, 1994.

Long-Span Glulam Roofs for Sport Facilities

Construction en bois lamellé collé pour salles de sport

Brettschichtholzbau für Sporthallen

Giorgio BIGNOTTI
Civil Eng.
Holzbau SpA-AG
Brixen, BZ, Italy

Giorgio Bignotti, born 1962, got his civil engineering degree at the Politecnico di Milano. Employed in Holzbau AG since 1987, he was in the first years involved in the structural design of glulam structures and then became responsible for the quotation and project management.

SUMMARY

The paper presents the state of the art in glulam constructions applied to long-span structures. The various aspects of glulam timber engineering are described including the following topics: material characteristics, production, structural systems, calculation methods, transportation, erection and cost analysis. A selected number of timber structures are presented in order to illustrate the application of this material to sports facilities constructions.

RÉSUMÉ

La communication porte sur l'état actuel de la technique dans la construction de structures porteuses en bois lamellé collé à grande portée. Elle traite des différents aspects relatifs aux charpentes en bois, puis examine les propriétés de ce matériau, sa fabrication, les systèmes porteurs, les méthodes de calculs, les problèmes du transport, du montage et de la rentabilité. A l'aide d'exemples bien choisis, l'auteur illustre les applications de ce matériau dans la construction des centres de sport.

ZUSAMMENFASSUNG

Der Beitrag beschreibt den Stand der Technik bei der Realisierung von Brettschichtkonstruktionen für grosse Spannweiten. Von den unterschiedlichen Aspekten des Ingenieurholzbaus wird auf Werkstoffeigenschaften, Herstellung, Tragsysteme, Berechnungsverfahren, Transport, Montage und Wirtschaftlichkeit eingegangen. Anhand ausgewählter Beispiele wird die Anwendung dieses Materials im Sportstättenbau illustriert.

INTRODUCTION

The need for covered areas in which large numbers of people can assemble has increased greatly in the last decades. To meet this demand structural engineers have to design long-span building structures taking into account economy, attractiveness and safety. Among the construction materials available for this purpose timber plays an important role. Glued laminated timber meets the requirements of a long-span structure material very well. It has a resistance similar to the best concrete, its weight is five times less, its attractiveness is well known, its cost is competitive and it is safe in the event of a fire even without additional protection. Glued laminated timber (GLULAM) is a valid material for the construction of sports centres, conference halls, shopping centres, atria of buildings, churches and all other constructions in which a large span and a beautiful roof are required. Places of assembly in general are a large field where glulam is used, but sport facilities still rapresent the most frequent application of this structural material.

2. TIMBER MATERIAL

2.1 Limitations of solid timber

When a designer wants to use wood in its original form for load bearing structures, he faces some difficulties: the large scattering in the mechanical properties make it necessary to apply high safety values; moisture content, distortion from knots, curved grain and other imperfections reduce the strenght, and dimension of the beams cannot exceed natural wood size.
Lamination solves all these problems.

2.2 Advantges of laminated timber

The technique of gluing together many lamellas reduces the scattering in the properties and improves the allowable strength and solves all dimensional problems. The grain becomes straight along the beam axis and the moisture content is reduced. The result is a material with a high bearing capacity, light, that can be ecologically produced in almost an unlimited range of shapes. For these reasons glulam is the main structural material used for large timber structures.

3. MANUFACTURE

3.1 Laminating stock

Glued-laminated timber (glulam) is obteined by gluing together a number of lumber laminations (lamellas). Usually lamellas are arranged so that the glue line plane is perpendicular to the long side of the cross section of the beam. Prior to gluing, the lumber is selected and dried so as to carry the moisture content under 15%. A large number of conifers are suitable for glulam. In Europe Spruce (Picea abies) is the most widely used species because of its good strenght and its great avilability. Except for special situations laminations have a thickness of 35mm. Spruce lumber is readily available in lenghts up to 4.5 meters, but glulam members always exceed this limit. Consequently, laminations are made of several pieces of laminating stock, end jointed together.

3.2 End Joints
The type of joint used for this purpouse is the "finger joint". Structural finger joints,about 50 mm long, are made by machining each mating surface and gluing the joints together under pressure.
3.3 Gluing
After grading and end jointig, the laminations are ready for assembly into structural members. They are structurally bonded together with one or two types of adhesives, depending on the service conditions. For interior applications a moisture resistant adhesive is used. For exposed exterior locations or continuously humid interior locations such as swimming pools, shower rooms, ice-arenas a waterproof adhesive (usually a resorcinol) is used. Adhesives must conform to the appropiate standards. After application of a controlled quantity of adhesive the laminations are assembled in a clamping jig applying a pressure of 850 kilopascal.The jig is adjusted to the desired shape of the member (if cambered or curvated is desired) before applying pressure.

3.4 Finishing
After the adhesive has set (8-10 hours), the member is removed from the clamping jig and surfaced both sides to remove squeezed-out glue and irregularities. The ends are then trimmed to provide a member of precise lenght. When necessary, the members are also drilled, dapped and grooved to accomodate connecting hardware. Unless otherwise specified an anti-fungus and anti-insects coating is brushed off.It is available in different tonalities according to the customer taste.

4. STRUCTURAL SYSTEMS
Glulam constructions are precast structures, but they are not standardized. Every time a customer orders a structure the manufacturer produces it according to the client's needs. Thus Glulam structures occur in a variety of types and morphologies. In the case of sport facilities roofing structures, the most frequent structural configurations are the following:

4.1 Cambered and tapered beams
It is the simplest structural system. Beams are supported by concrete columns and when possible are placed at a distance of 5-6 m. The span is usually between 25 and 40 m.The camber permits to design the beam with less conservative limits of deflection.

4.2 Three hinges portals
This system is used very frequently because permits to cover large areas with a simple and economic structures. Each half portal can be made of a single curved piece or of several straight pieces jointed together. The consequence of this kind of structures is that it produces big lateral thursts on the foundations. The span is usually between 20 and 70 m.

4.3 Trusses
Trusses are used expecially when the dimensions of the single glulam beam become too large. This may be caused by different reasons: manifacture difficulties when the beam is higher than 2-2.5 m or when its lenght goes beyond 40 m, economic convenience when the distance between the compression and the tension parts of the truss is wide enough. Usually trusses require less material and transport costs but more work for assembly.

4.4 Domes

Domes can be executed in two different ways: when their diameter is under 80 meters they usually are erected assembling together a number of radial arch members connected in the center to a steel compression ring. When their dimensions are bigger they usually are designed as geodesical systems whose general behaviour is the shell structural configuration.

5. CALCULATION METHODS AND STANDARDS

For calculation pourposes, standards are available in the individual countries. For example in Europe we have DIN 1052 in Germany, BS 5268 in the United Kingdom, Regles C.B.71 in France. All these are based on the permissible stress method. A new european code, harmonizing the differences between countries, is now available as an ENV European Provisional Standard, called Eurocode 5, based on the limit state method.

6. TRANSPORTATION

In general the transportation of glulam members does not present loading capacity problems but often the size of the beams create some difficulties. In this case the possibility of transport must be determined at an early design stage. This is important for a correct estimation of costs and for a rational choice of the structural systems. Curved beams with an overall height exceeding 3,6-3,8 m are not trasportable. In these cases structures have to be designed so that the beams can be manifactured in two pieces. Then at the site they can be end jointed with special steel or timber bolted connections.

7.ERECTION

During manifacture all the beams are cut and dapped to their final dimensions. If possible the steel connecting parts are fixed onto the main beams in order to reduce work on the construction site. This preparatory work makes the erection of a glulam structure usually a fast operation. Wire lines can be fixed through special posts on the main girders. When workmen have to walk on the erected beams they must tie their safety belt to the wire lines.
In order to prevent buckling it is particulary important to assemble the bracings as soon as possible.

8. COST ANALYSIS

The cost of a timber structure is important because often economic considerations determine the choice between glulam and other materials. It is often thought that glulam structures are very expensive if compared to conventional materials. This is often not true because a real comparison should take not only the cost of the roofing structure into consideration, but also the finishing grade or the foundation required. In many cases glulam has demonstrated that it is competitive. A rough cost analysis can be carried out as assuming:

8.1 Glulam
The amount of glulam must be established by a pre-calculation of the structural systems. Its cost (1000 DM/m3) has to be increased in the cases of curved beams (5-10%) and if special machining is required (10%).

8.2 Steel hardware
The quantity of steel necessary for the connections depends on the type of structural system. The steel cost can be estimated equal to 10-15% of the glulam's value in the case of simple structures, and to 20-25% in the case of complex structures with important connections.

8.3 Transportation
The transport cost depends on the dimension of the beams and on the distance between manifacturer and the site. When ordinary trucks 12 meter long are used the cost is equal to 3-5% of the glulam value. When special means of transportation are necessary it can increase to 10%.

8.4 Erection
Assembly and erection costs are usually considered for surface unit. A simple and linear roof can be erected with a cost of 20 DM/m2 but in the case of trusses or other complex situations the cost can increase to 30-35 DM/m2 (crane cost is not included).

9. EXAMPLES
Here are three exemples of glulam timber long-span structures. They were all erected in Italy in 1993:

9.1 " Sporting Club Milano 3" Tennis Center in Milan
This Sport Center has covered with glulam structures five tennis courts (three of them omologated for international meetings). The structure consists of arched beam with a span of 40 m. The corridor roof is hanging from the arches through gables.

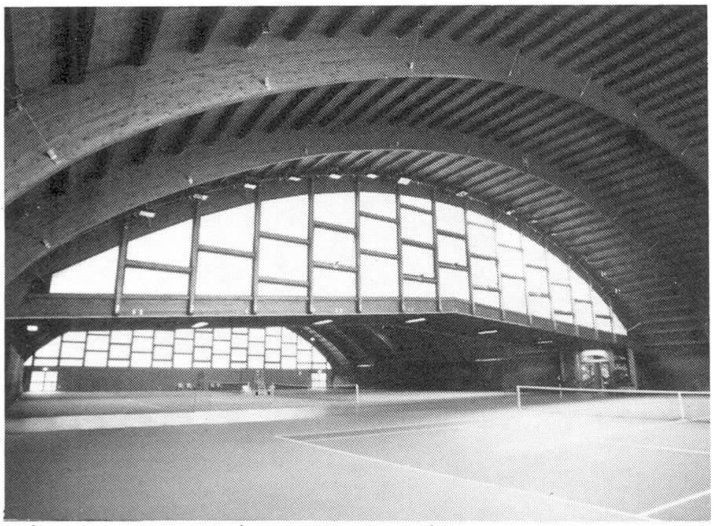


Fig.1 "Sporting Club Milano 3"
Arches cover five tennis courts.

9.2 The Casalecchio Dome near Bologna
This is the largest glulam dome in Europe. The free span between supports is 120x80 m. The geometry of the dome is made from a cylindrical vault and from two half-spherical caps. The triangular grid is made of glulam members having section 14.5x120 cm.

9.3 The "Wave" Ice Rink in Bolzano
This hall is the new ice hockey rink in Bolzano which holds 7000 spectators. The name "wave" comes from the shape of the main purlins which are double-cambered beams.

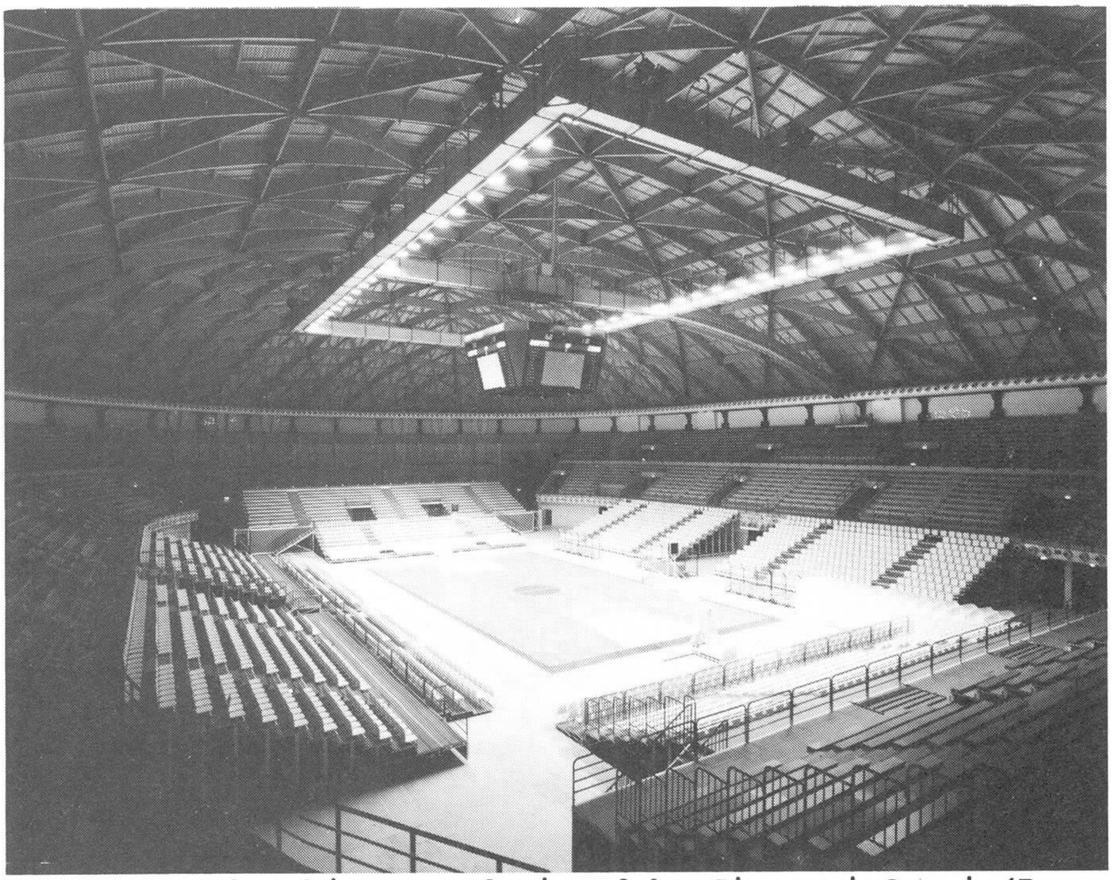


Fig.2 The Casalecchio Dome designed by Giovanni Cenci (Brunate-CO)

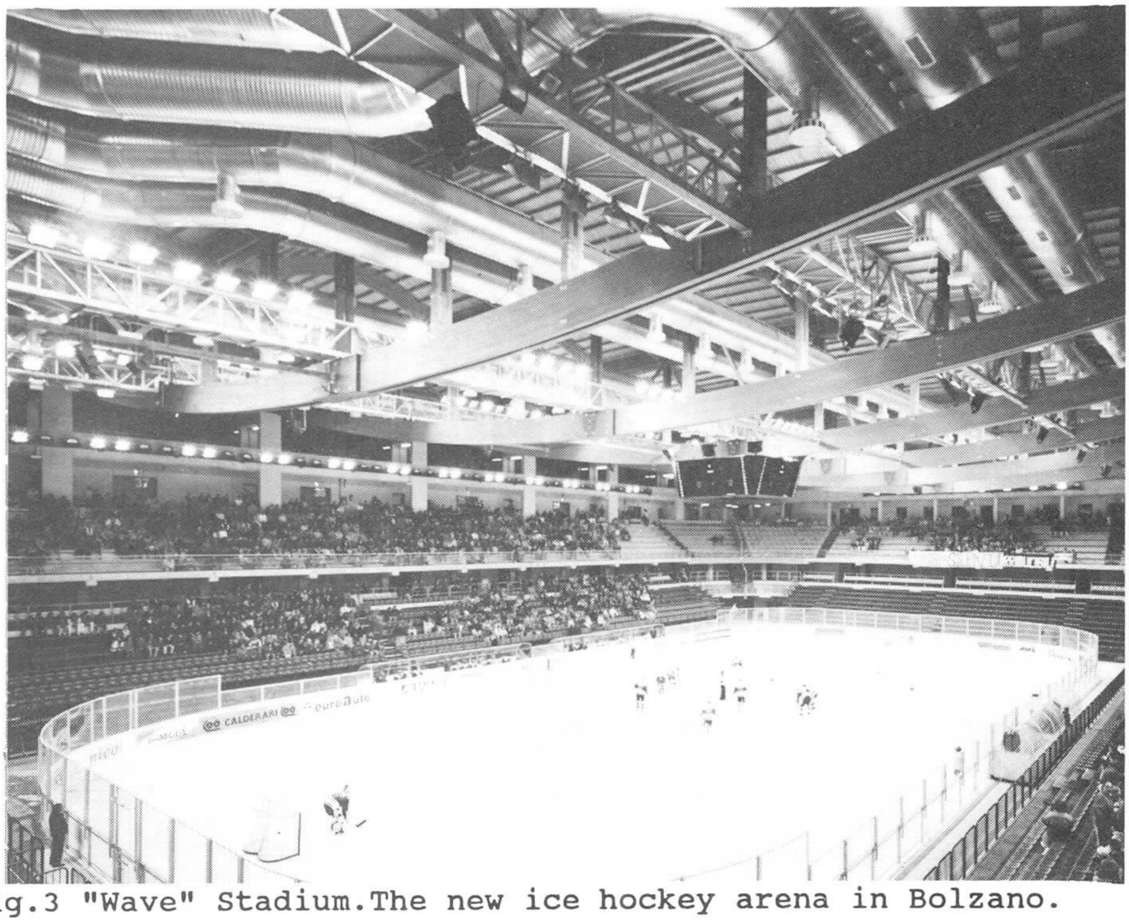


Fig.3 "Wave" Stadium.The new ice hockey arena in Bolzano.

Timber Structures for the School for Woodland Studies in Dorset
Structures en bois pour l'école des sciences forestières à Dorset
Holztragwerke für die forstwissenschaftliche Schule in Dorset

Edmund HAPPOLD
Chairman
Buro Happold
Bath, UK

Michael DICKSON
Founding Partner
Buro Happold
Bath, UK

Richard HARRIS
Associate
Buro Happold
Bath, UK

Prof. Edmund Happold was born in 1930 and ecucated at the Univ. of Leeds. He is Senior Partner and Chairman of Buro Happold, Bath, Uk.

Michael Dickson was born in 1944 and ecucated at the Univ. of Cambridge and Cornell. He is a founding Partner of Buro Happold, Bath, UK.

Richard Harris was born in 1949 and educated at the Univ. of Bristol. He is an Associate with Buro Happold, Bath, UK.

SUMMARY
This paper describes the use of roundwood thinnings in Norwegian Spruce to construct an arched form workshop and a prototype dwelling house within an historic wood as the Centre for the School for Woodland Industries, Hooke Park, Dorset.

RÉSUMÉ
L'article expose la mise en oeuvre de rondins en sapin de Norvège dans deux constructions à Dorset en Angleterre. Il s'agit d'un bâtiment en forme de plein cintre servant d'atelier et d'un prototype de maison d'habitation, situés dans une forêt historique et destinés au centre scolaire de Woodland Industries, Hooke Park.

ZUSAMMENFASSUNG
Der Betrag beschreibt den Einsatz von Rundholz aus norwegischer Fichte bei zwei Bauten in Dorset, England. Es handelt sich um ein tonnenförmiges Werkstattgebaüde und den Prototyp eines Wohnhauses in einem historischen Wald für das Schulzentrum der Woodland Industries, Hooke Park.

Introduction

Britain has a large number of lowland forests of which many were planted from the turn of the century up until the end of the Second World War, largely to produce timber for pit props for coal mining. This need has now gone and these forests, usually planted on land not readily utilised for other purposes, have become neglected due to the low commercial value of the timber.

Now British woodlands are an undervalued resource. Every year Britain imports some 85% of its timber requirements and it was the concept of exploring how to increase the value of small roundwood and of British timber generally which led to the founding of this school.

The School of Woodland studies at Hooke Park in Dorset was founded by the famous furniture design John Makepeace to teach students firstly how to maintain such forests and secondly how to make goods from the annual products from the forests which would enable them to make a living with the overall general ambition to regenerate woodland management and rural industry so the buildings for the school should be representative of imaginative experimentation in the use of forest wood.

In a typical well managed forest, from the total number of saplings originally planted only 10% should be allowed to reach maturity. To give these few trees the best growing conditions the remainder need to be gradually felled as thinnings. Conifers in the U.K. are normally planted with a density of 2500-4000 stems/hectare which will then yield between 200-300/hectare. The high density at planting is necessary due to the uncertainty of how any particular sapling will develop and as the production of straight, knot-free saw-timber is encouraged by keeping the trees crowded when they are young.

The vast majority of 'thinned' stems have trunk diameters between 50 and 200 mm and are too small for sawmilling. They are usually only suitable for low grade uses such as pulp, particle board and fencing. There is little commercial value in this operation and many smaller U.K. plantations are neglected because the cost of removing the thinnings is greater than their value. The effect of this neglect is to suppress the growth of the best trees and therefore to reduce their value.

The School is being built within Hooke Park Forest, a 140 hectare woodland, near Beaminster in Dorset.

Timber Technology:

Three main areas of interest formed the approach to the technology.

One has to be fascinated by the structural efficiency of trees and question whether we now over use sawing of timber. Sawing wastes some 40% of wood by volume, it causes warping fibres, so it make sense to, as much as possible, use timber in the round.

The main structure of a tree consists of cellulose fibres which run in the vertical direction and which account for some 50% of the solid content on the wood. They form cells so that the tree acts like a bundle of drinking straws glued together by a polymeric module called lignin (10-35%) and various other sugar compounds (15-40%). The other important component is water which not only affects the bending but also the possibility of bacterial attack.

The fibres are extremely strong in tension but have only a quarter of that strength in compression. This means that bending in timber is structurally very inefficient. However a small diameter thinning has a great capacity for carrying substantial loads as a tension member as long as efficient end connections to resist the resulting tension force can be produced.

Thicker diameter roundwood members also have greater capacity to resist buckling and to carry compression forces as slender struts due to prestressing of the outer fibres during growth.

The thinning process at Hooke Park yields a large number of Norwegian spruce poles 7m long 50-100mm diameter and a smaller number between 10-15m long with base diameters of 200mm. This is typical of many thousands of woodlands throughout the country.

Therefore, the available timber is well suited for components of building structures where the principle 'direct force' actions will be tension and compression.

The second interest is the protection of the timber from insect and fungal attack. With green timber this is of major importance but commonly available methods were not suitable due to the high moisture content and the resistance of Norwegian spruce to impregnation.

On the advice of Mr David Dickinson of Imperial College the internal timbers were treated on site by dip diffusion. This is a technique developed in Australia and New Zealand, but not popular in this country as it is only effective on freshly felled wood where the moisture content is high. The timber is dipped in a heated supersaturated boron solution (disodium octoborate tetrahydrate) for 15-30 seconds and then stored under polythene for between four to eight weeks. With an initial moisture content of at least 50% the boron solution is able to diffuse into the timber cross section. External timbers require a greater degree of protection and this has been provided by sap displacement using CCA (copper-chrome-arsenate). This has been developed from the Boucherie process, a method popular in France during the last century as a means of successfully displacing a newly felled tree's sap with chemical salts.

The final area of interest is the inefficiency of traditional tension joints. The joint that was developed for this project was based on research in the United States for fixing timber aerogenerator blades to a steel hub. The joint consists of a steel rod embedded in epoxy resin within a stepped drilled hole. The load is transferred into the timber partly by the direct bonding of the resin on to the exposed ends of the longitudinal cellulose fibres and partly by penetration into the capillaries A stepped hole allowed the ends of the capillaries to be exposed and readily available and easily sharpenable drill bits to be used.

Testing of these joints was carried out at Bath University. Initial tests compared the epoxy joint with traditional bolted flitch plate connections and proved the new joints greater efficiency. The first tests used a two part epoxy resin manufactured by Gougeon Bros. and lengths of reinforcement bar. In these tests failure was at about 30kN (11 N/mm^2) with the bar pulling out of the resin cone. Further tests were carried out with the SP110/210 system manufactured by Structural Polymer Systems Ltd. In both cases the epoxy resin had a very low viscosity to achieve penetration into the capillaries and also included cellulose microfibres. This gives gap filling properties to the mixture and also increase the elasticity of an otherwise brittle material. The results of tests on four of these joints are shown in Table A.

Joint	Dia. (mm)	Failure Load (kN)	Failure Stress (N/mm^2)
1	65	62.0	18.0
2	70	24.2	6.3
3	70	47.0	12.2
4	65	43.5	13.1

Table A

The result for joint 2 was attributed to poor workmanship in the construction of the joint as a layer of wood dust was found on the failed epoxy wood interface. The other three samples failed due to the bar pulling out of the resin core. These samples had large radial cracks in the epoxy core and splitting of the adjacent timber . This is probably due to the high shear stresses that develop at the point of entry of the bar into the joint. These shear stresses cause significant cross gain tensile stresses to develop in the timber around the core and the result is a splitting failure of the wood with pull out of the bar.

To prevent this type of failure the joints subsequently constructed for the Prototype House used threaded steel studding and a glass fibre band wrapped around the exterior of the joint. The later also has the effect of preventing splitting of the thinning at the end under eccentric loading.

Tests were also performed on two samples of the tension joints produced by the contractor during the Prototype House construction. The results are shown in Table B.

Joint	Dia. (mm)	Failure Load (kN)	Failure Stress (N/mm^2)	Factor of Safety on max. working load
1	68	42.8	11.8	5.9
2	68	41	11.3	5.7

Table B

The results give an adequate factor of safety and the failure loads compared very closely with those obtained previously. However, failure in both cases was by the epoxy cone pulling out of the timber. This may be due to the higher moisture content of these samples and that they were constructed in a horizontal position, rather than vertically as before, which led to pockets of air being trapped within the epoxy.

The compression joint that was developed is very similar to the tension joint. The steel rod acts as a locator during construction and helps load transfer by reducing the timber bearing stress. Cross grain strength of the timber will also be increased as the epoxy tends to fill the capillaries locally to the joint and strengthen them again crashing. Though these joints are designed principally to withstand compression they also have some tensile strength, which could be useful during frame assembly and in reducing the effective buckling length of compression members due to moment continuity in the joints.

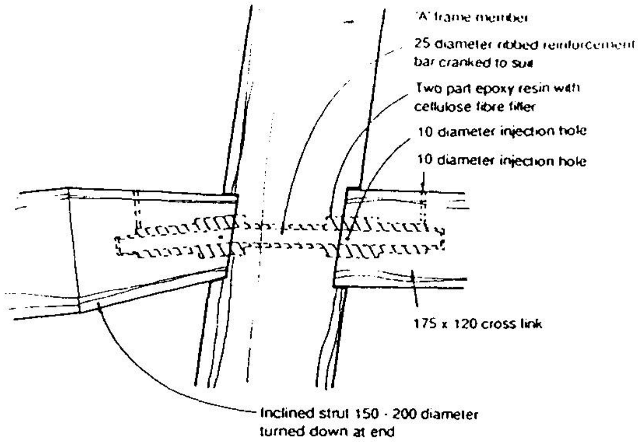


Fig. 1: Jointing technology

Design of a Prototype House

The Prototype House is 11.2m long by 8.5m wide and has six rooms each accessed from a central corridor. All the structural elements were fabricated from Norwegian spruce except for the highly loaded A-frames where corsican pine was used.

The roof structure used thinnings 60-90mm in diameter at 450mm spacing to span a maximum distance of 5.5m. Due to their small diameter the thinnings would be unable to support the expected loads in bending. However, with the thinning formed into a permanent catenary shape and the two ends restrained in direction it acted like a cable resisting the applied load in tension.

This tension is transferred into the building structure at the ridgeline through a tension joint attached to a wire cable hung between the heads of four A-frames. The tension at the eaves is transferred into a cable spanning between inclined side posts.

Work is now restarting on further staff houses and student residences which together with a programme of research work occupying the next two years, will be the subject of a further paper.

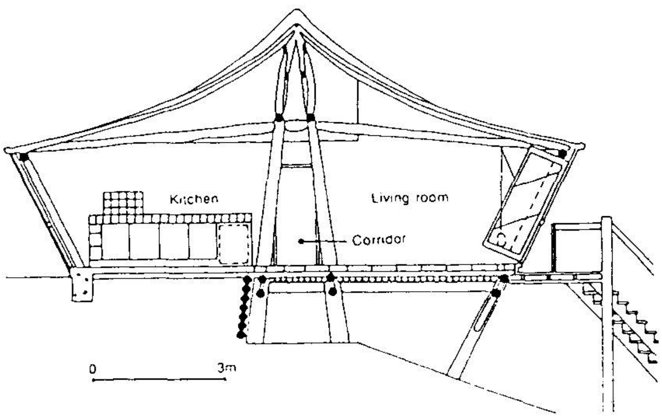


Fig. 2: Section through prototype house

Design of the Workshop Building

The requirements of the workshop building was that it provided a large open space where the machining tools can be situated and a separate area for lecture rooms and tutors offices.

Using an arch shape it is possible to span the 15m width of the building without any intermediate supports and also to obtain sufficient head clearance for a mezzanine floor. By varying the crown level of the arch along the length of the building the elevation. is seen as a series of undulating curves. Also by 'gathering in' the arch bases into groups a space can be created on the surface for window and entrance openings. All the structural timber was Norwegian Spruce.

The building actually constructed is three shells spanning 15m and forming a building 42.5 total length and 7m maximum height. Two of the shells form the workshop area, the third contains the library, seminar rooms, teaching area and offices, all arranged at ground floor an mezzanine levels. Access for visitors is by a bridge which enters the building at mezzanine level between the workshop and teaching areas.

The shells are formed using pairs of thinnings of nominal diameter 155mm at the base to 65mm approximately 9m long joined at the crown by a laminated crown arch member.

Both buildings have now been completed for two years and have preformed well.

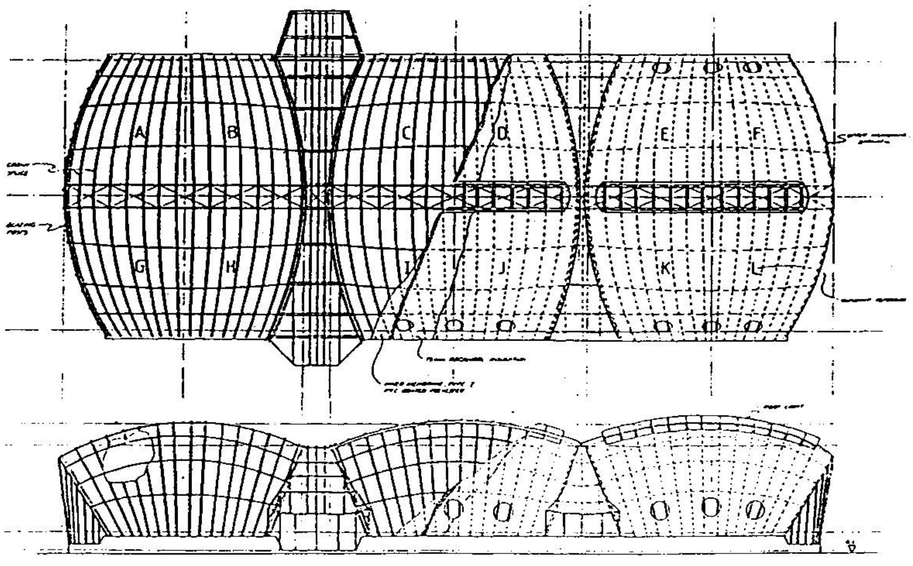


Fig. 3: Plan and elevation of workshop

Acknowledgments

The 'language' of the design of this project was based on the use of the 'wet wood' thinning timber from the site and from the development of a virtually 'full strength' tension joint. This joint was evolved from an introduction by Prof. Jim Gordon of Reading University to Gougeon Bros. of Minnesota and subsequent discussions. The relationship of the stiffness of the epoxy to that of the timber came from discussions with Prof. Bryan Harris of Bath University. The evolution of the building design was carried out with both the architects, Richard Burton of Ahrends, Burton and Koralek and Professor Frei Otto. The joints were then developed by Buro Happold and tested at the University of Bath.

Design of Overhead Sloped Glazing in Places of Public Assembly

Étude de lanterneaux inclinés dans les lieux de rassemblement

Entwurf geneigter Oberlichter in öffentlichen Versammlungsstätten

Neil NOBLE
Civil Eng.
Ove Arup & Partners
London, UK

Neil Noble, born 1949, obtained a civil engineering degree at the Univ. of Cape Town in 1972. He has worked in multi-disciplinary building engineering design teams in Ove Arup since 1974. Since 1988 he has concentrated on façade engineering and now leads a group specialising in this discipline.

SUMMARY

This paper discusses the factors which need to be considered when designing and specifying overhead sloped glazing in places of public assembly. The paper reviews the technical information and standards available to designers and proposes a minimum level of information to be provided in the specification and drawings. It also reviews criteria to be considered in the selection of the glass.

RÉSUMÉ

L'article traite des critères pour la conception et l'appel d'offres de fenêtres disposées en lanterneaux inclinés dans les lieux publics. Il passe en revue les données techniques et directives qui sont à la disposition du projecteur, ainsi que les normes minimales proposées pour établir le devis descriptif et les plans. Il indique en outre les critères servant au choix des vitrages.

ZUSAMMENFASSUNG

Der Beitrag diskutiert die Faktoren, die beim Entwurf und der Aussschreibung von geneigten Verglasungen in öffentlichen Versammlungsbauten beachtet werden müssen. Es werden die technischen Angaben und Richtlinien durchgesehen, die dem Entwerfenden zur Verfügung stehen, und Minimalstandards für die Leistungsbeschreibung und Plandokumentation vorgeschlagen. Ferner werden die Kriterien für die Auswahl des Glases besprochen.

INTRODUCTION

This symposium, entitled "Places of assembly and long span building structures", is focused on structural engineering matters; large structures, elegant structures, which provide a framework to support the covering to the public assembly space below. Whereas the consequence of failure of one of the elements of the covering is not normally as catastrophic as that of the supporting structure, the effects on serviceability, durability, usefulness and ultimately cost of maintenance and falling rental values is all to often overlooked during the initial design.

This paper considers the roof covering, and in particular the use of glass in that situation, and outlines the issues to be considered with respect to the design and specification of the elements concerned.

The total building envelope plays a vital role, not only in the architectural aesthetic but also in the technical or engineering performance of the building. On many projects, the requirements of the building envelope are becoming increasingly demanding as designs become increasingly sophisticated. The envelope is required to contribute actively to the building's performance in terms of energy control, whilst at the same time to let in as much natural daylight as possible. Supporting structures are required to span further and the roof covering needs to respond to this. Glass is being called upon to do more and more in terms of thermal performance and strength. Do designers actually have enough information or guidance on how to achieve these aims?

For a steel or concrete structure it is standard practice to specify grades of material to perform in accordance with particular requirements, to design the elements of the structure, in accordance with permissible stresses and deflections and to specify connection details, tolerance and standards of construction. Structural engineers are able to do this because there is a vast amount of information available in the form of standards, design guides and research papers.

Why is cladding, and glazing in particular, not procured in this way? One of the reasons is because there have not been until recently standards to cover the design, manufacture and installation of curtain walls or overhead glazing. The standard for curtain walls published last year by the Centre for Window and Cladding Technology has filled a very real need here, but specifically excludes internal vertical enclosures, overhead glazing systems with a slope of less than 75° to the horizontal and structural glass assemblies. Another reason is the fact that, in the case of the cladding, the contractor is normally responsible for the detailed design based on a performance specification produced by the original designer.

It is my view that, in the absence of any comprehensive standard, the original designer should be responsible for specifying not only the performance requirements, but also the materials to be used and all the design constraints relating to the cladding and in particular the interfaces with other elements to allow the contractor to carry out his duty. When one considers that the cost of the cladding of a building can be up to 20-25% of the total construction cost, as well as the consequences of failure, then surely the designer should specify in more detail.

However, in order to do this, a lot more information needs to be made available by the manufacturers of the various components. By way of contrast, structural engineers are continually being presented with new information, resulting from research and development programmes carried out by the steel and concrete industries. This research is going on in the glass industry, but it is difficult for design engineers to get the information they need. There is a fair amount of information available on U-values, shading coefficients, solar heat transmittance, and light transmittance, but very little is available on the structural performance and safety aspects of glass. More needs to be known about glass as a material in its different forms and strengths, its behaviour under imposed loads and the effects of weathering, aging, support conditions and edge conditions.

That said, what should the specification contain? How detailed should the design be without removing the opportunity for competitive tendering or undermining the manufacturers design responsibility?

Overhead glazing is more susceptible to falling objects and windblown debris, more likely to fall from its supporting framework when it breaks, exposed to greater levels of solar energy, frequently supported on adjacent structures which are subject to complex movements and required to support long duration snow loading, as well as wind loading. The design and specification needs to recognise these requirements which should be clearly set out on a set of tender drawings, including sections and details describing the preferred arrangements and constraints of the critical components. The contractor should use these as a basis for the detailed design.

Whilst each building needs to be considered on its own merits, the following may serve as a useful check list in the production of a specification and the development of design and tender drawings.

The PRELIMINARIES should include details of:

The Building

The role and function of the building, its appearance, form and size and its relationship to its neighbours should be described.

The Glazing System

The extent of the glazing, the geometry, the method of attachment to the building, the intended solidity, lightness, texture, contrast and colour should be defined. Expectations on weathertightness, drainage and staining, durability, life expectancy, maintenance and replacement should also be made clear. It is most important that the way in which the glazing is required to interact with other parts of the building is fully described, since it is at these interfaces that most problems occur. Finally, the issue of public safety needs to be addressed including principles to be adopted to minimise the risk of injury due to cladding failures.

Technical Procedures

The regulations, standards, codes and guides which define the procedures for design and analysis should be listed. The CWCT Standard and Guide to Good Practice contains a full list of relevant standards. In addition, there are a number of documents produced by the American Architectural Manufacturers Association (AAMA) covering, the structural performance of glass, sloped glazing guidelines and glass design for sloped glazing.

Technical Criteria

A detailed definition of internal and external climatic conditions, including temperature, humidity and acoustic levels should be included as well as an indication of the anticipated variation of external surface temperatures. Wind loading, snow loading, access loading, and other loads imposed on the cladding should be specified, with estimates of anticipated movements of the building structure under its dead loads, live loads, wind loads, settlement, shrinkage and creep.

The PERFORMANCE REQUIREMENTS should be clearly specified and, the tender documents should include adequate guidance in the form of drawings to illustrate the preferred method of achieving compliance with these.

Thermal Performance

Solar radiation on sloped glazing applications is normally substantially higher than on vertical glazing, due to the angle of inclination. This will affect thermal stresses within the glass and the performance of the glazing materials. Allowance for thermal expansion and contraction etc. are vital. Thermal performance criteria (U-value, shading coefficient, light transmittance etc.) need to be considered for the particular application.

Weathertightness

It is imperative in all glazing systems that water infiltration and condensation should be drained from the system, and in particular from the edge of the glazing unit. Drainage of the water in sloped glazing requires special design to the framing, since systems that work well in vertical situations are often unsatisfactory in sloped situations.

Design Loads

The strength of glass is a function of load duration, and long term loads, such as snow, must be treated differently to short term loads, such as wind. In addition to dead, wind and snow loads, consideration should also be given to the possibility of impact loading on sloped glazing installations, either due to airborne debris, missiles, building components or human accident.

Movement and Compatibility

All structures move and sloped glazing systems are no exception. In addition to deflections within the system itself, the relative movements between the overhead glazing system and its supporting structure under the various combinations of load needs to be thoroughly understood and the framing, supports and any joints must be designed to accommodate these movements. Inadequate provision for movement is a likely cause of failure of cladding components, particularly glazing.

Acoustic Performance

Any requirement for sound reduction through the glazing, and, consequently the level of acoustic insulation should be specified.

Fire Rating

Any requirements for fire rating of the glazing should be specified.

Safety

Safety of the public is of primary concern since the consequences of failure of glazing in overhead situations could be catastrophic. There is no single issue governing this, but a host of issues - specifying and design for the appropriate loads, movements, tolerances and thermal conditions, and ensuring that all parties are aware of these requirements at all stages.

Another, and equally important issue factor relating to performance in use, and safety concerns MATERIALS AND WORKMANSHIP. Critical materials should be prescribed in the specification, whether this is by specifying a particular type, or grade, or specific properties. Materials for use in the framework and their corrosion protection, fixings to the building, gaskets and sealants should all be specified, with respect to performance, compatibility and durability.

Glass Selection

The building regulations do not give any guidance as to what type of glass should be used in overhead situations. It is left to the designer to consider the particular application, the imposed loads, the thermal loads, the risk of impact from airborne missiles and the exposure of the public to injury from falling debris. Glass when procured in the U.K., would normally be supplied in accordance with BS 952 "Glass for Glazing". But there is a considerable difference between the performance of the types of glass included in the standard, particularly when it comes to strength and safety.

Any evaluation of the strength of glass must take into account its inherent structural characteristics. It is classified as a brittle material. It has no yield point, and fractures suddenly. The stress level required to break a pane of glass is related to surface compression, fabrication, surface quality, support conditions, type and duration of loading, size, thickness, geometry, edge quality, age and service history. Glass strength can only be expressed on a statistical basis, and fracture risk, while never entirely eliminated, can usually be reduced to an acceptable, practical level by appropriate selection. In determining the appropriate strength, a view must be taken on an acceptable statistical probability of breakage under design load.

In addition to strength requirements, the selection of glass, type is a fundamental issue when dealing with the safety of overhead glazed systems. Major building codes, both in this country and in the United States, are not in agreement on this matter. Both fully toughened as well as laminated glass are currently considered to be suitable for this type of use. Laminated glass, due to the influence of the PVB interlayer will tend to remain in one piece when fractured. Provided the interlayer is adequately retained in the frame, there should be little risk of the pane falling from its frame. However, if it is not adequately retained, the pane could fall as one complete piece causing significant damage. On the other hand, fully toughened glass breaks into small pieces when fractured and should fall to the ground in a shower of relatively harmless pieces of glass. However, it could fall to the ground before fragmenting.

In addition, in the case of toughened glass, the possibility of spontaneous breakage due to nickel sulphide inclusions should not be ignored. If toughened glass is specified, it is important to specify an additional heat soaking process in order to reduce the potential for spontaneous breakage due to this defect. Heat strengthened glass could be considered as an alternative to fully toughened, as this does not appear to suffer from the same phenomenon.

Overall, the final selection of glass must consider the type and duration of loading, the requirements for tolerance and movement, the thermal effects, support conditions, the degree of exposure, and the consequence of failure. A design and specification which thoroughly addresses these issues and their consequences, and the incorporation of these requirements into the design of the glazing and its support system, is essential if we are to instill the level of confidence into building owners and architects to allow them to incorporate overhead glazed structures into their projects in the future.

Leere Seite
Blank page
Page vide

Beautiful Structures and Spectator Comfort

Structures merveilleuses et confort des spectateurs

Wunderbare Bauwerke und Zuschauerkomfort

Rod SHEARD
Architect
Lobb Partnership Ltd
London, UK

Rod Sheard was born in 1951 in Brisbane, Australia where he completed his architectural training in 1974. He is Chairman of Lobb Partnership, a consultant firm of architects who specialises in sports and leisure design throughout the world.

SUMMARY

The future of sport depends on providing venues which satisfy the needs of its supporters. They must have character to enable the spectator to share the event with the crowd and experience the thrills of live sport but also offer a range of facilities, which will cater for all tastes and interests. With the help of modern technology we must find engineering and design solutions with a vision, so that the new stadium can provide an entertaining show based on the sporting event and, therefore, compete with television and attract a wider audience.

RÉSUMÉ

L'avenir du sport dépend de la qualité des stades répondant aux exigences des spectateurs. Ils doivent avoir un certain caractère pour permettre au spectateur de vivre l'événement sportif en symbiose avec la foule et d'éprouver toutes les sensations qu'il procure 'dans le stade mais aussi fournir toutes les installations qui répondent aux goûts et intérêts de chacun. Grâce à une technologie de pointe, architectes et ingénieurs doivent concrétiser leur vision, de façon à ce que le nouveau stade offre un spectacle complet et divertissant centré sur l'événement sportif et par conséquent, fasse concurrence à la télévision et attire ainsi un public plus grand.

ZUSAMMENFASSUNG

Die Zukunft des Sportes hängt von der Bereitstellung von Stadien ab, die den Ansprüchen der Veranstalter genügen. Stadien müssen Eigenschaften haben, die es dem Zuschauer erlauben, in der Menge den Geschehnissen beizuwohnen und die sportliche Leistung unmittelbar mitzuerleben, aber sie müssen auch über Einrichtungen verfügen, die unterschiedlichen Interessen und Bedürfnissen gerecht werden. Mit Hilfe moderner Technologie müssen konzeptionell neue bauliche und stilistische Lösungen gefunden werden, so dass neue Stadien auch dem jeweiligen sportlichen Anlass angepasste Unterhaltungsdarbietungen erlauben, womit das Fernsehen konkurrenziert und eine grössere Zuschauermenge angezogen wird.

"Study the past, if you would divine the future"
Confucius (551 - 479 BC)

The future of sport depends on satisfying the needs of its supporters.

Facilities which accommodate large numbers of spectators and require significant long span engineering solutions can sometimes be bleak utilitarian spaces. It seems that the emphasis on the engineering solution can sometimes distract the design team from the real agenda of providing beautiful buildings which are comfortable to the spectators. We are now well into the third sports revolution. The first, the 'rules' revolution, allowed the informal games then played by small communities and the landed gentry to be organised and the rules of modern sports were formed leading to a peak of spectators watching live sport in the mid twentieth century. The second 'television' revolution started in 1937 allowing the broadcast of live sport around the world to hundreds of millions of armchair spectators. Now the 'entertainment' revolution, the third in this sequence means sport is big business, competing for our leisure time along with all other forms of entertainment. But what do today's spectators seek from our sports venues and how well do our engineering and design solutions satisfy their needs ?.

Stadia are places of worship, elation, disappointment & sometimes money making.

People attend live sport to be part of the occasion, to be involved and join the crowd, to be at one with the event and feel comradeship and togetherness, to be part of something for better or for worse. They want to be able to experience the day and say 'I was there' and then relive it with their friends for days afterwards. Some individuals may only ever experience this sense of being a part of a community, part of a sports brotherhood at an event and it is not surprising therefore how devoted they can become.

Just like any other established community they will be attached to their traditions, they will be resistant to change and they will be violent if that is what their 'community' dictates. The long term aim of sports design must therefore be to provide all the facilities their 'community' requires, retaining those features which are special to their established traditions and eliminate those factors which can lead to their anti-social behaviour. Our building solutions must therefore also reflect these traditions using structural solutions which are not just 'finely engineered good solutions' but which are also appropriate solutions, solutions which have character.

Stadia are places where memories are made.

The expectations of a typical spectator starts to build well before the event and on the actual day steadily climbs to a peak, in the ideal circumstances, at full time. The climax of the event is the final whistle or the crossing of the line and after that single moment the emotions start to wane until they depart and eventually arrive home still full of the success or failure of the day.

During this emotional cycle there is a time when they become one with the crowd, when crowd behaviour takes over as evidenced in the good times by the Mexican wave now commonplace in our stadia but in the bad time by mob and eventually riot behaviour. The turning point is rarely one isolated event but usually a collection of events, established attitude and often media build-up, it is often planned but once it starts, it is rarely controlled.

Our sports venues reflect society, introspective & protective, flamboyant & expansive.

Sports stadia and arenas accommodate all of society, they are as much a part of our culture as our town halls, churches and cafes and cater to a wide cross section of people who are attracted to the extremes of physical effort and the precision of the human body and mind. More recently our stadia have become hosts to people attracted to other pursuits mainly in the world of music, but religious and other followings are also well represented.

These multi-user buildings will contain the population of a large town for a few hours or a whole day and they must be able to cope with the depth of problems our towns and cities experience every day of the year because when the spectators enter through the turnstiles these problems are not left outside. In the same way our engineering and design solutions must also reflect the diversity of culture in our towns and cities where comfort standards of the population are improving. It is no longer acceptable in our sports venues to place columns between the spectators and the playing area or leave vast areas of exposed concrete to weather and discolour over the years. Our stadia must become more enclosed, more controlled environments with engineering solutions which are subdued but no less ingenious. Our future stadia and arenas will require a wider range of services than in the past and the latest technology.

Technology will be a major design factor in the future, seats will be warmed by low voltage trace heating elements eventually even in the concrete structure itself. Cooling can be by chilled air outlets below the seats from high pressure chilled pipes and local fans. Even the seats are changing being ergonomically designed with integral padding of soft plastics bonded in manufacture. These will give the effect of a padded seat but in a seamless and therefore weather proof shell. In one arm will be the sockets for plugging in personal headsets to listen to 'Stadium Radio' or 'Stadium Television' if the receiver is hired. As the average population size increases the seat spacing will also increase and allow for slight adjustment of seat backs to suit the individual spectator. Pockets on the back of the seat in front will contain the free 'Stadium Catalogue' advertising products on sale by post or from the Stadium Retail Centre. Items will be able to be ordered using the hand held receiver hired for the day, purchases can either be waiting for you at the shop at the end of the match or delivered to your seat at half time.

Concessions & Support Facilities

Support facilities are providing amenities for all the family to enjoy as well as other entertainment areas for those not committed to the event. They will eventually include every type of function from business centres to video game arcades, similar to the range of facilities found in an international airport. Attractions will be designed to encourage spectators to arrive early and then stay on afterwards perhaps even sleeping overnight in the Stadium Hotel. Tomorrow's stadia will be places of entertainment for the family where sport is the focus but not the complete picture. It will be possible for five members of a family to arrive and leave together but in the intervening period experience five different activities. While the parents 'see' the live game their children may 'experience' the live game in the virtual reality studio where images from the 'in pitch' cameras provide close immediate action.

Technology

It wasn't until the 1932 Los Angeles Olympic Games that technology was used to determine the result of an event for the first time and the Kirby Photo finish Camera took one hour to produce the result, technology is now essential to the smooth operation of a sport and the venue. We expect races to be timed to thousandths of a second, drug samples to be analysed in laboratories to particles per million and video play backs provided instantly. This is only the tip of the technological iceberg. In terms of stadia development we are already benefiting from more efficient construction techniques allowing opening roofs, moving seating tiers and soon moving playing areas. The Japanese are developing a robot building system which will work 24 hrs a day 7 days a week which would suit stadia's repetitive forms.

The line between natural grass and synthetic pitches will merge in years to come with developments in plastic mesh root reinforcement, plastic turf support and plastic granular growing mediums with computer controlled nutrient injection. Combined with this, new hybrid grass types require less light, grow faster and are far more robust. These advances in pitch design allow a greater number of different types of events to take place at the same venue, increasing spectator type and numbers and adding to the venue's financial viability. Viewing standards defined by sight lines for most stadia are now calculated on computer and the creation of three dimensional computer models of stadia developments will allow spectators to see exactly the view they will have from their seat at the time of booking. This can be extended to see the whole stadium in three dimensions allowing detailed analysis of the safety system.

Information and Communication

Major advances are being made in communication and information technology which can now provide the spectator with all the advantages of the television viewer at home. Information must be provided to the live audience to keep them knowledgeable about the event and the stadium they are watching it from. This knowledge entertains and extends the attention span which prevents boredom and irritation in the crowd. It can be used to attract spectators into the stadium early and keep them later reducing the pressure on the circulation system. This peripheral information is particularly important when we recognise that the period of actual play is often only a fraction of the full time period for the event. One set of tennis at Wimbledon for example which lasts 30 minutes may only have four or five minutes of actual play. A five set match which lasts two and a half hours may only have twenty to twenty five minutes of play. A football match which lasts one and a half hours may again only have twenty minutes of action leaving plenty of time for the attention to wander. The speed of play is also increasing with Guy Forget serving at the last Wimbledon around 132mph, Malcolm Marshall the fast bowler for the West Indies cricket team has been timed at 90mph and Nolan Ryan pitcher for the Texas Rangers sending the ball at an amazing 104mph when in ice hockey the puck often travels at over 90mph. This is information the spectators want to know for them to be contented and occupied.

Television

Television is in direct competition with live sport, coverage is usually a seamless mix of live action, recorded highlights, action replays, interviews, previews and postscripts which are presented to the viewer in such a way that it is no longer simply the relay of an event but the production of an entertaining show based on the event. The idea is obviously to attract a wider audience than just the dedicated sports fan who would watch anyway. This should also be the aim of the modern sports stadia and one answer is to compete with television on equal terms, offering information equal to the professional broadcasters at facilities which are as safe and convenient as our own homes. Replays and information about players and previous matches should be automatic but so should highlights of other events, statistics on the game, expert commentary and perhaps even advertising.

This can be achieved through 'narrow casting' by the stadium's own CCTV network, eventually by satellite, not just to one or two large video screens but also small personal receivers with screens a few inches across. These receivers will be part of the ticket price or on hire for the day and will only receive the stadium channel; probably with ear phones and eventually with interactive controls allowing a choice of information. Press the 'statistics' button and type in your favorite player's name and his career statistics will be displayed, press 'action' and type in the date of the match and see the highlights of his match winning performance two years ago. All this is possible now with 'narrow casting' radio used at Wimbledon for the first time this year and Arsenal investigating an infra-red system for next season. The horse racing industry is tending to lead the field, The Hong Kong Jockey Club has developed its own hand held betting device which is about the size of a television remote control and will allow the spectator to place a bet on any horse in any race and also provide the latest betting information, their next step is to show the race on the same piece of pocketable equipment. Stadium supervision and security by television and remote camera is already operating in major stadia.

The perfect stadium is the one in which your team never loses.

As designers we must recognise the problems of the past and apply to new solutions our vision, a vision which sees a bright future for this very special type of building. A vision which believes in progress in sport and its venues and an understanding of how modern technology can help to find that solution. Our stadia of the past have often been barren, unfriendly, under-serviced and brutal places which have not kept pace with our societies, these solutions are no longer acceptable and a new breed is about to emerge. Like the little boy who always listens to sport on the radio because the images he creates in his head always look better, we must hold in our minds our own vision of the perfect sports venue and do our best to achieve it for future generations of sports fans.