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National Indoor Arena of Birmingham Designed for Flexible Operation

Stade national couvert de Birmingham conçu pour de multiples utilisations

Nationale Sporthalle von Birmingham konzipiert für Mehrzwecknutzung

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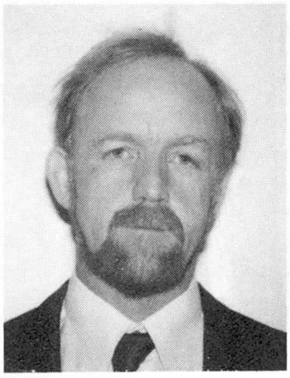

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SUMMARY

The National Indoor Arena was designed and built for the City of Birmingham to provide Birmingham with a first class venue for indoor sporting events and to accommodate a wide variety of non sporting events. The building complements the facilities at the International Convention Centre and the National Exhibition Centre. This paper describes how the key elements of the structure were designed to allow for flexible operation and to provide a low maintenance structure.

RÉSUMÉ

Le Stade national couvert a été conçu et construit pour la Ville de Birmingham afin d'offrir un lieu de première classe équipé pour accueillir des compétitions sportives en salle et un grand nombre d'événements non-sportifs. Le bâtiment est un complément aux facilités du Centre International des Congrès et du Centre National d'Expositions. Cet article décrit le projet des éléments clés de la structure en vue d'un fonctionnement flexible et d'un entretien minimum .

ZUSAMMENFASSUNG

Die Nationale Sporthalle wurde für die Stadt Birmingham entworfen und gebaut. Sie sollte Birmingham einen erstklassigen Zusammenkunftsort für die verschiedensten Veranstaltungen sportlicher und nicht sportlicher Art bieten. Das Gebäude ergänzt die Einrichtungen des Internationalen Kongresszentrums sowie des Nationalen Messegeländes. Dieser Bericht beschreibt, wie die Hauptelemente des Bauwerks entworfen wurden, um flexible Einsatzmöglichkeiten und ein Gebäude mit geringen Instandhaltungsanforderungen zu gewährleisten.

1. LOCATION

The National Indoor Arena is situated to the North West of the International Convention Centre and is bounded on the South and East sides by the Birmingham canal network and on the North and West sides by local roads. The site is split by the main London - West Coast Inter city rail line, the building forming an extension to the Monument Lane Railway Tunnel which passes under the International Convention Centre. A plan of the Arena and surrounding multi-storey car parking is shown in Figure 1.

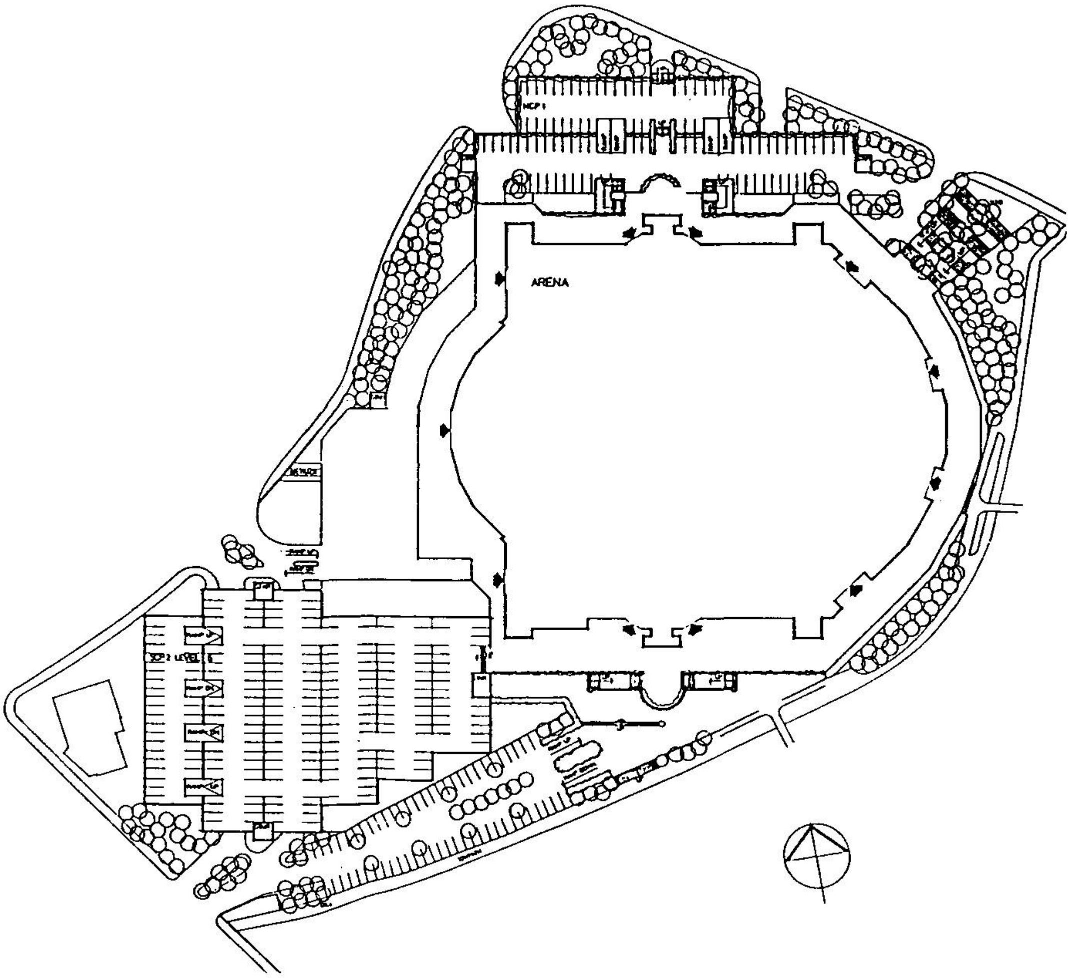

Fig.1 Plan of the Arena and car parks

2. OPERATION

2.1 Flexibility of Use

The National Indoor Arena is designed to be adaptable to enable many types of indoor sporting events to the held, typical uses are:

Athletics Tennis Badminton Squash Volleyball Basketball Boxing Wrestling Ice Skating Gymnastics Powerlifting Bowls

and many others.

The National Indoor Arena also caters for a wide variety of non-sporting events, typically:

Rock concerts TV shows (e.g. Gladiators) Conferences Opera Classical Concerts Company Meetings Exhibitions.

The National Indoor Arena is used in conjunction with the International Convention Centre to provide the facility for large plenary sessions at the NIA with smaller or breakout meetings and conferences at the ICC for major political, international and world conventions. If the requirement is large enough this can be expanded to utilise the facilities at the National Exhibition Centre as well as the ICC and the NIA as in the World Gymnastics held in 1993 and the planned Lions International Convention with 30,000 delegates later in the 90's.

2.2 Flexibility of size and seating arrangement

Events listed above range from major interest to minority sports and events requiring the maximum Arena floor space, e.g. Athletics, to those requiring the maximum seating capacity. This flexibility is catered for in two ways. First, the size of the area can be configured to suit the anticipated audience size for a particular event using full height curtains. Second, the seating arrangements can be varied between 3000 and 12000 seats and to suit end stage or central events as follows.

2.2.1 Fixed tiered seating

Approximately 6000 fixed, upholstered tiered seats are mounted from concourse level upwards on precast 'L' shaped concrete units spanning 10m between insitu concrete raker beams. The rake of this seating is designed so that for most events spectators have a clear view to within 1m of the edge of the performance area. Raker beam positions are used for aisles.

2.2.2 Retractable tiered seating

Approximately 1600 retractable tiered seats are mounted between concourse level and the arena floor. These seats comply with the same sight line criteria as the fixed tiered seats but have seat centre spacings of 500mm. These are mounted on laminated timber floors on steel subframes and are mounted in blocks of 5m width. Each block has a power operated retraction unit. In its retracted state each block is moveable by fork lift truck to allow for all round seating for appropriate events. Figure 2 shows a plan of the Arena with seating configured for a centre floor event.

2.2.3 Demountable temporary seats

A further 4000 demountable seats can be positioned in blocks on the arena floor for end stage events or similar.

2.2.4 V.I.P seating

Approximately 150 further seats are available in the VIP suites at high level on the South side of the area.

2.3 Ease of access

A major consideration in the design of a venue holding up to 12000 spectators is the need for easy access and particularly easy exit at the end of an event.

Reference to Figure 2 shows 12 exit doors at concourse level between the upper and lower seating tiers. Figure 1 shows 11 external exit doors (shown arrowed) exiting on to the external concourse surrounding the building. Figure 1 also shows 5 separate exits from the car park exiting on to 3 roads allowing rapid exit from the car parks.

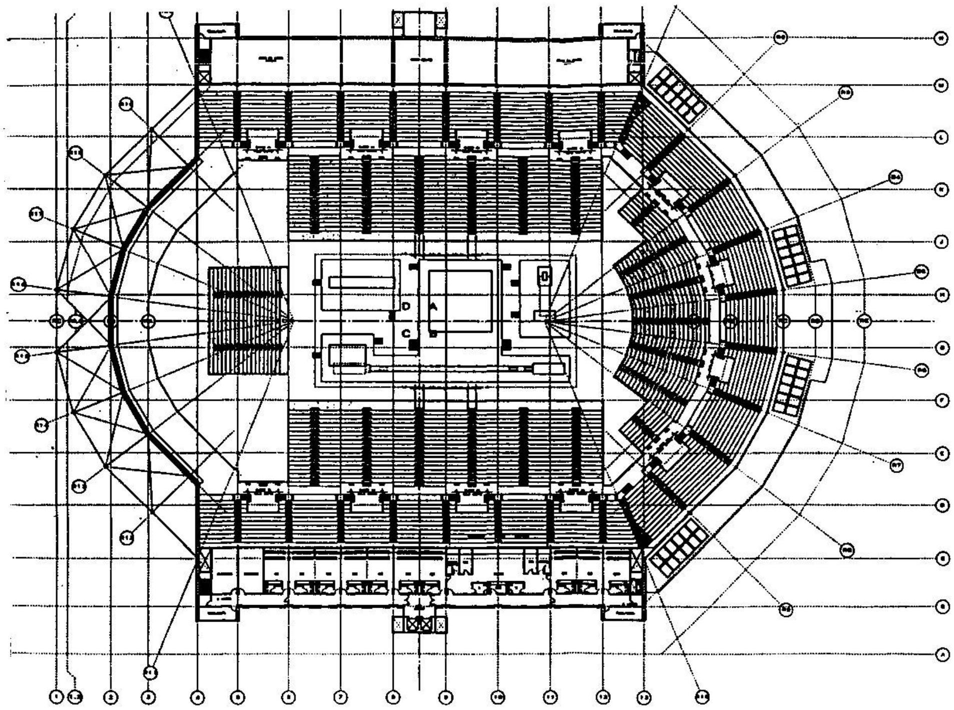

Fig.2 Plan of Arena showing tiered seating

3. STRUCTURE

The main elements of structure which allow the arena the flexibility to operate in the optimum format for a particular event and to be quickly altered from one format to another are the event floor, seating structure and roof.

3.1 Event Floor

The event floor is an insitu concrete slab, spanning over car parking, the main Birmingham - Wolverhampton Inter city rail line and the community sports hall. To allow rapid build up and take down for events and minimise 'dark time' the floor is designed to allow access for 38 tonne gvw heavy goods vehicles. The floor is a power trowelled slab, finished with a 6mm thick resin topping. The slab spans are 10m span two way spanning slabs over the car parking, 10m span one way spanning slabs onto 20m span downstand beams over the community sports hall and a non structural slab on 23m prestressed bridge beams over the railway line. All slabs are precambered so that during the design life of the building the flatness and slope of the floor complies with the limits for most sporting events. Specialist playing surfaces, include a 6 lane 200m banked athletics track are brought in.

The floor incorporates long jump and pole vault pits, and a full length duct, offset from the arena centreline to allow for cabling from an end stage event. These are infilled with concrete filled steel covers to match the surrounding floors when not in use. Apart from two rotational joints either side of the floor spanning over the railway track there are no other interruptions to the floor surface in the central area. Movement joints are situated around the perimeter of the floor under the retractable seating.

The floor is capable of taking drilled bolts and fixings for equipment, holes are made good after an event with epoxy filler to match the surface colour.

3.2 Seating structure

The seating structure has been described in 2.2 above.

3.3 Roof

The roof is a triple layer flat spaceframe, square on square on a modular grid of mainly 5m x 5m spanning 128m x 90m with an overall depth at the centre of 10m, reducing to 8m at the edges. Contained within the depth of the spaceframe are air handling ducts, cable trays and pipe work and walkways to access gantries for spotlights and television cameras.

Rigging to the roof is allowed for up to a maximum of 0.5 kn/m^2 over the central area and 0.25 kn/m^2 over the perimeter seating areas. Rigging is from M20 tapped holes in each bottom chord node. 0.5 kn/m^2 quoted above is equivalent to 12.5 kn per node if all the nodes in the central area are loaded. Higher loads are permissible on individual nodes if adjacent nodes are not loaded, up to 40 kn per note if every third node in each direction is loaded.

4. MAINTENANCE

The NIA is a low maintenance structure. In general, apart from offices, function suites and VIP suites the structural elements are exposed. Key elements are described below.

4.1 External and internal walls

External walls up to concourse low level are cavity walls with an external skin of stone faced concrete blocks with feature banding in engineering facing bricks. The inner skin is concrete blockwork with close textured dense concrete blocks in exposed areas, painted for ease of maintenance.

External walls to concourse high level are 60mm thick steel composite cladding panels with concealed lap joints fixed to a steel support system onto a reinforced concrete or blockwork inner skin. The external steel skin is hot dip galvanised at 275g/m^2 coated with 25 PVF2. The internal skin is galvanised and coated with 22 white polyester.

Internal partition walls are generally close textured dense concrete blockwork, painted for ease of maintenance.

4.2 Floors

Floors are generally insitu concrete with a power trowelled finish. These are painted in circulation areas. Seating tiers are high quality self finished precast concrete units. The event floor is described in 3.1 above.

4.3 Roof

The space frame roof structure members are hot dip galvanised with a zinc film thickness of 50 to 80 microns to provide inside and outside protection. Nodes are electroplated. Purlins, walkways, platforms and gantries are also galvanised.

The roofing consists of galvanised, coil coated trapezoidal steel sheets, approximately 30mm of acoustic mineral wool, a PVC vapour barrier, 100mm rigid mineral wool slabs and an outer surface of laminated, synthetic fibre reinforced, soft, high polymer PVC roofing membrane designed for permanent exposure in all climatic conditions.

Design of the Atrium Roof for the Imagination Headquarters, London

Projet de la converture de l'atrium du siège principal d'Imagination, Londres

Entwurf der Atriumbedachung des Imagination Hauptsitzes, London

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SUMMARY

This renovation design has transformed the new West End headquarters of a design and communications company. The revitalised building, comprising two parallel blocks, linked together with multi-level walkways through a central H-shaped atrium, has attracted critical acclaim from both the lay public and designers, winning several awards. This atrium roof cover is tailored to suit the complicated geometry of the old buildings.

RÉSUMÉ

Dans le West End de Londres, ce programme de rénovation a transformé le nouveau siège principal d'une société spécialisée dans la communication. Le bâtiment, rajeuni, composé de deux blocs parallèles reliés sur plusieurs niveaux par des passerelles enjambant un atrium en forme de 'H' a reçu les louanges du public et du monde professionnel et a obtenu de nombreuses récompenses. La couverture du toit de l'atrium s'adapte à la géométrie compliquée de ces anciens bâtiments.

ZUSAMMENFASSUNG

Der Bericht beschreibt die Renovierung des neuen Hauptsitzes einer Design- und Kommunikationsfirma im Londoner West End. Das Projekt umfasst das Zusammenfügen von zwei bereits bestehenden parallelen Bürogebäuden unter Verwendung von Fussgängerpassagen auf verschiedenen Ebenen und einer Membrandachkonstruktion, welche der komplizierten Geometrie der alten Gebäude angepasst werden musste und zu einer hellen Atriumsatmosphäre beiträgt.

Introduction

The existing building of Staffordshire, a rather drab, Edwardian Ministry property, presented an imposing five storey brick facade in a slight crescent off Store Street. Six to eight metres behind, and linked by a brick-built toilet block, stood a second four storey block, reconstructed after the Second World War. The unused and extremely bleak brick faced gap between the buildings absorbed most of the lighting penetrating the space. Ron Herron of architects Herron Associates, put forward proposals to transform the building, including demolition of the connecting link between the blocks and replacing it with skeletal metal bridges. (Fig. 1) This would emphasise the narrow gap which he tentatively suggested could be covered with translucent fabric wrapped down the ends to create a unified atrium space. With the construction of ground floor and partial first floor slabs within the atrium, the basements of the two buildings could be combined to create a very large floor area for many of the client's technical functions, including some recording studios and video production units. With alterations to the drab interior, an eighty year old building could then adequately provide the flexible, open plan space required by a high tech company, whose image is of prime importance. The client was most enthusiastic, and Buro Happold was appointed structural, fire and services engineers for the project.

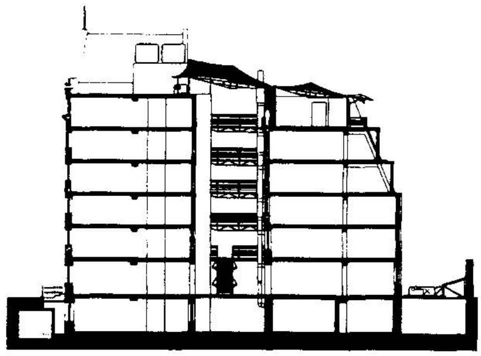

Figure 1 : Cross Section

Choice of Atrium Roof

Initially a glazed roof was considered to enclose the inner courtyard between the two buildings. However, not only was this an expensive solution requiring a substantial support structure, but the complicated geometry of the opening would have led to a very irregular structural layout. Use of a stressed fabric structure gave greater freedom to cover the space with the minimum of support structure necessary to accommodate the complicated geometry. Such a roof has a very low self-weight, the fabric weight of 1 kg/m^2 being only 5% that of glass. Typically such fabrics let in between 10% and 15% of visible light, sufficient to give a surprisingly strong light inside the atrium but also reflecting to the outside a sufficient part to avoid excessive heat gain.

The scheme which was developed provided a roof across the inner courtyard which continued over the roof slab of the rear building. This created a gallery space on the roof, thereby providing a useful and economical additional floor area. It was also planned to continue the roof down both ends of the newly created atrium to give an impression of a fabric wrapping to the building Christo-Style'.

Roof Structure and Detailing

Even with reasonable double curvature, for fabric to the stable under applied wind and snow loads it must be prestressed typically to 150 or 100 kg/m2. Under wind these stresses will increase to about 600 kg/m2, and must be resisted at the boundaries of the fabric. As it was felt that the existing structure would not be able to resist such a level of force between the buildings, a lattice strut supporting structure was developed across the building. A gutter and beam supported on columns divides the membrane on the line between atrium and gallery, and glazing on this line separates the stress (Fig.2). The lattice struts are also supported on this line and a second set of struts span over the gallery to the rear elevation. The whole roof is anchored in the transverse direction to the front block only (Fig.3).

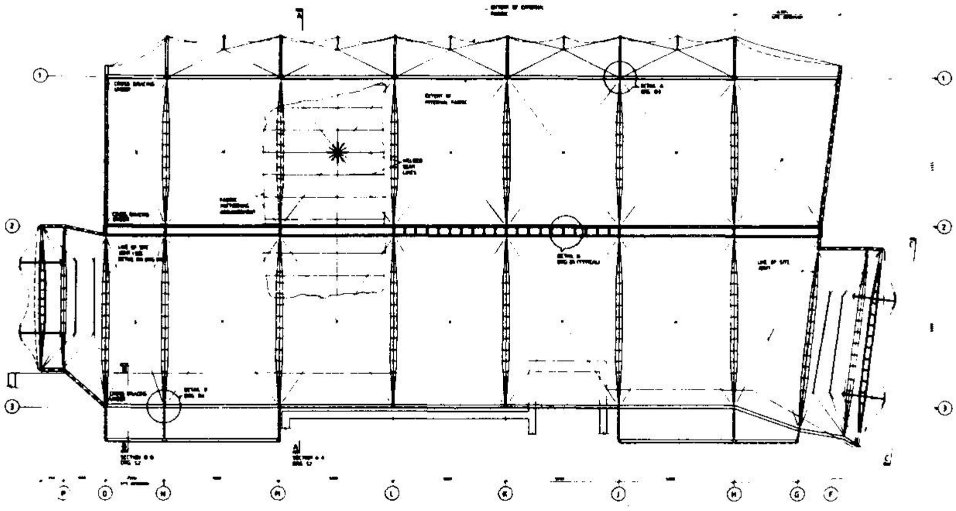

Figure 2 : Roof plan of Atrium

The fabric is supported by umbrellas on flying masts which stand on bridle cables running back to the ends of the struts. Generally there are single masts in each bay but at the ends the masts are in pairs to achieve the required shape and level of support to the membrane. At the ends of the atrium the fabric is attached to a triangular lattice truss designed to take the fabric forces in bending. Beyond the gallery the fabric oversails the glazed wall, creating a canopy. This narrow band of fabric is given shape by a series of flying struts which push the edge of the fabric down between cantilevering arms.

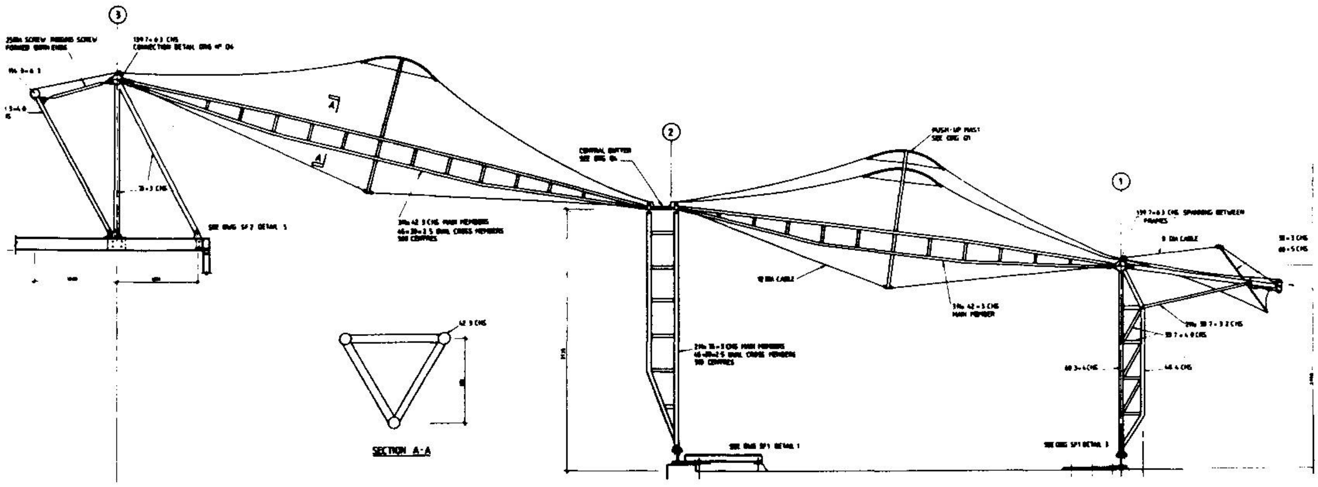

Figure 3 : Section through Atrium roof

Details had to be designed to enable the fabric to be prestressed, and had to include means of pulling or pushing the fabric into shape and maintaining it under load. Where it is attached to steel perimeter members the fabric passes over the structural tubes. Small steel tubes running inside pockets in the membrane are screwed down to small blocks on the side of the perimeter tubes, the screws providing adjustment at the edge to ensure a good fit of the fabric. Final tensioning of the fabric is applied by jacking up the flying masts, achieved by turning a threaded shaft which passes through the lower support point.

The normally flexible fabric is stiffened to resist loads by virtue of the double curvature and prestress induced by the boundary and support conditions. To achieve this condition, the fabric has to be accurately tailored to the prestress geometry, determined by a computer form-finding process where stresses are specified in the membrane surface which then moves to its equilibrium position (Fig.4). The resulting numerical model is used for load analysis and finally to produce the cutting patterns to which each panel of cloth is tailored. This total process is carried out in-house using specially written software so allowing the designers total control of the fabric shape and detailing essential for such a complex roof.

Fabric was patterned with 1m wide cloths cut from a 2m wide roll to conform to the well developed curvature of the roof. Cross seams were introduced at the mast heads to create the required dome shaping in those areas. Stretch under load was measured by laboratory tests and fabric was consequently adapted by 0% in the warp and 2% in the fill direction to compensate.

During the design stage, consideration was given to using either PTFE-coated glass cloth, silicone-coated glass cloth or PVC-coated polyester for the atrium roof membrane. It was difficult at this stage to obtain approval for PTFE glass because of concern over possible production of toxic fumes during a fire. Such worries have since been dispelled and it would probably now be permissible to use this long lasting material. As an alternative, silicone-coated glass is difficult to obtain, consequently very expensive, and problems would have arisen in obtaining an adequate supply for the project. Consequently it was decided to use PVC-coated polyester fabric with fluoropolymer lacquer on the grounds of cost and expediency.

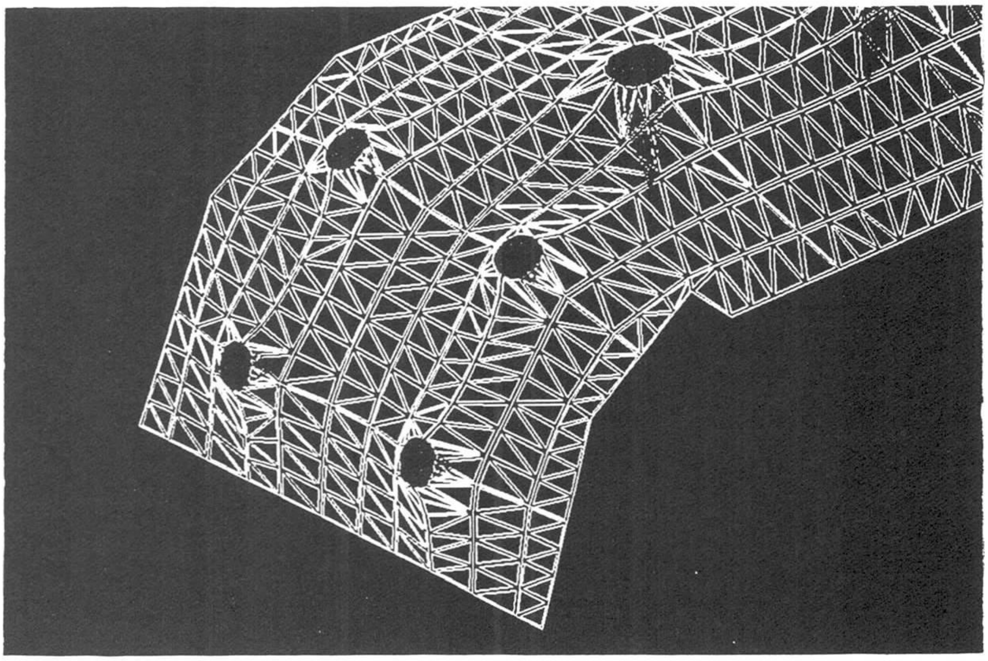

Figure 4 : Computer analysis of roof form

Fire Engineering Considerations

Buro Happold was also commissioned to carry out a fire appraisal of the spaces beneath the membrane roof of the atrium and roof gallery. A report was submitted to the local authority - London Borough of Camden - and its recommendations formed the basis for the fire strategy in these spaces.

The atrium was intended for transient use with a low fire load at least four storeys below the roof. It does not form part of ny fire escape route as the front an rear buildings both have their own independent means of escape. Furthermore, the buildings on either side are isolated by half hour fire glass in all windows and fire doors. In the event of an atrium fire, optical and smoke detectors will trigger an alarm and bring into action the smoke ventilation system. This comprises low level inlet panels which open automatically in the vertical side screens to the atrium, and high level louvres with extract fans.

The roof membrane itself does not support combustion, and when impinged on by flame does not drip. It is merely burnt and vaporised locally around the impinging flame, and is then self-extinguishing. If air temperatures in the atrium reach around 250°C it is anticipated that the PVC adhesion at welds would begin to reduce and seams would slide apart and open releasing the pre-stress, so improving ventilation of smoke. If temperatures then rise to much higher levels, and flames reach the membrane, local burning will take place and ventilation would be improved even further. With such a high level of ventilation assured, internal temperatures could not reach those more common in restricted compartment fires.

Whilst this type of membrane has been in use in structures worldwide for at least thirty years, it had not been used in such a way before in London or indeed anywhere in the UK. Despite the arguments of the fire report, Camden Building Control were not willing to set a precedent by making a favourable decision on the use of membrane for the atrium covering.

On application to the Department of the Environment for a determination there was no hesitation in accepting the use of the material for a roof. However, as no large scale tests are known to have been carried out to simulate the behaviour of vertical walls of fabric was not acceptable without demonstration. As there was insufficient time to conduct such tests, the intention to continue the roof membrane down on either side of the atrium to second floor level was deferred. As an alternative, the side walls are designed as more conventional fire panels supported on a steel grillage spanning between the two buildings.

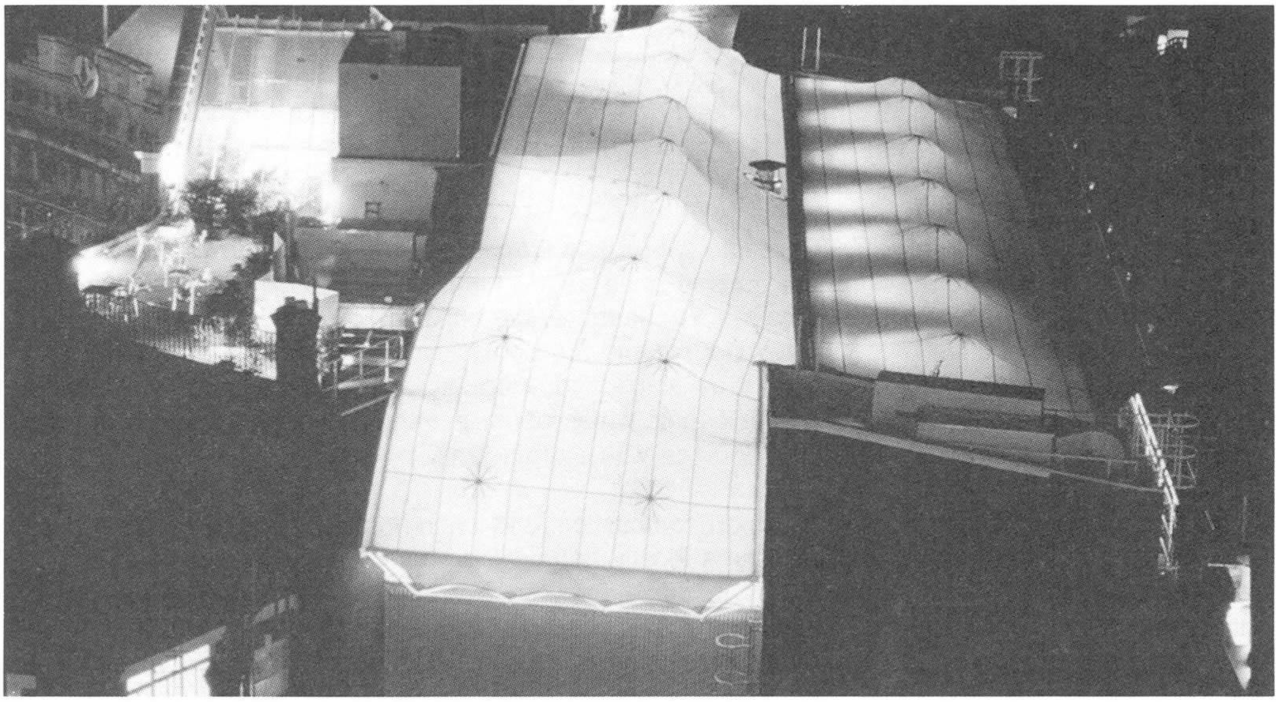

Figure 5 : Atrium at night

Conclusion

An extremely rigorous £5m refurbishment programme was thus completed in less than one year, with contractor R M Douglas Construction working on upper and lower floors simultaneously. In the words of the judges at the British Construction Industry Awards Staffordshire House (Fig. 5) is a 'landmark scheme to bring new lift to old buildings against which other refurbishment projects are likely to be judged for years to come'.

Straight Tensioned Cable Roof Structures

Structure de câbles rectilignes en tension pour des toitures

Dachtragwerke mit vorgespannten geraden Drahtbündeln

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SUMMARY

The use of straight tensioned cables with fabric of foil cladding can result in translucent roof structures with useful environmental properties at very competitive costs. This paper describes two projects designed on this principle one of which is now under construction.

RÉSUMÉ

L'emploi de câbles rectilignes en tension, supportant des toiles, peut contribuer à des toitures translucides aux propriétés intéressantes et à des prix très compétitifs. Ce document décrit deux projets conçus selon ce principe, dont l'un d'entre eux est actuellement en construction.

ZUSAMMENFASSUNG

Der innovative Einsatz von Gewebefolien in Verbindung mit Seiltragkonstruktionen für transparente Dachtragwerke ist als eine sehr ökologische Tragwerksoption anzusehen. In diesem Beitrag werden zwei Projekte beschrieben, bei denen diese Tragwerksvariante eingesetzt wurde. Eines dieser Projekte befindet sich zur Zeit im Bau.

INTRODUCTION

The now traditional constructional systems for cable roofs use one set of cables to carry uplift loads with a second set of cables to carry down loads. These cables are either arranged at right angles to each other to form a surface with anticlastic curvature, or the cables can be separated vertically as in a cable truss. There are considerable advantages in using single straight cables, particularly in association with fabric or foil cladding.

Compared with a two way cable net, one set of cables is eliminated along with the cross clamps and the anchorages.

Whether the load is upward or downward the cable tensions are in the same direction which can be a great advantage if the tensions are taken by a funicular arch or ring beam.

Connections to the foil or fabric cladding can be greatly simplified.

Taken together these benefits can result in very economical roof structures. This paper describes two such designs, one of which is now under construction.

Harlow Velodrome

The requirement was to provide a 500 metre oval covered cycling track with seating for some 3000 persons. The velodrome would be used by the existing cycling club for training and inter-club competitions. Occasionally international meetings would be held. The potential income from this activity was low, but additional income could be generated by using the hall for other sports events (e.g. tennis) and for music concerts. Three tennis courts could fit in the central area but the track caused problems with seating and access for concerts. The facility was to be jointly funded by the cycling club who were to gain some capital from the sale of their existing site and the local authority. For the facility to be viable the capital costs had to be minimised.

The proposed structure was a cable roof with straight cables anchored to a horizontal ring beam and supported by a funicular arch (Figs. 1 & 2). The ring beam was 90 metres by 120 metres to be constructed in precast concrete. It sat on top of a series of A-frames, the sloping leg of which carried the seating outside the cycle track. The cable forces were largely taken directly into the A-frames, with some residual compression and bending in the ring beam. The supporting arch was a four chord tubular steel truss spanning 160 m on to concrete abutments which were tied together by a ground beam.

The cable lines were at 10 metre spacing, the longest cables being 50 metres long. They were designed to be prestressed to 50kN under zero external load conditions. Each cable line consisted of a 42 mm diameter wire rope cable. Under extreme loads the tension in the longest cables would rise to 500 kN.

Because the cables were designed as prestressed even under zero load conditions they are able to provide lateral stability to the spine arch. This resulted in considerable economies in the design of the arch.

The cladding was to be PVC coated polyester fabric panels. The panels were double layer, the outer layer of type III PVC/PES cloth being prestressed and carrying the loads. The inner layer of a lower strength PVC/PES cloth acted as a liner which provided an air gap and would be unstressed. The two layers were to be made up together and finished with a roped edge which slid into grooved aluminium extrusions on the cable lines.

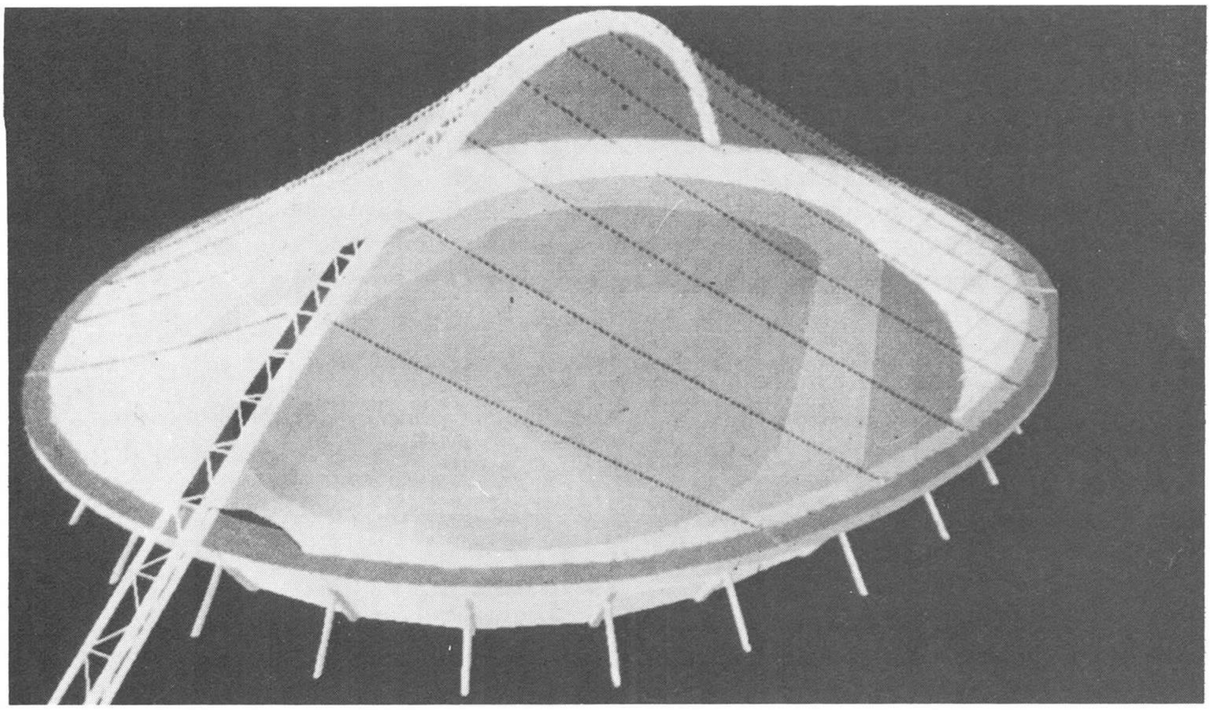

Fig. 1. Computer model of Velodrome

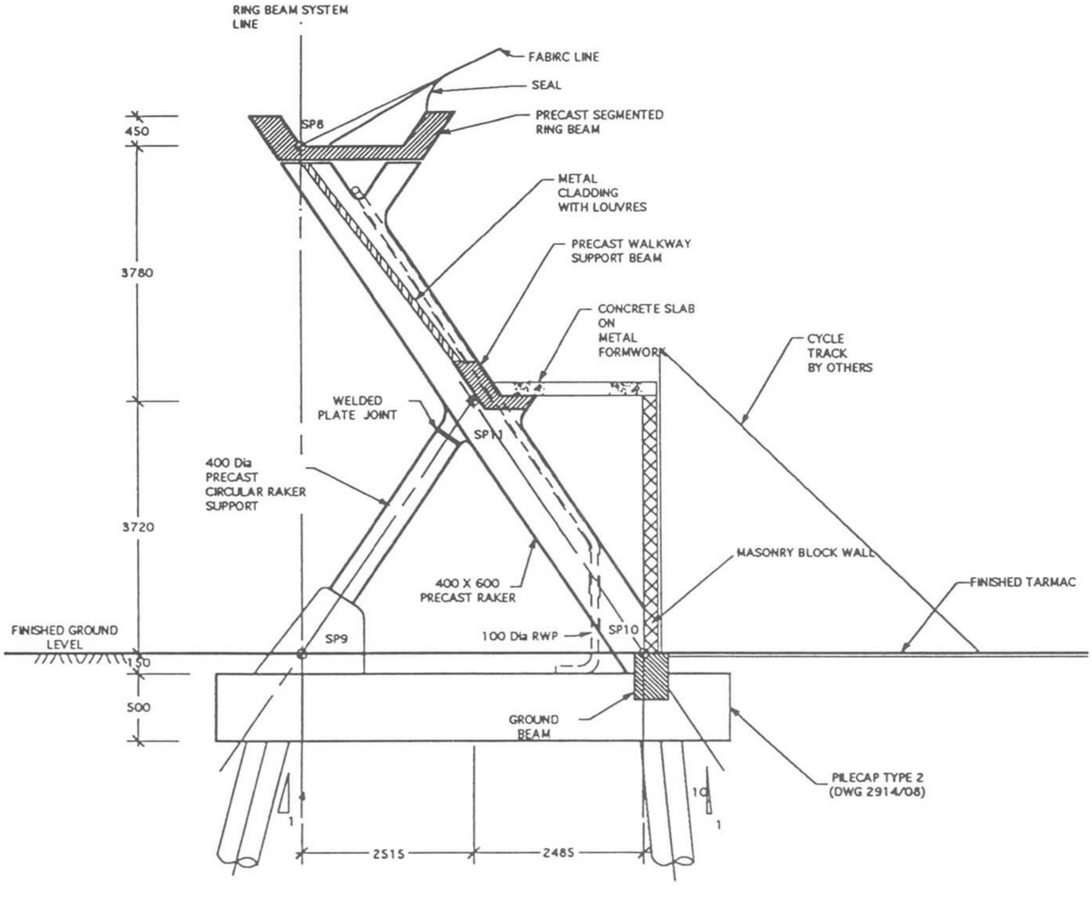

Fig. 2. Structural section

Tennis Halls

The second structure is a 10 court tennis hall for a tennis and health centre with a plan area of approximately 6000 square metres. Again there was a strong financial pressure to minimise costs. In this case the structure was being compared with standard portal frame halls normally provided as design-build packages. The objective was to use as cladding triple layer ETFE foil inflated cushions. This roofing system offers a wide range of translucencies, together with a reasonable degree of insulation by virtue of the triple layer construction. The foil cladding is relatively expensive and the system could only be made viable if the structure costs were minimised.

The cushions are supported on pairs of 18 mm diameter cables at 3 metre centres supported from a longitudinal ridge cable which is in turn supported by opposed external masts and ties (Fig. 4). Each of the two ridged structures is 78 metres x 36 metres, separated by a steel portal frame spine. The entire structure is stabilised via a system of external ties and ground anchors. Under permanent load conditions the parallel cables are stressed to 20-50 kN, rising to 100-150 kN under applied load.

The ETFE foil panels are approximately 3 metres x 20 metres. The outer and inner layers are 150 μm and the middle layer is 30μm in thickness. The cushions are pressurised to 300 to 400 Pa. This pressure causes the initially flat foil to stretch and curve out so that the cushions have a rise of about a tenth of the span. In this condition they are able to resist full wind and snow loading. The foil panels are connected to the cables via an aluminium extrusion clamping system which allows for full movement of the cables under wind loading and thermal variations (Fig. 3). The foil edges at top and bottom are terminated on aluminium edge channels which house air supply hoses. Differing from the connections of foils in rigid frame roofs, the details for the foil to cable connections have been designed to take into account the movements of the cables. Rotations in the end connections to perimeter steelwork necessitate the use of sliding panels at the edges to accommodate for in-service movements and easy installation.

ETFE foils have been commercially developed for construction purposes over the past 15 years, primarily in Germany and the UK. The leading manufacturing firm is Vector Foil Gmbh. Buro Happold have been involved in developing lightweight foil enclosure designs since the early 1980's when cushion construction was first considered for several large enclosure projects including the design of a covering for a township in Northern Canada. Currently, foil cushion roof are being used in applications where controlled environments, but high sunlight transmission is desirable, such as sporting halls, swimming pools, or greenhouses. However, advances in manufacturing processes, detailing and general acceptability have opened up wider possibilities in the use of foil roofs. Due to their insulation and high light transmission properties, ETFE foil cushion constructions have been increasingly introduced as economical alternatives to glass panel systems and planar glazing for roofing enclosures such as atria, sports halls, swimming pools and retail areas.

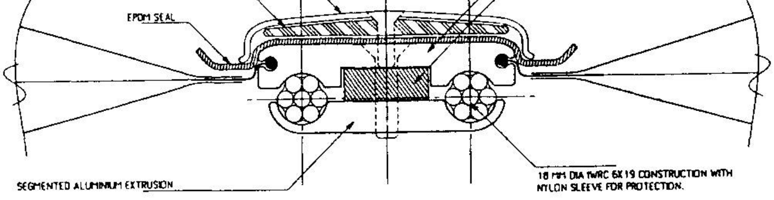

Fig. 3. Flexible extrusion connection detail

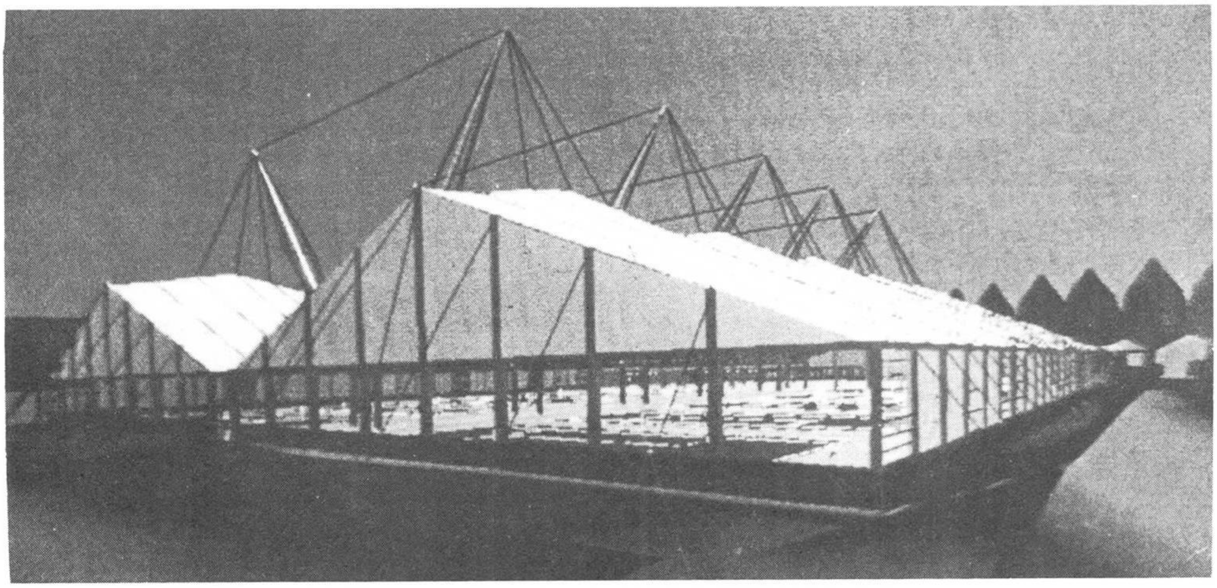

Fig. 4. Computer modelling of tennis halls

Analysis and Design Considerations

Under static load the cables stretch with increasing tension which allows the curvature to increase until equilibrium is reached. The deflections under peak loads may be a metre or more. To calculate the forces in a single cable it is possible to solve the nonlinear equation by hand or with a calculator. However it is easier to use a nonlinear computer program such as TENSYL which gives the forces in all the structural members. Such a program can also be used for the form finding stage in which the geometry of the structure is optimised.

Of greater interest is the behaviour under dynamic loads as from wind. For small oscillations where the cable tensions do not vary significantly the natural frequency $= \sqrt{(T/w)}/2*L$. For the tennis hall example this results in a frequency of ≈ 3 Hz if w is taken as the self weight of cables, clamps, foil, etc. At this frequency there is little energy in the wind. The response is affected by damping from the added mass of the air, from acoustic energy given to the air at a distance from the roof, and from the material properties of the cables and cladding. The result of this is that a resonant response should not occur. This concurs with the experience of similar flat tensioned structures e.g. marquee tents. However the roof will move a lot and this must be taken into account in the detailing.

As indicated above, the structural form resulting from the adoption of straight cables is simplified when compared with alternative two-way cable nets. In a two-way cable net the formfinding is a complex iterative process, in which the individual link lengths of the net must be adjusted to find an equilibrium form. In the proposed forms using prestressed straight cables, the cable forces dominate the resulting form and allow rapid determination of the shape of the roof given suitable boundary conditions.

In the case of the Velodrome, the A-frame, ring beam and supporting arch define the roof enclosure and the cables are essentially slightly curved generators of the form between these boundaries. Both vertical and horizontal curvatures are controlled by the amount of prestress in the cables. The fabric acts primarily to distribute loads to the main cables, which can lead to high lateral forces in the event of failure of a single panel of fabric. In roof forms such as this it is also possible to extend the structural system such that the fabric panels are retractable along the line of the main cables (Figs. 5 &6).

In the case of the Tennis Halls roof, the central spine and external tie-backs define the boundaries. The roof shape is controlled by the interaction of the single ridge cable and its supporting mast with the parallel cables at 3 metre centres acting as stringers for the foil cushions. The inflated cushion system results in high lateral forces at the clamped edges, which must be considered, particularly in conditions where the loading is not equalised on both sides of the main cable.

Both of the above structures have been successfully assessed and analysed using the TENSYL computer software developed by Buro Happold for the design and patterning of tensioned fabric structures. This integrated system ensures that the designer has full control of the analytical model and the system geometry at all stages of the design and fabrication processes. The provision in TENSYL of cable elements under force control (i.e. the behaviour of the cable is governed by a defined tension overiding its elastic properties) enables rapid assessment of the effects of changes to the prestress levels in the cables. The final scheduling of cables and boundary geometry is aided by the facilities provided in TENSYL for the calculation of linear and angular geometries.

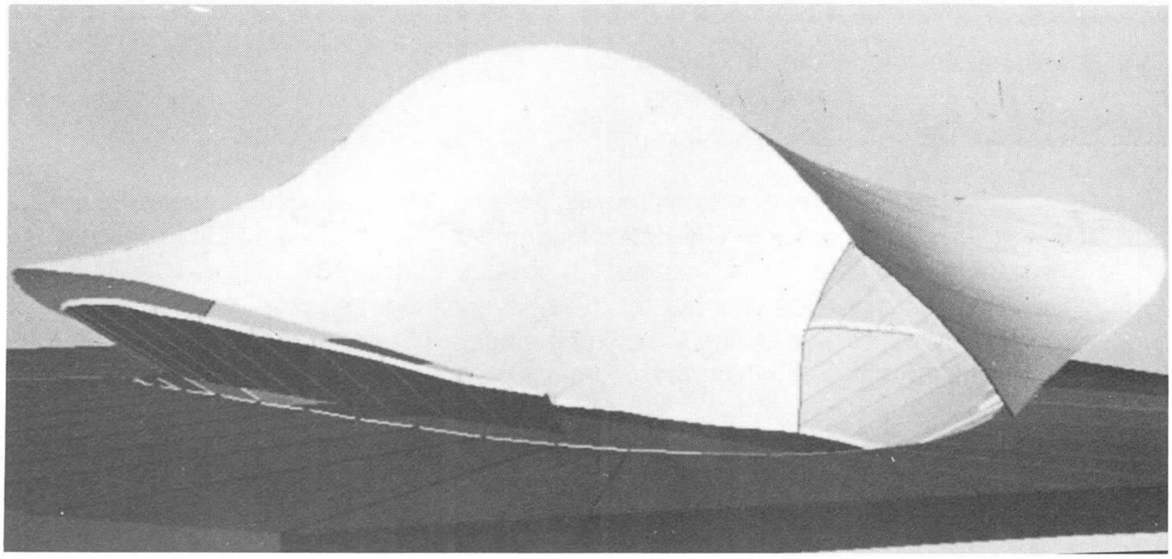

Fig. 5. Computer model of Stadium Roof with fabric closed

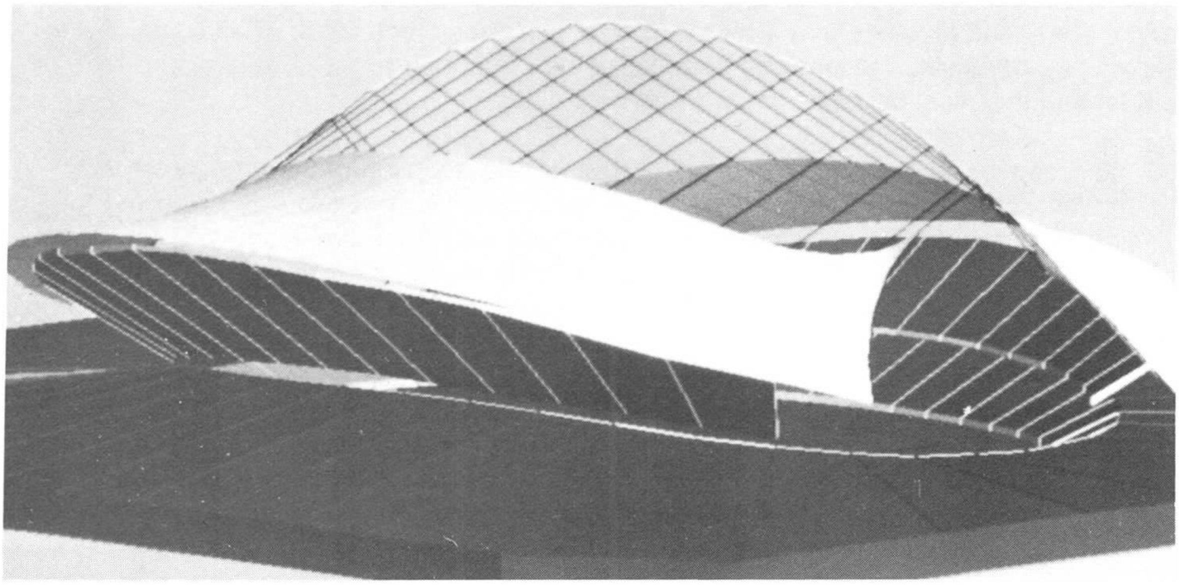

Fig. 6. Computer model of Stadium Roof with fabric retracted

Toiture de l'hémicycle du Parlement européen à Bruxelles

Bedachung der Europaparlamentshalle in Brüssel

Roof of the European Parliament Hall in Brussels

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En septembre 1993, la première session extraordinaire du Parlement Européen s'est déroulée à Bruxelles inaugurant ainsi le nouvel hémicycle du Parlement Européen anciennement dénommé Centre International de Congrès.
Cet article est consacré à la description de la toiture de ce nouvel hémicycle.

1. IMPOSITIONS POUR LA TOITURE

Pour couvrir la nouvelle salle hémicycle, le bureau d'études de stabilité (association momentanée Verbeeck Fraiture Dumont et *b* GROUP) a établi avec l'aide de l'architecte (association momentanée Bontinck, CRV, Vanden Bossche) un cahier des charges reprenant les différentes impositions pour la couverture.
Ces impositions étaient de plusieurs types : impositions de forme, d'étanchéité à l'air et à l'eau, d'isolation thermique et acoustique, d'accessibilité, de résistance au feu et également bien entendu de résistance structurelle et de déformabilité.

1.1 Impositions architecturales

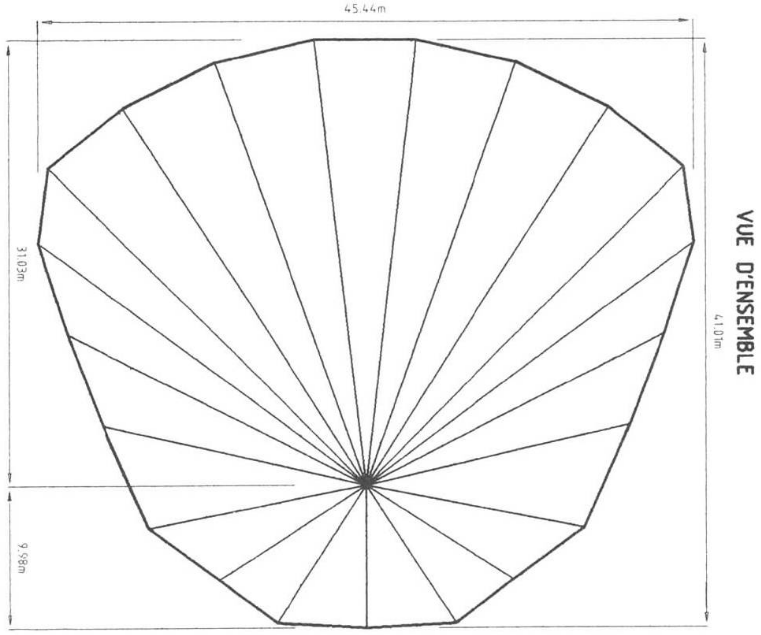
La surface supérieure de la couverture était imposée suivant un profil particulier pour un bon écoulement des eaux pluviales . De plus, la toiture étant visible des bâtiments adjacents qui la surplombent, son esthétique était primordiale pour les architectes qui ont choisi le zinc comme matériau de couverture et d'étanchéité.
La surface inférieure devait, elle, être profilée pour que le faux plafond suspendu à la structure assure une acoustique satisfaisante pour la salle ; de plus, le faux plafond devait avoir une forme en concordance avec le plancher de la salle, ainsi, le point haut du faux plafond devait se situer au-dessus du siège de la présidence et donc de manière tout à fait excentrée.
Le volume intérieur de la toiture devait, quant à lui, être aisément accessible notamment pour assurer la surveillance et l'entretien des nombreuses techniques contenues dans ce volume et en particulier pour pouvoir assurer la maintenance de l'éclairage de l'hémicycle.

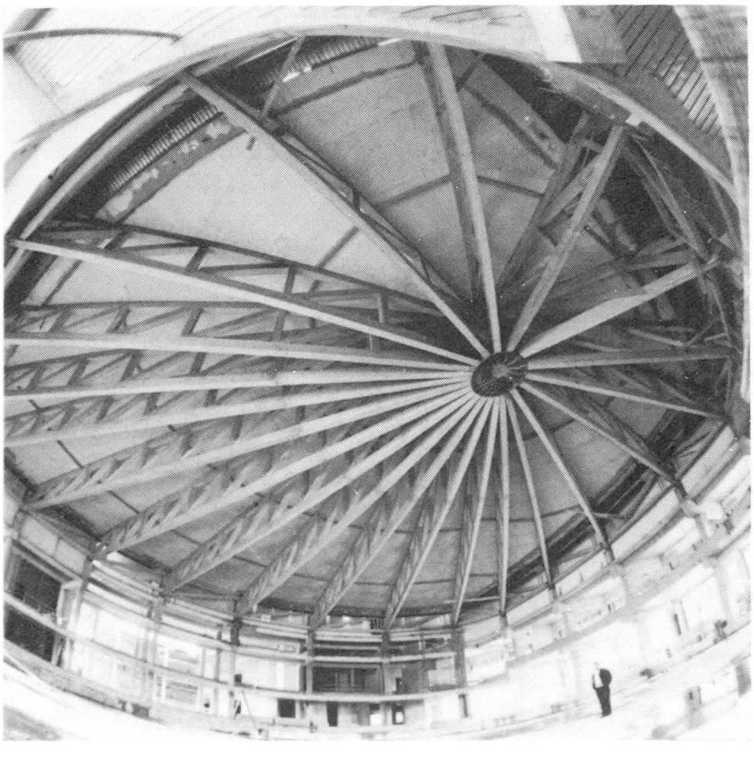
1.2 Impositions structurelles

La position des 22 points d'appui de la structure était évidemment imposée de même que les réactions maximales sur ces appuis . Ces 22 points d'appui étaient répartis dans 3 zones distinctes du bâtiment séparées par des joints de dilatation ; le dispositif d'appui de la structure de toiture devait évidemment tenir compte de cette particularité.
La structure de la toiture était imposée en bois lamellé collé.
La portée de la toiture est de l'ordre de 42 mètres, la surface couverte étant de +/- 1400 m2.
L'ensemble de la structure de toiture devait avoir une résistance au feu de 2 heures.
La surcharge à reprendre par la structure de toiture est de 3000 N/m2 en plus de son poids propre ; cette surcharge reprend notamment le poids propre des panneaux de couverture en bois, le zinc avec son voligeage de ventilation, le faux plafond de la salle, un plancher type caillebotis pour la circulation dans le volume de la toiture, les surcharges de techniques comprises dans le volume de la toiture et enfin les surcharges climatiques.

2. SOLUTION RETENUE POUR LA TOITURE

La solution retenue pour la toiture est celle de l'association momentanée Prefalux Lamcol calculée par le bureau d'études Van Wetter.
Il s'agit d'une structure tridimensionnelle de poutres planes en treillis en bois lamellé collé.
21 fermes en treillis lamellé collé, toutes différentes les unes des autres, reposent sur 21 des points d'appui périphériques et convergent vers un noyau métallique situé au droit du point haut de la toiture, donc de manière tout à fait excentrée. De cette façon, les fermes ont une longueur projetée en plan variant de 8.91m à 30.05m.
La hauteur des fermes est de l'ordre de 2m sur les colonnes d'appui et de 3.5m au droit du noyau métallique.
Les membrures supérieures et inférieures des poutres en treillis ont une section de bois lamellé collé de 28 x 45 cm et même à certains endroits de 28 x 58 cm ; elles sont réalisées autant que possible par une pièce de lamellé collé unique sauf les fermes les plus longues qui possèdent un joint d'assemblage pour une question de transport.
Les entretoises et les diagonales du treillis ont une section de 28 x 29 cm.
L'assemblage des diagonales et des entretoises sur les membrures est réalisé par le système breveté BSB.
Les différentes poutres en bois sont assemblées par l'intermédiaire de tôles en acier perforées pour permettre le passage de goujons en acier. Les tôles sont placées dans une fente réalisée au coeur de la pièce de bois ; de cette façon, elles sont invisibles et protégées contre l'incendie.
Le noyau métallique a un diamètre de 2.15m et une hauteur de 3.68m ; il est réalisé en tôle de 12 mm d'épaisseur


45.44m
31.03m
9.98m
41.01m
VUE D'ENSEMBLE



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Long-Span Girder Using Pre-cast Concrete Beams

Poutres à grande portée constituées d'éléments préfabriqués en béton

Entwurf weitgespannter Träger aus Betonfertigteilbalken

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1. INTRODUCTION

This report presents the design of the roof of Matsuyama Community Center. It also includes the results of the research performed for the deflection due to time of the long span girder used in this roof during and after construction. The roof is placed in an area of 32 x 32 m and, is composed of pre-casted concrete members that are connected and then post-tensioned.

2. OUTLINE OF THE STRUCTURE

Fig. 1 shows the two types of pre-casted girder members used. This roof is composed of 48 pieces of pre-casted elements which consist basically of two crossed beams connected to each other. Post-tension is applied in both, X and Y directions. Concrete strength is 450 kg/cm^2.

3. TEST MEASUREMENTS

3.1 Strain-Time story

Fig. 2 shows the variation of strain when post-tensioning, after taking off the roof supports (jack down) and, 1.5 month after jack down. Here, it is possible to observe that the concrete compressive strain due to tension in the cable, the flexural strain after jack down and, the flexural strain due to creeping increase with the time. The compressive strain when post-tensioning is 179 μ (average of M-1, M-2 curves) and is 205 μ (average of M-3, M-4 curves) from test results. And the compressive stress is 54 - 62 kg/cm^2. The compressive stress is 57.4 kg/cm^2 from analytical calculations. Bending occurs at jack down and the strain produced at the lowest part of the center of the roof is 130 μ. The compressive stress condition remains after jack down. And tension does not occur even if load is applied, thus having excellent post-tensioning conditions.

3.2 Deflection-Time Story

Fig. 3 & 4 show the deflection when applying post-tension, at jack down, and after creeping. The vertical deflection of the central part is zero before applying tension and is 2 mm when applying it. The deflection of the central part is 12 mm after jack down, and the deflection due to creeping is 32 mm three months after jack down, which is approximately 2.5 times the deflection at jack down. The span-deflection ratio is 1/750. And the final maximum deflection due to creeping is 40 mm, remaining almost constant from then on.

3.3 Horizontal Displacement and Rotation

Horizontal displacements are produced as shrinkage occurs after post-tensioning. These are 4.14 mm for X direction and 4.37 for Y direction. The horizontal displacement calculated analytically is 5.2 mm. Considering a 10 % of losses due to friction. The external girders rotated about 1/400 after jack down.

4. CONCLUSIONS

The test results permitted us to have a clear understanding of the processes of post-tensioning, jack down and creeping.

(1) The concrete compressive stress calculated analytically is close to the value obtained from the tests.

(2) There is an excellent post-tensioning condition as tension is not produced at the lowest part of the center of the roof girder.

(3) There is no problem due to deflections, including creeping effects, as the span-deflection ratio is 1/750.

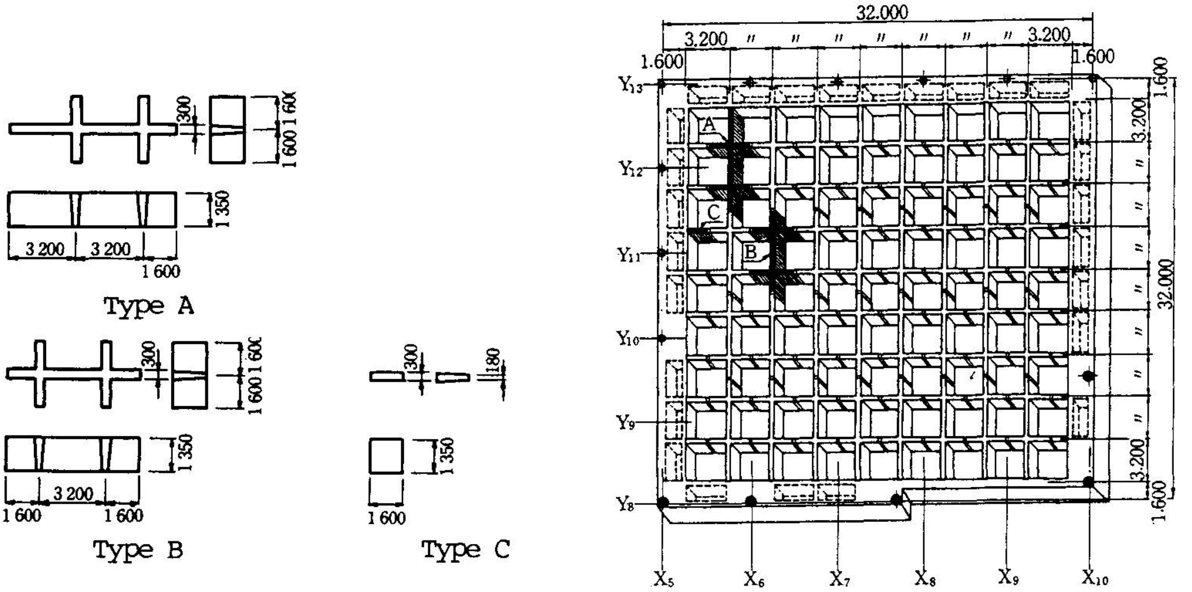


Fig. 1 Types of Pre-casted members and plan of the roof

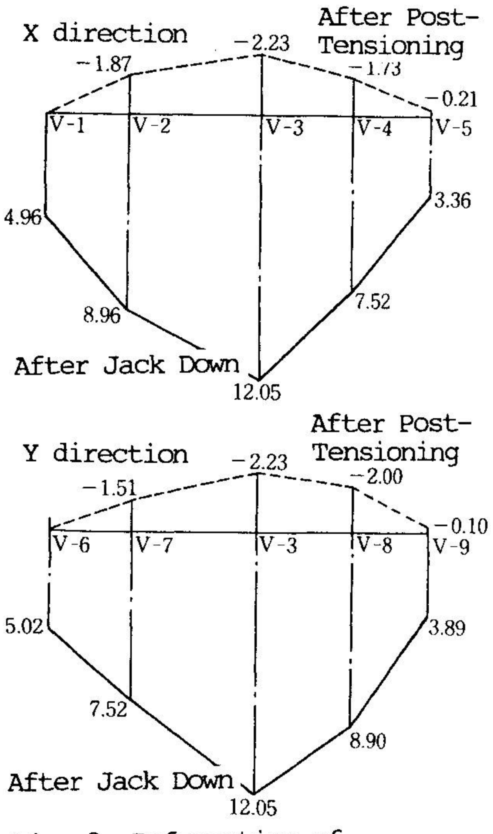


Fig. 3 Deformation of the long span girder

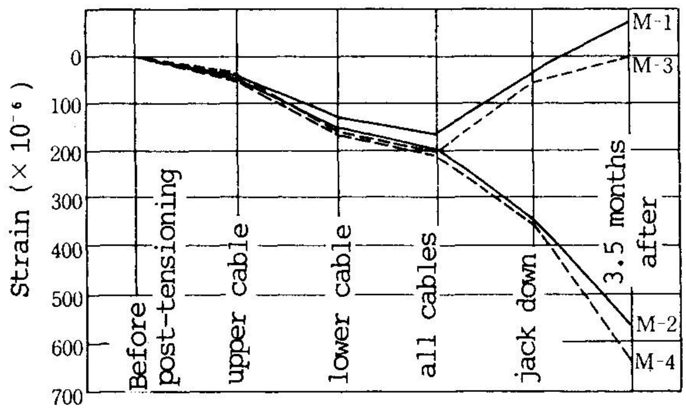


Fig. 2 Strain Time-Story

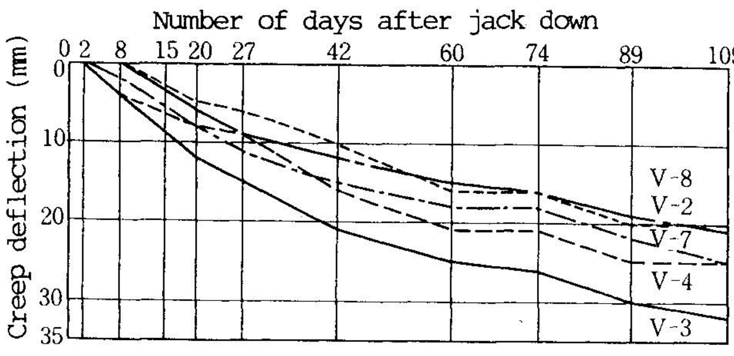


Fig. 4 Creep deflection after jack down

Sports Hall "Kreuzbleiche" in St. Gallen, Switzerland
Halle de Sport "Kreuzbleiche" à Saint Gall, Suisse
Sporthalle "Kreuzbleiche" in St. Gallen, Schweiz

Hansruedi SIGNER
Civil Eng.
St. Gallen, Switzerland

1. DESCRIPTION OF THE BUILDING

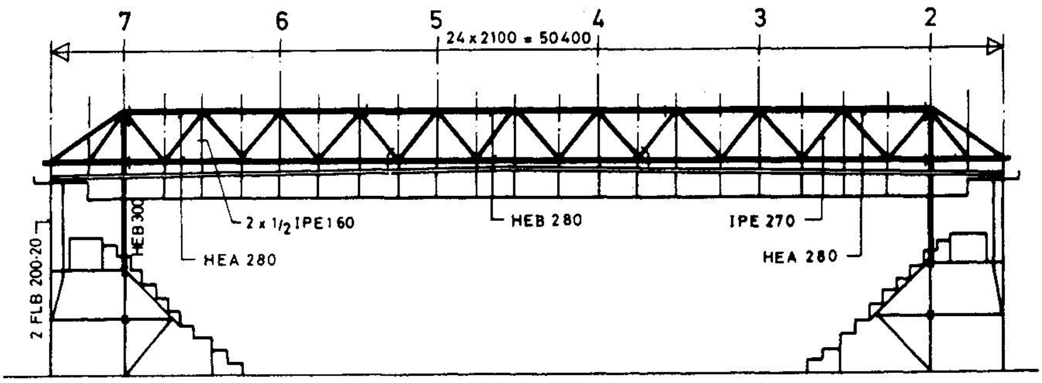
The sports hall (triple sports and aparatus gymnasium) was designed for both sport events and functions. The hall is also used by a nearby vocational school during the day and by sports clubs in the evenings and at weekends. It is complemented by a two-storey car park and outdoor parking facilities. Conveniently located for the school it is also easily accessible by public transport. As a result of the selected positioning of the main body of the building, the communal character of the 'Kreuzbleiche' as a recreation facility close to the towncentre is not impaired. The architecture of the sports hall is not only distinguished by the interior and exterior design but by the steel construction resting on the spectator entrance ramps. The colourful filigree framework, the main supports over the roof and the secondary girder system, which is visible in the hall and overhangs the whole façade, give the building its compelling architecture.

The hall is ideally suited for both sporting activities as well as all kinds of events with audiences. It has been presented the Gold Award 1989 of the International Working Group for the Construction of Sports and Leisure Facilities (IAKS).

2. STRUCTURAL STEELWORK

The roof is suspended from three exterior double lattice works of 42 metre span whose overhangs are anchored by tie rods. The engineers decided to use lattice work for the whole roof-construction. Thus lattice work was consistently used with all structural elements such as purlins, passages and bracings whereas the supporting structure of the stands comprises beams and rectangular hollow sections. However, since the grid of the supports of the car park does not match the one of the hall, the main columns also had to be supported by lattice work which was integrated in the stands structure. The structure is coated with a zinc dust primer and coloured finishing coats.

Cross Section



3. FIRE ENGINEERING AND SAFETY

It was not necessary to protect the entire steel structure as the large entrances and emergency exits as well as two exterior firestairs alongside are fully sufficient to evacuate the hall quickly. Moreover the fireload of the hall is very small.

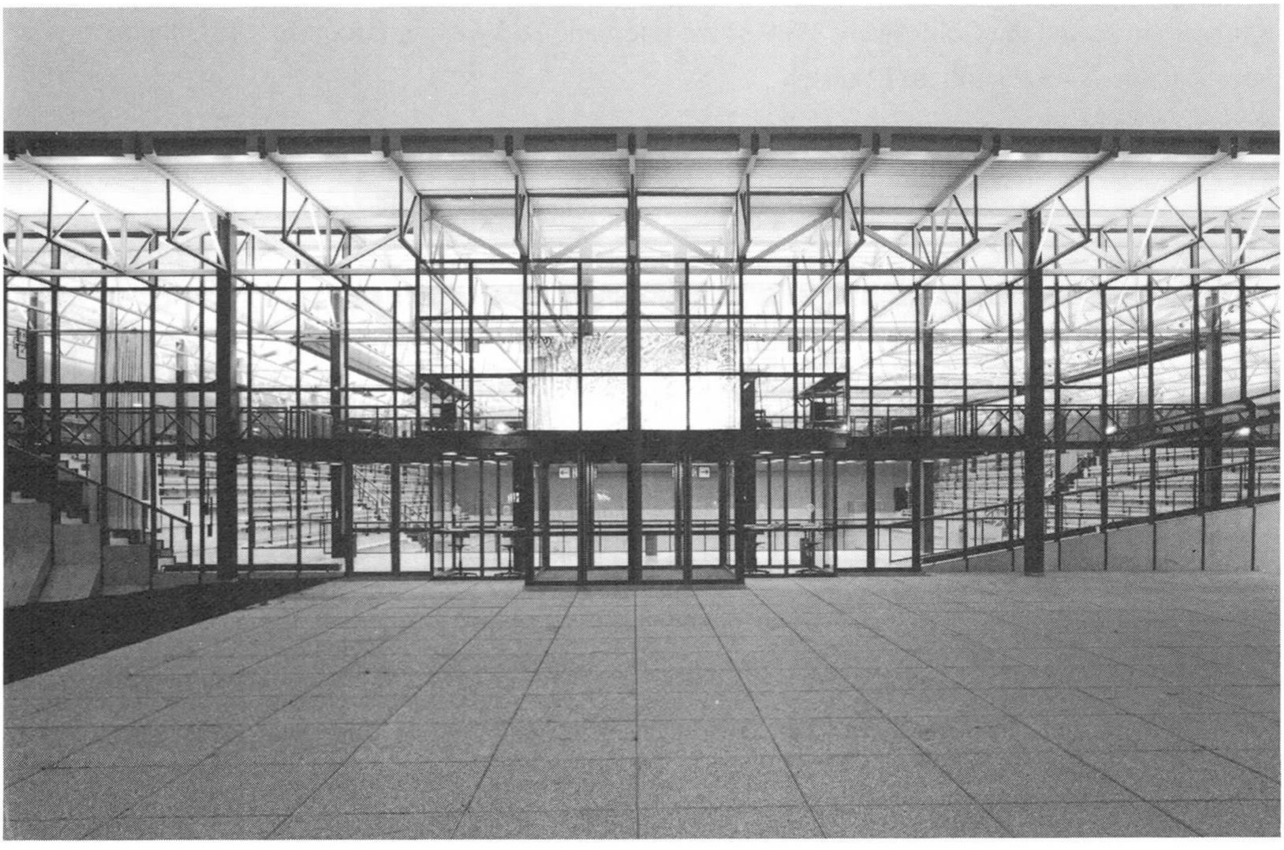
4. ROOFING AND INSULATION

The roof, whose slope amounts to 1%, has to carry a snow load of 2.2 kN/m^2 and consists of perforated trapezoidal sheets, a thermal insulation of 80 mm in thickness and a waterproofing PVC sheet. It is also loaded with a layer of gravel which is 50 mm thick.

The exterior steel structure, overhanging purlins and the frame of the stands are thermically separated by PVC of a compressing strength of 80 N/mm^2. The glass façade in the east of the building comprises termically separated façade sections supported by exterior supporting steel structure.

5. TECHNICAL SPECIFICATIONS

Area without columns:	42 x 50.4 m
Area of pitch:	27 x 48.5 m, the sports hall can be divided into 3 smaller gyms by folding partitions
Overall length of main girder:	50.4 m, maximal span 42 m
Roof surface:	3410 m^2
Constructional Steelwork:	370 t
Spectator capacity:	3600 for sport events 1500 for cultural activities such as concerts and theatre performances
car park:	spaces for 382 cars



View from East with Main Entrance

Aircraft Maintenance Hangar, Cardiff, Wales

Hangar pour l'entretien des avions, Cardiff, Wales

Hangar für die Flugzeugunterhaltung, Cardiff, Wales

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Ove Arup & Partners
Cardiff, UK

1. INTRODUCTION

A new British Airways 3-bay hangar is sited at Cardiff Wales International Airport (Fig 1.) and provides an outstanding facility for the heavy maintenance of Boeing 747 aircraft. The £75M project encompasses a double and single bay 22,000m² hangar with a 6,000m² mezzanine floor, a 15,000m² support building, an aircraft ground run pen and a 35,000m² concrete apron. The hangar (Fig 2.) is equipped in each bay with full aircraft access docking, overhead cranage, aircraft undercarriage lifting platforms and specialist ground support services fundamental to the effectiveness of the maintenance operations.

The profile of the building steps down to suit the function within each part of the facility. The height of the hangar was reduced by utilising external tubular triangular shaped trusses (Fig 3). This structural form minimised the enclosed volume and heating costs. The approach also reduced the impact of the hangar on the surrounding area by softening the roof profile and providing an industrial facility of some distinction.

2. HANGAR FRAME CONCEPT

From the commencement of the project the supply and erection of the hangar steelwork was identified as a critical element of the works. Consequently, the frame concept was developed to allow elemental fabrication and erection of the main steelwork members. Full space structures would have provided a lighter structural solution but minimum weight in this instance did not achieve minimum cost or programme.

3. STRUCTURAL DETAILS

In the hangar the main members are two continuous tubular steel space truss girders. One located over the hangar doors is 9m deep x 5m wide, and the spine girder located at the step between the high and low level roof areas is 14.5m deep and 8m wide. These girders

Fig. 1 - Aerial View Feb 1993

weigh 600 tonnes and 1000 tonnes respectively and are 232.5m long with spans of 153.75m and 78.75m. The total weight of structural steelwork in the hangar is 4000 tonnes and on the project as a whole is approximately 6000 tonnes.

The infill low level roof structure is formed with steel hollow section trusses two way spanning and supported by externally exposed triangular girders. The high level roof is formed with single span trusses. A detailed review of the structural systems is given in a paper by S. Luke [1].

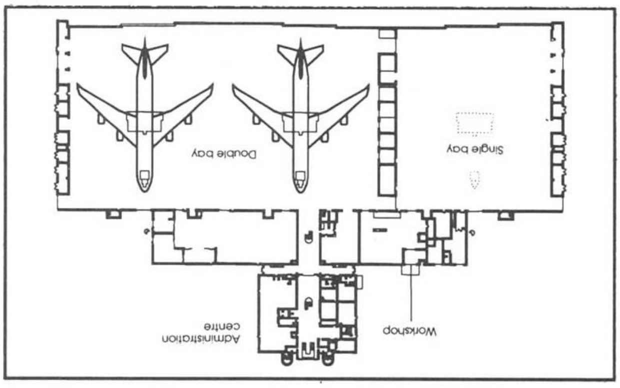

Fig. 2 - Plan

4. CONSTRUCTION

The lifting procedure chosen by the steelwork fabricator for the main girders provided a notable event since both were lifted into position supported at two points approximately 200m apart using hydraulically operated lifting towers (Fig 4.). It is believed the structural roof members are the longest to be lifted by this method in Europe.

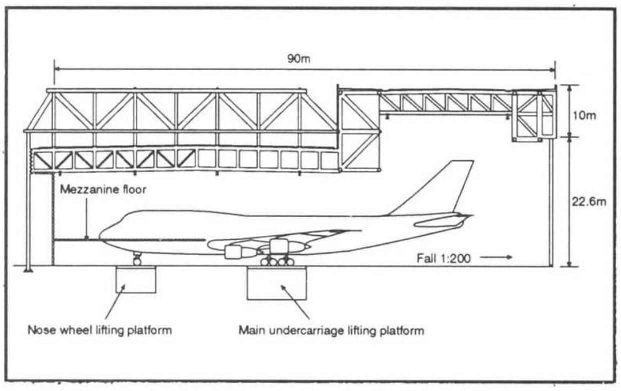

Fig. 3 - Hangar Section

Construction commenced in May 1991 and was complete in April 1993. Following the Client fit out the first aircraft arrived for a maintenance check on the 1 June 1993, 38 months after commencement of the design, as planned at the beginning of the project.

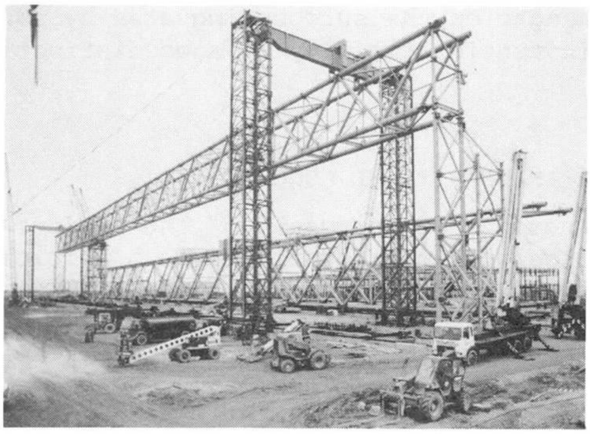

Fig.4 - Door Girder Lift Jan 93

5. CONCLUSION

The Cardiff base is a 'state of the art facility' and is designed to make sure that the highly skilled workforce achieves maximum efficiency. When an aircraft enters the hangar, the power-operated docking platforms (Fig 5.) close around it, so that the engineers can reach every part. As a result, overhaul work can begin within 20 minutes and this has been made possible by careful integration of design with maintenance operations.

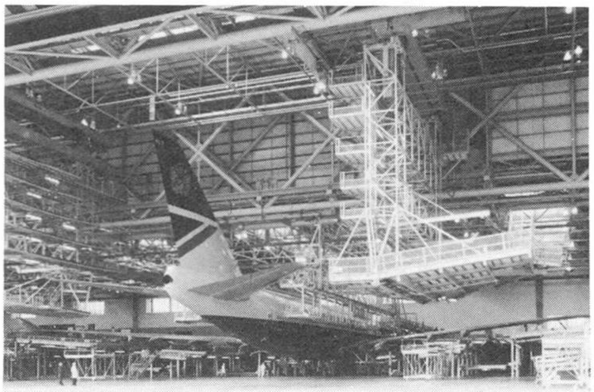

Fig. 5 - Access Docking

Reference

1. LUKE S.J. British Airways Maintenance Hangar, Cardiff Wales Airport Proc. Instn Civ Engrs Structs and Bldgs 1993, 99, Nov, 439 - 453

Swimming Complex Spartak in Sofia, Bulgaria
Centre de natation Spartak à Sofia, Bulgarie
Schwimmanlage Spartak in Sofia, Bulgarien

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The project solves a problem for covering the two existing swimming pools 12,50/25 m and 25/50 m as well as the adjoining building supplied with sittings for 500 spectators (Fig. 1).

The main bearing structure consists of five single-storeyed steel frames with span of 63,80 m. The distance between them is 18,80 m. The calculations are done for a biconstruction which consists of two plane solid frames integrated with rod lattice. The solid frames bear all vertical loads and the horizontal loads, acting in the planes of the frames. The rod lattice bears the loads, acting out of the planes of the frames. The frames are clamped into foundations in two directions by means of four anchor devices for every foundation. Every anchor device consists of eight anchor bolts M42x1500 mm.

The secondary bearing structure is the roof construction which consists of trusses, purlins and bracings. The trusses are with a span of 16,00 m; a step of 5,60 m and a height of 1,60 m. The roof purlins are with a span of 5,60 m and a step of 2,00 m. The statical scheme of the trusses and purlins is a simple beam.

The columns from the transverse facades are with variable height and step of 4,00; 4,20 and 6,00 m. The columns from the longitudinal facades are with broken axis and variable height.

The main bearing frames are fabricated in a workshop and transported as space blocks with length of about 18 m and weight of 150 kN. At first are assembled two main bearing frames and the secondary bearing roof construction between them. The assembling part with weight of about 5500 kN is pulled in a design position by means of hydraulic cranes. The rest part of the construction is erected by the same way (Fig. 2).

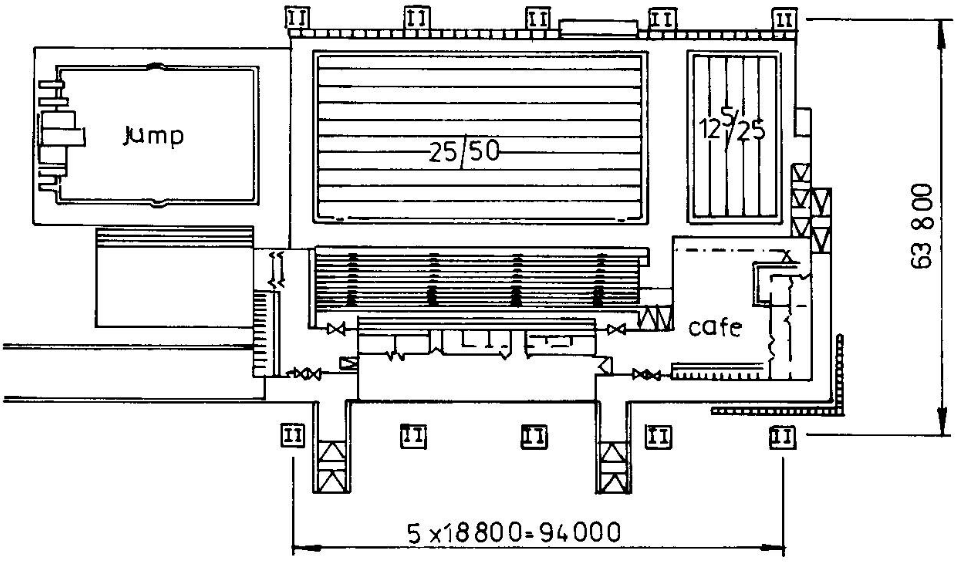

Fig. 1 Plan of the swimming complex

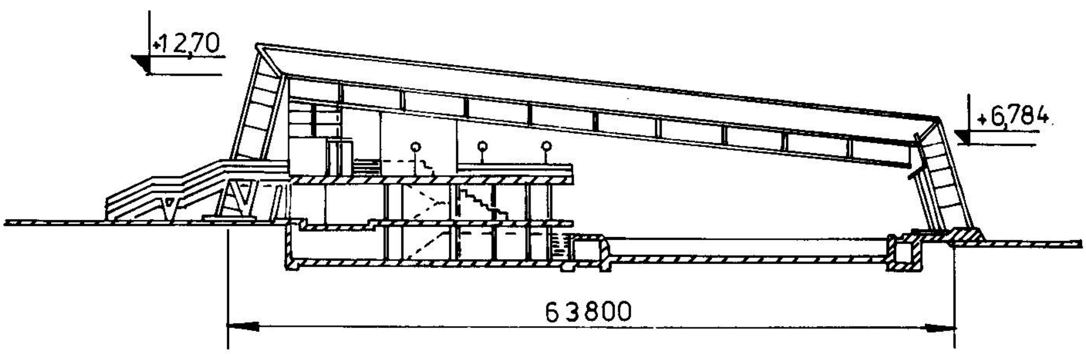

Fig. 2 Cross section