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SESSION 2
OUTSTANDING LONG-SPAN STRUCTURES

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Application of Pantadome System to Long-Span Roof Structures

Application du système Pantadome aux structures spatiales

Anwendung des Pantadome-Systems für Raumtragwerke

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SUMMARY

Spatial long-span structures are very efficient to cover wide areas without columns after their completion, but are never efficient from constructional standpoints. The principle of Pantadome System which the author developed to solve the problems of spatial structures is explained. Examples of the system to various long-span structures are described.

RÉSUMÉ

Les structures tridimensionnelles à grande portée sont très efficaces quand il s'agit de couvrir des grandes surfaces sans piliers intérieurs, mais ne sont jamais faciles à construire. Le principe du système Pantadome, développé par l'auteur afin de résoudre les problèmes des structures spatiales est décrit. Des exemples sont présentés pour l'application de ce système pour des structures tridimensionnelles à grande portée.

ZUSAMMENFASSUNG

Weitgespannte Raumtragwerke haben eine erprobte Einsatzfähigkeit beim Ueberdachen grosser Flächen ohne Innenstützen, aber auch Schwächen im Hinblick auf die Herstellung. Das vom Autor zur Lösung derartiger Konstruktionsprobleme bei Raumtragwerken entwickelte Prinzip des Pantadome-Systems wird vorgestellt. Beispiele werden beschrieben, wo dieses System für die Abdachung weitgespannter Tragwerke angewandt wurde.

1. PROBLEMS IN CONSTRUCTION OF DOMICAL LONG-SPAN ROOFS

A domical space frame, once completed, is one of the most efficient spatial roof structures capable of covering a very wide area. It is not always efficient, however, from the viewpoint of construction, because it requires big amount of scaffoldings, labor and time and often encounters difficulties in terms of accuracy, reliability and safety of work during its erection. Modern erecting methods such as lifting systems which are very often adopted in erection of double-layer grids of plate type can not equally be applied to a domical space frame.

Buckminster Fuller tried to solve this kind of problems in a few ways when he encountered them in building some of his geodesic domes. For construction of one of his domes in Honolulu in 1957 he adopted a system in which a temporary tower was erected at the center of the dome from top of which concentrically assembled part of the dome was hung by means of wire ropes. As assembly of the dome proceeded the dome was gradually lifted, enabling the assembling work to be done along the periphery of the dome always on the ground. He also adopted another method when he built a huge dome of 117m in diameter at Wood River, U.S.A., in 1959, where the assembled part of the dome was raised on a balloon-like enclosure. Some other cases have also been reported where different lifting methods have been applied to different domes. However, none of the above methods for lifting domes have become popular unlike many lifting methods which became widely used to raise plate-type space frames.

2. PANTADOME SYSTEM AS ONE OF THE SOLUTIONS

A patented structural system called 'Pantadome System' which had been developed by the author for a more rational construction of domical space frames was successfully applied to the structure of a sports hall completed in Kobe 1984. Pantadome System was then applied to the Sant Jordi Sports Palace in Barcelona and the National Indoor Stadium of Singapore and are now being applied to a few other long-span structures.

The principle of Pantadome System is to make a dome or a domical space frame geometrically unstable for a period in construction so that it is 'foldable' during its erection. This can be done by temporarily taking out the members which lie on a hoop circle(Fig. 1). Then the dome is given a 'mechanism', that is, a controlled movement, like a 3-D version of a parallel crank or a 'pantagraph' which is generally applied to a drawing instrument or a power collector of an electric car (hence the name, 'Pantadome').

Since the movement of a Pantadome during erection is 'controlled one' with only one freedom of movement (vertical), no means of preventing lateral movement of the dome such as staying cables or bracing members are necessary during its erection. The movement and deformation of the whole shape of the Pantadome during erection are three dimensional and may look spectacular and rather complicated, but they are all geometrically determinate and easily controlled. Three kinds of hinges are incorporated in a Pantadome system which rotate during the erection. Their rotations are all uni-axial ones, and of the most simple kind. Therefore, all these hinges are fabricated in the same way as normal hinges in usual steel frames.

In Pantadome system a dome is assembled in a folded shape near the ground level. As the entire height of the dome during assembling work is very low compared with that after completion, the assembly work can be done safely and economically, and the quality of work can be assured more easily than in conventional erection systems. Not only the structural frame but also the exterior and interior finishings, electricity and mechanical facilities are fixed and installed at this stage. The dome is then lifted up. Lifting can be achieved either by blowing air inside the dome to raise the internal air pressure, or by pushing up the periphery of the upper dome by means of hydraulic jacks. When the dome has taken the final shape, the hoop members which have been temporarily taken away during the erection are fixed to their proper positions to complete the dome structure. The lifting means such as air pressure or hydraulic jacks can be then removed, and the dome is completed.

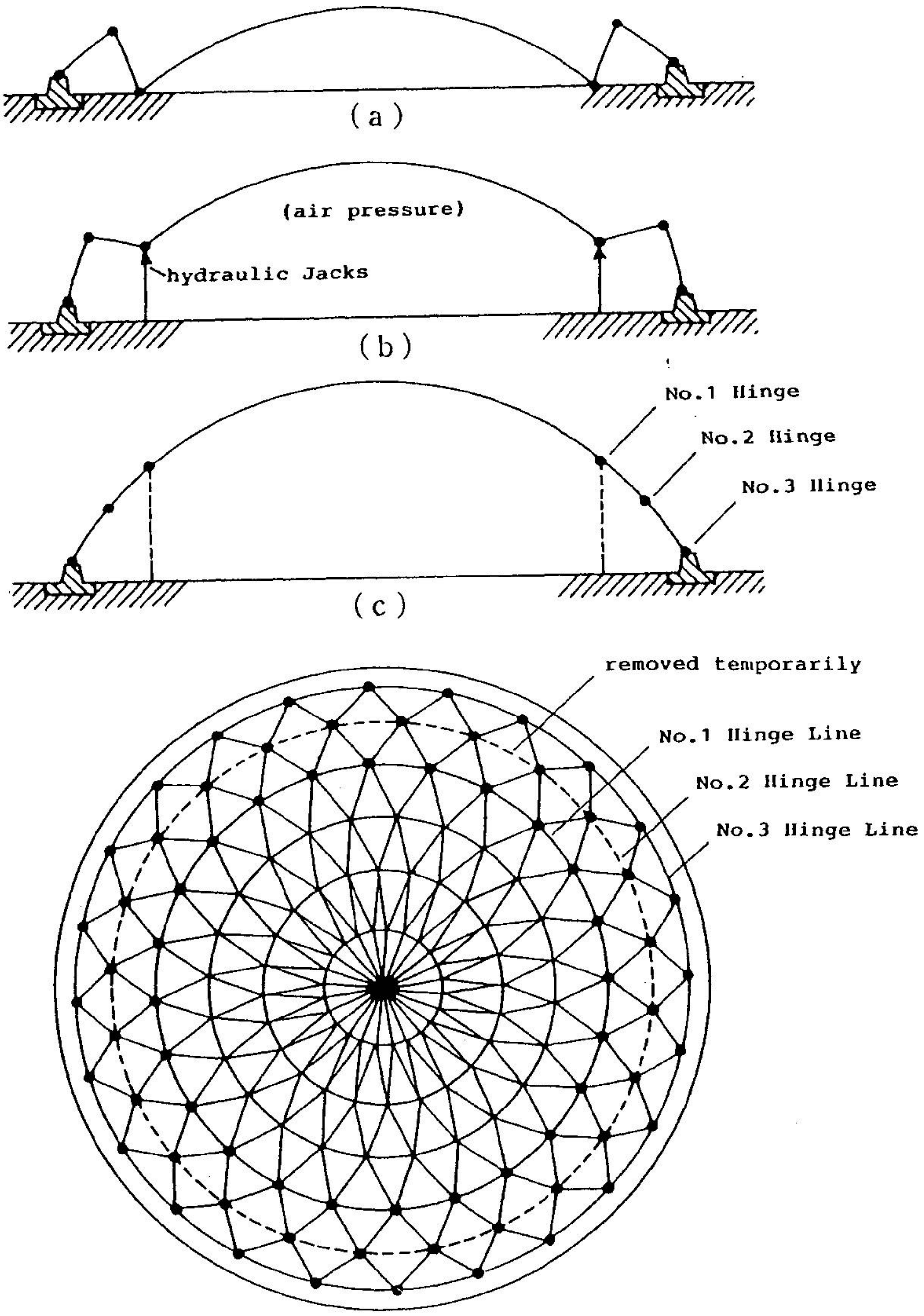

Fig. 1 Principle of Pantadome System

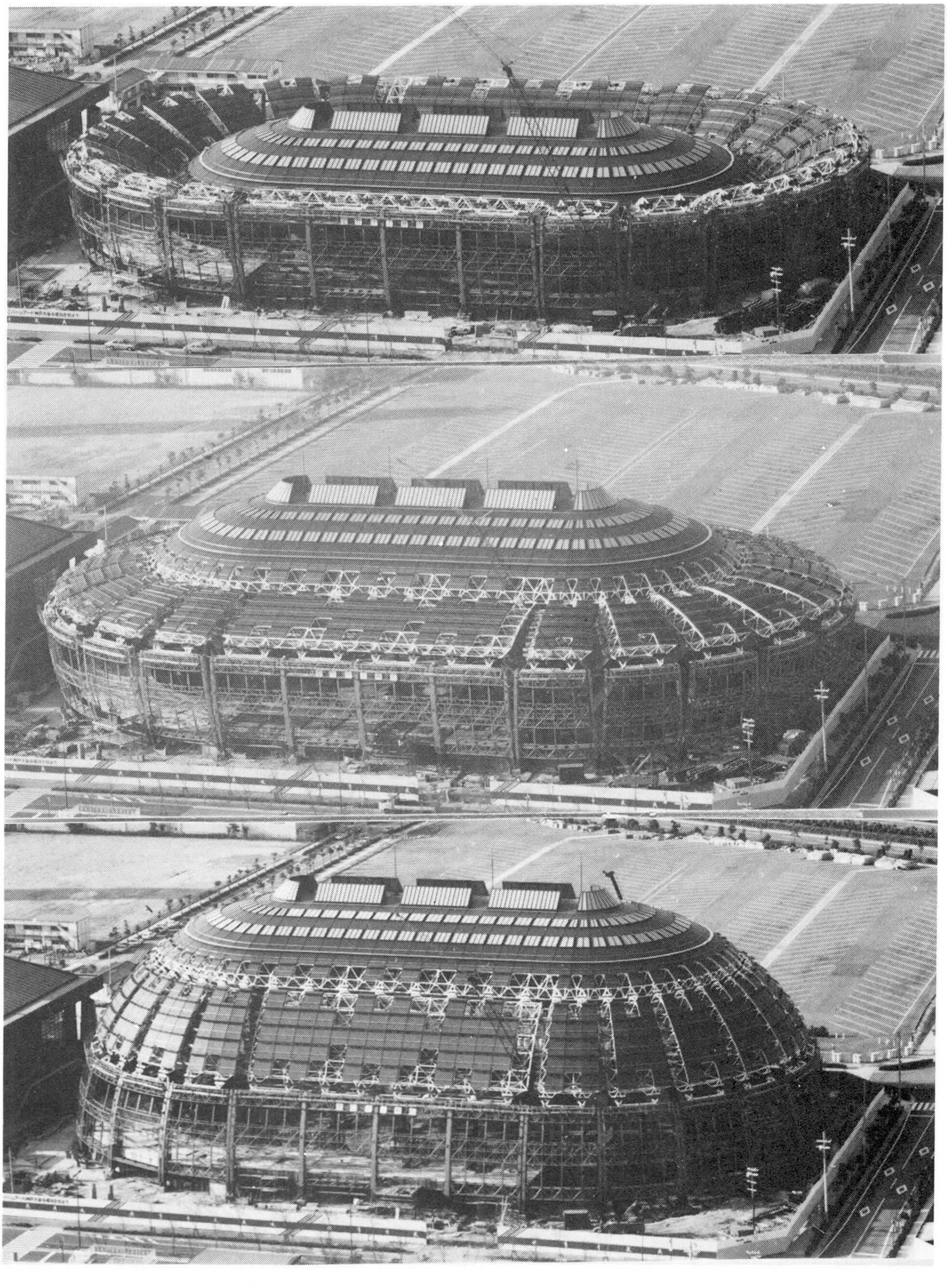

Fig. 2 Lifting sequence of World Memorial Hall

3. APPLICATION OF PANTADOME SYSTEM

Pantadome system can be successfully applied to domical frames of various configurations. World Memorial Hall in Kobe in which the Universiad '85 was celebrated has an oval plan of 70m x 110m. It was designed by the author in cooperation with Architect Mitsumune, and it was constructed by means of Pantadome System as shown in Figure 2. It was the first dome to which Pantadome System was applied.

The second example of Pantadome application is the Singapore National Indoor Stadium having a rhombic plan of 200m x 120m in the diagonal directions, the structure of which was designed by the author in cooperation with Architect Kenzo Tange(Fig. 3). The National Indoor Stadium has an arena of 3000 m^2 and grandstands for 12000 seats. The geometry of the roof is constituted by four cylindrical surfaces, each convex inward, having the axes of the cylinders parallel

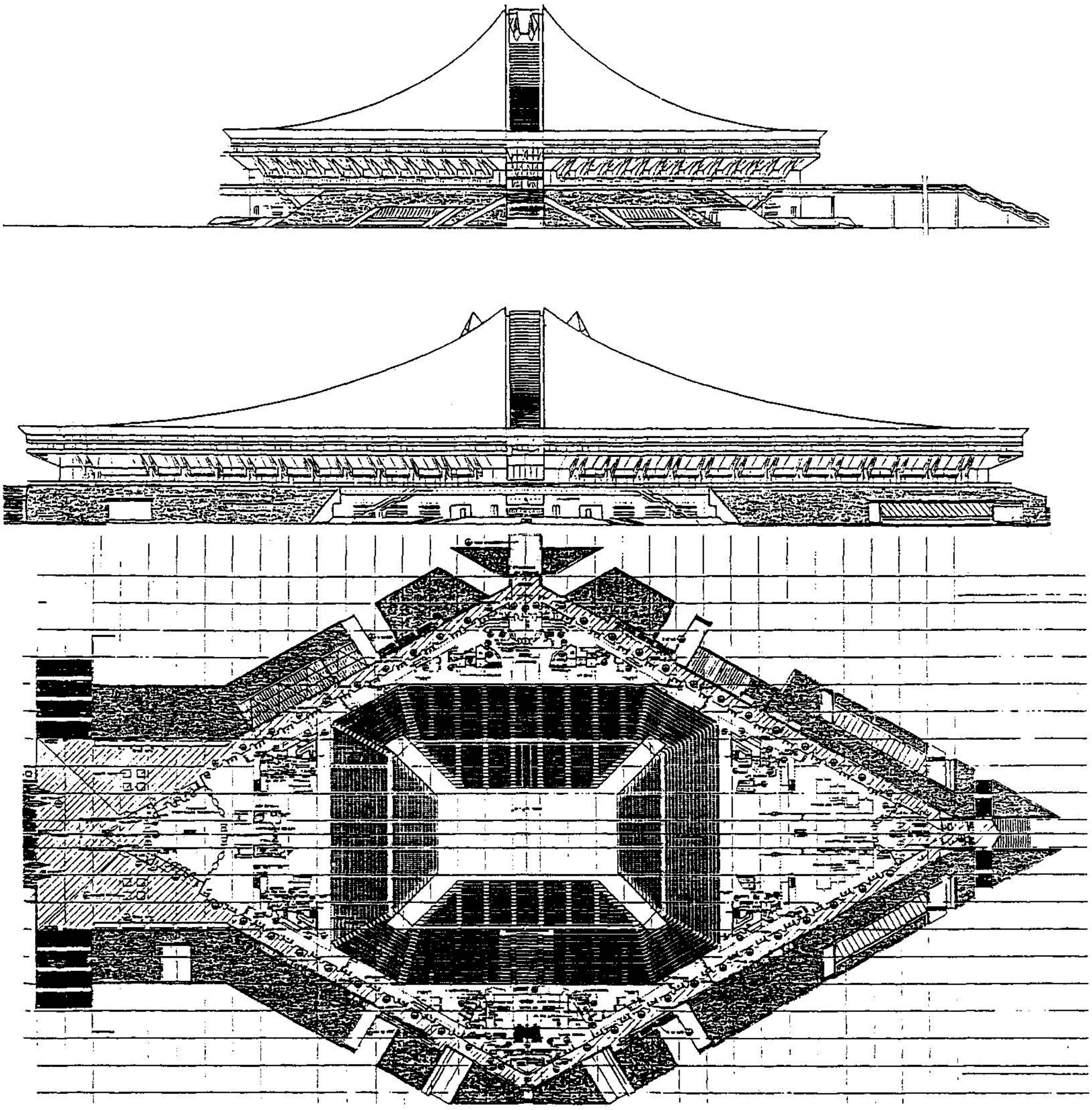

Fig. 3 Singapore Indoor Stadium

to the four sides of the rhombic plan. Although the roof surface which is convex inward gives the visual impression of a hanging roof, actually it has a sufficient dome effect. The hinge lines for Pantadome mechanism were set along the straight lines parallel to the boundaries of the roof plan.

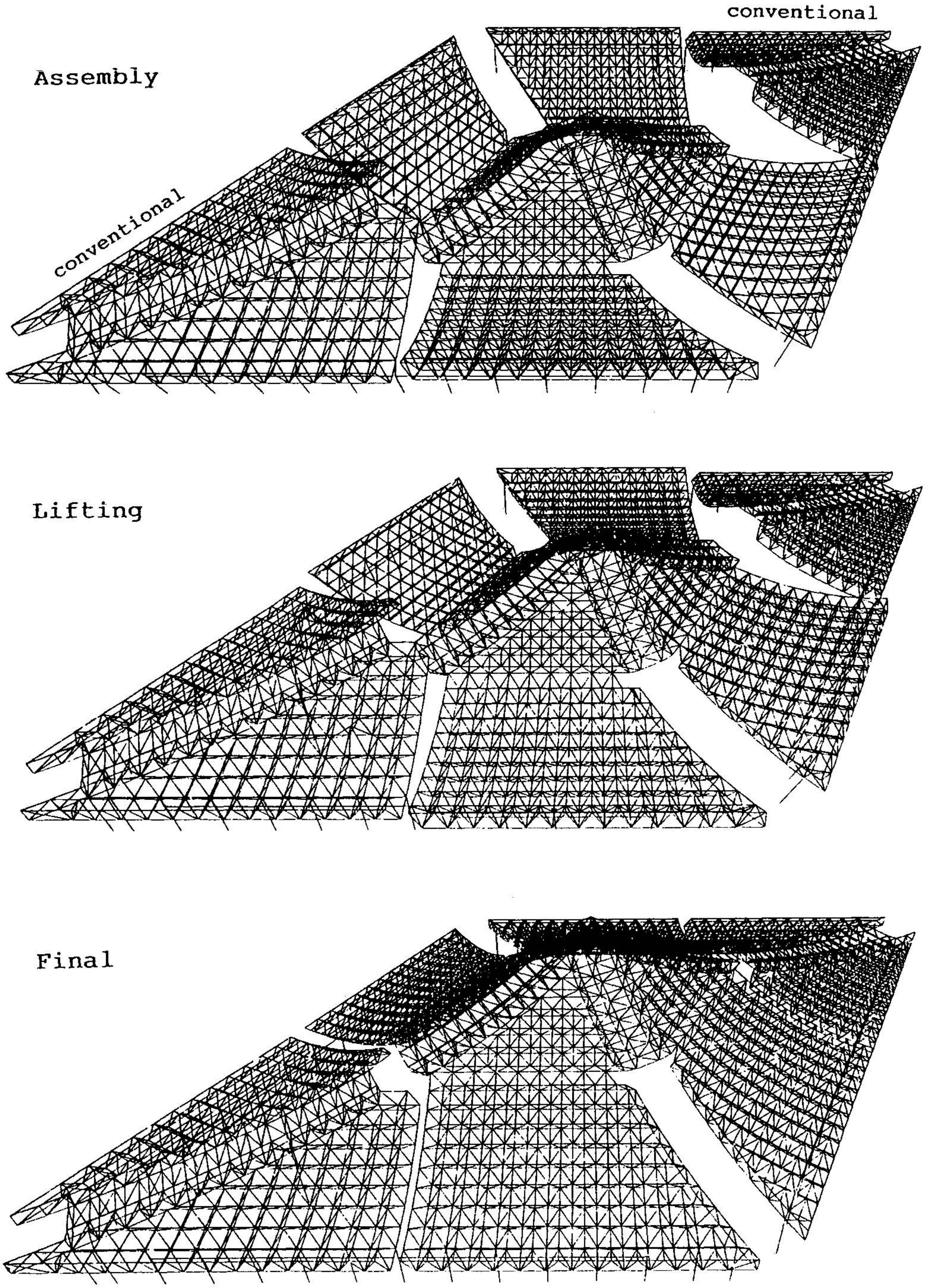

Fig. 4 Lifting sequence of Singapore Indoor Stadium

The processes of erection of the roof of Singapore National Indoor Stadium is shown in Figure 4. Around the two opposite corners of the rhombic plan the floor levels are elevated, and the height of the roof is lower. So it was considered that the erection of the roof could be easily carried out by means of conventional methods around these areas. This is why the Pantadome System was applied only to the central part of the roof.

4. SANT JORDI SPORTS PALACE IN BARCELONA

Sant Jordi Sports Palace in Barcelona was one of the venues for the Olympic Games '92. As a result of an international design competition the author had the opportunity of designing the structure of this important building (Fig. 5).

The roof structure which covers an area of 128mx106m for the arena and the grandstands seating 15,000 is constructed by a steel space frame consisting of 9,190 tubes connected by 2,403 joints. The shape of the roof structure is constituted by the central domical part and the four toroidal parts surrounding it. The area of the whole roof surface is 13,460 m^2. The type of the space frame is what is called a double layer grids structure having a depth of 2.5m. The diameter of the steel tubes of the space frame ranges from 76mm to 267mm with exceptions of bigger tubes for valley and ridge members(406mm) and for the peripheral members(508mm).
The roof has a rise of 21m as a dome. The weight of the whole roof including finishing is about 3,000 tons, of which 1,000 tons are the weight of the steel space frame. the roof structure is supported by 60 columns which stand on the reinforced concrete substructure along the periphery of the grandstand. The height of the roof is 31m above the base of the columns. Since the floor on which the columns stand is 14m high from the arena level, the maximum height of the roof from the arena floor is about 45m.

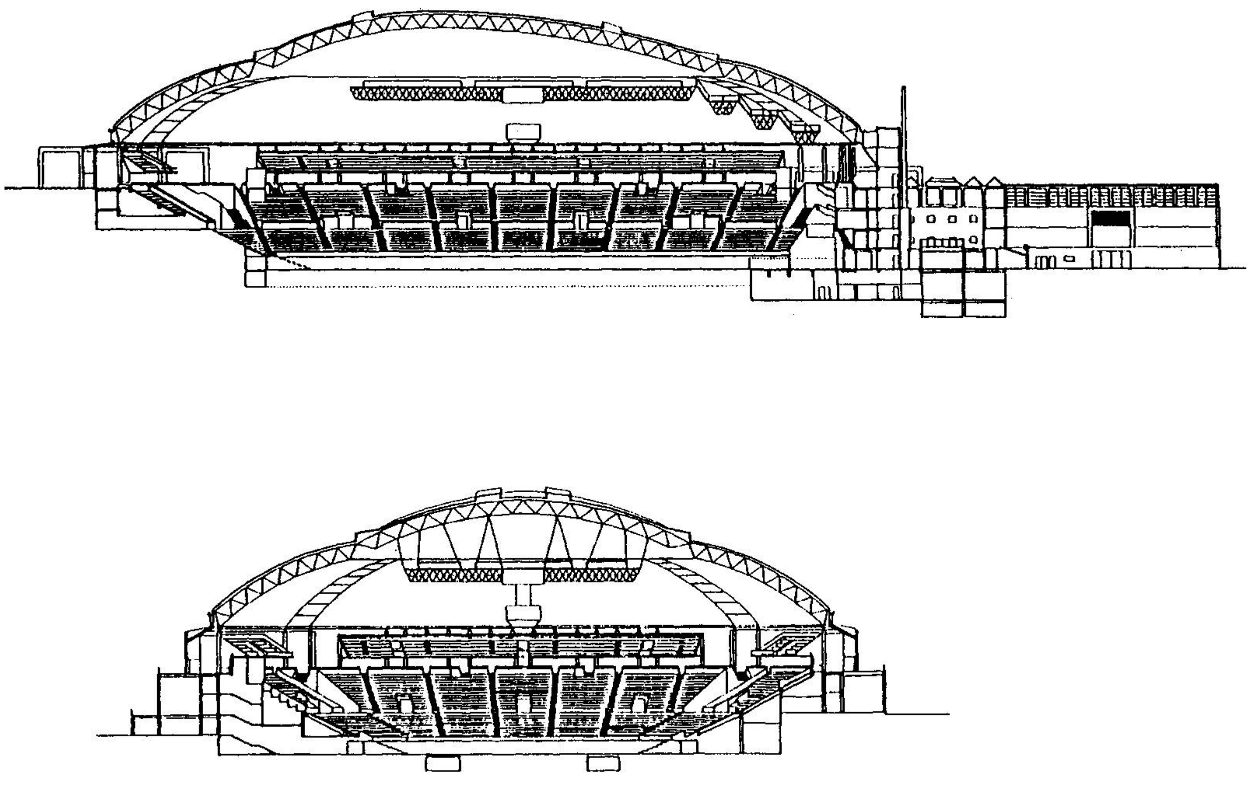

Fig. 5 Sant Jordi Olympic Sports Palace

Each adjacent pair of the 44 columns is rigidly connected to each other at their tops by means of a lattice girder to form a portal frame in the peripheral direction. Since all the columns are kept hinge-connected at their tops and bottoms even after completion so that the roof is free to expand and shrink without producing thermal stresses in the structure when temperature changes, the lateral resistance of the roof structure is exclusively given by those 22 portal frames (14 in the longitudinal and 8 in the transverse directions, respectively) along the periphery of the roof.

Assembly of the roof structure was effected in the site by means of hoisting systems. It consisted of the following two different stages(Fig. 6):

a) Assembly of the central dome on the arena, supported by provisional light steel columns.
b) Assembly of the 16 peripheral segments of approximately $400m^2$ weighing up to 40tons.

The equipment for pushing up the roof was composed of 12 lifting towers.

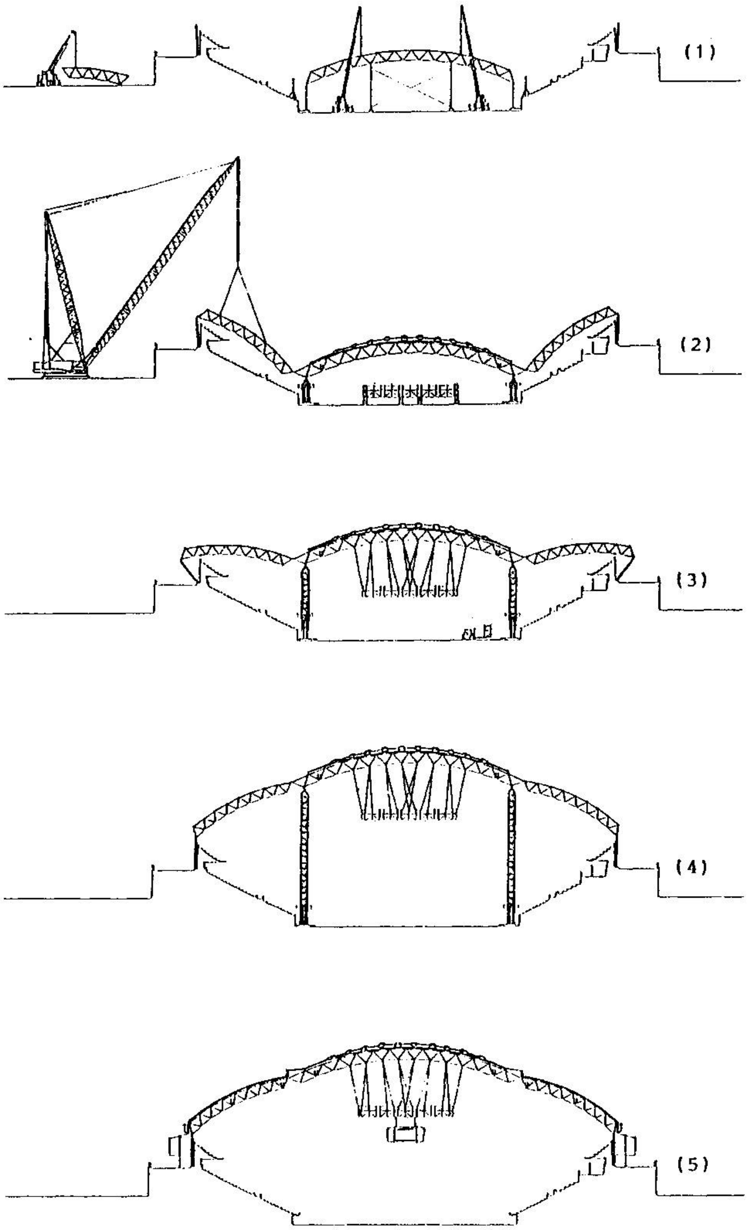

Fig. 6 Erection of Sports Palace

When all the specified roof members had been assembled, and necessary lifting equipments had been installed, the lifting operation began. The lifting operation was carried out in stages of 3.04m, which corresponds to the height of one full tower element.

The lifting operation began on November 21st, 1988. The load on the 24 lifting units were initially 760 tons. After 2.74m of lifting, a platform measuring 60x20m and weighing 80tons which carries electrical and ventilation equipment was hung from the central part of the roof by tensioning the suspension cables. In 10 days the roof was pushed up the height of 9 full and one half tower elements, which corresponds to 28.83m. Once the final height had been reached, the towers were set down onto steel plate packers of 16cm height. Thereafter, lifting units, pumps and controls were dismantled and removed.

The members which had been kept away from their positions during the lift were then fitted in and welded to complete the roof structure. At the same time the installation works of insulation, roofing and interior finishing continued. After completion of the structural system the loads on the temporary towers were relieved successively in small steps by jacking up one tower at a time and by removing gradually the packer plates. The vertical displacements of the center of the roof due to removal of the lifting towers was 140mm, showing a close agreement with the result of analysis.

5. EXAMPLES UNDER CONSTRUCTION

Out of three other buildings which are now under construction on the Pantadome System, two major buildings are described here.

5.1 Sun-dome Fukui

One of them is Sun-dome Fukui being built in fukui Prefecture, Japan.
This building is a multipurpose hall to be used for the World Championship Games of Athletics in 1995. It has a circular plan of 116m in diameter, and the maximum height of 40m. The building is located in a heavily snowed region with a snow load of 6kN/m^2. For functional reason the roof of the building was so designed that it keeps snow which has fallen on it. Consequently the structure itself is also rather heavy, the weight of the steel frame being 2.5kN/m^2, and the total roof including finishing 4kN/m^2. The lifting process of the roof is as shown in Fig.8 in section and Fig.9 for a model study. The lifting of the roof is scheduled for August 1994.

5.2 Osaka Sports Center

Another application of Pantadome System is being tried for a Osaka prefectural sports hall to be built in Osaka. It has an oval plan of 110mx125m and the maximum height of 42.65m. The main structural feature of the building is that the roof is slightly(5 degrees) tilted, and so the lifting direction is not vertical. This roof is to be lifted in November 1994.

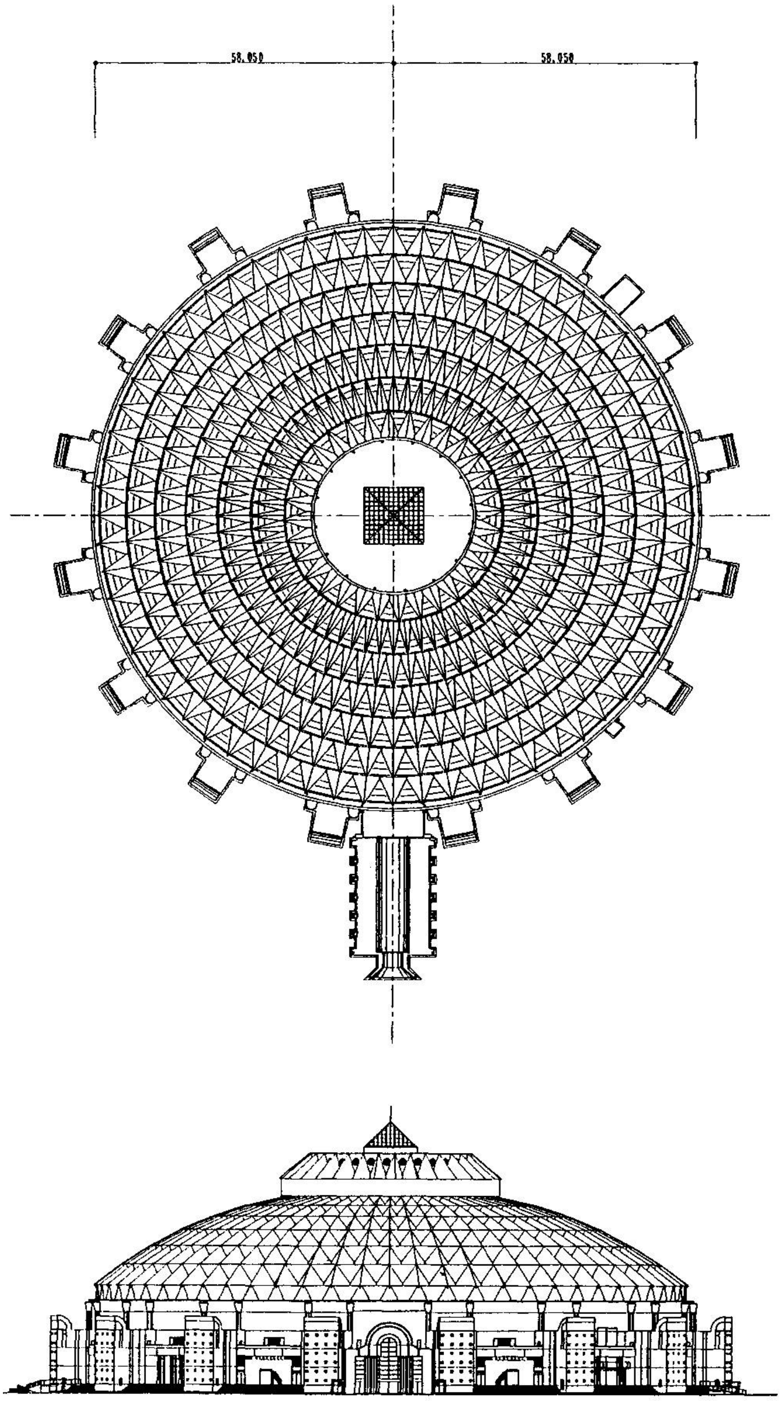

Fig.7 Sun-dome Fukui

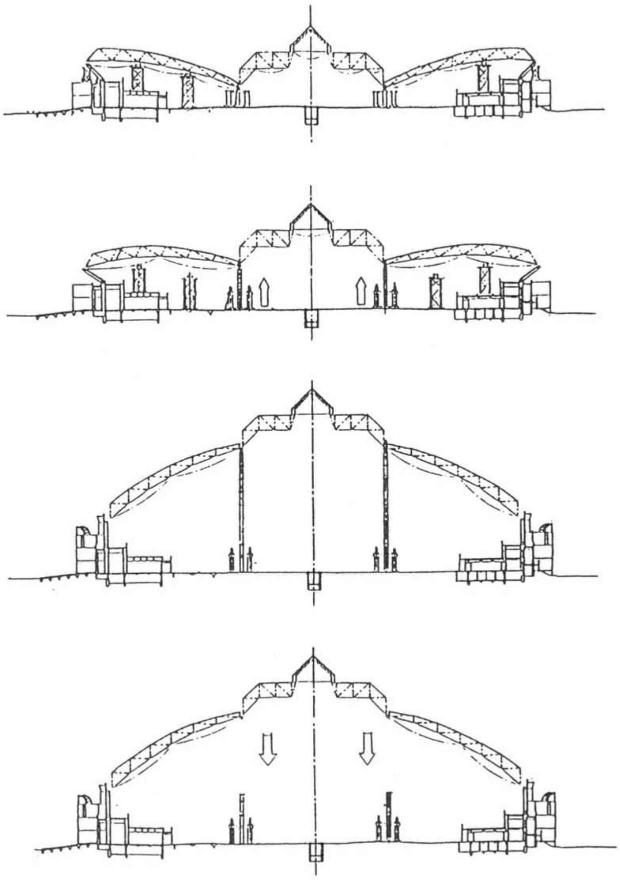

Fig. 8 Erection process

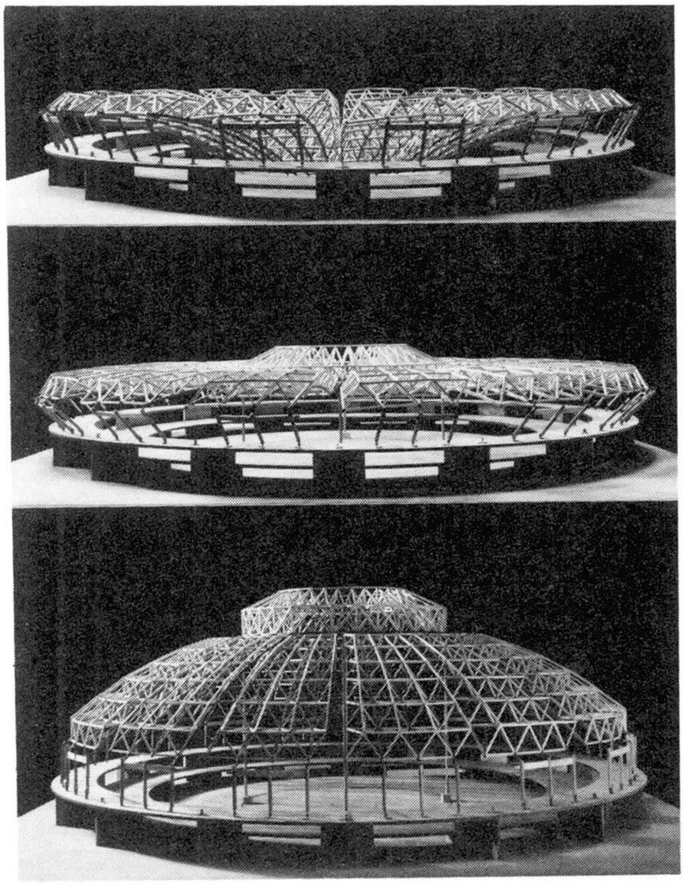

Fig. 9 Model study of lifting

6. CONCLUSIVE REMARKS

As a solution for the construction problems of domical space frames a patented structural system named Pantadome System has been presented. Pantadome System has been applied to large-span domes of a few different geometries. It has so far been successfully applied in a few different countries (Japan, Singapore and Spain) where local building conditions are different from each other. It seems that these examples support rationality of the system in terms of safety, construction speed, quality of built structures and economy of space frames.

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Some Indian Examples of Large Span Building Structures

Quelques exemples de structures à grande portée en Inde

Beispiele einiger weitgespannter Tragwerksbauten in Indien

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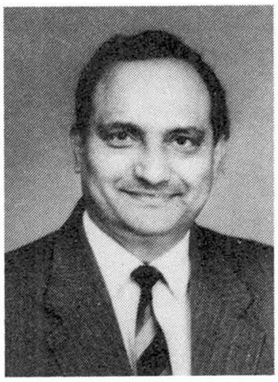


A. Ramakrishna, has a Masters in Civil Engineering. Joining L&T as a junior engineer, he is now member of the Board. He is an acknowledged expert in precast concrete technology and has won many national and international awards. He is a member of various national and international professional bodies.

SUMMARY

This paper deals with some large span building structures which have been constructed in India. The report details the application of precast and prestressed structures using 'A' frames, asymmetrical 'A' frames, and parabolic arch-shaped concrete spans. The use of shells, folded plates and ribbed slabs made of concrete as roof elements for large span structures is also described.

RÉSUMÉ

Ce rapport traite de quelques bâtiments à grande portée construits en Inde. Le rapport décrit l'application de structures préfabriquées et précontraintes utilisant des cadres 'A', des cadres 'A' asymétriques et paraboliques en forme d'arc. L'utilisation de coques, plats ondulés et travées à nervure en béton comme éléments de toiture à grande portée est également décrite.

ZUSAMMENFASSUNG

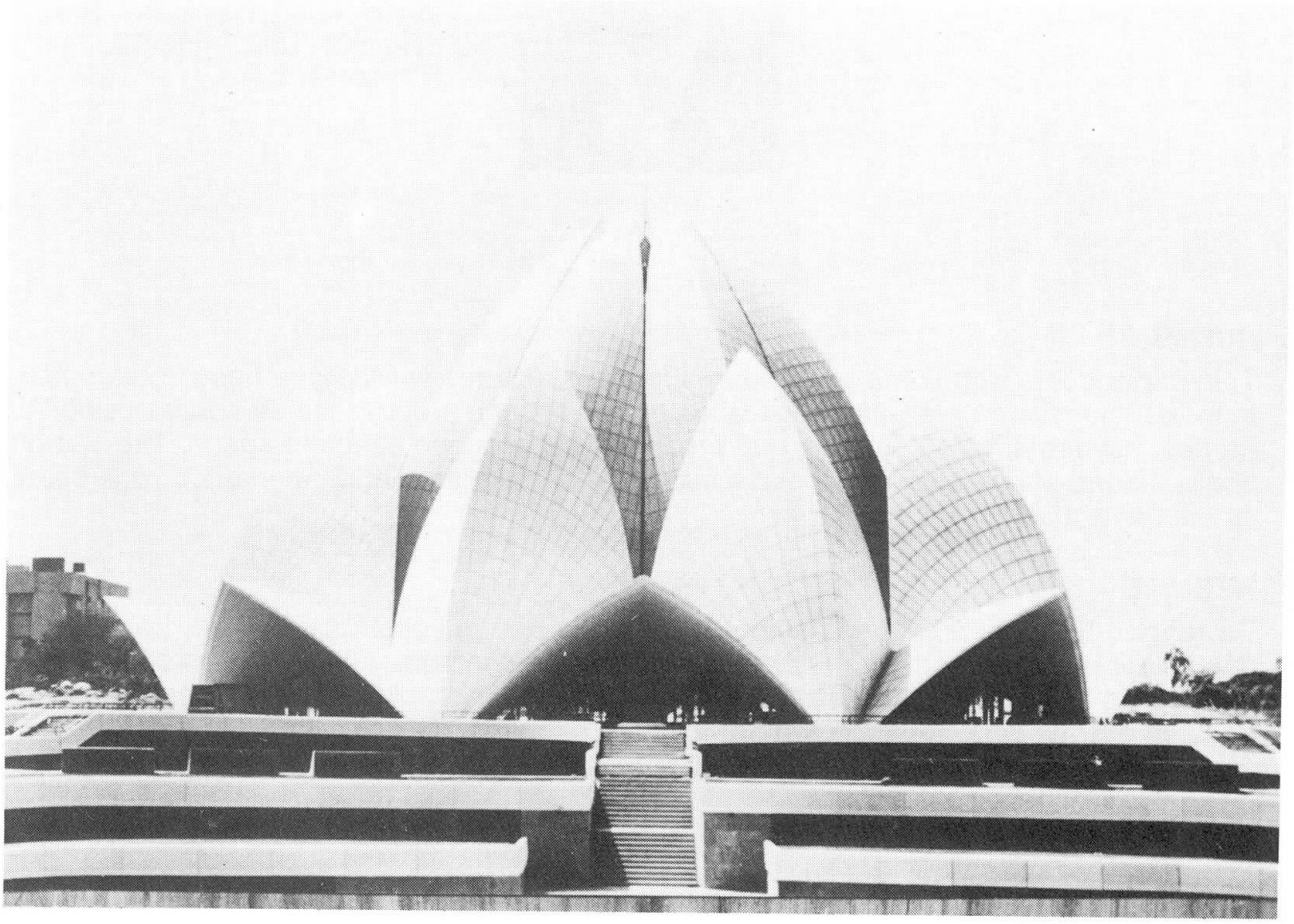
Dieser Bericht behandelt einige weitgespannte Tragwerksbauten, die in Indien realisiert wurden. Der Bericht schildet die Anwendung von vorfabrizierten und vorgespannten Tragwerkselementen, wobei 'A'-Module, asymetrischen 'A'-Module und hohlbogenförmigen Fertigbetonelemente benutzt wurden. Der Einsatz von Schalen-, gewellten Platten- und gerillten Balkenelementen aus Fertigbeton für die Bedachung wird ebenfalls beschrieben.

1. HOUSE OF WORSHIP - LOTUS TEMPLE

The temples of Bahai faith are well known for their architectural splendour and the Mother Temple constructed in Delhi is only a continuation of this rich tradition.

The temple complex, consists of the main house of worship, the ancillary block which houses the reception centre, the library and the administrative building as well as the toilet block. The temple proper comprises a basement to accommodate the electrical and plumbing services and a lotus shaped superstructure to house the assembly area.

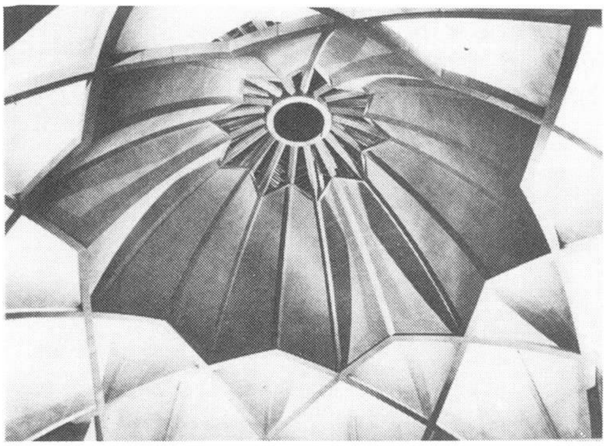
All around the lotus are walkways with beautiful, curved balustrades, bridges and stairs which surround the nine pools representing the floating leaves of the lotus. Apart from serving an obvious aesthetic function, the pools also help in the ventilation process of the building (Fig. 1)



The lotus, as seen from outside, has three sets of leaves or petals, all of which are made out of thin concrete shells. The outermost set of nine petals, called the 'entrance leaves', open outwards and form the nine entrances all around the outer annular hall. The next set of nine petals called the 'outer leaves' point inwards. The entrance and outer leaves together cover the outer hall. The third set of nine petals, called the 'inner leaves', appear to be partly closed. Only the tips open out, somewhat like a partly opened bud. This portion, which rises above the rest, forms the main structure housing the central hall. Near the top where the leaves separate out, nine radial beams provide the necessary lateral support. Since the lotus is open at the top, a glass and steel roof at the level of the radial beams provides protection from rain and facilitates entry of natural light into the auditorium.

Below the entrance leaves and outer leaves rise nine massive arches in a ring, through each one of which is a row of steps leading into the main hall.

The inner leaves enclose the interior dome, a canopy made of criss crossing ribs and shells of intricate pattern. When viewed from inside, each layer of shells and ribs, as they rise, disappears behind the next lower layer. Some of the ribs converge radially and meet at a central hub. The radial beams emanating from the inner leaves described earlier, meet at the centre of the building and rest on this hub. A neoprene pad is provided between the radial beams and the top of the interior dome to allow lateral movement caused by the effect of temperature changes and wind (Fig. 2)



1.1 Materials:

The petals of the lotus are in white concrete using specially graded dolomite aggregates form the Alwar mines near Delhi and white silica sand from Jaipur. The white cement used is imported from Korea. The reinforcement used in white concrete, as well as the binding wire for it, is entirely galvanised so as to prevent the long-term effect of rusting of reinforcement on the white colour of concrete. Specially designed steel spacers were used, to keep the reinforcement in place. No plastering painting or any other type of surface finish was done. A lightly bush-hammered, exposed concrete surface, with the pattern of formwork joints form the final finished surface of the interior of the temple.

On the exterior, the petals are clad with white marble panels fixed to the concrete surface with specially designed stainless steel brackets. The marble has been quarried from the Mount Pentilikon mines of Greece and thereafter sent to Italy, where each panel was cut to the required size and shape before transporting them to the site in Delhi. The flooring inside the temple in white marble and the finish of the walkways and stairs of the outer podium is in red sandstone.

The Bahai temple in New Delhi is one of the most complex project undertaken and is also probably one of the most outstanding contemporary structures in the world.

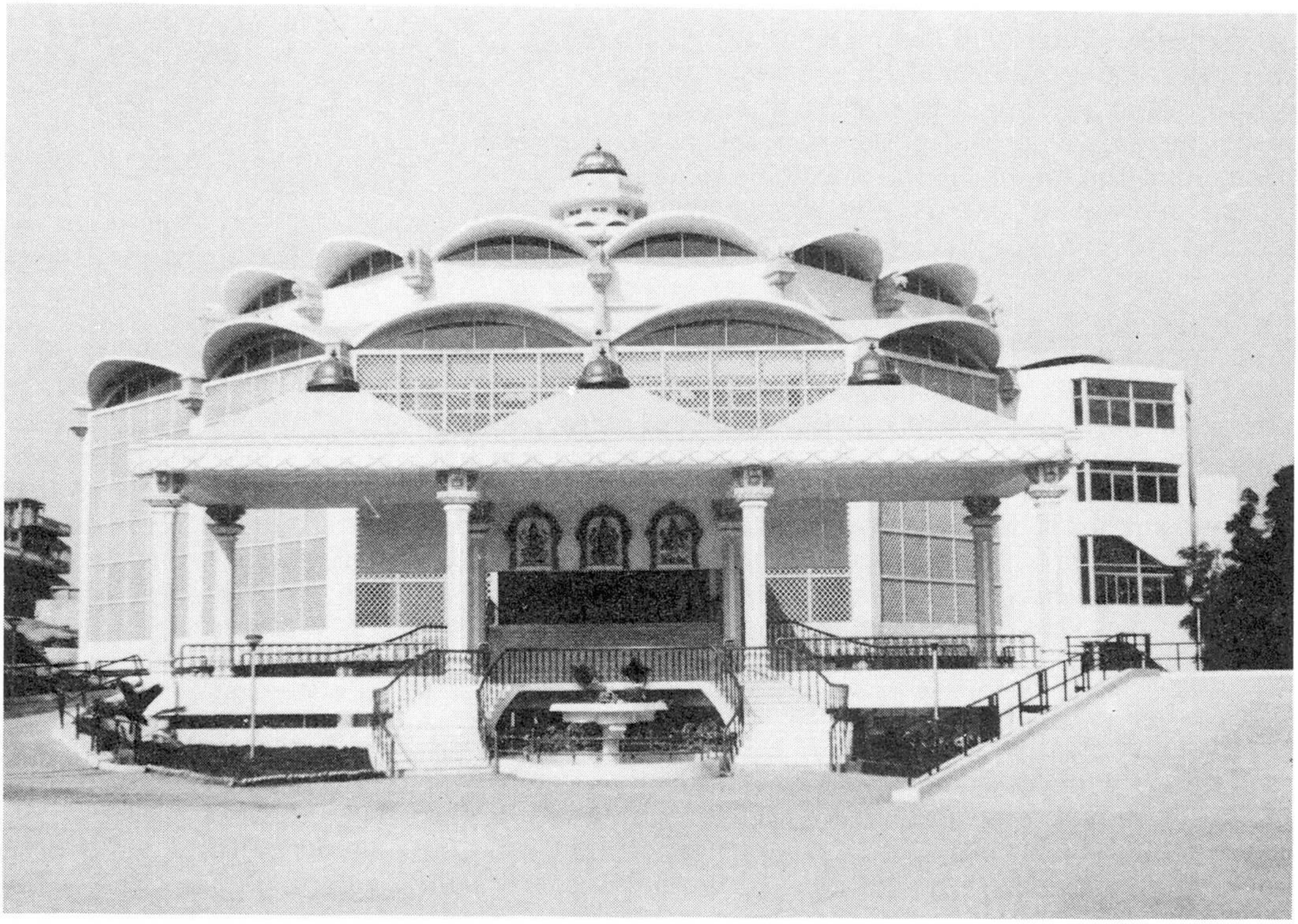
2. MULTIPURPOSE AUDITORIUM - HYDERABAD

2.1 Sri Sathya Sai Nigamagamam

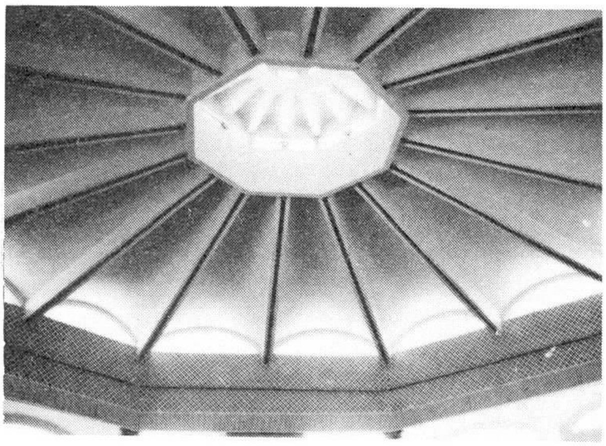
The Sri Sathya Sai Nigamagamam, a modern Kalyanamandapam (marriage hall) cum multipurpose auditorium was built for Sri Sathya Sai Central Trust at Hyderabad.

The auditorium and balcony, arranged in an octagonal shape, has a seating capacity of 1500.

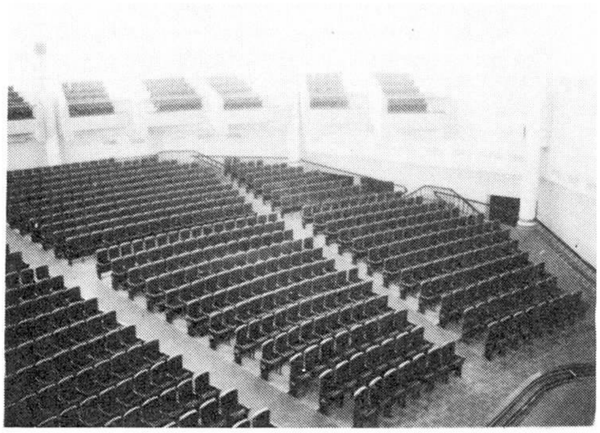
An interesting design feature of the auditorium roof is that it consists of an octagonal concrete compression ring having a height of 3 m. rising to a height of 16.5 m above the auditorium floor level and connected with sixteen concrete beams spanning between columns/circumferential beams and compression ring. An octagonal dome and 16 upper curved shells form the roof of the auditorium. Sixteen lower curved shells form the balcony roof (Fig. 3).



The dining hall roof consists of an interesting grid slab pattern of radial beams and intermediate circumferential beams. The site being located on a rocky terrain, heating of rock using firewood for excavation without disturbing the adjacent structures was resorted to. Single hold drill blast at a time for rock excavation was also done (Fig. 4)



A central scaffolding tower and sixteen radial beam towers were simultaneously erected for stability. Concreting was completed in sequence, taking up compression ring, radial beams and curved shells one after another. The whole staging was deshuttered only after completion of sixteen shells. The air volume



covered within the auditorium / balcony was 1.5 lakh cu.ft. (Fig. 5)

The multipurpose auditorium consists of centrally air cooled dining hall, auditorium and balconies. The main auditorium has an annexe block consisting of kitchen, marriage party rooms and green rooms, plant room and electrical sub-station buildings to cater to air cooling and power requirement of the auditorium.

3. AIR HANGAR FOR INDIAN AIRLINES, BOMBAY

The new Engineering Complex of Indian Airlines at Santacruz, Bombay, has a unique hangar which is being extended to accommodate additional aircraft. The unique 62 m long cantilever folded plate roof of the hangar is considered a world record for such a structure. There was a requirement for expanding the hangar laterally for housing more aircraft. Hence the cantilever scheme was chosen for the roof so that no side supports which hinder free lateral expansion are required. The unobstructed column free space available for the aircraft is 12,000 sq.m in the existing hangar and a further area of 8,000 sq.m is now being added within the full area where six A-300 aircraft are planned to be housed.

3.1. Roof Structure:

The new roof structure will measure 152 m x 60 m in plan, symmetrically divided by an expansion joint located at half the width. The 152 m length consists of two cantilevered hangar roofs of 62.3 m each and a central 27.4 m length over the Engine Hospital Building. The roof system is a continuous multiple folded plate system. The transverse section of the fold consists of 8 modules of 7.62 m width each having a corrugated plate arrangement with horizontal top and bottom plates between webs inclined at 45^{o}.

The cantilevered roof is supported by prestressed concrete suspension ties. The main columns supporting the load at the roof structure are located on the flanks of the central Engine Hospital building and are extended upwards above the roof as concrete pylons. At the lower flange level there is a clear discontinuity in the pylon with the entire roof system sup-ported on reinforced neoprene bearings resting over the top surface of the lower columns. This support system permits free movement of the roof structure under temperature, creep and shrinkage movements. At the upper flange level, a "Freyssinet" type concrete hinge is provided in the pylon to permit free rotations in the longitudinal plane.

Two main transverse diaphragms are provided. The main "forward diaphragm" is located at the front end of the suspension ties near the tip of the cantilever where the ties are anchored. The main "rear diaphragm" is located at the rear end of the ties at the foot of the pylon where the back-ties are anchored. Additional transverse diaphragms spaced at about 9 m centres help in retention of geometrical shape and for carrying transverse wind loads.

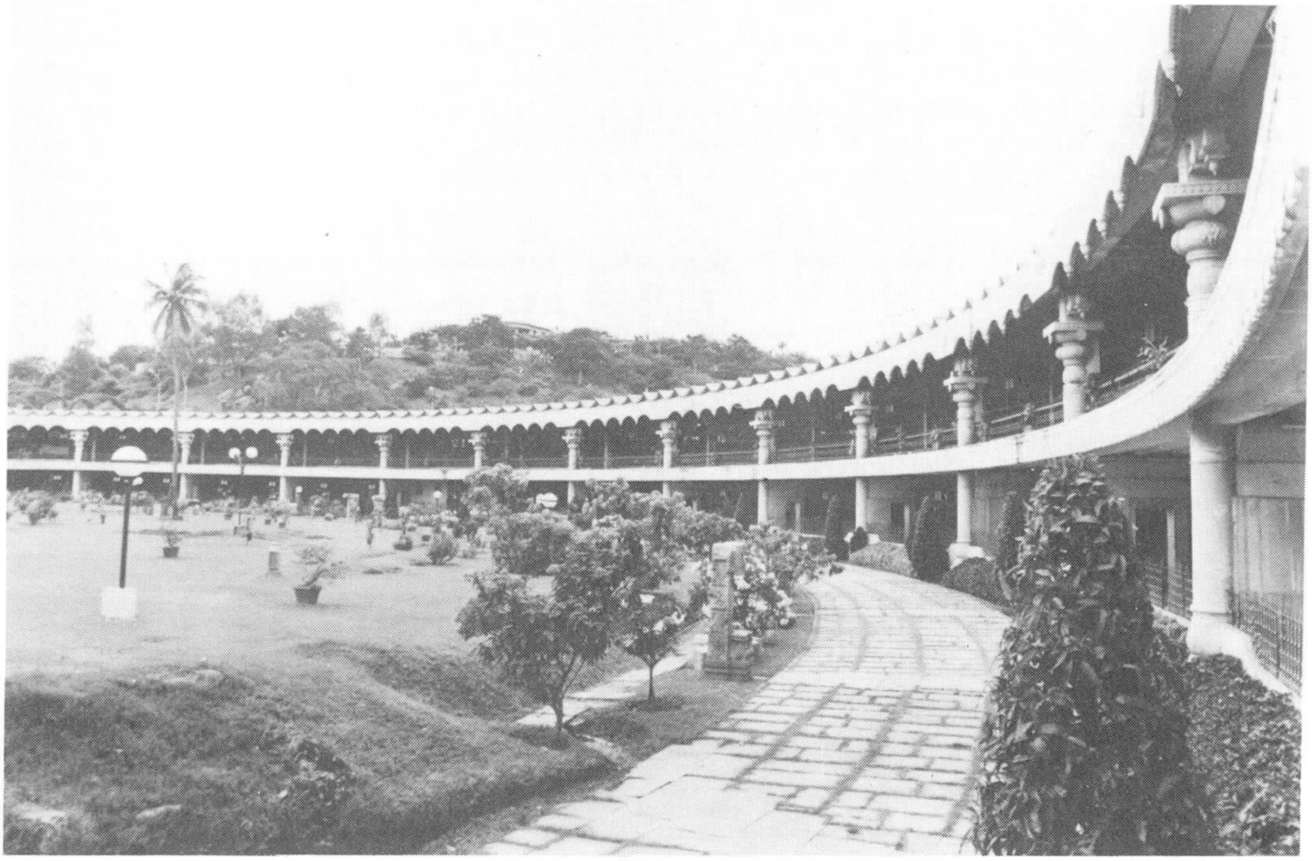
3.2 Construction of the roof:

The construction of the thin folded plate elements of the cantilever structure poses a considerable challenge for construction. The sequence of construction is essentially as follows:

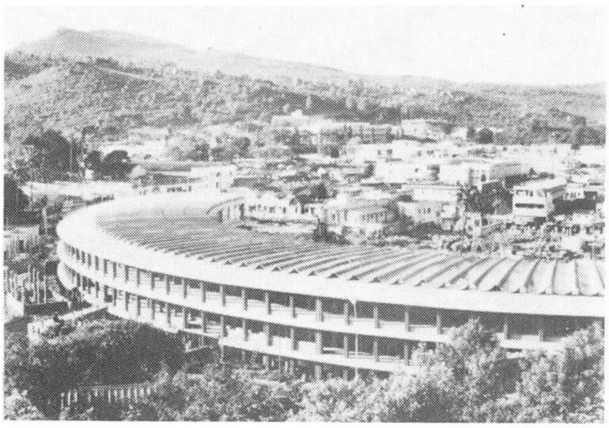
- Construction of P.C.C/R.C.C base of flooring
- Erection of staging and shuttering for the folded plate for four folds.
- Casting of roof in the central and cantilevering portions in a sequence specified by designer.
- Casting of pylons and ties
- Prestressing the ties
- Decentering
- Shifting staging to next four folds and repeating above steps.
-

4. MODERN QUEUE COMPLEX AT TIRUMALA:

The complex is meant for devotees waiting to have a darshan of Lord Venkateswara at Tirupati. It comprises seventeen compartments and is like a semi-circular stadium with a bottom and an upper gallery (Fig. 6)



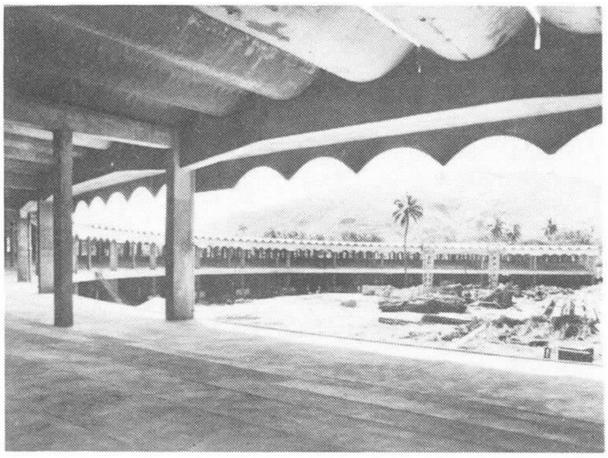
Each compartment contains a reading room, canteen and other facilities for devotees. The upper gallery is made of 323 pre- cast pre-stressed concrete channels and roof consisting of 206 pre-cast reinforced concrete Hyperbolic Paraboloid shells. To facilitate natural lighting, gaps have been left between the roof shells, which are covered with translucent fibre reinforced plastic (FRP) materials. The prestressed channel shaped gallery units are varying in length from 12m to 15m with a width of 915 mm and a depth of 700 mm. High tensile wires of 7 mm dia were used and pretensioning and steam curing was adopted



The pre-stressed channels are supported on brackets projecting from the RCC core walls, in a step-like fashion and form the upper gallery. The gallery steps are finished with marble. The lower gallery is formed by filling with earth and topping with PCC. Cut-outs in the core walls serve as passage for connecting the different compartments. (Fig. 7)

Each of the seventeen compartments can accommodate 400 persons (Fig. 8)

5. ASTHANA MANDAPAM, TIRUPATI:



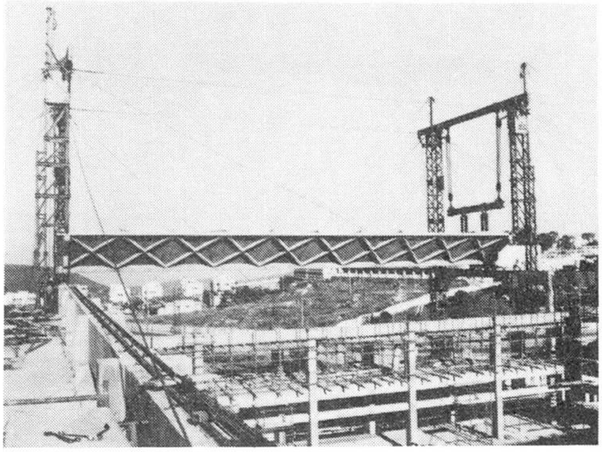
For cultural and religious programmes to take place a complex which has a seating capacity of over 1,500 has been built at Tirupati for Tirumala Tirupati Devasthanams (TTD).

The exclusive feature of this building is a column-free area of 1,700 m^2 achieved by a unique roofing system in precast, prestressed concrete, consisting of nine folded plate elements, each of 30 m span, 6.3 m wide and 60 t in weight. This element is cast on the ground in a timber mould and then erected / placed in position by means of a sophisticated erection scheme.

5.1 Casting:

The mould consisting of timber and plywood has got slab panels which can be lowered after casting, facilitating easy demoulding, refixing and avoiding friction

Prestressing was done by post-tensioning two cables, one of 21 m length comprising six 12.7 mm strands and another of 30 m length consisting of six strands using Gifford-Udall system. Each strand was given a force of 15T(Fig. 9)



5.2 Erection:

Each element was lifted by using four 10 t capacity winches with wire ropes suspended from pulleys hanging from atop the gallow beam. The supporting system comprised four derricks, two of which were on the main columns. Once it was lifted and placed on the trolleys over decking beams, one leg of the smaller derrick on each side was removed so that hauling was done on the rails provided on the beam. The hauling was done by using a chain pulley block and once it was positioned, the unit was again jacked up and placed back in position after removing the rails. The folded plate were waterproofed using two layer tarfelt treatment. (Fig. 10)

5.3 Special features:

The diamond patterned ribs on the bottom of the 'V' type folded plate unit offer a beautiful look when seen from the ground. The economy of this type of roof can be seen from the fact that the slab is only 40 mm thick. Casting being on the ground, the cumbersome scaffolding and staging work was eliminated as the roof is at 10.06 m above the ground level.

6. PRECAST, PRESTRESSED CONCRETE LARGE SPAN BUILDING STRUCTURES

There are many examples of structures built in India in structural steel, cast-in-situ concrete and precast concrete. Some of these structures built in Precast Concrete are described hereunder:

6.1 Storage Structures with Sheet Roofing over "A" Frames

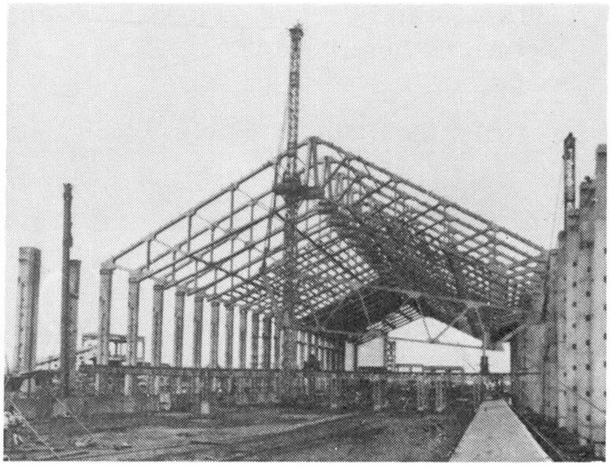
For fertilizer plants in Goa and Tuticorin Precast concrete "A" frames proved to be an economical solution. "A" frames of nearly 40m span were cast in three pieces with two inclined legs and one triangular piece welded together with in situ concrete grouting at a point near the point of contraflexure. Structurally efficient "I" sections were advantageously utilised for the "A" frames to reduce weight for handling. The purlins supporting asbestos sheet roofing with particle board consisted of precast concrete trussed or angle shaped purlins with a thickness of 5 cm. The maximum weight of each element does not exceed 20 tons.

The cross members of the "A" frame besides reducing bending moments in the frame also support the conveyor system as well as walkway slabs. While the conveyor supporting beams are of precast concrete in pairs, walkway slabs are in the form of a ribbed slab with a flange thickness of 4 cm. Compared to structural steel conveyor supporting system and chequered plate walkway platforms, these precast concrete elements proved to be much more economical and durable for the corrosive atmosphere inside fertiliser storage structures.

For storage structure in a cement plant at Satna, unsymmetrical "A" frames were used to match the location of the feed in conveyor. In all other respects it is similar to symmetrical "A" frame storage structure constructed in Goa and Tuticorin (Fig. 11)

6.2 Storage Structure with sheet roofing over trusses:

In the 48m span storage structure for Larsen & Toubro's Cement Plant at Awarpur, the trusses were cast in two pieces outside the building on one side parallel to column grid. The bottom chord of each truss was prestressed at ground level. A specially designed gantry on wheels was used to tilt the trusses and shift the same to the respective places of erection. From the gantry the trusses which were in vertical position after tilting, were hauled inside the building above the conveyor level using two sets of gantries. Three derricks two at the column grid and one at the centre with a platform to receive both the trusses were used for hoisting. After hoisting to the required height, the trusses were made to rest on hydraulic jacks placed over the central derrick and aligned. Two short cables of 3m length were introduced in the sheathing already left in the trusses at the upper end of the lower chord. The central joint was grouted with epoxy and the cables were stressed and grouted. Central jack was released and the truss was made to span 48 m as a single piece. The roof purlins and bracing beams were erected using derricks. An average erection cycle of 10 days per bay was achieved (Fig. 12)



6.3 Storage Structures with Concrete Roof:

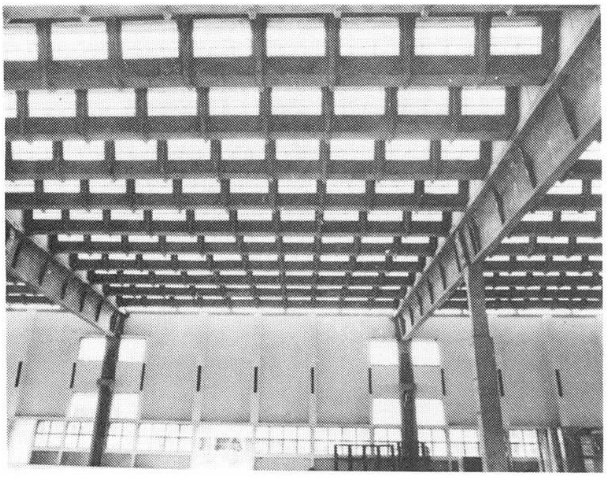
The 55m span 250m long silo at Jagdishpur uses a catenary arch as the main structural system and is 250 m long. The roofing system consists of precast "I" shaped arch units cast in three pieces and channel slabs of 1.5 m width and 8m length. The foundation is a continuous raft with pedestal to support the arches spaced at 8m centre to centre. The beams supporting the conveyor and the walkway slabs are supported on frames suspended from arch units. The entire structure above ground level is in precast concrete. The arches were cast inside the silo area in three pieces, each approximately 30m long and weighing about 20t using a specially designed timber mould supported on steel frames to facilitate vertical casting. The units were picked up sequentially by crawler cranes and made to rest on hydraulic jacks supported on special trestles moving on rails. After alignment, the units were connected by welding and grouting. After assembly, the jacks were released and the arch was made to span 55m as a single unit. The suspender frames were also erected using the same trestle. The roof slabs and beams supporting the conveyor and the walkway slabs were erected by a crawler crane. An average erection cycle of 5 days per bay was achieved (Fig. 13)

6.4 Shells, Folded Plates and Ribbed Slabs as Roof Elements:

Hyperbolic Paraboloid (HP) shells, barrel shells, folded plates and ribbed slabs, both reinforced and prestressed have been advantageously utilised as roof elements for many large span applications in India.

At Larsen & Toubro's hydraulic excavator factory in Bangalore, precast prestressed HP shells 20m in length and 2.5m in width have been used as roof elements together with prestressed concrete roof trusses and lattice type prestressed concrete gantry girders forming a 20m square grid. HP shells were 5 cm thick at the bottom most point and 10 cm at the edges with diaphragms at the end. The weight of each element was 9 tons.

Totally 32 Nos. prestressing cables of 7 mm size arranged in a criss crossing fashion formed the reinforcement in the HP shell. This apart a welded mesh of 10 gauge. 3.25 mm dia at 150mm centres formed the non prestressed reinforcement. These shells were cast in a concrete mould with top surface of the mould having welded mesh and chicken mesh reinforcement in high strength concrete. The top surface was rubbed with carborundum stone and finished with bees wax to give a smooth surface for casting. The end beams of the mould were made in reinforced concrete to support the cross beams transferring the force. These beams help closure of the space by an insulated hood to facilitate steam curing. In a 24 hour cycle of production two HP shell elements were achieved with two moulds. For vibrating the concrete the steel leveller running on the rails on the longitudinal beams of the moulds was used to achieve accuracy in vibrating the concrete during casting (Fig. 14)



For the building of Motor industries company Limited (MICO) at Bangalore 20m span, 3.31m wide folded plate elements were used for the roof. The folded plate has openings for northlight glazing on the one side. This is, one of the largest RCC Folded Plate shells in India. The large depth and small thickness of plate made it economical also. The folded plate has a shell thickness of only 4 cm and the webs of the folded plate were inclined to an angle of 60^{o} to horizontal. The steep inclination warranted shutters not only at the bottom but at top also.

An unique mould was designed and fabricated which had the facility for tilting up/lowering the soffit shutter of the webs. The webs were cast in the horizontal position first. The placing of concrete was very simply accomplished by using skips and gantries and the concrete vibration was carried out using screed vibraters. Two hours after concreting, the entire length of web (22m x 2m size) was tilted to its final inclined position. The reinforcement in the valley was tied next and the concreting completed by casting the valley. Steam curing was resorted to, to attain early strength. The soffit shutters of the webs were lowered and the folded plate removed. The tilting-up and lowering operations were carried out by hydraulic rams specially designed for this purpose. Even though a 36 hour cycle for production of one precast folded plate has been achieved, for operational convenience a 48 hour cycle was adopted. The element weighs only 14.5 tons (Fig. 15)

From the considerations of economy of materials which are relatively more expensive than labour in India and quality, durability, and speed of construction, innovative use of precast prestressed concrete has proved to be extremely successful in India for large span building structures.

CONCLUSION :

The innovative techniques and methods used for large span building structures above coupled with structural design concepts and speed has resulted in construction of elegant and economical structures. This has benefited the world of concrete to a large extent. The varieties of uses to which concrete has been put to for building of large span building structures in India are unique and the adaptations done are economical and pioneering in nature.

Structural Design of the Osaka Dome

Projet du Dôme d'Osaka

Entwurf des Osaka-Doms

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SUMMARY

The Osaka Dome, to be completed in 1997, is a large domed stadium intended for sports events such as baseball and football games and also for concerts and large assemblies. The Dome is composed of a doughnut-shaped stadium with an outside diameter of 200 m and a roof 166 m in diameter. This paper describes the structural features of the stadium, the basic concept for the dome frame, seismic design, wind-resistant design, and the construction process for the stadium.

RÉSUMÉ

Le dôme d'Osaka, qui doit être achevé en 1997, est un stade couvert immense prévu pour les manifestations sportives, musicales et autres grands rassemblements. Le dôme est de forme torique avec un diamètre à la base de 220 m et un diamètre au sommet de 160 m. La communication présente les caractéristiques essentielles des structures porteuses du stade, le principe du cadre du dôme, le dimensionnement contre les effets des séismes et du vent, ainsi que la méthode de construction.

ZUSAMMENFASSUNG

Der Osaka-Dom, der 1997 fertiggestellt sein soll, ist ein grosses überdachtes Stadium für Sportveranstaltungen, Konzerte und andere Grossanlässe. In Gestalt eines Torus hat der Dom einen Aussendurchmesser von 220 m und einen Dachdurchmesser von 160 m. Der Beitrag beschreibt die Hauptmerkmale des Stadiumtragwerks, das Grundkonzept des Dom-Rahmens, die Bemessung gegen Erdbeben und Wind, sowie das Bauverfahren.

1. OUTLINE OF OSAKA DOME

The Osaka Dome is planned as a multi-purpose dome with a seating capacity of 44,000 persons for sports events and a maximum seating capacity of 55,000 persons for other events. Visually, the dome's exterior features "Fiesta Mall", which creates an image of a floating skyline suggestive of waves and clouds that surround the dome. This "Fiesta Mall" is in a class by itself as it can serve as a place for various events separate from the dome.

The dome is provided with a mechanized system that can change the arena/seating space configurations to those most suitable for the event taking place in the dome.

The ceiling shape is also variable, being composed of layers of ring-shaped elements (called "Super-rings") that can be raised and lowered as necessary to create the internal space configuration desired.

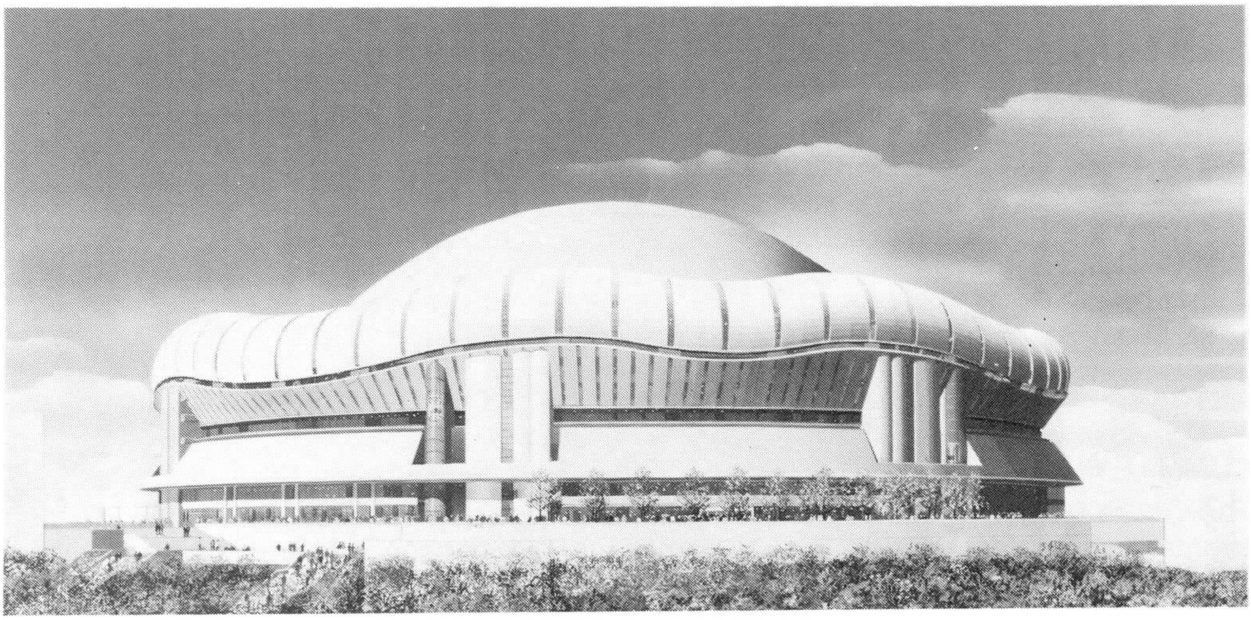


Fig. 1 Exterior Appearance of Osaka Dome

2. OUTLINE OF STRUCTURAL DESIGN

The Dome's roof consists of a 134m diameter center dome portion and a 16m wide perimeter portion which resembles a brim of a hat in shape. This perimeter portion is more gently sloped than the central dome.

Structurally, the roof framing consists of the central dome which is designed to form a uniform geometry of steel lamella and a perimeter portion composed of uniformly laid out 36 pairs of Y-shaped steel girders. The bases of these girders are located on the top of the stands.

As for the stresses developed by the dead loads, compressive stresses are caused in both the radial and the perimeter directions of the center dome portion while compressive and bending stresses are developed in the perimeter portion.

Intensive stresses developed at the borderline area between the center and the perimeter are taken care of by the compression ring beam.

The dome's deadweight which is about 7,000 tons is carried to the substructure by way of the hinged dome bases. Since these hinged bases are interconnected by the tension ring beam and great lateral force is carried by tension hoops, almost no lateral force is transferred to the stand structure below.

The stand structure under the domed roof is of steel framed reinforce concrete construction. From the viewpoint of architectural planning as well as exterior and interior design effects the structure in the radial direction consists of Y-shaped frames which have comparatively low rigidly. On the other hand, the frame in the circumferential direction is provided with shear walls to have high rigidity and strength. The frame in the radial direction and that in the circumferential direction are integrated into one by the floor slab that extends in the circumferential direction to form a rigid and strong structure which looks like a big doughnut.

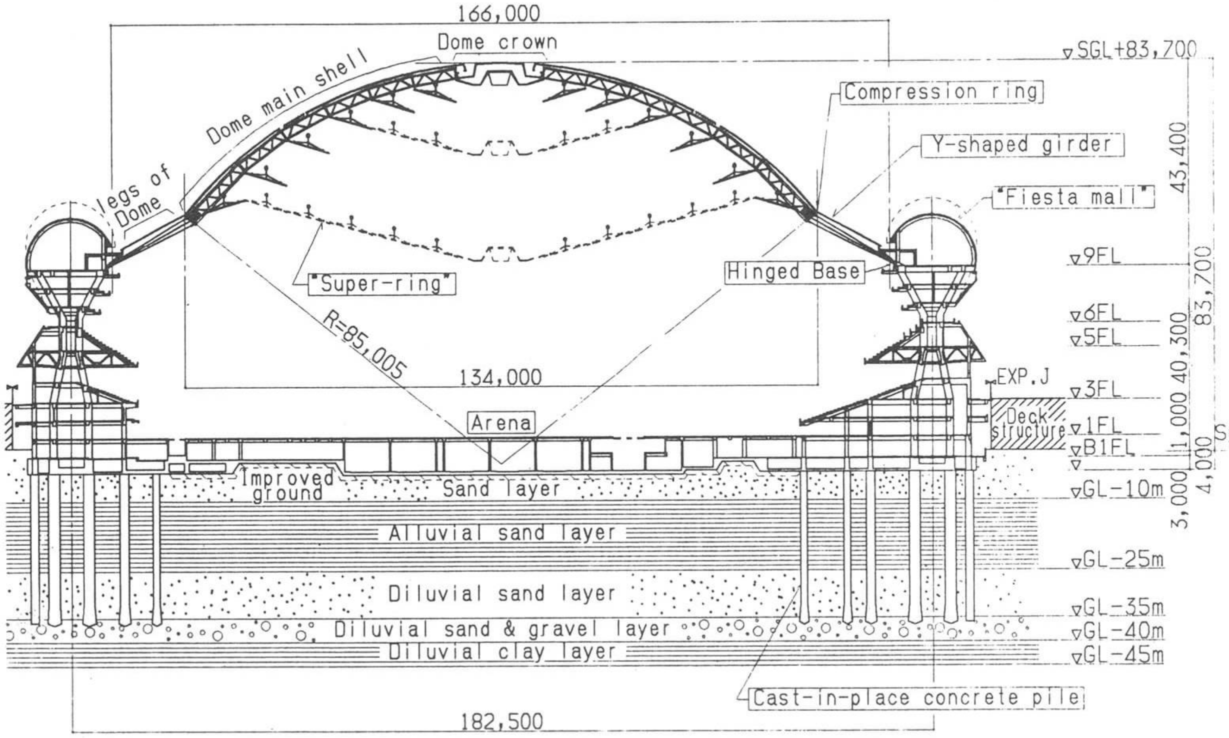


Fig. 2 Outline of Structural System

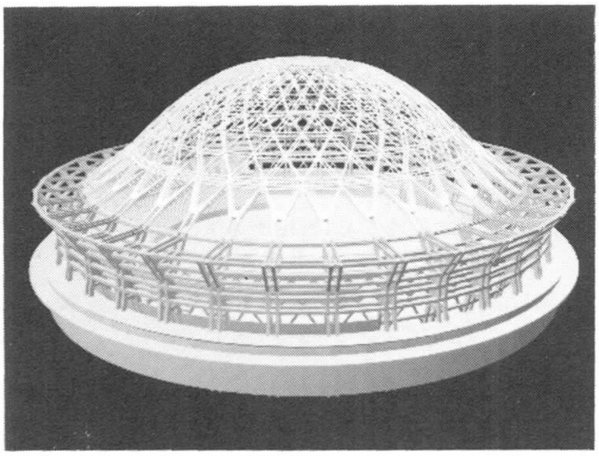


Fig. 3 Structure in perspective

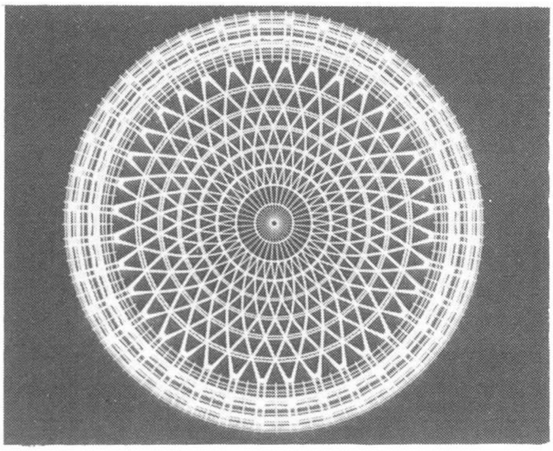


Fig. 4 Structural steel members arrangement

3. DESIGN OF THE DOME

3.1 Design based on dead load

The load per unit surface area of the dome is 250kg/m^2 of which 150kg/m^2 is the weight of the steel frames. Stresses and deflection induced in the dome by this load is as shown in Fig.5

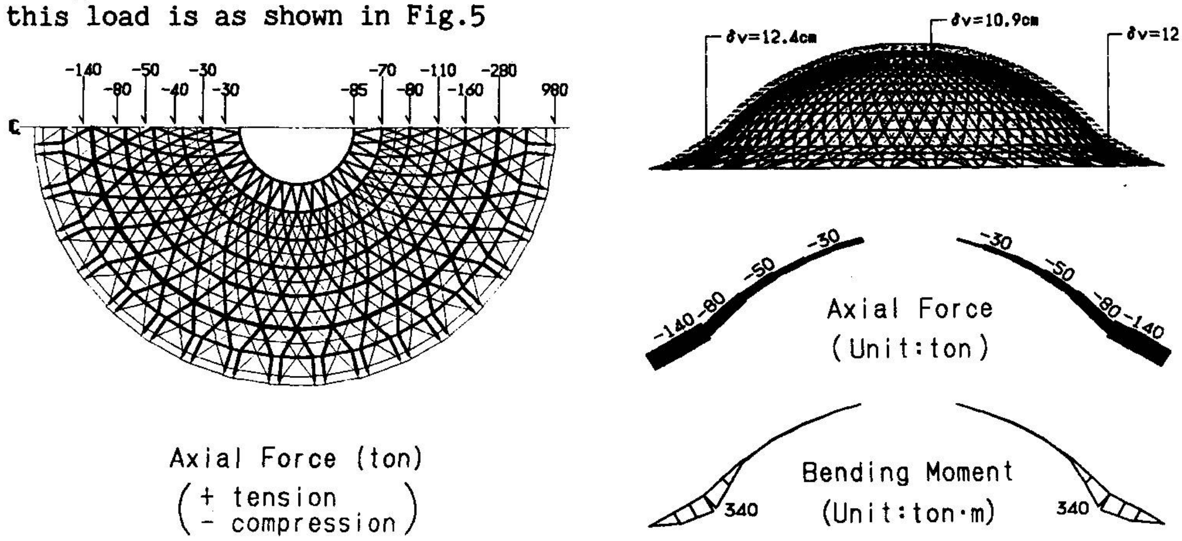


Fig. 5 Stress and Deflection Induced by Dead Weight

The spherical dome portion is subjected to compressive force both in the radial direction and in the circumferential direction. Since Y-shaped girders are subjected to bending moments in addition to compressive forces, these girders are designed to have an H-shaped cross section which has high rigidity against bending (see Fig. 6). As the circumferential direction of the boundary portion formed by the Y-shaped girders and the spherical dome must carry large compressive force, a compression ring made up of highly rigid trusses are located at this portion (see Fig. 6).

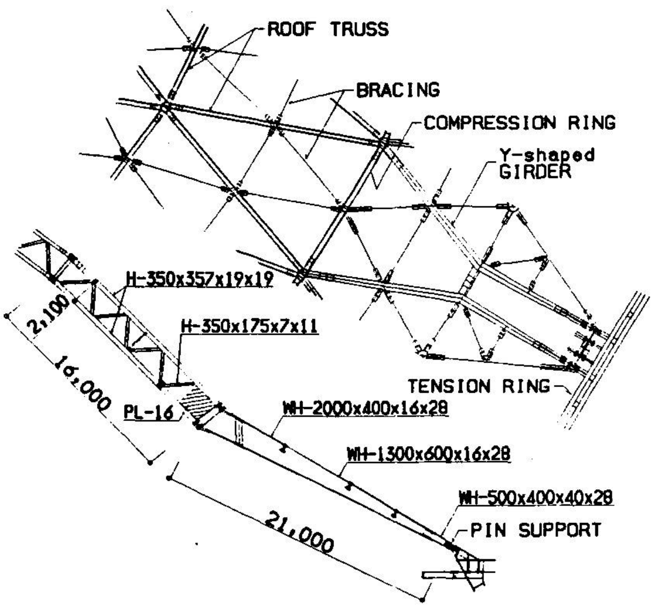


Fig. 6 Y-shaped Girder and Compression Ring

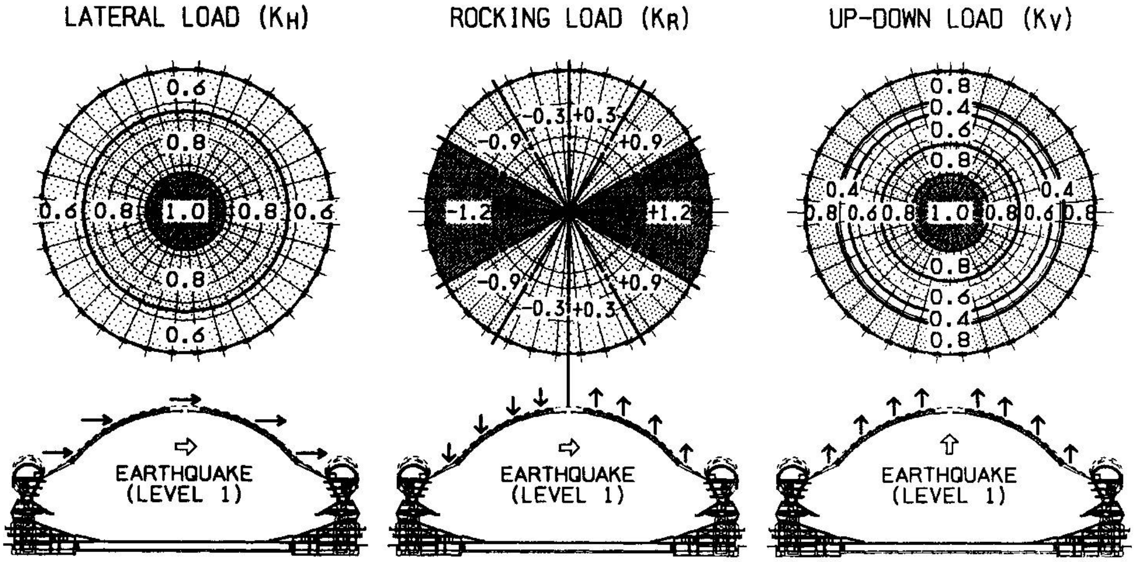
3.2 Seismic design

Earthquake response analyses were conducted to compute the seismic forces acting on the dome during Level 1 and Level 2 earthquakes. The analyses indicated that the dome should be subjected to a lateral force combined simultaneously with rocking motions due to seismic force, and, in addition, up-down motion of the ground should act on the dome. Shown in Fig. 7 is the design seismic loads. Stresses of the dome induced by seismic loads are shown in Figs. 8 and 9.

Notes Level 1 earthquake motions are those which are caused by earthquakes of such magnitude as the structure will encounter a number of times during its life span. (They correspond to Seismic intensity 4 - 5 on Japan's Standard Scale.)

Level 2 earthquake motions are those which will be caused by exceptionally great earthquakes that the structure might encounter during its life span. (They correspond to Seismic intensity 7 on Japan's Standard Scale.) Level 2 load is two times that of Level 1.

As an seismic design principle, the members were required to stay in the elastic region under Level 2 earthquakes and a buckling safety factor of not less than 1.2 was to be secured under Level 2 earthquake.



NOTE: K_H, K_R, K_V are co-efficients of seismic load to be multipled to the weight of roof.

Fig. 7 Design Seismic Loads

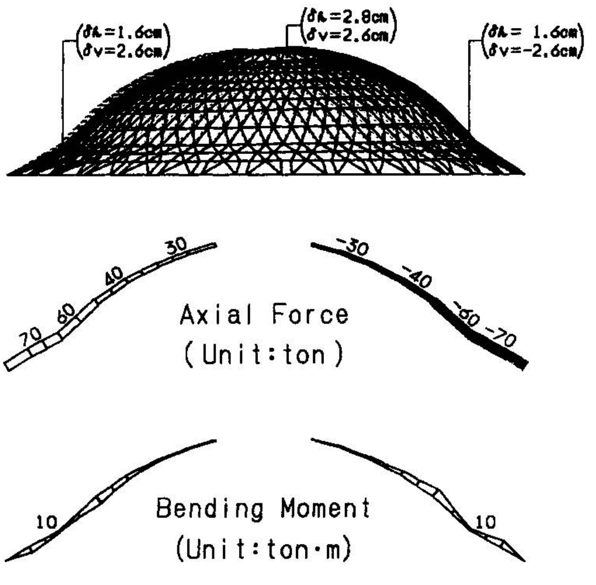


Fig. 8 Stress & Deflection due to Lateral Load

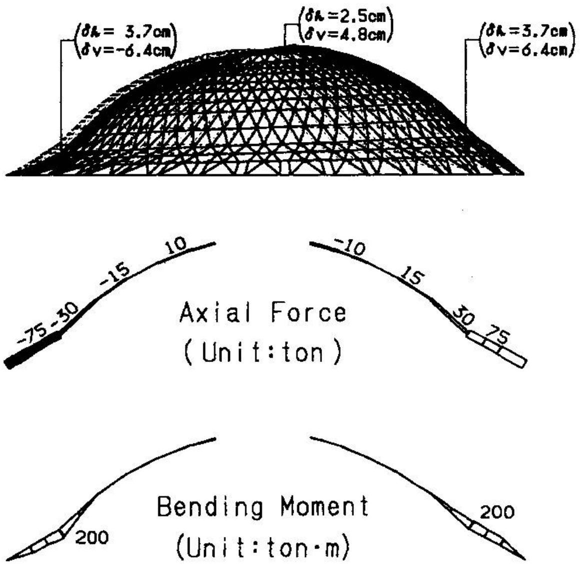


Fig. 9 Stress & Deflection due to Rocking Load

3.3 Wind resistant design

As a design principle, the dome members were required to remain in an elastic region under Level 1 and Level 2 wind forces.

Notes Level 1 wind is such a typhoon that occurs at a 100-year interval and has a wind velocity of 35m/sec at the crown of the dome.

Level 2 wind is such a typhoon that occurs at a 500-year interval and has a wind velocity of 41m/sec at the crown of the dome.

In computing the wind loads for this dome, the external pressure coefficients were established by wind tunnel tests (see Figs. 10 and 11 and also for reference Fig. 12).

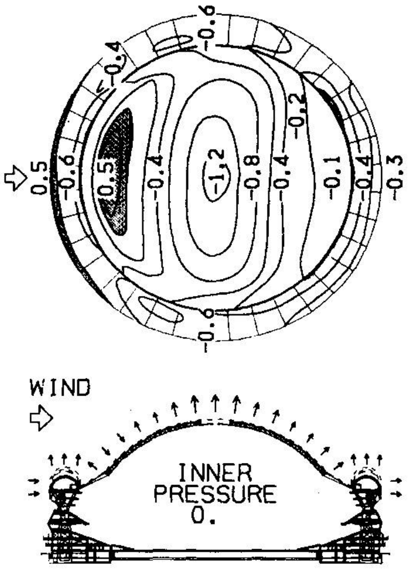


Fig.10 Average External Pressure Coefficients

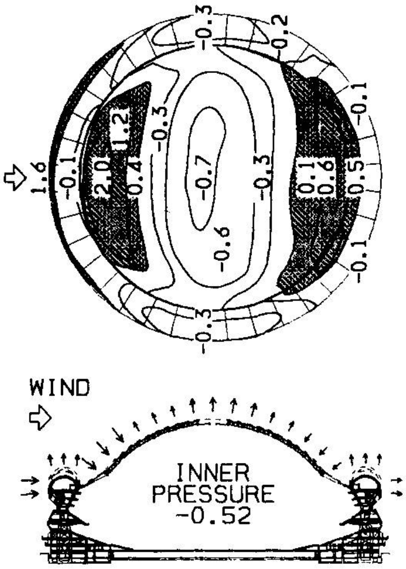


Fig.11 Max. Positive External Pressure Coefficients

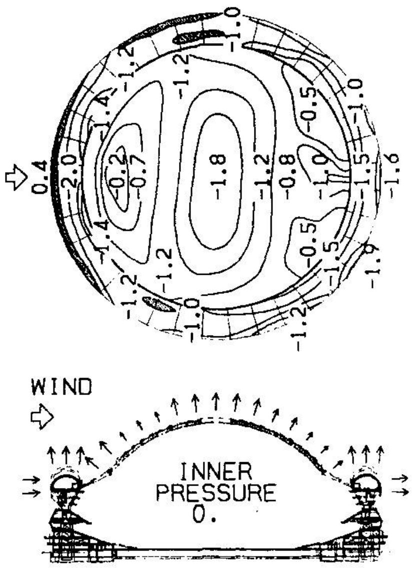


Fig.12 Max. Negative External Pressure Coefficients

It should be noted that the wind loads being smaller than the seismic loads do not constitute a predominant factor for the structural design of this dome.

4. OUTLINE OF CONSTRUCTION

The steel members of the dome will be erected by the lift-up method (see Fig. 13). They will be lifted into place by wires manipulated from 36 erection platforms. Stresses and deformation of the members at principal locations will be measured while these lifting operations are performed.

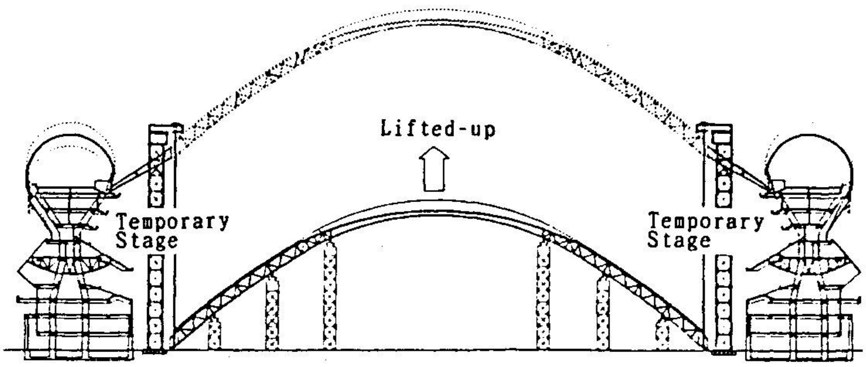


Fig. 13 Construction method

Design and Construction of the Sports Hall Roof Structure, Belgrade
Étude et réalisation de la structure du toit de la salle de sport, Belgrade
Entwurf und Konstruktion des Dachtragwerks der Belgrader Sporthalle

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SUMMARY

A universal Sports Hall for more than 20'000 spectators is under construction in Belgrade. The roof structure of the Hall with spans of 133/103m, is a prestressed shallow lens shaped by the two-chord orthogonal grillwork of externally prestressed reinforced concrete girders with tendons outside the cross section of the concrete. The complete roof structure is prefabricated and is very rational and economical. It covers about 1,5 hectare with the equivalent thickness of concrete in the grillwork of the main roof girders of less than 8 cm/sqm and the weight of prestressing tendons of 6,5 kg/sqm.

RÉSUMÉ

Une salle de sport de 20'000 places et d'usage polyvalent est en construction à Belgrade. La structure du toit, de 133 x 103 m de portée, à la forme d'une lentille plate précontrainte, formée par une grille de poutres en béton armé avec précontrainte extérieure. La toiture est entièrement préfabriquée, de manière rationnelle et économique. La grille principale de poutres couvrant env. 1,5 hectare a une épaisseur équivalente de moins de 8 cm/m2 et le poids du câble de précontrainte est de 6,5 kg/m2.

ZUSAMMENFASSUNG

In Belgrad ist eine Mehrzwecksporthalle für 20'000 Zuschauer im Bau. Das Dachtragwerk mit einer Spannweite von 133 m x 103 m besteht aus einer vorgespannten flachen Linse, die durch einen zweigurtigen orthogonalen Trägerrost aus Stahlbetonträgern mit aussenliegenden Vorspanngliedern gebildet wird. Die komplette Dachkonstruktion wird vorgefertigt und ist dadurch sehr rationell und wirtschaftlich. Bei Ueberdachung von etwa 1,5 Hektaren betragen die äquivalente Dicke des Hauptträgerrostes weniger als 8 cm/m2 und das Gewicht der Spannkabel 6,5 kg/m2.

1. INTRODUCTION

The construction of the Sports Hall for 20.000 spectators has been planned for the Basketball World Championship 1994, the organization of which was awarded to Belgrade before the desintegration of the former Yugoslavia.

The preliminary design of the Hall, made by the Company ENERGOPROJEKT - MDD *Urbanizam i arhitektura*, the author of which is architect Vlada Slavica, has been selected at the competition. The first four authors of this paper, professors of concrete structures the Faculty of Civil Engineering of the University of Belgrade, are the authors of the Hall structural system.

The design of the structure is realized in cooperation with the Faculty of Civil Engineering at the University of Belgrade. The next two authors of this paper are the assistants at the Faculty of Civil Engineering in Belgrade and, together with the authors of the Hall structural system, they are the designers of the Hall roof structure.

The main contractor is the consortium of two large construction companies from Belgrade: ENERGOPROJEKT - MDD *Visokogradnja* and the Construction Company GP NAPRED DD. The last two authors of this paper are the Managing and the Technical Directors of that Consortium.

The execution of prestressing was entrusted to the Institute for Testing of Materials of the Republic of Serbia IMS from Belgrade.

In spite of the fact that in the meantime, due to the United Nations sanctions against Federal Republic of Yugoslavia it has been decided for the Basketball World Championship 1994 not to take place in Belgrade, the construction of the Sports Hall has been continued but in a somewhat slower rate.

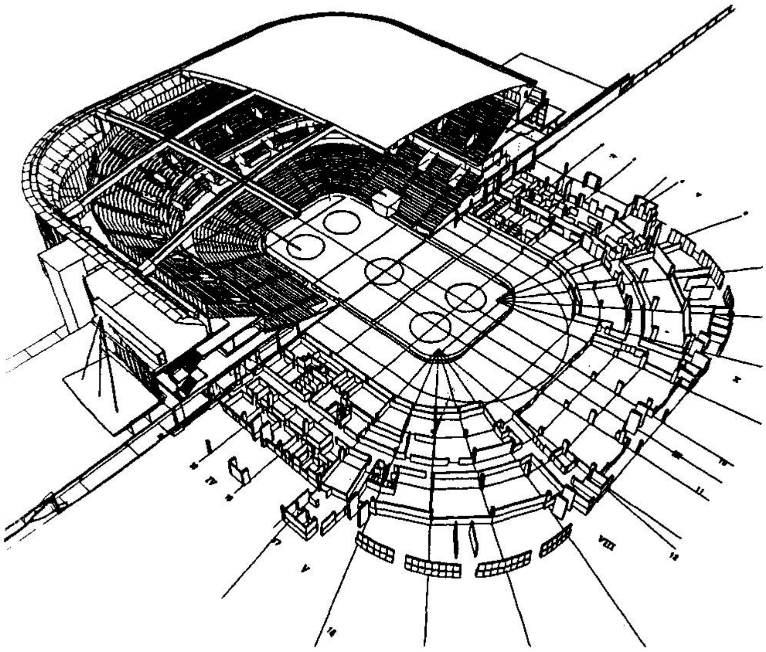

Fig. 1 Axonometric view of the Sports Hall

2. THE ROOF STRUCTURE OF THE HALL

The dimensions of the structure have been designed to provide a possibility for athletic competitions, too. The Hall is of a rectangular base with rounded corners, with spans 132.7 x 102.7 m. The building is 36 m high and the total area of the roof is more than 15.000 m^2. Figure 1 shows the computer made axonometric view of the Hall.

The roof is supported by 14 main columns in facade walls and is structurally completely independent from other parts of the building.

The main roof structure is a two-chord orthogonal grillwork made of 7 externally prestressed RC girders, 3 in the longitudinal and 4 in the lateral directions. Upper compressed concrete chords of the girders polygonally follow convex paraboloid surface with the elevation of +8.00 m with regard to the horizontal supporting plane on the tops of the main columns. Lower chords are designed as prestressing tendons, free in space, which polygonally follow the concave parabolic surface with the sag of -4.00 m with regard to the supporting plane. In 12 cross points of the grillwork the constant distance of the chords is provided by pyramid-shaped "chairs" composed of 4 RC columns of the 35/35 cm section. The RC deviator blocks are on their lower ends. Figure 2 shows the disposition of the roof stucture.

The roof structural system can be understood as a discretization of a prestressed shallow lens the compressed surface of which is composed of polygonal RC girders, its tensioned part of prestressing tendons and its constant geometry, namely the spacing between the surfaces of the system, provided by RC chairs. However, the system can be more simply understood as an orthogonal convex RC prestressed grillwork, elastically supported in its cross points by deviation forces depending on the configuration of the tendon system and on the intensity of external actions and prestressing. Excellent efficiency of such a two-chord structural system comes as a results of a very big eccentricity of the prestressing tendons, which are "taken out" from the section and have incomparably higher eccentricity of the prestressing force than classically prestressed structural elements with tendons inside the cross section of the concrete.

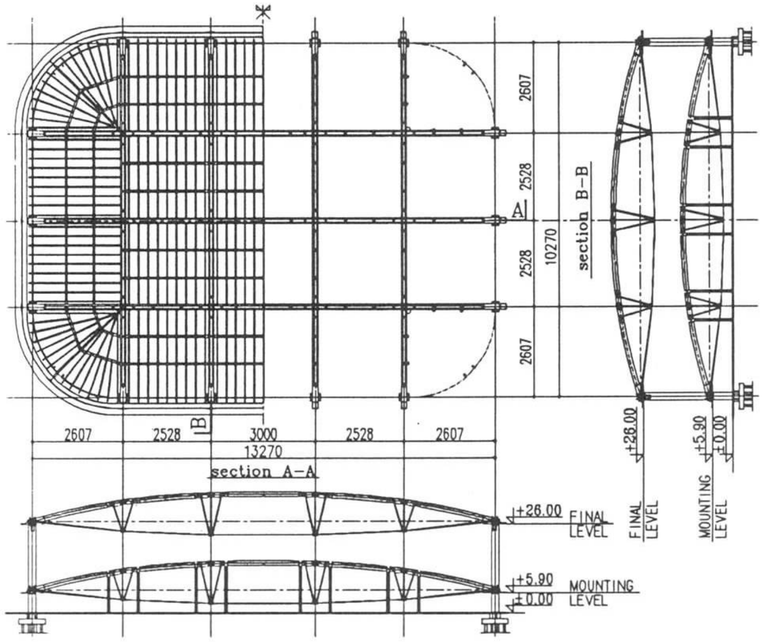

Fig. 2 The roof structure with main roof girders on mounting level on temporary scaffolds and on final level, on the tops of the main columns

The RC chords of the main roof girders are all of the same cross section, constant along the whole span, composed of twin girders of the rectangular cross section, 140 cm high and 40 cm wide, at a distance of 80 cm. The supporting parts of the girders, at the length up to 5.0 m from supports, are full rectangular sections 1.60 m wide, shaped as anchorage blocks for prestressing tendons of the lower chord. The design concrete grade of the main girders is C 50.

The lower chords of the main roof girders are composed of 8 prestressing tendons each. The tendons are made of 11 Neptun grade 1860 strands, ϕ 15.80 mm S, with nominal steel area of 150 mm^2, made of low relaxation steel, permanently corrosion protected with grease and the HPE coating. Anchors type SPB of the Strands Prestressing System, developed in the Institute for Testing of Materials of the Republic of Serbia IMS, have been used for anchorage of tendons. Deviator blocks, shown in Figure 3, are made of cast in situ RC.

Secondary roof girders are also two-chord RC systems, of the spans 23.60 and 28.40 m. The purlins are T section RC girders, with spans 7.20 to 7.60 m.

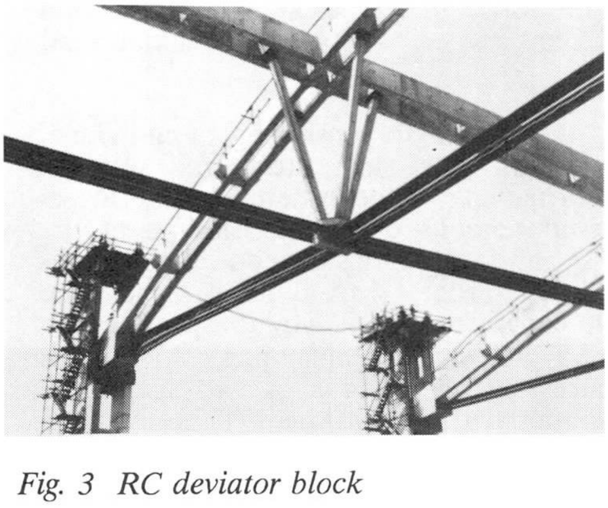

Fig. 3 RC deviator block

The main columns of the roof in facade walls are designed as two RC walls grade C 40 concrete, of rectangular cross section 50/220 cm at the clear spacing of 2.0 m, just necessary for the main girders supports between them.

3. DESIGN AND CONSTRUCTION OF THE ROOF STRUCTURE

3.1 Construction of the main roof girders

Only the 14 main columns were cast in situ. The complete roof structure has been prefabricated on site. The grillwork of main roof girders has been divided to a total number of 43 precast elements - 12 "crosses" that included the parts of two orthogonal girders in the cross zone above the "chairs", the total length of each being about 8.0 m in both directions and the weight about 50 t, 17 beam elements between "crosses", their length being 16-20 m and the weight about 50 to 60 t, and 14 supporting parts, between the first "crosses" and the columns, with anchorage blocks, about 22 m long and almost 100 t heavy.

The mounting level has been on +5.90 m above the floor of the Hall, Figure 2. The precast elements were temporarily supported by light steel tube scaffold towers around 12 chairs with 80 cm free joints left between them. In such a way, before the prestressing started the roof girders grillwork was supported by a total number of 220 supports: 12 x 8 x 2 = 192 temporary supports on scaffolds in the span and 14 x 2 = 28 supports on neoprene bearings in the axes of the columns at the level +5.90 m. The "chairs" RC columns have been also prefabricated. Only the deviator blocks, although designed to be prefabricated, were cast on scaffolds.

3.2 Prestressing

The prestressing of the main girders has been designed to take place in two stages. In the first stage, while the structure has been still on scaffolds, the tendons had been tensioned to the level of approximately 60% of the total force for complete prestressing. Due to prestressing the main girders have been gradually lifted from their temporary supports on the scaffolds. At the end of that prestressing stage the structure had been lifted about 16 cm in the central part of the roof, remaining supported only on 28 supports in the axes of 14 main columns, so that the scaffold were no more necessary.

After the first stage of prestressing the roof structure has been lifted to the tops of the main columns, to the design position. The construction is continued by the erection of the secondary roof girders and purlins, the roof cover and the permanent part of the equipment which is hung to the roof. Upon the application of this additional permanent load the second stage of prestressing will be carried out. The total maximum force in the tendons at the total loading is about 12.000 kN, or 54% of the breaking load of the tendons.

High sensitivity of the system to tensioning of individual tendons is characteristic for such structural systems. Because of the deformability of the system "elastic" losses (losses in prestressed tendons due to tensioning of the succeeding tendons) can many times exceed elastic losses in classical prestressed girders with tendons inside the cross section of the concrete. That is why the prestressing has been performed with 16 hydraulic jacks so that in each step of prestressing all tendons of two symmetric roof girders were simultaneously tensioned. Thereby it has been provided that forces are the same in all tendons of a girder, regardless of big elastic losses.

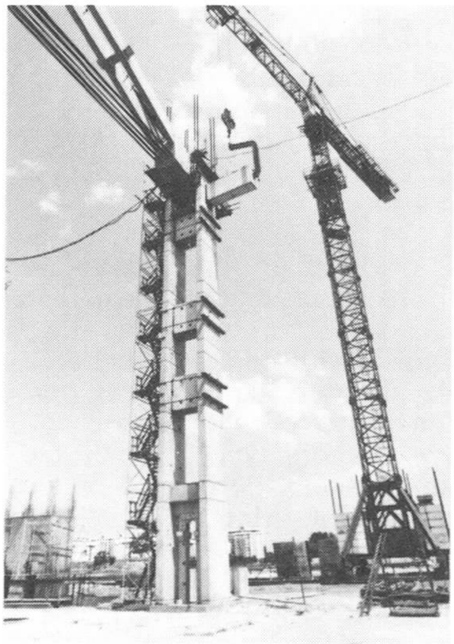

Fig. 4 Pushing of the precast supporting beam through the openings on the top of the column

3.3 Lifting of the main girders to the design position

Lifting of the main roof girders from the mounting level +5.90 m to the final design level of 26.0 m has been done using 96 special hydraulic jacks by pulling the whole grillwork from the tops of the main girders. The rate of lifting was 2 m per hour plus the necessary time for dismounting and remounting the temporary bracings of the column walls, Figure 4. After the level of about 20 cm above the final position has been achieved, the precast 12 tons RC supporting beams have been pushed through the openings on the tops of the columns and the whole structure has been lowered onto the neoprene bearings, Figure 4.

Figure 5 shows the main roof girders in the design position on the tops of the main columns.

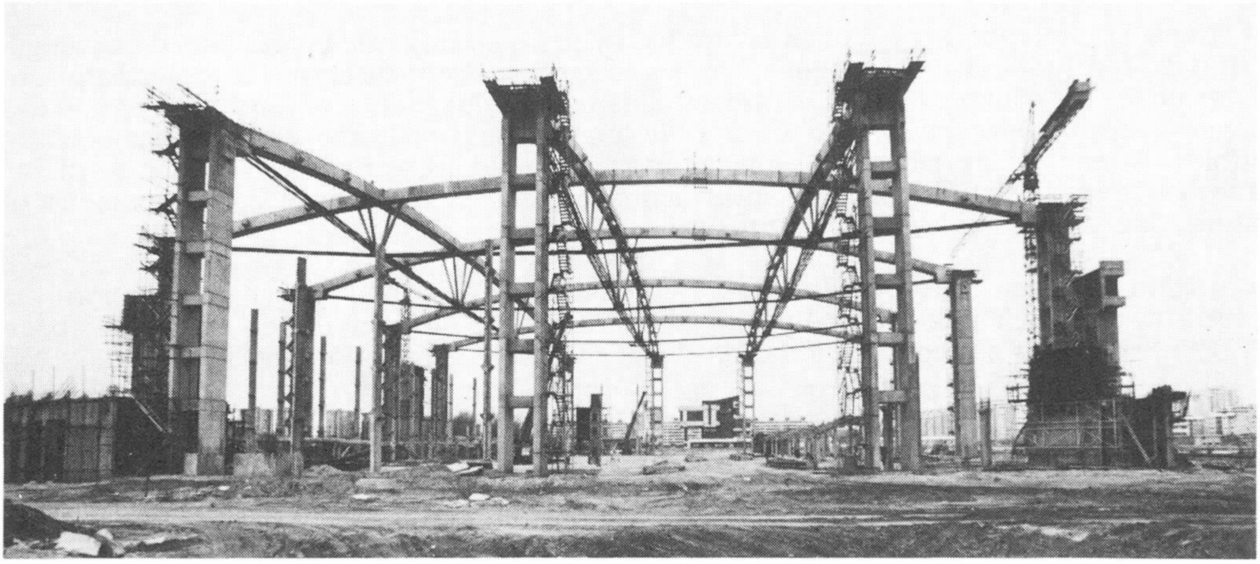

Fig. 5 Main roof girders in the final design position on the tops of the main columns

3.4 Design models

Because of the significant influence of the building technology, the design models had to strictly follow the construction stages.

The first design model covered only the self-weight of the precast elements supported by the temporary scaffolds, before they have been made monolith and without the influence of tendons. The next model has already been a monolith structure of the grillwork with tendons, but still supported on 192 temporary supports on the scaffolds, beside 28 supports in the axes of the columns. In each stage of prestressing that design model was continually transforming into a new structural system because of gradual lifting from scaffold supports, transferring the structure, towards the end of the first stage of prestressing, to the final system of the grillwork supported on 14 main columns only.

The detailed design models of the structural system in all stages of its gradual constituting have enabled the establishment of the optimum shape of the structure together with the most rational sequence and intensities of prestressing. By the optimum analysis it has been achieved for all girders to have equal cross sections, constant along the whole span and the same number of tendons. Besides the obvious technological advantages, such solution has provided the total utilization of the material to be very near to the optimum one.

The previous analyses and experiences with similar linear two-chord big span systems [1] to [3], have shown that the effects of the Second Order Theory slightly deviate from those calculated without taking into account the deformations of the system. This is clear keeping in sight that vertical displacement of the roof lens as a whole is of little importance as the external loading is mainly dead load, the depth of the lens remains practically constant and axial forces "travel" with the system, remaining axial in the deformed system, too.

A similar conclusion refers to the effects of creep. As the structure is suitably "load-balanced", so that there is no significant elastic deflections for the main part of the permanent load, there will be no important deflections due to time dependent deformations of concrete as well.

3.5 Control measurements of stresses and deformations

The complexity of the roof structural system, its sensitivity to relatively small changes of prestressing forces, and especially permanent chainging of the system during the first stage of prestressing required the detailed control of the behaviour of the structure during construction.

The forces in the tendons have been controlled by pressure in jacks, by the pulled-out length of strands and by direct assessment of forces in tendons by measuring the deformation of strands under the transverzal load. However, data on the elongations of tendons could, in this case, give reliable information on the forces in tendons only while the structure has been on scaffolds. After that the measured elongations of the tendons were a result not only of the change of stress in the tendons but, much more, a result of the change in the geometry of tendons, due to deformation of the structure.

The friction has been measured directly on the tendons of two main roof girders, before the prestressing started. The loss of the prestressing force along the span due to friction has been only 5 to 8%, which has confirmed the results of the preceeding laboratory research.

The deformations of the structure have been geodesically observed on 354 points on the main roof girders and on the main columns and the deflections during the prestressing were also continually monitored by deflectometers on all 12 crossing points of the main girders.

The obtained results have very precisely confirmed the design analyses and have enabled the planned building procedure to be conducted with full reliability.

4. CONCLUSION

The prestressed concrete roof structure of the Sports Hall in Belgrade is a very rational and economical structural system. By external prestressing with high eccentricity of tendons both the dead and live loads have been resisted by relatively small forces in the tendons. With such systems the deformations from dead load may be very easily governed with suitable geometry and the choice of intensity of prestressing forces while the live load deformations remain significantly below the permitted limits. The total snow and wind deflection in the middle of the roof of the Hall is lower than 18 cm, namely about L/600 with regard to a smaller span.

The complete RC roof structure is prefabricated on site. It has been relatively quickly mounted and is very lightweight. It covers the area of about 1.5 hectare, with the equivalent thickness of concrete in the main roof girders of less than 8 cm/m^2, the weight of prestressing tendons of about 6.5 kg/m^2 and the weight of reinforcement lower than 12 kg/m^2.

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Design and Construction of Kupolen Exhibition Hall and Sports Arena

Conception et construction de la salle d'exposition et de sport de Kupolen

Entwurf und Bau der Ausstellungs- und Sporthalle von Kupolen

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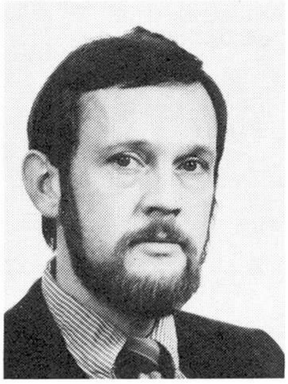

Hakan Sundquist, born 1944, graduated at the Royal Inst. of Technology, Stockholm. After having worked for over 20 years as consultant and contractor, he received a chair as a professor in Structural Design and Bridges at the Royal Inst. of Technology, Stockholm in 1992.

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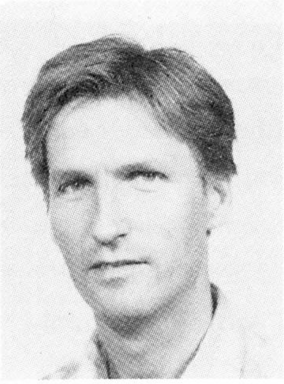


Torbjörn Granaht, born 1950, graduated at the Royal Inst. of Technology, Stockholm. Since his examination he has worked as a chief engineer at the Design Dept of SIAB Contractors.

SUMMARY

This article describes some of the interesting features of a multi-purpose project containing a huge dome structure with a free span of 123 m. The construction of the dome was done using an innovative construction process using no false work or other temporary framework. The dome has a rather great slenderness ratio. The article describes some of the special problems, such as the large amounts of snow avalanching from the sides of the dome.

RÉSUMÉ

L'article présente quelques aspects intéressants d'une installation à usage multiple, dont les particularités d'une structure géante en forme de coupole de 123 m de portée. La mise en oeuvre s'est déroulée selon une méthode novatrice, sans recours à un échafaudage ou autre support temporaire quelconque. Les auteurs soulignent le remarquable élancement de la coupole, et présentent des problèmes spéciaux tels que celui du glissement possible d'importantes masses neigeuses de la surface courbe.

ZUSAMMENFASSUNG

Es werden einige interessante Aspekte einer Mehrzweckanlage beschrieben, die als Kennzeichen ein riesiges Domtragwerk mit 123 m freier Spannweite enthält. Der Bau wurde in einem innovativen Vorgehen ohne jegliches Lehrgerüst oder anderer temporärer Unterstützung errichtet. Der Dom besitzt einen bemerkenswerten Schlankheitsgrad. Unter anderem werden spezielle Probleme angesprochen wie die grossen Schneemengen, die vom Dom abgleiten können.

1. Project

Kupolen in Borlänge, situated about 250 km north-west of Stockholm, Sweden is a result of an idea that came up during discussions between the town of Borlänge, an exhibition arranger and the real estate developer and contractor Siab. Their common interest was to combine the towns need for an indoor sports arena at a low cost, the arrangers need of a larger exhibition space an the developers interest for a real estate development. In exchange for the building rights for the Kupolen project and also some other building rights, the town got their sports arena large enough to contain a soccer plane with full international size and with 3000 seats. **Figure 1** shows two perpendicular sections through the project and **Figure 2** gives an impression of the finished structure.

The 7000 m^2 floor area of the main floor can also be used for large exhibitions, **Figure 3**, with a back up area 5000 m^2 for lounge, conference centre etc. To complete the project and to help financing the project, it also contains a 15000m^2 shopping mall under the main floor, a hotel and an office block with 4 stories. These 2 buildings are both situated under the cupola and along the long sides of the sports area but with no structural connection to the dome. The Kupolen sports and exhibition hall was constructed in 1989.

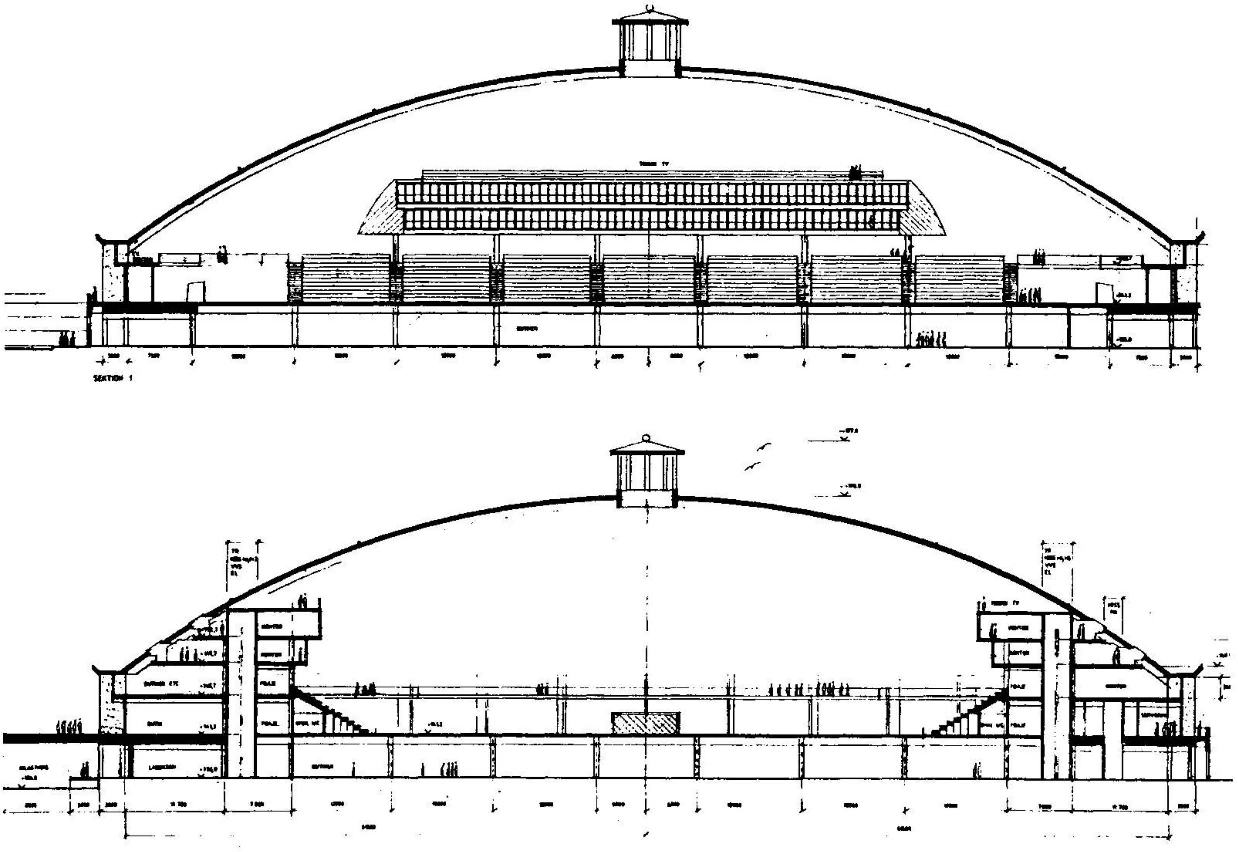


Figure 1 A longitudinal and a transversal section through the project.

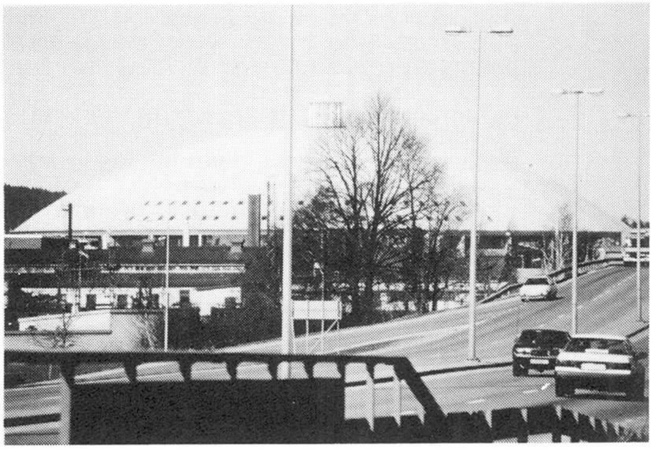


Figure 2 Outside view of the dome.

Figure 3 Inside view of the dome.

2. The dome

2.1 Structural design

The dome structure is supported by 40 generously proportioned columns. On top of the columns there is a post tensioned ring beam. The effective area of the ring beam is about $1m^2$ and the effective tension force is approximately 10 MN. The length of the ring beam is about 400 m. The tension system used is 6 cables each with 12 0,6" strands and with lengths of about 90 m. They were set out two by two with displaced joints. The form work for the ring beam situated about 7 m above the main floor area was done using prefabricated curved shell concrete elements. These structures that form a part of the final ring beam were completed with reinforcement and cables before placing the concrete, see **Figure 4**.

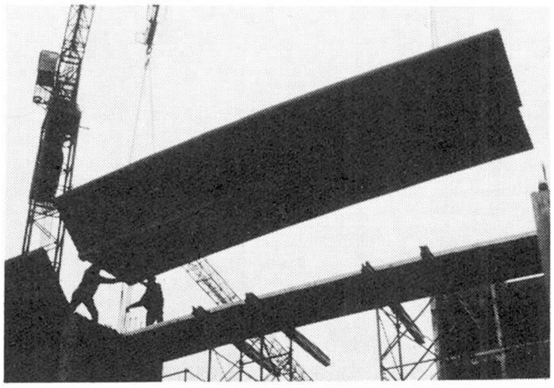


Figure 4 The ring buttress beam was constructed using prefab shell elements also forming the water drainage system.

The actual cupola structure is made up by steel beams forming a Schwedler dome. The total thickness of this structure is just 350 mm and all beams were made using welded H-beams. Since the circumference is approximately 400 m the distance between the main beams is 10 m the first turn. The ring structures are also made up using welded steel H-beams and were placed with a distance of about 6 to 7 m. Diagonals made up with hollow square welded steel pipes completed the net of the dome. The thickness of the plates forming the beams were varied from thick close to the buttress to thinner closer to the top to take account of the variation of moments and forces.

One reason for choosing such a thin structure is that there are many windows belong to the hotel and the office block in the cupola. The schwedler net of beams were covered by elements made up of thin steel box sheeting and plywood. The elements were complete containing insulation and had a bottom perforated corrugated steel sheeting for noise absorption. The element had prefabricated roof topping of white Sarnafil rubber coating.

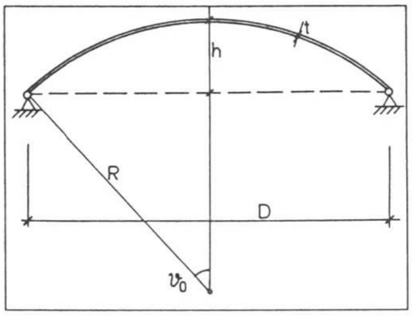


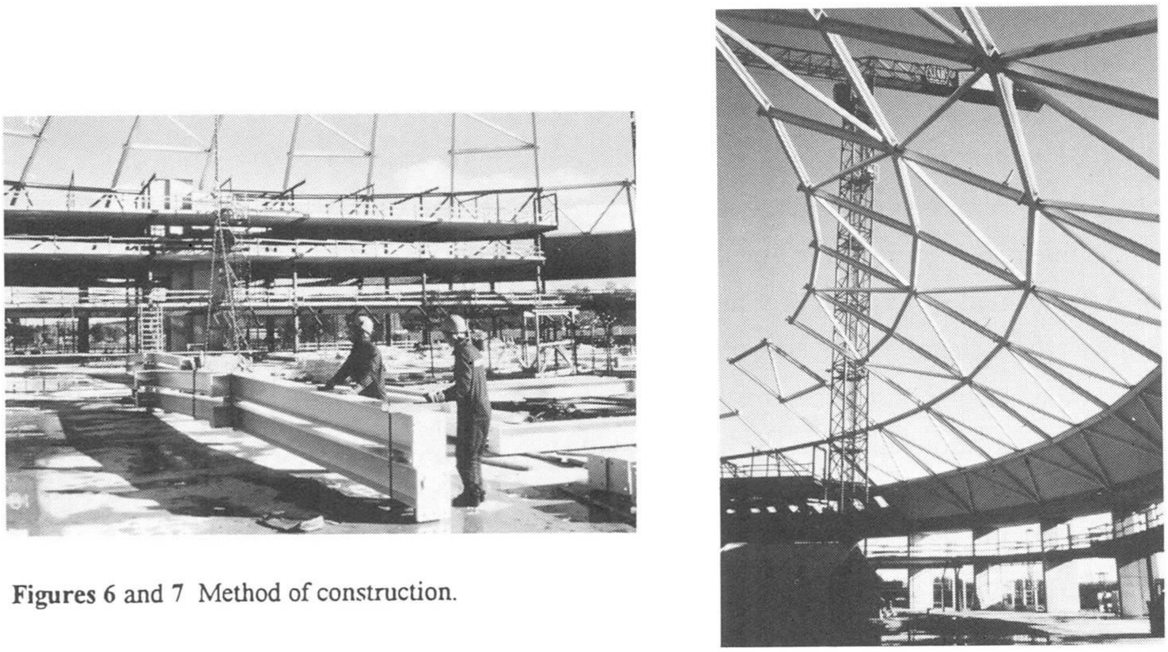
Figure 5 Measures for the structure. R = 100m, D= 123 m, h = 21m and t_{eff} = 80mm.

The radius of curvature of the structure is 100 m and the free span is 123 m. The stiffness of the finished structure could be calculated having a equivalent thickness of 80 mm leading to an slenderness ratio (R/t) of 1250 for the shell, (**Figure 5**). The structure were calculated for different symmetrical and non symmetrical snow and wind loading combinations both using standard simple hand calculations and second order FEM analysis using a system with more than 10000 degrees of freedom. The structure is also calculated to be able to carry point loads in different combinations from the exhibition in the joints. The structure is also controlled for the accidental situation were one main beam has been taken away.

2.2 Method of construction

The dome structure is made up of 1140 beam elements. Due to the geometry of the system chosen, it was possible to reduce the amount of different elements to only 32 different types. Between the joints all beams are straight, so all changes in angels are made at the joints. The elements are screwed together using high strength frictional bolting mainly with end plates at right angels to the direction of the beams.

The erection of the dome was done using no false work or other temporary strutting. First were elements containing two main beams with length approx. 12 m and the ring and diagonal beams between these two erected from the concrete ring beam, **(Figure 6)**. After all elements of the first turn and the ring beams connecting these had been erected the second turn was erected from the first, **(Figure 7)**. The method used, was like constructing a snow igloo. Directly after one turn was completed it was covered by the roof elements.



Figures 6 and 7 Method of construction.

3. Structural system for the lower floors and the foundation

3.1 Framework

The main floor of the building is cast in situ concrete using a new method for composite action between steel sheeting form work, steel beams and concrete. The system allowed the need for very little strutting of the form work despite that the columns had the spacing of $12*12\ m^2$. The floor is designed to carry 54 ton lorries.

On this floor were cast the 40 columns carrying the ring beam that founds the base for the dome structure.

The office and hotel blocks was constructed using steel framework and pretensioned hollow core slabs. These structures have there own framework and have no structural connection to the dome.

3.2 Foundation

The soil strata underlying the project consists of silt to great depth, more than 50 m. Piling was considered but should have been very costly due to the great depth to the rock. Although the soil investigation showed the risk for rather great both total and differential settlements, a continues concrete footing was chosen. The risk for differential settlements was great, due to the existing ground slope of about 5 m from one end to the diagonal end of the building. Vertical drainage in combination with surcharge loading was recommended in the soil investigation report for compacting the upper layers of the silt, but the time needed for this operation was to short due to the tight schedule of the project. An other problem was the risk for water upheaval during excavation because the ground water table was just a few meters below the excavation level. The final solution chosen was the installation of vertical sand drains with centre distances of 12 m to a depth of about 10 m the minimise the pore pressures and to use a very careful and well planned method for excavation. The excavation started in the low end allowing the water to drain out in ditches with a centre distance of 12 m. The ground was directly cowered with fibre texture and a rather thick layer of gravel carried out by crane. After compacting the gravel, the concreteing of the concrete slab started. The bottom slab with a minimum thickness of 250 mm and a thickness of 850 mm under the columns has no contraction joints and is accordingly constructed in one piece $135*150m^2$. The structural design of the foundation slab is by assuming the ground acting as an elastic media with point loads from the columns spread out by the elastic slab.

4. Special problems

4.1 Acoustics

The sports arena is for multi-purpose use so it could also be used for concerts and other musical events. Since the office and hotel blocks are situated inside the dome but structurally independent, it was a difficult task to design the sound insulation between the dome structure and the wall of the above mentioned buildings. Also the windows directed into the exhibition hall had to be insulated for high noise levels.

An other interesting problem has been detected when there are heavy pop concerts in the Kupolen. At a few times the whole building including soil strata below has come into slow vibrations. These vibrations have not caused any problems but have been noticed in buildings close to the project.

4.2 Fire protection

Only the part of the steel structure close to the inside buildings have been fire protected using fire protection painting. Thorough investigations have shown that the temperature of the main part of the dome could not reach dangerous temperature levels.

4.3 Snow problems

In the wintertime the Dalkecarlia area has lots of snow. The large area of the dome collects lots of snow and there is a risk that the snow is avalanching down with high speed from the dome and can be a problem for all the people visiting the sports arena and the shopping mall. The snow precautions are made up first by rings of steel wires allowing the snow to come down in small flakes and also reducing the speed. As a second and hopefully final stop the concrete buttress ring is formed with a 3m wide and 1 m deep snow and rain drainage canal, see **Figure 4** . The drainage canal is at the outer edge completed with a snow stopping fence. The canal is also fitted with heating. Despite all these precautions there still has been snow coming over the edge when the canal is filled over the fence and the weather is so cold that the heating could not melt the snow.

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Design of a Long-Span Prestressed Concrete Spherical Shell Roof
Projet d'une coupole sphérique de grande portée, en béton précontraint
Entwurf eines weitgespannten Kuppelschalendaches

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SUMMARY

This paper deals with the structural design of a large-span reinforced and prestressed concrete dome shell, which will be constructed underground in a park and will support large loads including soil. The structure of 110 m in diameter and 16 m in rise has a roof of an arena of an underground gymnasium facility, and supports all the loads (about 5 tons/m2) on the roof. Its tension ring is made of prestressed concrete, while the shell is a composite structure consisting of precast prestressed concrete beams and slabs, as well as cast-in-place concrete.

RÉSUMÉ

Les auteurs présentent l'étude d'une coupole de grande portée, en béton armé et précontraint, destinée à recouvrir une salle de sport souterraine. D'un diamètre de 110 m et présentant une flèche de 16 m, elle a été conçue pour supporter de fortes charges (y compris la couverture de terre), à savoir 5 t/m2. L'anneau tendu est réalisé en béton précontraint, tandis que la coque se compose de poutres et de dalles précontraintes préfabriquées, complétées par du béton coulé sur place.

ZUSAMMENFASSUNG

Der Beitrag beschreibt den Entwurf einer weitgespannten Stahlbeton-/Spannbetonkuppel, die in einem Park eine unterirdische Sportstätte überspannen wird. Mit 110 m Durchmesser und 16 m Stich ist sie für grosse Lasten (einschliesslich Ueberschüttung) von 5 t/m2 ausgelegt. Für den Zugring wird Spannbeton verwendet, während die Schale aus vorgefertigten Trägern und vorgespannten Platten mit ergänzendem Ortbeton besteht.

1. INTRODUCTION

The building introduced in this paper is a city gymnasium intended to serve as the core of various sports facilities in the city of Osaka, Japan. The building now under construction comprises two circular arenas, one large and one small, and is planned to have most parts located underground. The project is quite unique in that it intends to integrate a large-scale structure with a green-rich park in an urban area of one of Japan's large cities, which have a green-area deficiency problem.

The aforesaid large and small arenas were planned so that their roofs would support soil fill planted with trees and vegetative cover to form a part of the park's landscape while creating large-scale spaces under these roofs. The main arena and the sub-arena have diameters of 110m and 52m respectively.

To cover these two arenas, spherical concrete shell roofs provided with prestressed tension rings at the perimeters were proposed and adopted. The present paper introduces some structural design features of the spherical prestressed concrete shell roof that covers the main arena.

2. BUILDING OUTLINE

- Primary intended use : Gymnasium
- Owner : The City of Osaka
- Site area : 123,986m^2
- Total floor area : 38,425m^2
- No. of floors : 3 floors underground
- Height : Building - 30m above the datum G.L
- Foundation : - 11.5m below the datum G.L.

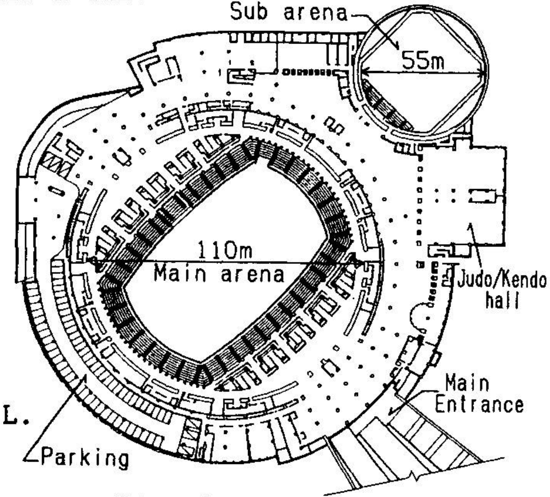

Fig. 1 General Plan

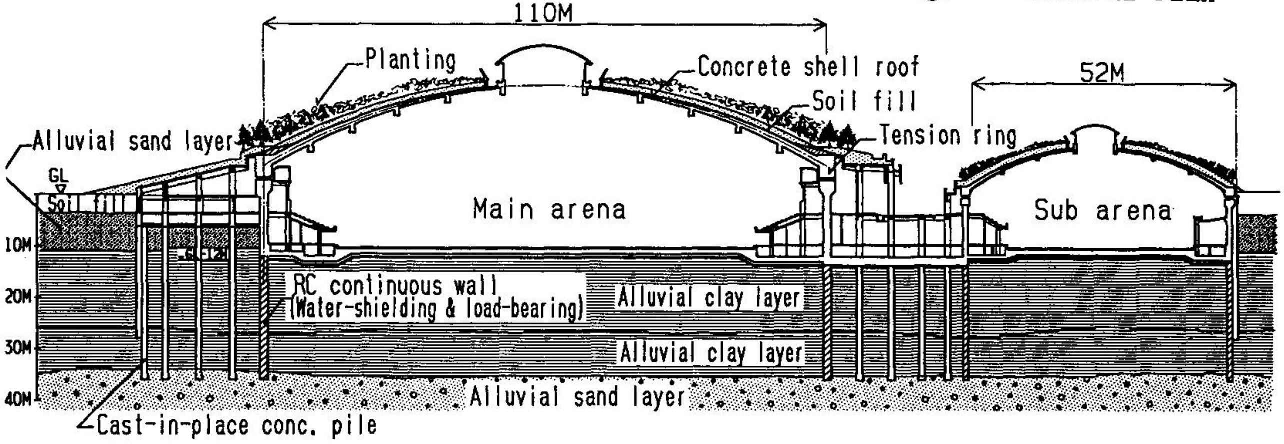

Fig. 2 Structural Cross-Section

3. STRUCTURAL DESIGN

3.1 Structural Design of Spherical Shell

(1) Structural Features of Spherical Shell

The prestressed concrete spherical shell used for the roof of the main arena of this project has the following structural characteristics.

1) The spherical shell roof of the main arena is a long-span structure having a diameter of 110m and a rise of 16m.
2) As has been mentioned, the shell forms the roof of a large-size gymnasium located underground in a city park and is overlain by soil fill of about 1.0m in average depth (0.6 - 1.5m). Thus, the roof must support a total load of about 5 - 6 tons/m^2.
3) Reinforced concrete is used for the main part of the shell structure with cast-in-place prestressed concrete (PC) used to form the perimeter tension ring into which tensile force of about 20,000 tons is to be introduced.
4) To facilitate construction and in the interest of economy, composite construction using precast PC slabs and beams and cast-in-place concrete elements was adopted for the shell portion (see Fig. 4).

(2) Structural Design of Spherical Shell

Fig. 3 shows a conceptual drawing indicating the flow of force in the spherical shell. As shown in the drawing, when a vertical load is applied to the shell, compressive force is produced in the radial (radius) direction of the shell. This compressive force is transmitted to the tension ring and causes a tensile force which tends to expand the tension ring and the shell perimeter. This tensile force is countered by the prestressing force introduced in the circumferential direction of the tension ring. In view of this, it was decided that this portion would be designed to form a prestressed concrete structure in which compressive force is pre-applied to concrete by way of prestressing strands which are located in the concrete along the shell circumference.

Where a long-span roof structure must support a heavy load, as is the case with this project, the use of a spherical shell made of concrete that has high compressive strength provides a highly economical and rational solution because in such a shell, most loads imposed on it would be supported as if they were compressive loads. By introducing prestresses into the tension ring, the low tensile strength which is a disadvantage of concrete can be compensated for and this makes it possible to utilize concrete in compression effectively over the full sectional area. If, for instance, a similar shell roof is designed using structural steel, the steel members will have to be designed for extremely high stresses. This will result in an uneconomical structure which is also subject to large deformation. Further, the present roof structure has an additional advantage because it enables prestresses to be introduced stage by stage during construction as concrete placement and soil filling progresses and this makes it possible to control stresses in the concrete and to minimize deformation at each stage of construction.

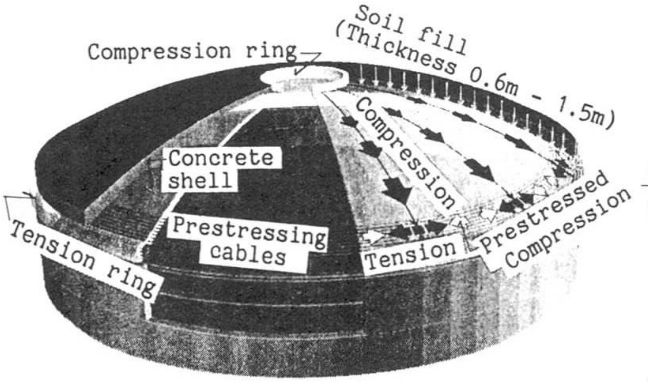

Fig. 3 Conceptual Drawing Indicating Flow of Forces in the Shell

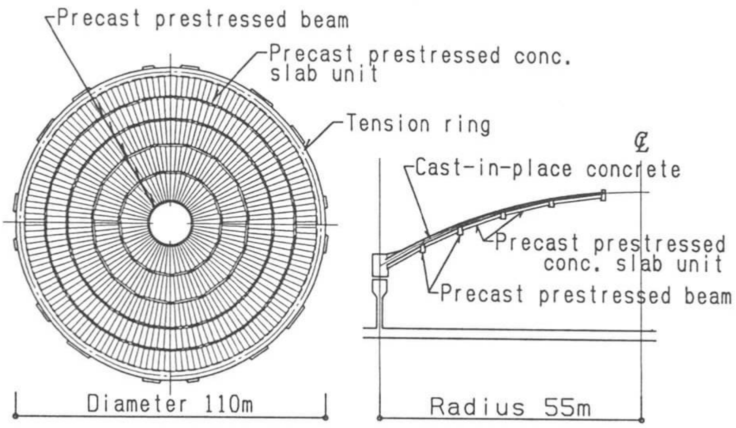

Fig. 4 Simplified Drawing of Main Arena's Shell Roof

The roof of the main arena is composed of a spherical shell portion which varies in thickness from 480 - 1200mm, a tension ring beam (a cast-in-place concrete rectangular beam 4.8m in depth by 3.0m in width), and a compression ring (a precast PC beam 16m in diameter) located at the crown of the shell. As shown in Fig. 4, the spherical shell portion is of a composite construction consisting of five precast PC ring beams all concentrically related to the perimeter tension ring, precast PC slab units (DT slab units) laid between the aforesaid ring beams, cast-in-place concrete elements.

The reasons for using precast prestressed slab units and beams in this structure are as follows: Precast concrete slab units serve as formwork for cast-in-place concrete; hence, scaffolding work and formwork can be almost entirely eliminated. This also enable precast PC units to be used more effectively for structural purposes. Moreover, since these precast units are supported by precast PC beams which have high rigidity, supporting elements may be concentrated at the support points at the ends of these precast beams. Further, because the deflection of these precast beams is small, the support points may be widely spaced.

Fig. 5 shows the detail of the support points near the tension ring of the main arena. As shown, rubber elements are used at the perimeter tension ring of this spherical shell to enable the shell to deform in the direction of the radius. These rubber elements are used to relieve the spherical shell of lateral deformation and long-lasting deformation due to creep that are likely to occur when a vertical load is imposed on the shell or a tensioning force is introduced into the prestressing strands.

3.2 Planning for Construction of the Spherical Shell

Fig. 6 shows the construction sequence for the spherical shell roof of the main arena. The work performed at each stage in this sequence is as follows.

(1) Install rubber supports, place steel reinforcements, and then pour concrete. When concrete has cured well, tension three cables that correspond to about 10% of all the cables. (Primary tensioning)

(2) Erect props to support precast PC beams and place the PC beams (about 7.6 - 9.4m long) on the props in the direction of the radius. Then, lay precast PC slabs units between the PC beams.

(3) Place steel reinforcements in the upper surface portion of the shell

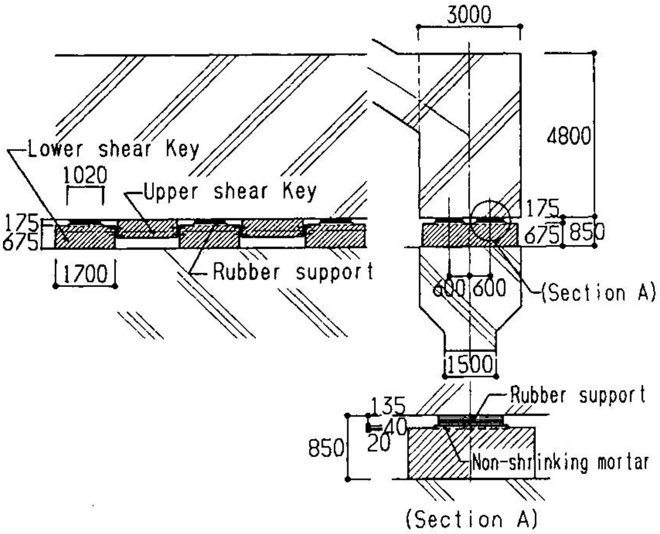

Fig. 5 Detail of Support Points

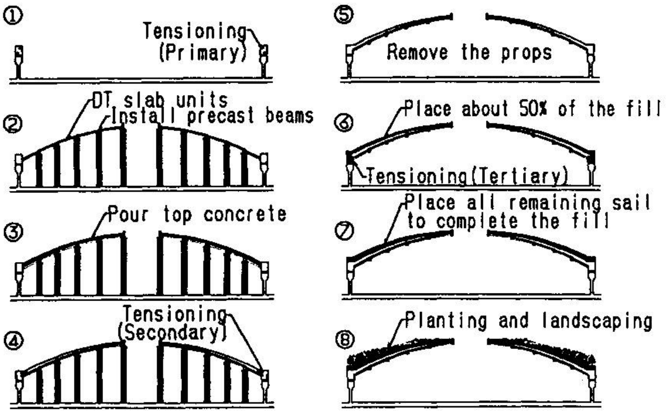

Fig. 6 Construction Sequence

where concrete is to be poured in place. Pour concrete and let it cure. Start pouring of concrete at the lower edge near the tension ring and proceed gradually to the crown of the shell.

(4) Perform secondary tensioning of the tension ring PC cables. Tension about 50% of all the PC cables at this stage.

(5) Jack down the props supporting the precast PC beams and remove the props.

(6) Waterproof the spherical shell, place soil to a depth equal to about 50% of the total fill depth, and then perform tertiary tensioning of the tension ring PC cables (i.e., tension all the cables not tensioned in the primary and secondary stages).

(7) Place all remaining soil to complete the fill.

(8) Perform planting and landscaping to finish the work.

3.3 Spherical Shell Design Principle

(1) Principle for Dealing with Stresses due to Long-Term Loads

The cast-in-place portion of the shell was designed so that it would be subject to compressive stresses of about 20 - 40kg/cm^2 and no tensile stresses would be produced in it. In particular, the tension ring was designed to be prestressed so that compressive stresses produced in it would be about 20kg/cm^2.

Further, based on the concept of limit-state design, the sectional area was designed so that the compressive stress developed in the shell concrete by a loading equal to 1.7 times the long-term loading would not exceed the short-term allowable compressive stress (Fc x 2/3) as prescribed in the applicable structural standard of the Architectural Institute of Japan and the tensile stress would not exceed the crack-causing tensile stress (Fc/10).

(2) Studies on Stresses due to Seismic Loads

The shell was designed in such a way that the long-term allowable compressive stresses developed by the design seismic load (horizontal seismic coefficient K=0.3) would not exceed the long-term allowable compressive stress (Fc x 1/3) with no tensile stresses developed by the seismic load. Further, in terms of limit-state design, the shell was designed so that the compressive stress developed in it by a load equal to 1.5 times the design seismic load would not exceed the short-term allowable compressive stress used for the shell design and so that such a load would not produce any tensile stress in the shell greater than the crack-causing tensile stresses.

In addition, analyses were conducted to verify the shell's hysteretic response to earthquake motions and it was found the aforesaid limit-state principle would be satisfied when the shell was subjected to severe earthquake motions.

3.4 Structural Analyses

(1) Stress and Deformation Analyses of the Spherical Shell

1) Analysis method

The static stress and deformation analyses of the shell as a whole were performed by conducting three-dimensional elastic analyses of quadrangular shell components by means of the finite element method.

2) Results of the analyses

The stress of the shell under long-term loading are as shown in Figs. 7 and 8. Fig. 7 shows the stresses in the shell due to the axial force; Fig. 8, the stresses due to the bending moment. In Figs. 7 and 8, the left half of each stress diagram show the stresses in the direction of the circumference (or the latitude) and the right half shows those in the direction of the radius (or the longitude). In Figs. 7 and 8, the top left diagrams indicate the stress state that would occur in an assumed case where the full vertical load was applied while the tension ring was not prestressed at all whereas the top right diagrams shows the stress state in an assumed case where the tension ring was fully prestressed with no vertical loading applied to the shell. At the bottom of each figure, these two types of diagrams are superimposed to indicate the actual state of stresses. (The diagrams were prepared by disregarding the effects of cracking of the concrete and assuming that the same degree of rigidity was maintained on the tension side as on the compression side.) It is known from the axial force diagram of the shell in Fig. 7 that if no prestressing force is introduced, particularly in the latitudinal direction, a large tensile force is developed near the tension ring. It also is known that a prestressing force introduced into the tension ring effectively counters such tensile force and in practice causes the compressive stresses to be dominant throughout the entire shell. Further, from the bending moment diagram in Fig. 8, it is known that the bending moment caused by vertical loading is offset by the bending moment generated by the prestressing and in consequence the shell is not actually subject to a large bending moment.

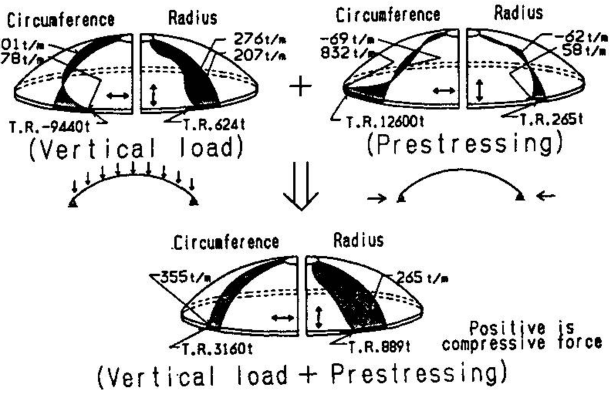

Fig. 7 Axial Forces in Shell

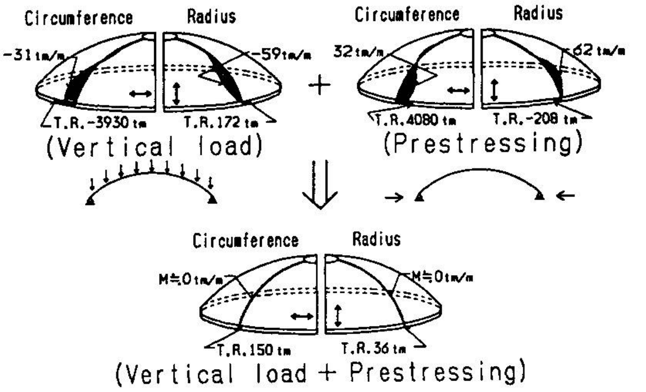

Fig. 8 Bending Moments in Shell

4. CONCLUSION

The construction of a gymnasium covered by the spherical shell roof described in this paper is now in progress with the completion slated for 1997. In the course of the spherical shell construction, further deformation and stress measurements at various locations are planned, which are expected to lead to still more interesting data. The authors wish to make a thorough study of the structural behaviors of this spherical shell by continuing on-site observation and data analyses on a comparative basis.

5. ACKNOWLEDGEMENTS

By taking this opportunity, the authors wish to express their cordial thanks to Professor Kazuo Suzuki of Osaka University for his cooperation and advice in designing the spherical shell.

Design of Steel Shell Frames of the Fukuoka Exhibition Hall

Projet de la couverture en acier de la salle d'exposition de Fukuoka

Entwurf der Schalenstahlrahmen der Fukuoka-Ausstellungshalle

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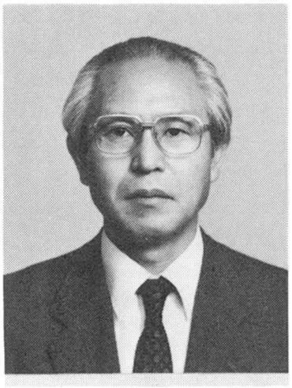

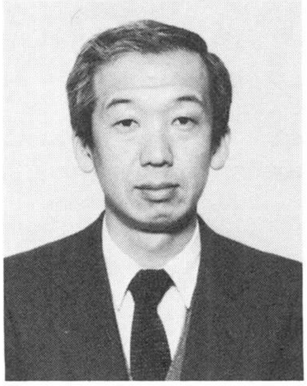

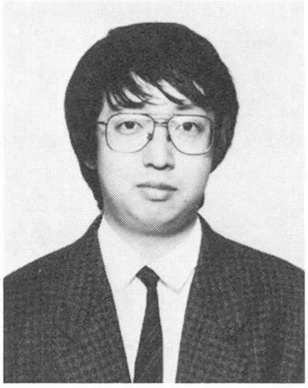

SUMMARY

The roof structure of the Fukuoka Exhibition Hall Building is composed of large scale steel vaulted shells. Prestressing is used to reduce vertical displacements. The design method for metal prestressing shell structures and the tests to check the safety of the bearing plates that fix the prestressing wire and, to confirm the relaxation are being presented in this paper.

RÉSUMÉ

La structure porteuse de la toiture de la salle d'exposition de Fukuoka se compose d'éléments de voiles cylindriques en acier afin de réduire les déplacements verticaux. La communication traite des données de base servant à la conception des structures porteuses de coques métalliques ainsi que des essais de charge ultime des plaques d'ancrage et de la relaxation des fils de précontrainte.

ZUSAMMENFASSUNG

Die Dachkonstruktion des Fukuoka-Ausstellungsbaus besteht aus grossformatigen Stahl-Tonnenschalen, die zur Verminderung der Durchbiegung vorgespannt werden. Der Beitrag behandelt die Entwurfsgrundlagen für vorgespannte Stahlschalentragwerke sowie die Versuche zur Sicherheit der Ankerplatten und der Relaxation der Spanndrähte.

1. INTRODUCTION

The architectural design of the roof was undertaken by imagining sea waves, because the structure is located in front of the sea in Fukuoka city (see Fig. 1). The Fukuoka Exhibition Hall Building consists of 4 stories (two basement floors) with a total architectural area of 40,581 m^2, a maximum height of 31 m and seating capacity for 15,000 people. This long span building will be mainly used as an exhibition hall; there is also a small gymnasium and council room. The dimensions of the roof structure are 144 x 120 m and the rise of the vault is 7.5 m. This roof is composed of three continuous steel latticed vaults and two steel shell wings arranged at both sides of the vaults as shown in Fig. 2 & 3. The roof structure is supported by four big columns, whose spans are 36 m and 100.8 m, with other smaller columns at the shell wings. The steel latticed vaults and wings are composed of welded built-up H-steel sections (BH- 850 x 200 mm) arranged in the longitudinal direction, and steel trusses (H = 850 mm) in the transverse direction as shown in Fig. 4. The steel latticed vault is reinforced by steel ribs arranged every 14.4 m in the transverse direction in order to avoid buckling, general instability and to reinforce against non-symmetrical loads such as seismic and wind loads. Additionally, these ribs are utilized as maintenance routes and for lighting and mechanical uses. Instability behavior and the stiffening effect of ribs, together with the dynamic behavior under vertical and horizontal earthquake vibrations, and prestressing effects on displacement and stress, have been analyzed in order to design this beautiful shell structure. Simple connections are used and wind tunnel tests were performed to determine the wind force coefficient.

2. STRUCTURAL ANALYSIS

2.1 Design Load

The design live load of the roof is 980 N/m^2 which allows for the suspension of various exhibits, and also the possibility of having concentrated loads of 20 kN at the frame nodes. Wind load is determined using the results of the wind tunnel test. The model scale is 1/300, and roughness of the surrounding condition of the site is considered. Finally, 0.9 to 5.5 kN/m^2 of wind load is adopted allowing for the effects of gust and a 150 years return period. As for the aseismic design, vertical and horizontal loads on the roof structure are considered. Vertical and horizontal seismic load coefficients are determined by numerical dynamic analysis in time domain, using three actual earthquake records. The first natural period in the vertical direction of the roof is 0.71 sec. and the first natural period in the horizontal direction, of the whole structure including the main frame that supports the roof structure, is 0.33 sec.

2.2 Stress Analysis

Seven cases of structural frame analysis have been undertaken to design the roof structure as shown in Table 1. There are two analytical models; one considers the whole model with some secondary members thinned out and the second, considers a quarter of the structure taking into account all members and the symmetry conditions. Combinations of the stresses used to design the section are shown in Table 2. Boundary (support) conditions are set considering the order of the construction. The support condition against self weight of the steel roof structure is initially considered as roller (free) in the long span direction in order to avoid large internal forces in the supporting columns due to horizontal movements of the roof, when the temporary supports used for erection are removed. The support conditions against other loads such as live loads, seismic loads, etc. are considered as spring supports. The results are shown in Table 3, where the vertical deflection at the center of the roof and horizontal displacement at the supported point of the roof are presented. The design of the sections of the members is performed considering a combination of axial force and two directional bending moments.

One example of the analysis results is shown in Fig. 5.

3. PRESTRESSING

It is too difficult to pre-camber this roof structure, therefore, prestressing is used in order to get the least possible deflection. Details of the prestressing are shown in Fig. 6. Unbonded tendons to avoid rust are installed in the steel pipe at the upper and lower chords of the edge beam in the bottom of the steel vault. The tensile forces in these prestressing cables are restricted by the following equation for any load case.

$$s_0 \leq 0.6\ Pu \text{ and } 0.7\ Py$$
$$s_{max} \leq 0.7\ Pu \text{ and } 0.85\ Py$$

where

s_0 : Stress by prestressing
s_{max}: Stress by $s_0 + s_{\wedge}$
$s_{\wedge}$: Stress due to seismic load, thermal stress, live load, etc.
Pu : Ultimate tensile strength of tendon
Py : Yield tensile strength of tendon

Stresses due to prestressing are shown in Fig. 7, and the deflection of the center of the roof is +7.2 cm. The maximum tensile forces in the prestressing cables are obtained considering the worst load case condition and are equal to 0.67 Pu and 0.79 Py. The anchoring plates for these prestressing cables are analyzed using the Finite Element Method. These anchoring plates are designed such that the maximum stress does not go beyond 0.75 Fy; where Fy is the yielding strength of the anchoring plate. Thus the plate thickness for a prestressing force of 3000 kN is 90 mm, and for the case of 1500 kN is 70 mm. Relaxation of the total system and stress of the anchoring plate have been confirmed by experimental tests of the actual size test piece shown in Fig. 8. The maximum relaxation during 100 days of the total system composed unbonded tendon, anchoring plate and steel pipe was 2.6 %, as shown in Fig. 9. Distributions of the strain at the surface of anchoring plate are shown in Fig. 10.

4. CONCLUSIONS

The roof structure of the Fukuoka Exhibition Hall Building is composed of large scale steel vaulted shells. Prestressing is used to reduce vertical deflection. Studies investigating the effect of the rib reinforcement to increase the stability behavior; the effect of prestressing to reduce vertical deflections and, the experimental tests of the prestressing system were performed in order to design the large-span thin steel vault structure reported in this paper. And the design method for metal prestressing shell structures, the test methods to check the safety of the anchoring plate that holds the prestressing wire and to confirm the relaxation are also presented in this paper.

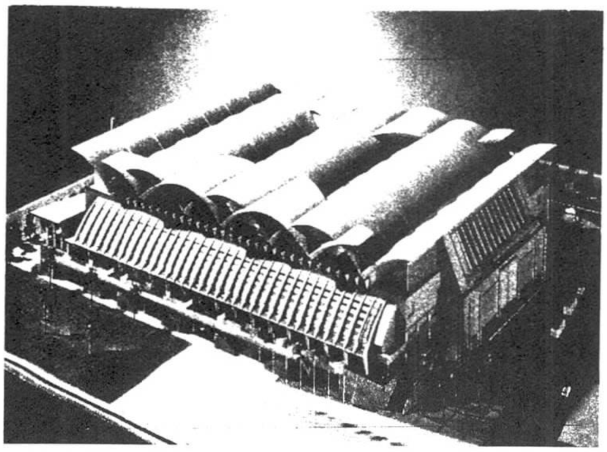

Fig. 1 Fukuoka Exhibition Hall

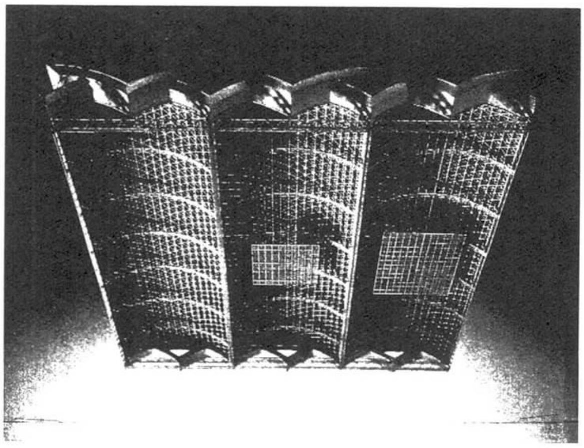

Fig. 2 Photo of the roof framing

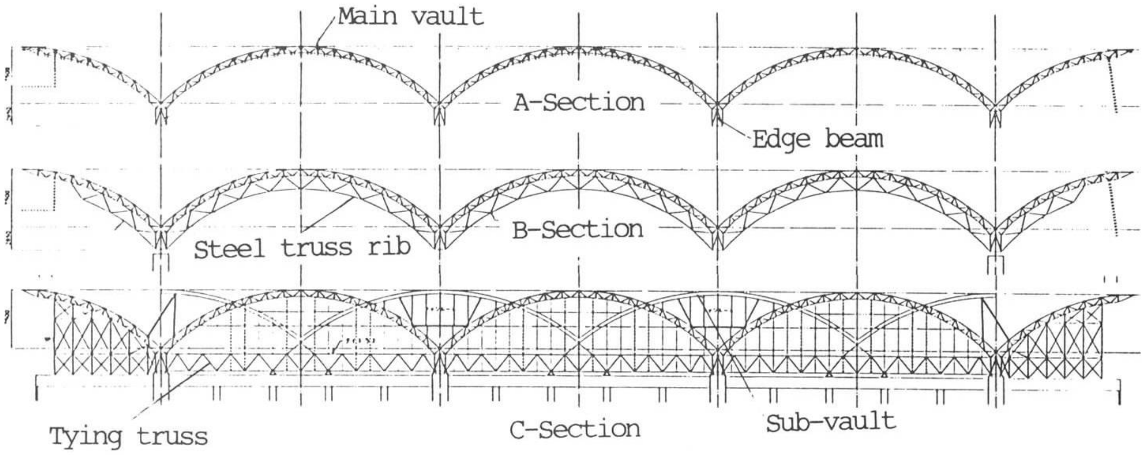

Fig. 3 Elevation of the roof framing

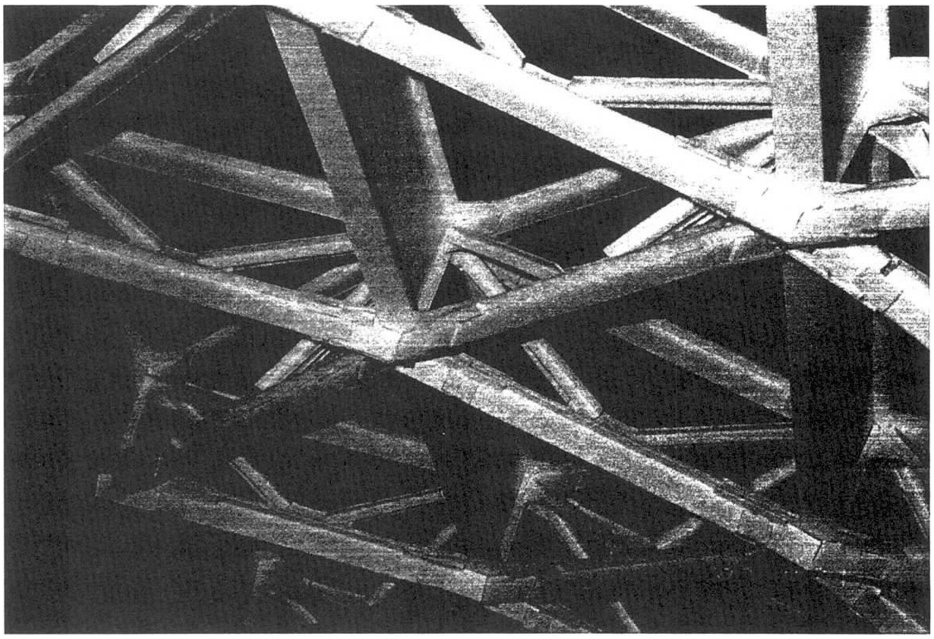

Fig. 4 Photo of the main shell vault

Table 1 External load cases

No.	Load case
1	Vertical load (Steel weight only)
2	Vertical load (Live load etc.)
3	Thermal stress (± 15℃)
4	Thermal stress (± 30℃)
5	Earthquake
6	Wind forces
7	Prestressing

Table 2 Stress combinations for designing section

No.	Combination	Allowable stress
1	1+2+3	long term
2	1+2+3+7	long term
3	1+2+4	short term
4	1+2+3+5	short term
5	1+2+3+5+7	short term

Table 3 Vertical deflection at the center of the roof

No.	Vertical deflection (cm)*1	Horizontal movement (cm)*2
1	− 8.6	
2	− 17.0	2.25
3	± 0.63	± 0.66
4	± 1.23	± 1.31
5	0	1.73
6	+ 8.74	
7	+ 7.2	

*1 − : downward, +:upward
*2 longitudinal direction (supported point)

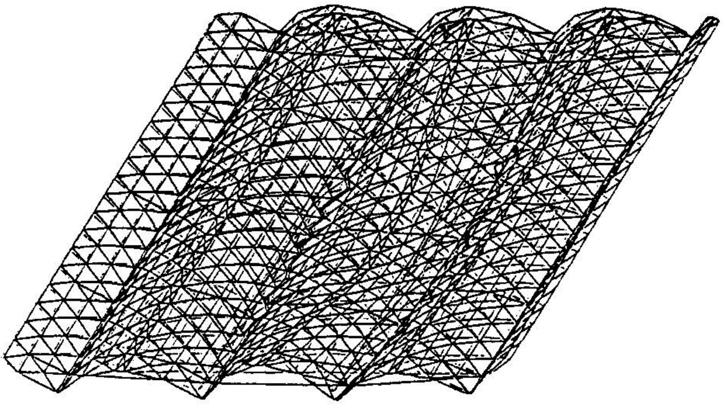

Fig. 5 Results of the analysis

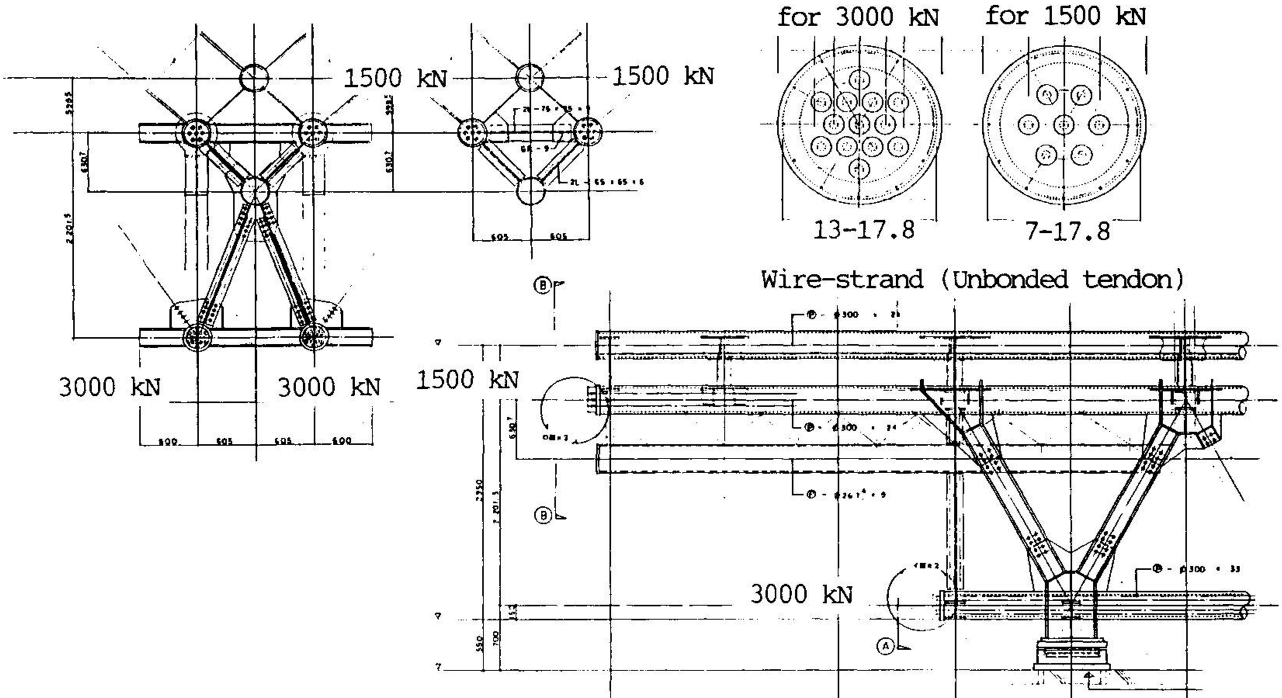

Fig. 6 Prestressing details

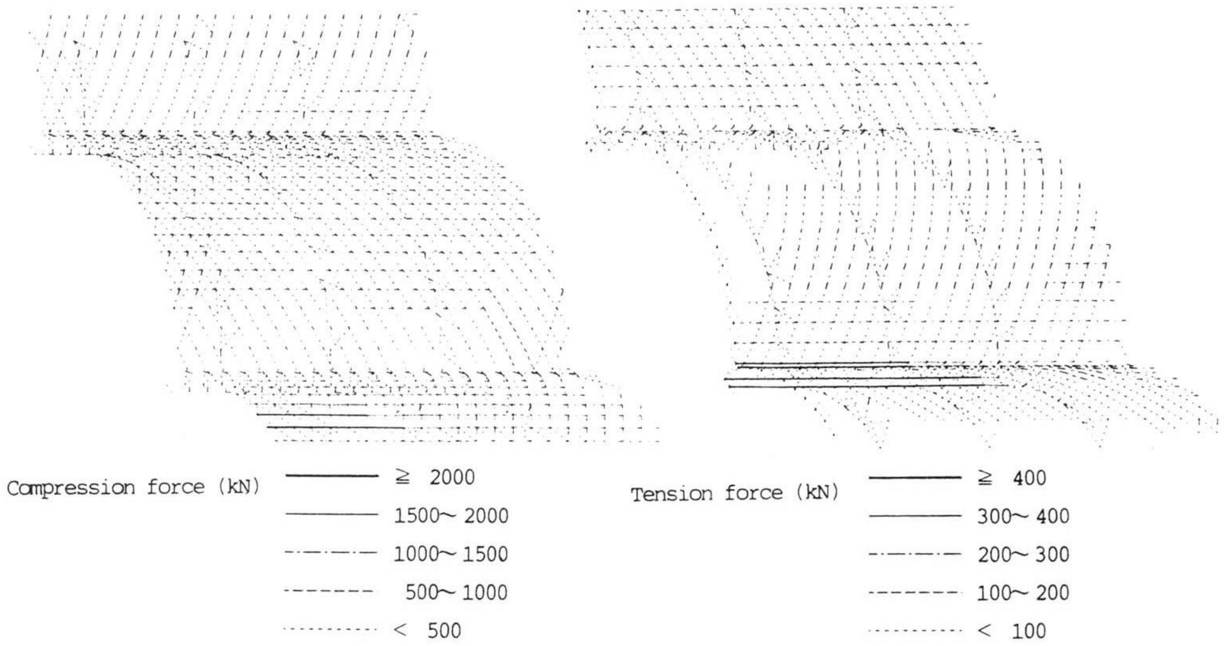

Fig. 7 Frame stresses against prestressing

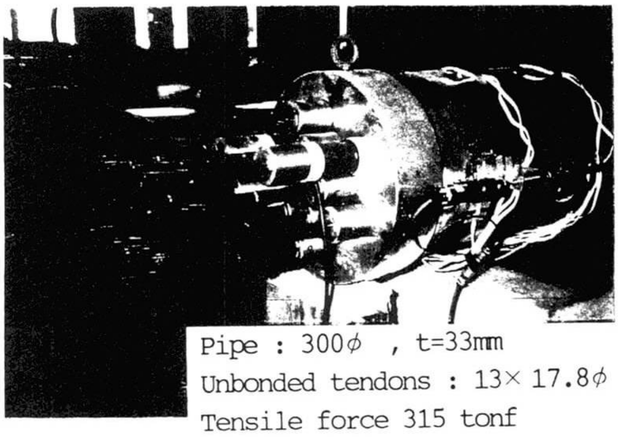

Fig. 8 Test sample

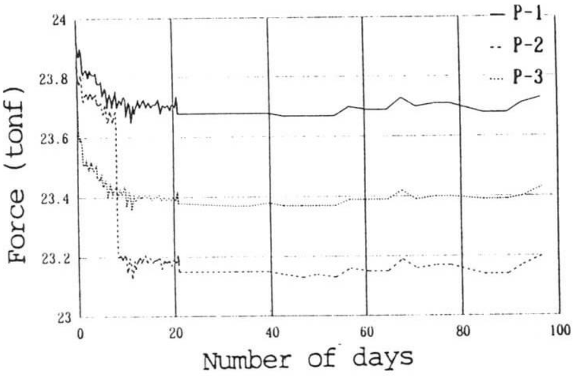

Fig. 9 PC Post-tension time-story

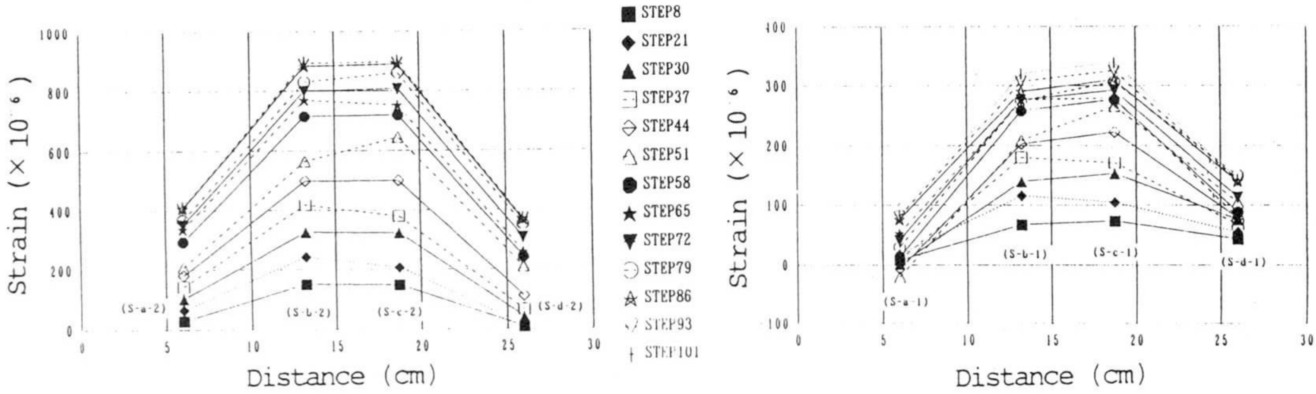

Fig. 10 Anchor plate strain when Post-tensioning

Roof Structure for the New Singapore Convention Centre

Toiture du nouveau centre de congrès à Singapour

Das Dachtragwerk des neuen Kongresszentrums in Singapur

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SUMMARY

The roof of the convention centre has overall dimensions of 173 m x 144 m. This paper focuses upon the design of this structure, which is believed to be unique in terms of the configuration of the space frame modules.

RÉSUMÉ

La toiture du centre de congrès a des dimensions extérieures de 173 m x 144 m. Ce rapport traite en particulier le concept de cette structure, qui est unique en ce qui concerne la configuration des modules d'espace du cadre.

ZUSAMMENFASSUNG

Das Dach des Kongresszentrums hat Aussenmasse von 173 m x 144 m. Dieser Bericht konzentriert sich auf den Entwurf dieses Tragwerkes, das hinsichtlich der Gestaltung der Module des Raumfachwerkes als einmalig gilt.

1. INTRODUCTION

The new Singapore International Convention and Exhibition Centre is a major feature of the Suntec City Development, the building is due for completion in 1994.

The total development comprises (refer to Figure 1):

- Convention and Exhibition Centre
- Four 45-storey high rise office towers
- One 18-storey rise office tower
- Extensive low rise retail podium

The Convention and Exhibition Centre will provide world class facilities aimed at maintaining Singapore as one of the major convention and exhibition centres in the world. The Convention Hall alone will be capable of accommodating up to 12,000 people.

The roof of the convention centre has overall dimensions of 172.8m x 144m. It is a special feature of the development and this paper focuses upon the design of this structure, which is believed to be unique in terms of the configuration of the space frame modules.

2. LAYOUT AND GEOMETRY

The roof structure comprises an external, fully exposed, space frame (exoskeleton) and a series of secondary roof structures suspended from the exoskeleton that support the roof cladding systems.

The roof exoskeleton is essentially a single layer 7.2m deep space frame with a square on square diagonal topology, with a node spacing of 20.36m. The basic frame extends to form two intermediate spine trusses which optimise the aspect ratio of the frame throughout the central 172.8m by 86.4m clear span. The frame also partially extends around the perimeter of the roof to facilitate the suspension of the secondary roof panels along the edge of the building.

The roof frame has a clear span of 172.8m by 86.4m; it has a total of 28 supports, and comprises tubular sections ranging from 400 to 900mm diameter. It is a fully welded structure using purpose designed node assemblies.

The overall configuration of the roof exoskeleton is illustrated in the views shown on Figure 2, 3 and 4.

The secondary roof is composed of three different forms of structure designed in response to the requirements of cladding systems (refer to Figure 5). These are as follows:

a) Type 1 -- Opaque roof structure, supporting a full acoustic roof and ceiling cladding system, with a depth of 1250mm sufficient to accommodate the air-conditioning ducting for the Convention Hall.

b) Type 2 -- Louvre roof structure supporting louvred roof panels above the service driveway and M&E Rooms, and fully exposed to the atmosphere. Structural depth is 1000mm.

c) Type 3 -- Skylight roof structure supporting glazed roof panels above the atrium area. Structural depth is 1000mm.

All secondary roof structures are essentially pyramidal 'pseudo' space frames with a base dimension of 20.36m x 20.36m and an apex height of 7.2m. All panels, except on the perimeter, were designed to be fully assembled as basic frames. Each frame was lifted from the roof apices by jacks fixed to the exoskeleton nodes.

Both the louvre and skylight structures are fabricated from circular hollow sections which were chosen for both appearance and ease of maintenance. The opaque structure is constructed from standard rolled sections.

3. DESIGN OF EXOSKELETON

3.(a) SPACE FRAME

The space frame was analysed using the Structural Analysis Program SAP90, and checked using the Structural Analysis Computer System SACS.

The frame was analysed assuming fixed concentrically loaded nodes except for the spine truss top chord nodes which were required to have a 200mm vertical eccentricity. Vertical loading was applied at the top and bottom nodes of the space frame to represent the actual loading pattern from the secondary roof structures. Support conditions simulated the actual roof articulation, which is fixed against translation about Grid lines D and 10.

Lateral wind loading on the roof is applied eccentrically to the nodes via the secondary roof structure hangers and the extended support stubs.

Tubular members were designed using purpose-written software for compliance with the requirements of the current edition of API (RP2A) 'Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms'. This approach, although perhaps more stringent than BS 5950, was considered appropriate in view of the unconventional scale of the roof frame. The effective length of compression elements was taken as the full distance between node centres in view of the node detail adopted.

Tubular member sizes range from 400 to 900mm diameter with wall thickness varying from 10mm (taken as a practical minimum for general construction robustness and weldability) to 40mm. The sections were fabricated from a mixture of Grade 43A and 50B steels to achieve minimum overall weight.

The frame was also checked to ensure that there would be no progressive or catastrophic collapse if one tubular was removed or one weld totally failed.

3.(b) NODE ASSEMBLIES

The connection of frame tubes of this size required the development of a special node assembly, comprising a vertical thick walled cylinder with plate stub tube connectors. The final arrangement, as shown in Figure 6 was chosen for its simplicity of fabrication and architectural merit.

The roof frame consists of a total of 109 of these standard nodes and 38 non standard nodes confined mainly to the spine trusses and the perimeter roof hangers. At the spine truss locations, additional in-plane inclined and vertical members lead to the use of a more conventional gusset plate connection, with a non-structural node can for architectural purposes.

The diameter of the structural node cylinders was chosen to be as compact as possible within the constraints of having sufficient space for up to 10 tubular connections and the architectural requirements. The following diameters were chosen:

a)	600mmΦ x 65 thick	-	for the standard case
b)	770mmΦ x 65 thick	-	for the bottom chord at the spine truss
c)	300mmΦ x 25 thick	-	for the perimeter top chord at hanger locations.

The design of the node was initially developed by preliminary analysis as a ring frame subject to point loads and then refined by finite element analysis using the program SACS. Furthermore, because of the unconventional nature of the structure full-scale load testing was be undertaken on typical nodes to verify the design in respect of the strength and stiffness of the overall node assembly.

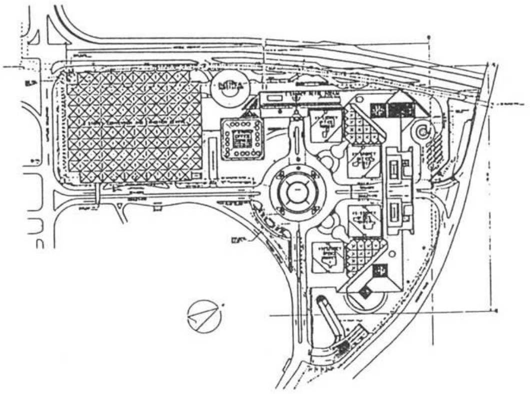


FIG. 1 SUNTEC CITY PROJECT

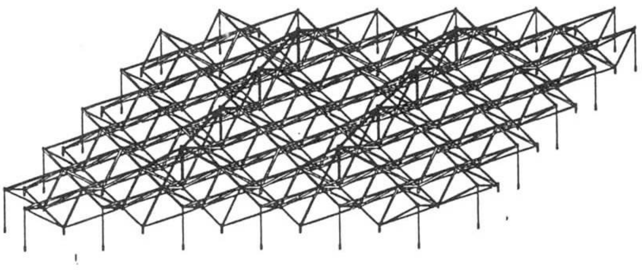


FIG. 2 ROOF FRAME – ISOMETRIC VIEW

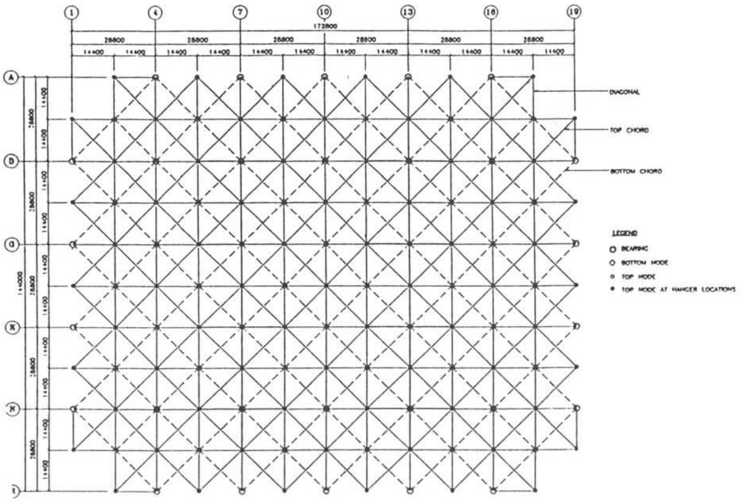
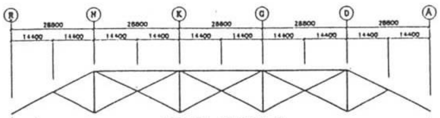
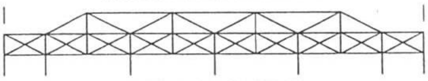


FIG. 3 ROOF FRAME – PLAN VIEW

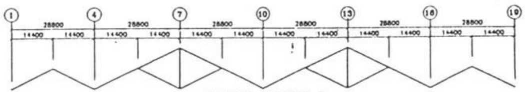


SECTION AT GRID 7

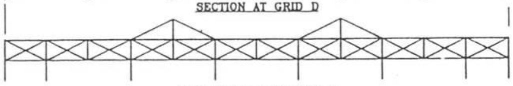
SECTION BETWEEN GRIDS 10 & 13



ELEVATION ALONG GRID 19



SECTION AT GRID D



ELEVATION ALONG GRID R

FIG. 4 – TYPICAL SECTIONS AND ELEVATIONS

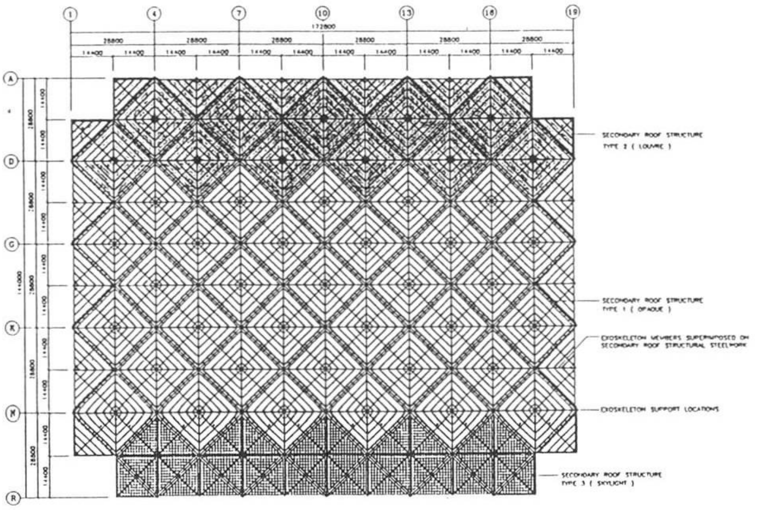
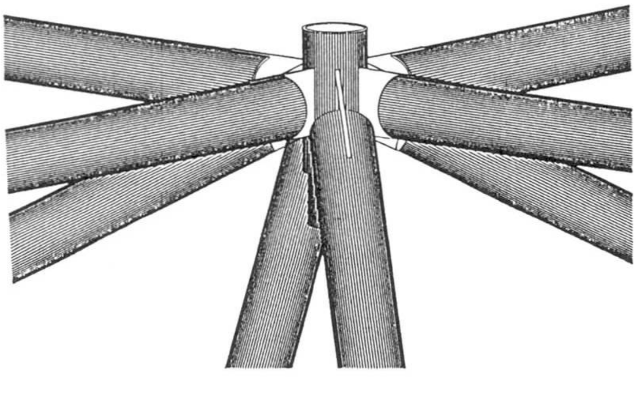


FIG. 5 – SECONDARY ROOF LAYOUT PLAN

FIG. 6 – TYPICAL TOP NODE ASSEMBLY



3.(c) ROOF SUPPORTS

The roof exoskeleton is supported at a total of 28 supports, 18 around the perimeter of the building and 10 internally. At each of these locations, the frame is supported on a single disc or confined elastomeric bearing with load capacities up to a maximum of 1000 tonne (spine truss supports).

The bearings are required to accommodate rotations and translations due to vertical loading and thermal changes of the exoskeleton estimated to be in the order of 60°C (-5°C+55°C about a mean of 25°C). The overall roof articulation arrangement is shown on Figure 7.

As the secondary roof structure is suspended below the exoskeleton, the supports for the main frame were required to accommodate eccentricities, from the nodes, approaching 2000mm. The moments generated by this eccentricity, because of horizontal wind loading and bearing friction effects (taken as 5% of DL reaction), must also be accommodated by the frame.

4. DESIGN OF SECONDARY ROOF STRUCTURE

4.(a) FRAMEWORK

Each of the three types of secondary roof structures are designed to support vertical loading appropriate to the type of cladding system they support. They also support M&E services and the general live loading required in accordance with BS 6399.

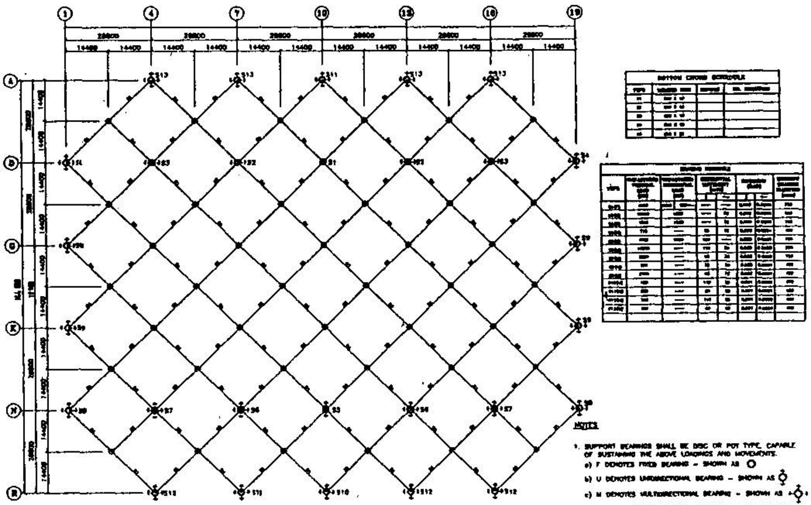
Structural analysis of the roof frames was carried out on three dimension models using SAP90, and individual members are designed to BS 5950.

Each secondary roof structure pyramid was designed to be lifted from its apex as a basic frame and then hung from the exoskeleton nodes at the apex and four lower corners. All loads applied to the secondary roof are subsequently distributed to all five supports. This beneficial use of construction staging results in a saving in overall member sizes. The special secondary roof panels, around the perimeter of the building, are designed to lift from their permanent supports under certain load combinations.

The basic configuration chosen for the louvre and opaque roof structures is a welded two-way truss system comprising primary ridge and perimeter elements connected by square bolted connections.

The skylight structure, on the other hand, is designed as a welded two-way vierendeel truss system with triangulated ridge and perimeter elements; all trusses are orientated vertically for aesthetic reasons.

Tertiary steelwork is provided between the roof pyramids to accommodate gutters, services and ceilings, etc.



1. SUPPORT BEARINGS SHALL BE DISC OR POT TYPE, CAPABLE OF SUSTAINING THE ABOVE LOADINGS AND MOVEMENTS.
   - a) F DENOTES FIXED BEARING – SHOWN AS ○
   - b) U DENOTES UNIDIRECTIONAL BEARING – SHOWN AS
   - c) M DENOTES MULTIDIRECTIONAL BEARING – SHOWN AS
2. BEARING MOVEMENT REQUIREMENTS ARE FOR A TEMPERATURE RANGE OF 60°(MEAN 30°C +60°C –10°C)
3. ± 65 DENOTES MOVEMENT OF 65 IN EACH DIRECTION. TOTAL MOVEMENT 130 mm.

FIG. 7 – ROOF ARTICULATION

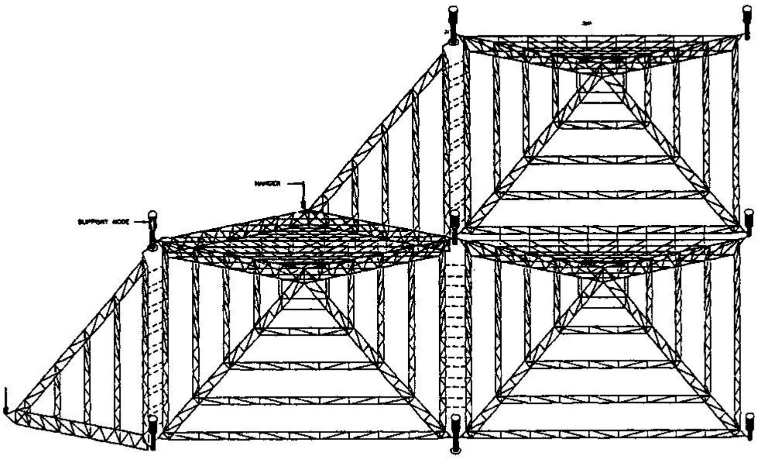
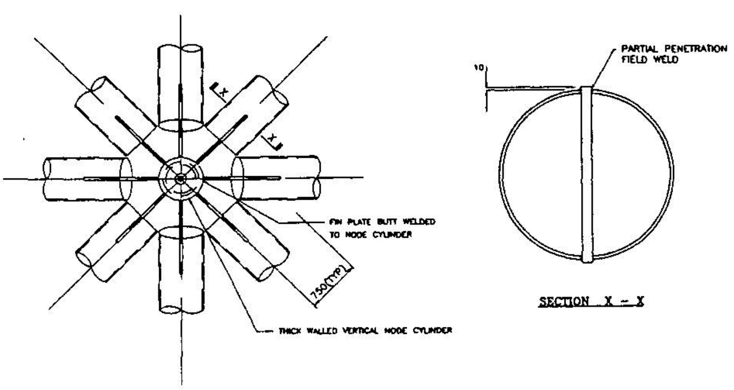


FIG. 8 – PART ISOMETRIC VIEW AT CORNER OF OPAQUE SECONDARY ROOF



SECTION X – X

FIG. 9 – TYPICAL CONNECTION NODE TO TUBE

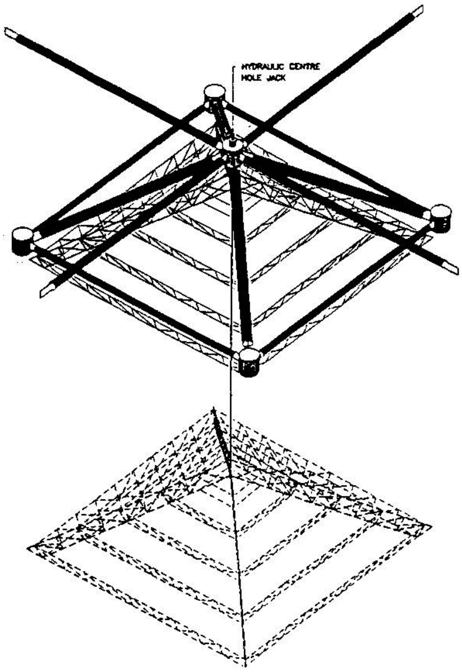


FIG. 10 – ERECTION OF SECONDARY ROOF STRUCTURE

4.(b) SUPPORTS

Secondary roof structures are supported from hangers that project below the exoskeleton nodes. The hangers and associated branch elements are arranged to suit the geometric requirements arising from the secondary roof structural types, and in response to their location within the main frame.

In order to provide sufficient articulation for each pyramid under vertical loading and differential thermal effects a simple support detail was devised for the lower node supports, consisting of:

a) Dual 20mm diameter prestressing bars with spherical washers top and bottom to accommodate lateral translation.

b) A shear plate assembly to control direction of movement and provide restraint to lateral wind loading. (A minimum of two shear assemblies will be activated regardless of the direction of the wind.)

The central apex connection is a fixed bolted connection utilising eight M24 H.S bolts in all cases.

The secondary roof around the perimeter of the building is composed of half or quarter inverted pyramids as shown in Figure 8. The lower corners of these panels are suspended from the top chord of the main roof frame by 100 mm diameter hangers. The hangers are provided with stainless steel prestressing bars concealed within the top roof frame node in order to facilitate adjustments in the level of the roof structures. At the lower end of the hangers, the secondary roof is connected by means of a concentric pin connection.

5. NODE TESTING

Because of the unusual nature of the nodes it was decided, early in the project, to carry out full-scale prototype testing of the nodes to test the assumptions and results of the finite element analyses. This testing was done by Det norske Veritas Industry at their facilities in the Singapore Science Park.

Testing was completed on three full size nodes, to ensure that the roof possessed adequate stiffness to resist the expected load patterns without unusual or excessive deformation of the node assemblies or the completed roof.

An 85 tonne rig was specially designed for the testing of the nodes which were fabricated with eight stub tubes in three dimensions to simulate the real conditions as closely as possible. Testing took place over a period of five days and included maximum loads equal to twice the design load. Data was logged from 300 points on each node.

The stiffness test (deformation measurements) required a force equal to the dead load plus 1.5 times the live load. The strength test was taken to twice the total load. Excellent correspondence with the analysis was achieved.

6. CONSTRUCTION

6.(a) ROOF EXOSKELETON

All of the steel for the roof was manufactured in Korea and Japan with fabrication taking place at Hyundai Heavy Industries in Ulsan, South Korea.

Tubular members were shop fabricated to their full length of up to 26m and weighed up to 22 tonnes. Nodes assemblies were also shop fabricated with fin plates and secondary roof hangers attached before being painted and shipped to Singapore. The remainder of the frame was field welded as shown in Figure 9.

The connection of the tubular members to node fin plates was developed with a view to simplicity of welding and maximum accommodation of angular tolerances.

In order to control the node positions such that they remained within the specified tolerance of ±5mm it was necessary to rigorously control when tubes could be connected to the node finplates. Tight tolerances were achieved by fixing the nodes at grid N in position and progressively working away from that grid. Tubes were tack-welded to node fin plates when the average temperature of the tubes was in the temperature window of 28 to 32°C. Final welding could then be carried out at any time of the day or night.

The 6th Storey slab was designed for a live load of 17.5 kN/m^2 allowing the roof exoskeleton designed to assume assembly on the structural slab. The roof was lifted by jacking of the complete frame from the 10 internal supports.

The jacking operation requires the installation of temporary towers between roof frame elements and a total jacking load of approximately 2400 tonnes.

Because of the number of members connected to each of the 10 nodes used for lifting, the temporary towers were designed to have 1 level of horizontal braces to the columns temporarily removed during the lifting operation. This need to frequently remove and reinstall braces meant that the 8m lift was carried out over an extended period after the critical lift off and survey checks. The jacks were capable of operating at a stroke speed of 1.5m/hour.

6.(b) SECONDARY ROOF

The secondary roof structures were assembled upon the 6th floor slab following lifting of the roof exoskeleton. Each internal pyramid structure was, after assembly, lifted from its apex by jacks placed upon the roof frame top chord nodes as shown in Figure 10. The maximum weight of secondary roof frame lifted is 25 tonne.

Perimeter half and quarter structures were erected by mobile crane. Following erection of all secondary roof structures the roof cladding systems and M&E services were installed to complete the roof of the Convention Centre.

6.(c) PROTECTIVE COATING

The protective coating system of the exoskeleton and the louvre secondary roof structure consists of

1)	Blast clean to Swedish Standard Sa 2.5 grade	
2)	Inorganic zinc	DFT 65-80 microns
3)	High build epoxy MIO	DFT 125-150 microns
4)	Aliphatic polyurethane	DFT 50-75 microns

The internal exposed skylight structure has a similar system except that the undercoat DFT is reduced to 75-100 microns.

The opaque secondary roof structure has surface preparation comprising of

1)	Blast clean to Swedish Standard Sa 2.5 grade	
2)	Epoxy Zinc Phosphate Primer	DFT 75-90 microns
3)	Epoxy Enamel	DFT 50-70 microns

A Few Large Span Structures in India

Quelques structures à grande portée en Inde

Einige weitgespannte Tragwerksbauten in Indien

Mahendra RAJ
Managing Director
Mahendra Raj Consultants
New Delhi, India

Mahendra Raj, born 1924, received his civil engineering from Punjab Univ., Lahore and masters degree from Univ. of Minnesota. He has been involved with various type of large span structures in India and USA.

SUMMARY

India has seen significant construction activity in the last 25 years. A number of large span structures have been built during this period. The author has been involved in several such projects in the capacity of structural consultant. Two such projects are described here, one of them is an exhibition hall while the other one is an indoor stadium with a seating capacity for 5'000 persons.

RÉSUMÉ

Pendant les dernières 25 années, l'Inde à fait preuve d'une activité considérable dans le génie civil. Plusieurs structures à grande portée ont été réalisées pendant cette période. L'auteur a participé à la réalisation de plusieurs de ces projets en tant qu'expert. Deux de ces projets sont décrits dans ce rapport, l'un étant un hall d'exposition et l'autre un stade couvert de 5'000 places assises.

ZUSAMMENFASSUNG

In den letzten 25 Jahres verzeichnet Indien eine rege Bautätigkeit. Eine Anzahl weitgespannter Tragwerksbauten wurden während diesr Zeit errichtet. Der Autor wurde als Konstruktionsberater bei mehreren Projekten beigezogen. Zwei solche Projekte werden in diesem Bericht beschrieben, das eine ist eine Ausstellungshalle und das andere ein gedecktes Stadium mit einer Kapazität von 5000 Sitzplätzen.

1. HALL OF NATIONS AT NEW DELHI

An international trade fair to be held at New Delhi in 1972, required large exhibition halls. The main hall was required to have a free and unobstructed space of 6700 sq. m. with an approximate height of 30 m. and was named as the "Hall of Nations".

1.1 Shape and Form

For covering such a large area several options e.g. shells, folded plates, hypar shells were considered and rejected on the ground of economics. Eventually the solution narrowed down to a double layered space frame. The "Hall of Nations" had a base dimension of 73mX73m which reduced to almost half at the roof. While investigating the configurations of the space frame it was identified that the most appropriate system to create space frame was the one which used pyramid as the basic element. Geometry for the "Hall" is shown in figure 1.1. Steel and concrete were given due consideration and it was found that concrete was the most economical material, with structural steel being 30% more expensive. Thus concrete was the final choice. For cladding, gunited triangular plates were provided while roof of the hall was covered with precast light weight concrete planks.

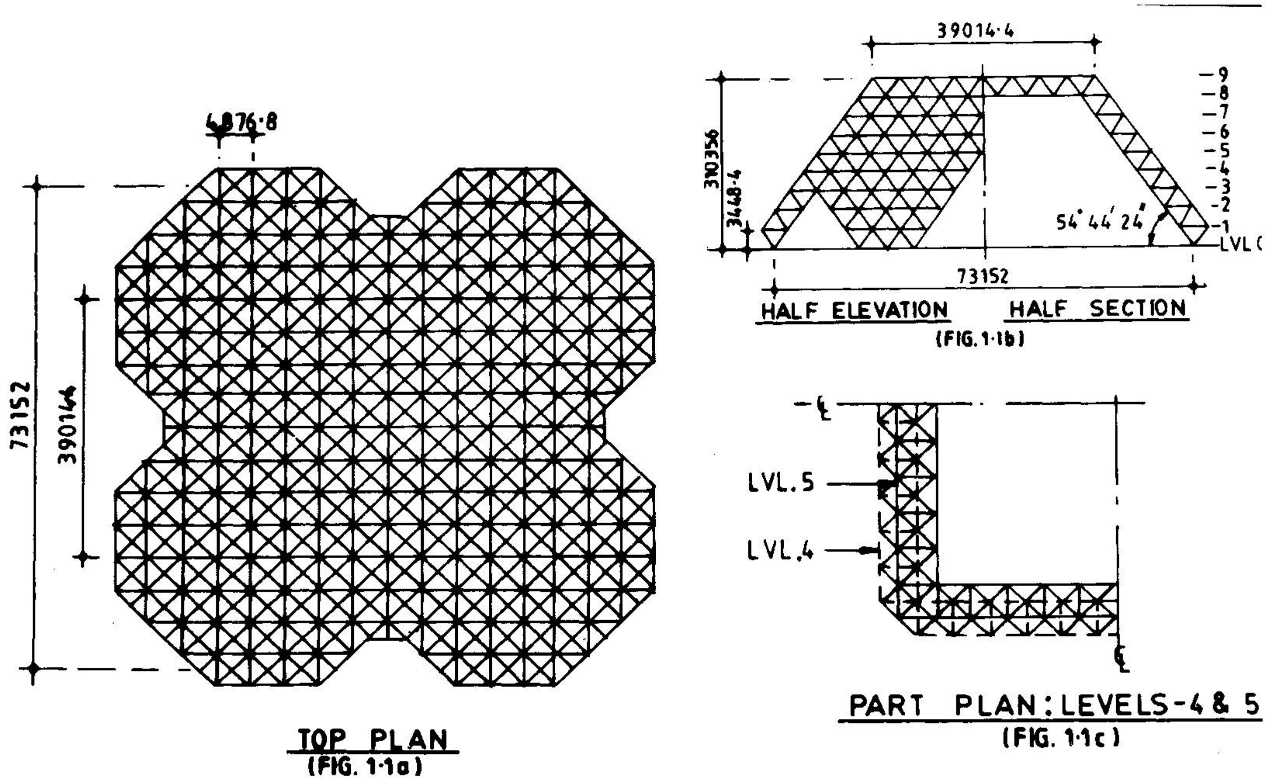
1.2 Analysis Design and Construction

A study of the configuration revealed that the structure would be stable only after construction had reached at level 5, where it received the first alround continuity (fig. 1.1) and before that it would rest on scaffolding erected from ground. However this resulted in five different configurations with introduction of each additional ring beyond level 5. Therefore analysis was carried out for all five different configurations to fix final member sizes. The analysis was done assuming pinned joints, as the members were slender and it was believed that significant moments would not develop. An independent analysis done later confirmed this assumption as the moments in members were very small. The space frame was analysed for earthquake and wind loads using a three dimensional pin jointed model. Most important point for selecting a member shape was strength and ease of fabrication. Several alternatives for member section were considered and finally a rhombic section with chamfered edges having an area of 585 sq. cm. was adopted. The analysis also revealed that horizontal deflections of nodes near central line of symmetry would be large. To contain these deflections, horizontal diaphragms were introduced between inner and outer faces of the space frame at levels 3, 6 and 8.

The obvious choice of construction was precast construction technique. It was visualised that the members and joints would be precast separately and put together with the help of bolts and field welding of shim plates. Details of such a precast joint are shown in figure 1.2. Unfortunately no contractor came forward to construct this structure using precast technique, hence in-situ concrete was adopted. The development of in-situ joint was difficult owing to congestion of bars at the joint element. On an average, nine members met at one joint and even with four bars per member, there would be thirtysix bars meeting at the joint, some carrying tension while others were in compression. In the precast joint some assistance was available from steel plates embedded in concrete, but in the in-situ joint that would not be possible. Finally a system was evolved wherein only twenty bars passed through the joint element. Bars from lower four members were lapped with four upper member bars, thus catering to eight members. Bars for ninth member were taken through the joint and anchored in one of the eight members. Figures 1.3 and 1.4 show arrangement of reinforcement at one such joint and a typical joint profile respectively. The sequence of construction was to cast straight length of member from joint to joint, place in position partially pre-assembled form work of the joint alongwith placed in position short length curved bars. These bars were welded with straight bars of members, formwork of the member upto next joint erected and the member and joint concreted. This sequence was followed upto roof level. The construction of space frame was taken on all the four sides simultaneously.

1.3 Foundations

The structure was supported on cast in-situ driven piles tied together with grade beams. Horizontal forces in grade beams at any stage were dependant on the extent of structure erected by that time. To contain horizontal forces some grade beams were post-tensioned in stages as the structure went up. The structure is in service since 1972. Figure 1.5 shows completed structure.



TOP PLAN
(FIG. 1·1a)

(FIG. 1·1b)

PART PLAN: LEVELS-4 & 5
(FIG. 1·1c)

Fig. 1.1 Plan and Section of Hall of Nations

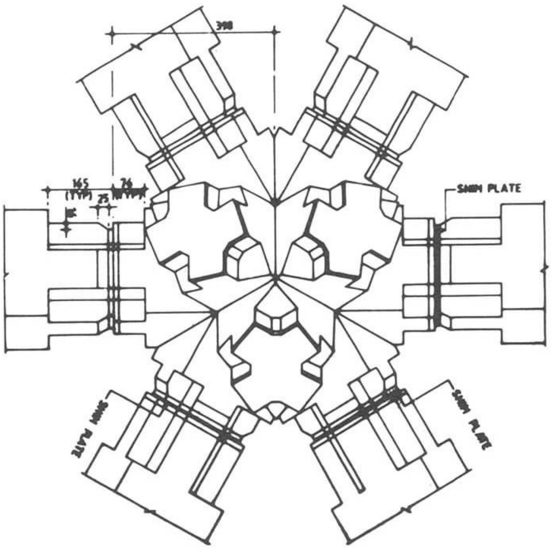


Fig. 1.2 Precast Joint Detail

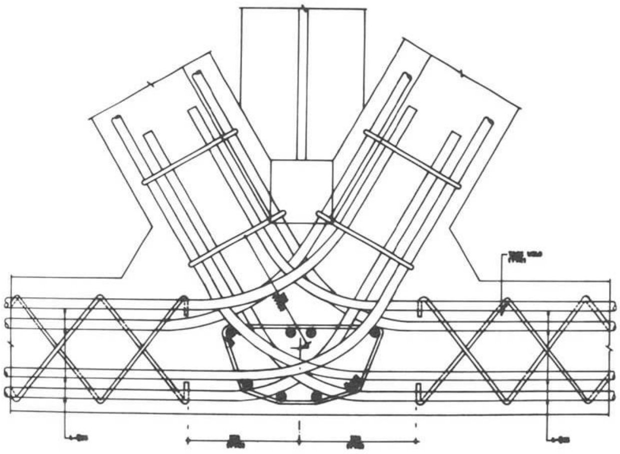


Fig 1.3 In-Situ Joint, Typical Reinforcement Detail

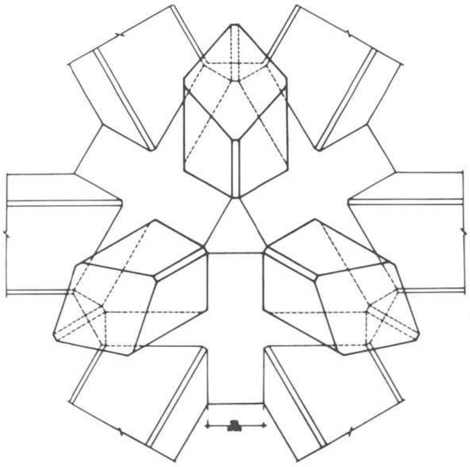


Fig. 1.4 In-Situ Joint Profile

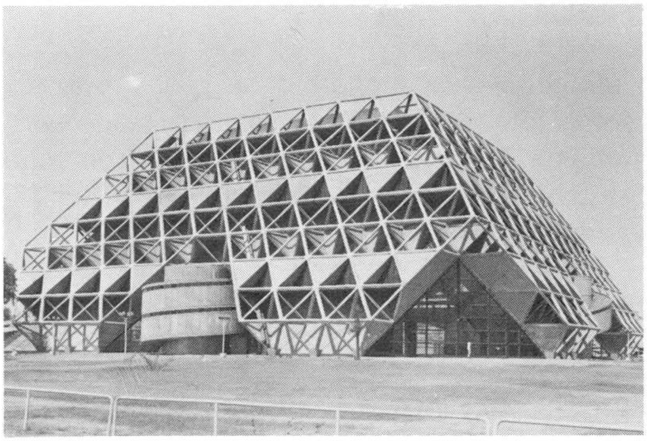


Fig 1.5 Hall of Nations Completed Structure

2. INDOOR STADIUM AT SRINAGAR

This stadium was constructed during 1980-84 at Srinagar in Jammu and Kashmir for holding indoor games tournaments. Planned in the shape of a cross with maximum dimension of 62.0 m, the stadium is provided with a seating capacity of 5000.

2.1 Plan Geometry

Site of this stadium is seismically active and dictated a simpler plan geometry. Box like rectangular or square pattern would be simpler but architecturally they were not acceptable. Finally a cross shaped plan was chosen as it was better than any other box like shapes and possessed desired seismic qualities too.

2.2 Structural System

For covering this large area our effort was to evolve a system which could create the entire enclosure without any intermediate support. The structural system for the stadium consisted of inclined plates connected together with seating frames. The plates on outer periphery were inclined outward from the foundations at level 0 to the elbow at level 3, and then inward till level 5. Plates on each internal corners were inclined inward. These plates were created with intersecting precast reinforced and post-tensioned concrete members on a triangular grid. The triangles thus formed were infilled with gunited and glazed cladding plates. Figure 2.1 and 2.2 show plan and elevation of stadium,respectively. Seating frames carrying precast seating elements were provided around the central arena and connected with the inclined plates at level 3. Thus seating frames were an integral part of the system. For the roof of stadium, steel lattice frame was chosen which echoed the configuration of main structure. Analysis of the structure was carried out for different stages. The analysis revealed that the plates transferred all the loads axially but had significant out of plane bending moments too. Further plates on outer periphery caused large tensile forces in the inwardly inclined plates at each fold. To contain these tensile forces post-tensioning was adopted. Figure 2.3 shows typical cross section of a member. All members except the edge members were precast and all joints were in-situ. Details of one such joint are shown in figure 2.4.

2.3 Foundations

This structure is supported on bored cast in-situ piles tied together with plinth beams. Large horizontal forces are transferred to foundations by folded plates. These forces are carried by a network of post-tensioned plinth beams. Post-tensioning was carried out in stages to balance the thrusts. The structure is in service since 1984. Figure 2.5 shows completed structure.

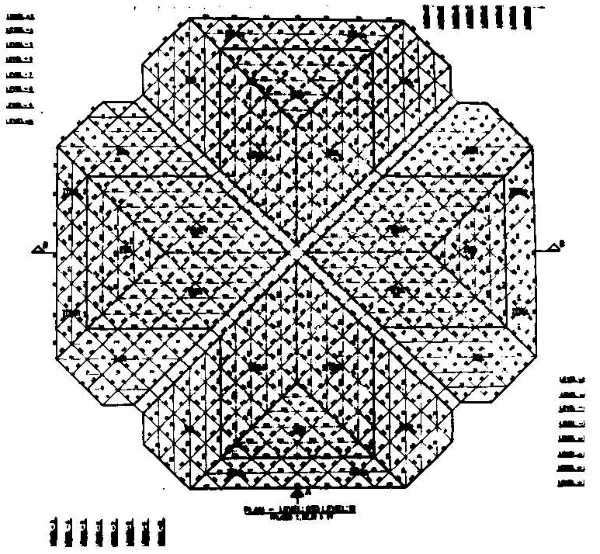


Fig. 2.1 Plan

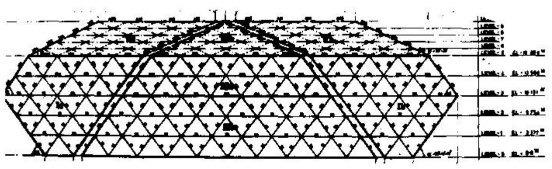


Fig. 2.2 Elevation

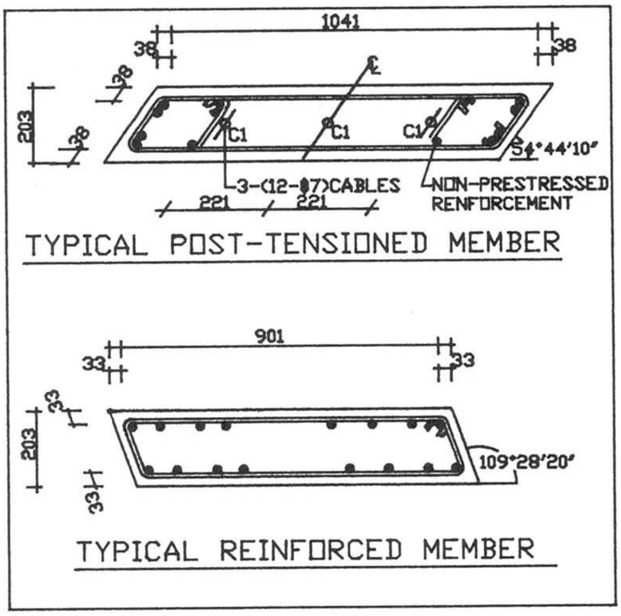


Fig. 2.3 Cross Section of Members

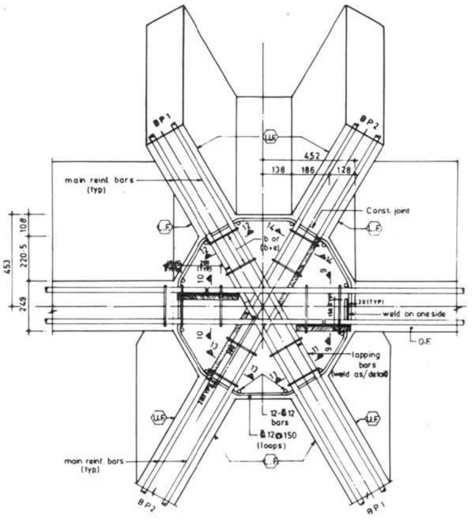


Fig. 2.4 Joint Details

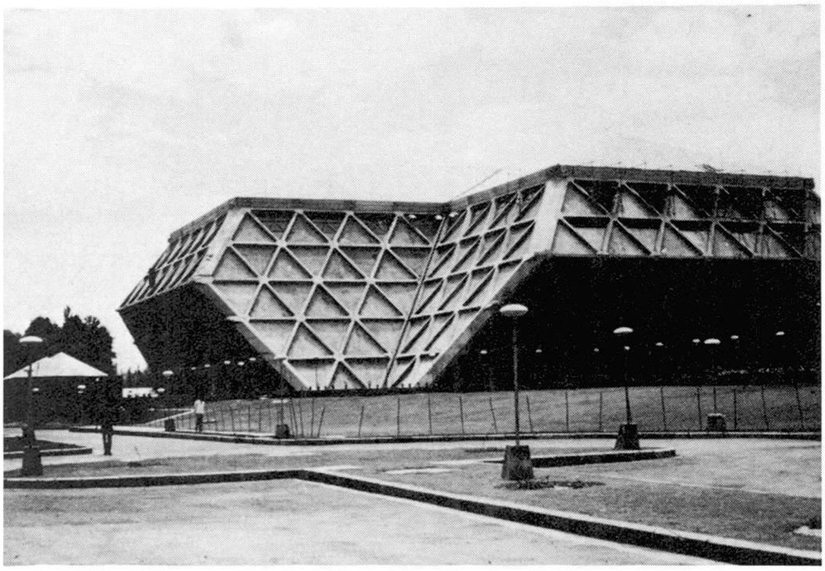


Fig. 2.5 Exterior View of Stadium

Point Theatre, Concert Hall and Exhibition Centre

Le Théatre Point, salle de concerts et centre d'expositions

Das Point Theater, Konzerthalle und Ausstellungszentrum

Brian J. MCCANN
Civil Engineer
Thorburn Colquhoun
Dublin, Ireland

Brian J. McCann, born 1944, received his degree in Civil Engineering from Univ. College Dublin. For the past twenty years he has been in consulting engineering specialising in the refurbishment and redevelopment of older buildings.

SUMMARY

This facility located in the Docklands area of Dublin is the venue for theatrical events, concerts and conventions. Originally constructed in 1878 as a quayside railway goods warehouse, it was converted in 1988 into a multipurpose concert and exhibit venue with an auditorium capacity of up to 3'750 seated and 8'000 standing. The central auditorium covering 2'850 sqm was created by the removal of internal columns after the roof had been supported on external beams spanning 55 m between supports over proscenium arch some 34 m span.

RÉSUMÉ

Ce bâtiment dans la zone portuaire de Dublin abrite des représentations théâtrales et musicales et des congrès. Construit en 1878 pour servir d'entrepôt des chemins de fer portuaires, il fut transformé en 1988 en une maison de la culture à usage polyvalent (3'750 places assises et 8'000 places debout). D'une surface de 2'850 m2, la création de l'auditorium a impliqué la démolition des piliers intérieurs, après avoir suspendu et ancré la toiture à des poutraisons extérieures qui, avec une portée de 55 m, enjambent une scène courbe de 34 m de longueur.

ZUSAMMENFASSUNG

Dieses Gebäude im Dubliner Hafengelände ist Schauplatz für Theater- und Konzertaufführungen und Kongresse. Im Jahre 1878 ursprünglich als Warenlagerhaus für die Eisenbahn am Hafenkai errichtet, wurde es 1988 in einen Mehrzweckkulturbau umgebaut, mit 3'750 Sitzplätzen und 8'000 Stehplätzen. Das zentrale Auditorium mit 2'850 m2 Grundfläche wurde durch Entfernen der Innenstützen geschaffen, nachdem das Dach an externen Trägern verankert wurde, die mit 55 m Spannweite einen Bühnenbogen von 34 m überspannen.

1. INTRODUCTION

As originally constructed for the Great Southern and Western Railway of Ireland in 1878, the Point Depot comprised a 52m wide nine track three bay goods warehouse 112m long. The 12m high centre bay was flanked by full height mezzanine floors 8m over ground level. Transfer of goods to and from the upper levels was effected by a series of chain operated swing derricks mounted on the mezzanine columns and external walls. Prior to the conversion, the Point Depot stored permanent way materials following the rationalisation of railway freight services in the early 1970s.

2. CLIENT BRIEF

The twin aims of the brief to the design team were to provide a venue for modern entertainment activities while preserving the unique ambience of the building particularly the delicate detailing of the wrought iron roof and mezzanine steelwork (Fig.3). The major structural elements of the conversion undertaken between 1988 and 1991 were:

Phase 1: (1988)	-	removal of columns to provide a 34m wide x 55m deep central column free area thereby creating an amphitheatre type space.
	-	the provision of an over stage grid to support sound and visual equipment up to 18 tonnes in weight.
Phase 2: (1989)	-	the addition of an 80 line fly tower with flying height of 21m and load capacity of 27 tonnes.
	-	improved load capacity of up to 30 tonnes over the central auditorium for events 'in the round'.
Phase 3: (1991)	-	side and rear balconies with fixed seating capacity for 1,574 persons to reduce labour costs associated with demountable seating.

3. STRUCTURAL ASSESSMENT OF ORIGINAL BUILDING

The structural elements of the building were in good condition with no significant defects in the masonry walls, columns or roof trusses. At the time of original construction both wrought iron and cast iron were in general use (1). The materials used in the trusses and beams were identified as wrought iron following tests undertaken by University College Dublin (Yield Strength : 234N/sq.mm. : Ultimate Tensile Strength: 351N/sq.mm.).

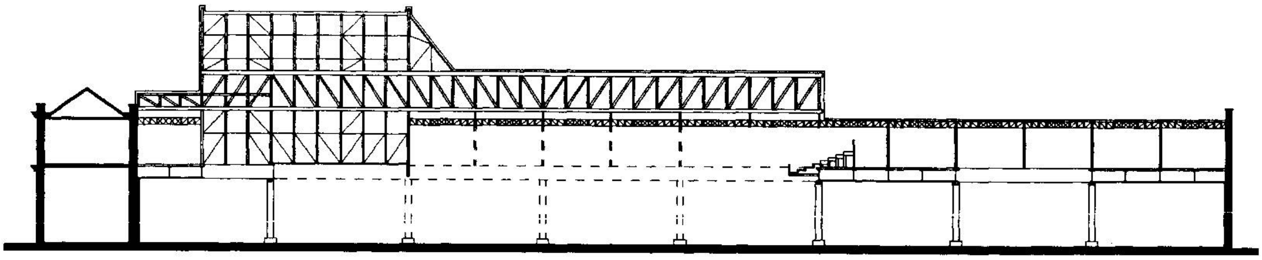
The material of the cast iron columns was confirmed by their orange skin appearance and the wall thickness measured at three points around each diameter. The load bearing capacity of 6500 kN was calculated from the lowest of the Euler, Rankine, American and Goodman formulae. The structural assessment based on research and tests was substantially confirmed by local folklore which related that during World War II some 6,000 tons of sugar had been stored on the mezzanine floors with a combined area of 3,500 sq.m. giving a superimposed live load of 17 kN/sq.m.

Fig.1: Erection of Main Roof Girders - Phase 1.

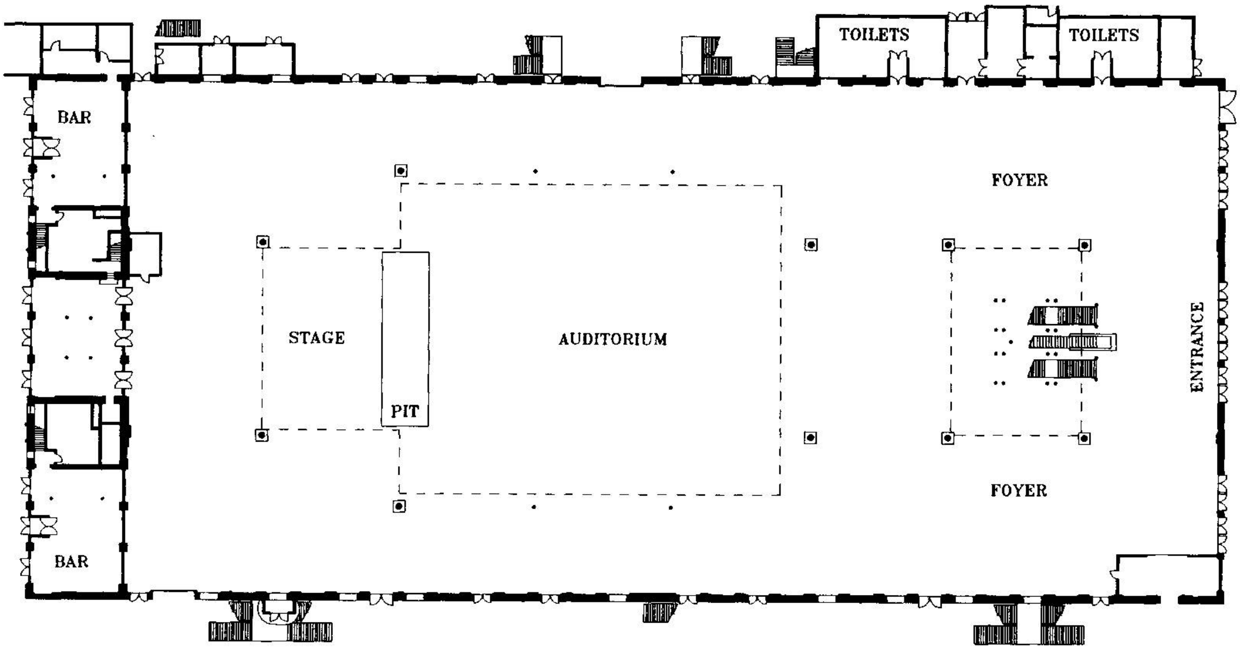
4. STRUCTURAL SCHEME FOR CONVERSION

The major element of the conversion from goods warehouse to multi-purpose arena was the opening up the centre of the building to form an amphitheatre and the linking of the two mezzanine floors in the side bays to form circulation, access and upper exhibition areas. The enlarged central well containing the seating area measures 55 metres long by 34 metres wide.

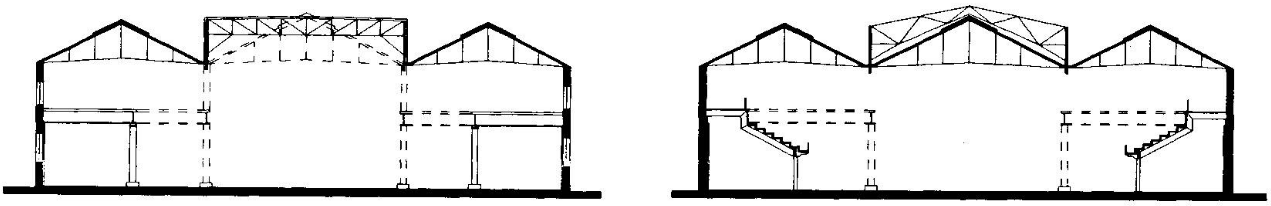
In order to retain the original roof form, the solution devised by consulting engineers Thorburn Colquhoun, provided for the replacement of support lost by the removal of interior columns by transferring the roof load to an external structure i.e. literally hanging the roof from it. The solution enabled the new complex to accommodate heavy equipment when hosting concerts or other events without the need to erect a special scaffolding as is the case in some other concert halls and venues. The external structure located in the roof valleys comprised twin lattice girders, 55m long, 3.5m deep and each weighing 38 tonnes, supporting the roof from 36mm dia, high strength hangers at 6.87m ctrs. A camber was designed into the girders both for the sake of appearance and to shed rainwater externally. Lateral stability to the girders was provided by the over stage grid and by external cranked and tapered lattice beams over the seating area.



LONGITUDINAL SECTION – Phase 1 & 2

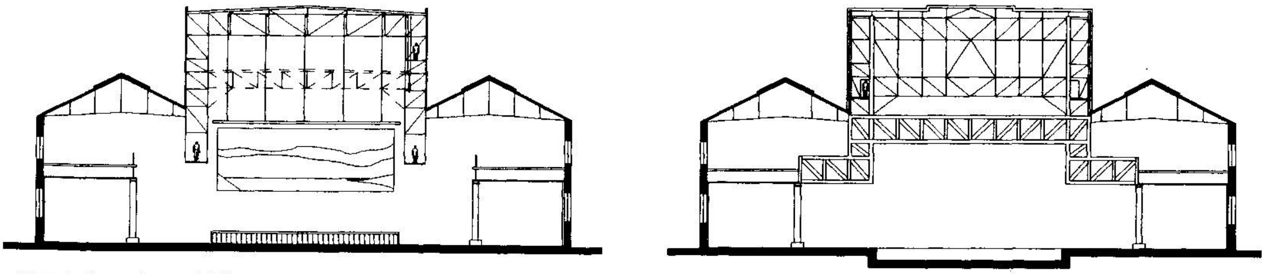


FLOOR PLAN



SECTION STAGE – Phase 1

SECTION PROSCENIUM – Phase 3



SECTION STAGE – Phase 2

SECTION PROSCENIUM – Phase 2

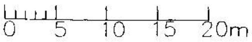


Fig. 2

5. CONSTRUCTION - PHASE ONE (Summer 1988)

Because of the size of the two primary support girders, the Contractor opted to fabricate them on site. After fabrication, on site x-ray checking was carried out on all butt welds. While this work was being executed the mezzanine floor beams adjacent to the four primary support columns were propped and the columns suspended from the beams while new piled foundations were installed under the original columns and their stone bases. A noteworthy feature of the erection procedure was the contractors decision to temporarily remove the roof trusses over the auditorium to facilitate erection of the main girders (Fig.1). This decision was undoubtedly influenced by the fact that a new roof covering on new steel angle purlins was being provided to replace the original timber boarding.

Once the roof loading had been removed from the columns at mezzanine level, the wrought iron beams at the central area mezzanine level were propped and cut back by 7 metres with the remaining six cast iron columns replaced on new piled foundations incorporating the original stone bases. The original fascia beams were also replaced on the columns before removal of the temporary props.

Within the foyer area, the central well at mezzanine level was infilled with reinforced 914mm x 419mm Universal Beams to create landing areas for the stairs and escalator.

Fig.3 Auditorium from Stage - Phase 1.

6. STRUCTURAL ASPECTS - PHASE TWO.

Because of the requirement for clear floor space under the fly tower, a double purchase system, employing 27 tonnes of counterweights on one side only, to balance the flying loads of 13.5 tonnes, was required. The major structural challenge was to support the fly tower on the large valley girders installed during the previous year to create the central well area. Particular attention was paid to the stability of the valley girders following removal of the original grid, to the load carrying capacity of the valley beams and to supporting structures. The addition of the fly tower generated an increase from 2263 kN to 2595 kN on each valley beam, inclusive of drifting snow against the wall of the fly tower.

The core of the design finally evolved, after consideration of strength, stability and sight lines, involved a cranked lattice beam 2.8m deep spanning 34m between two of the relocated columns. This beam in addition to providing an intermediate support to the valley girders has the additional advantage of creating a natural break between stage and auditorium at the front of the fly tower without unduly restricting the open space aspect of the building during exhibitions.

7. CONSTRUCTION - PHASE TWO (SUMMER 1989)

Phase two was successfully completed in 1989 to an erection sequence devised by Thorburn Colquhoun. No exceptional difficulties were encountered during construction. The main supporting beam under the proscenium arch was fabricated in three sections in the workshop and assembled by site welding. Erection proved awkward due to the weight (28 tonnes) and lack of headroom for the crane jibs.

8. PHASE THREE (SUMMER 1991)

Operating experience during the first two seasons highlighted the requirement for permanent seating where possible. To facilitate installation of the two side balconies, the cut back mezzanine in the central well was removed and replaced with precast concrete terrace units on cranked 457mm deep Universal Beams supported on new 300mm dia. circular hollow section columns. The side balconies also incorporated a full length horizontal wind girder at mezzanine level. The primary supporting member in the rear balcony was a 19.6m long composite steel section comprising 254 x 254 UC on 914 x 419 UB 343 stiffened by 2 No. 36mm dia wire cables giving total depth at the centre of 2,400mm. Such a low span/depth ratio of 8:1 was required to limit vibration by foot-tapping patrons on the rear balcony.

ACKNOWLEDGEMENTS

The author wishes to thank everyone associated with the project with particular credit to Mr. Harry Crosbie for his vision and foresight.

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Central Hall of Liederhalle Culture and Convention Centre Stuttgart

Salle principale du centre culturel et de congrès Liederhalle à Stuttgart

Der mittlere Saal des Kultur- und Kongresszentrums Liederhalle in Stuttgart

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Reinhold Braschel, born 1942, graduated at the Technical Univ. of Graz in 1979. He founded his own engineering office in 1975 established The IFB Planning Group in 1989.

Frank STELLER
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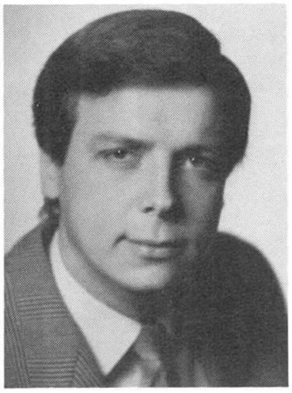

Frank Steller, born 1958, got his Dipl-Ing. at the Univ. of Stuttgart. Since 1985 he has worked with the IFB Planning Group, where he is a project manager for structure design.

SUMMARY

The roof structure of the Culture and Convention Centre Liederhalle had to consider concentrated high loads. Because of the sensitive glass construction, minimal deformation was allowed. While providing for a variety of interior equipment to be suspended from the roof structure, all inadmissible penetrations of the steel frame through the hanging ceiling had to be avoided. This required a comprehensive three-dimensional global analysis, which could only be achieved by means of a CAD system

RÉSUMÉ

Dans le projet du Centre culturel et de congrès Liederhalle, de fortes charges ponctuelles sont ancrées dans la zone de la toiture. Seules de faibles déformations sont tolérables en raison des sensibles superstructures en verre. L'interaction entre la structure métallique et le plafond suspendu a posé quelques problèmes intéressants, dont la solution ne fut possible que grâce à une approche globale, conséquente et tridimensionnelle recourant à l'informatique.

ZUSAMMENFASSUNG

Beim Bauvorhaben "Kultur- und Kongresszentrum Liederhalle" mussten grosse Einzellasten im Bereich der Dachkonstruktion verankert werden. Wegen der empfindlichen Glasaufbauten waren nur geringe Verformungen zugelassen. Bei der Vielzahl von Abhängungen im Zusammenhang mit dem Innenausbau durften keine unzulässigen Durchdringungen der einzelnen Bauteile auftreten. Dies war nur durch eine konsequente dreidimensionale Gesamtbetrachtung mit Hilfe der EDV möglich.

1. INTRODUCTION

To widen the many-sided cultural offer and at the same time provide the province capital Stuttgart a building for international conventions, Stuttgart decided therefore to build the Cultural and Convention Centre Liederhalle. In the beginning of the year 1988 the construction started and by August 1991 it was finished. The building consists of the Centre Hall with up to 1840 seats, the Small Hall with 425 seats and a garage with three storeys. The construction of a First Class Hotel with 300 rooms and direct access to the Convention Centre had to be considered during the planning of the Small Hall. Together with the neighbouring Liederhalle Concert Hall from 1956, which is under monumental protection, the capacity of the Cultural and Convention Centre will be about 5300 seats for various events like conventions, exhibitions and seminars.

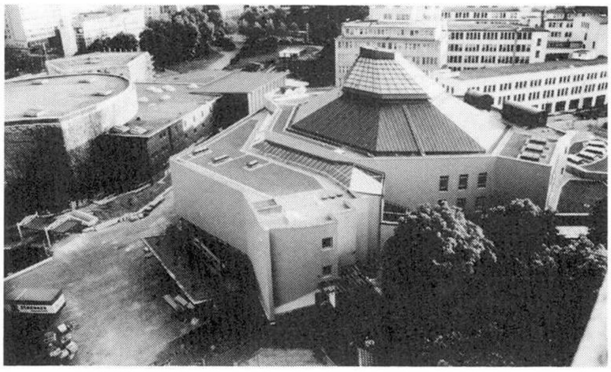

Fig. 1 Culture and Convention Centre Liederhalle

With the core of the building, the so called Centre Hall and its characteristic glass cupola the Cultural and Convention Centre gets an architectural highlight. Around the septagonal hall, the lobby areas, seminar and administration rooms are coordinated as the technical centre and the kitchen in the ground-floor. The seats in the hall and in both the galleries are stepped up and back. Moveable platforms can be slid out to accommodate tables in the galleries. The floor of the hall can be raised or lowered to meet various functions.

The roof of the hall, approximately 35 m in diameter, is a seven sided steel pyramid stump. The structural system consists of spider web placed I-profiles which are stabilized by a septagonal compression ring below the glass pyramid. To meet acoustical requirements the roof is covered with pre-cast reinforced concrete panels which were grouted afterwards. The steel construction of the roof carries a load of approximately 3000 kN. Therefore a shell structure was not possible. It was also important to minimize the deformations of the structure supporting the cupola to protect the glass and the shading elements from breakage.

The Centre Hall with its different structural systems and complex geometry and function made the engineering work highly demanding.

2. CAD-INSERTION

In order to make the planning and the structural design of the roof construction with its complicated geometry and crooked areas, an intensive CAD-Insertion was required. At that time IFB was one of the first Planning firmes in Stuttgart to work successfully with CAD-software. In that way it was made possible to tackle the complicated geometry of the floors with Post-Processors and then transfer the data into Finite Element Program.

A decisive advantage was obtained by using coordinate plans to measure out the building. In the beginning it was not planned to work with this sort of plans. To measure out the building in the traditional way turned out to be too complicated and nearly impossible. Therefore we had to work out a coordinate plan which shows all the characteristic points of the building with their x and y coordinates. Since the plans of the building were already digitalized in our

CAD-System we could develop the coordinate plans in a relatively short time. Using those plans, the survey engineer could determine exactly the layout of the building at the site with electro - optical methods. The coordination plans were therefore extremely important in order to maintain the accuracy between the steel production and the construction site.

By making the coordinate plans we gained specially two things:

- highest possible accuracy in measures for the structure
- extended use of the coordinates for the interior construction

Through this positive experience by using the coordinate plans for the reinforced concrete structure we decided to make coordinate plans also for the steel structure which will be discussed later.

3. THE STRUCTURAL SYSTEM OF THE BUILDUNG

3.1 The Reinforced Concrete Construction

Beside the roof construction all carrying elements are made out of reinforced concrete. The building itself contains no expansion joints. The floors in the basement are partly 90 m lang without expansion joints and are like the upper storeys calculated as slabs.

The loads from the roof construction are carried with corbels on to the supporting walls. Beside the high dead loads the support had to take up horizontal loads of 2000 kN from the compression ring. The reinforcement for the structural system was determined with the help of strut and tie models.

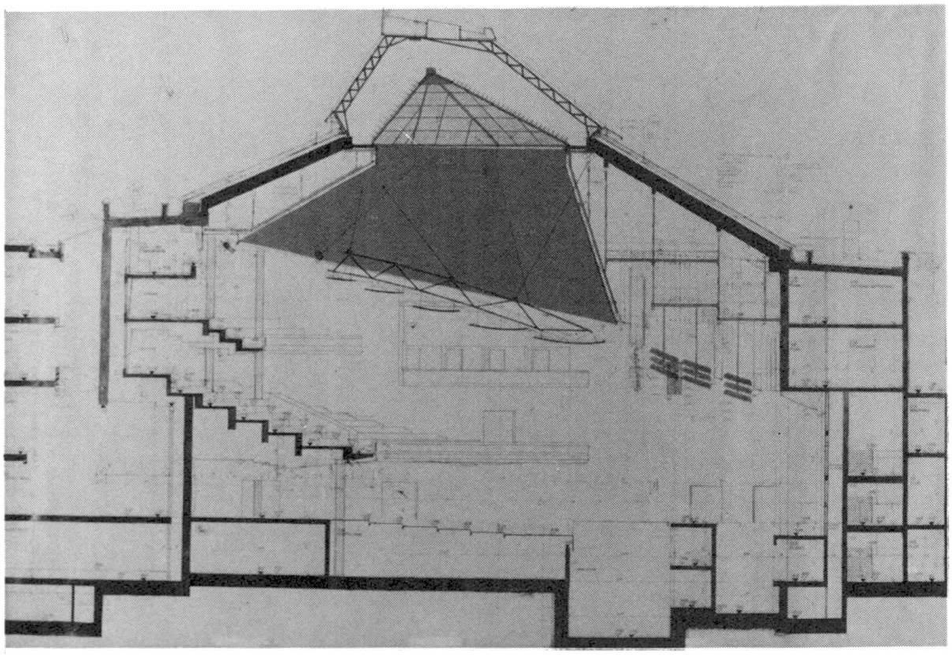

Fig. 2 Cross-Section of the Centre Hall

3.2 The Steel Construction

As portraied it was not possible to formulate the roof construction as a shell structure because of high loads. Therefore the roof was constructed as a steel frame pyramid stump with many connection points for the various demands.

Many complicated technical details placed high demands on the planning. Especially the many-sided hanging construction with its complicated technical details placed high demands on the planning. Because of that the main emphasize was on the roof construction.

4. THE HALL CEILING AND THE HANGING ELEMENTS

4.1 The Roof Construction

The geometrical form of the roof corresponds to an irregular septagonal pyramid stump with curved sides. Characteristic for this structure is the septagonal compression ring which is supported by twenty-one steel girders running from the ring down to the supporting walls. The compression ring supports then the visible glass cupola. The main girders are so arranged that the curved trapezoidal areas are devided into plane triangular areas.

That way it was possible to use pre-cast reinforced concrete elements for the covering of the roof which had also the function as an acoustical system. To minimize deformations in order to protect the sensitive glass structure all connections were made moment-stiff. The structural analysis was done with a three dimensional truss program. For acoustic reasons the natural frequency of the roof structure had to be less than 15 Hz. Therefore a dynamic analysis was carried out. This subject led later to a Dipl. thesis which examined the resonance behaviour of the structure as well as the behaviour under earthquake loads.

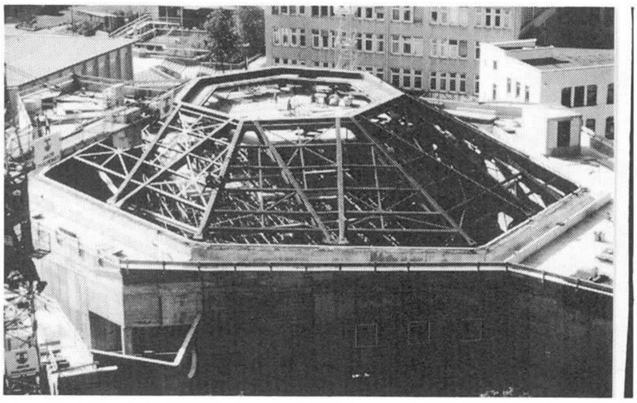

Fig. 3 General View of the Roof Construction

The core of the steel work planning was again the coordinate plan. The entire nodes, breaks and purlin connections were listed there with their x, y and z-coordinates so that all axis of the steel profiles were defined in a three-dimensional system. This was important to the planning of all the hanging components.

The compression ring consists of four parts which were assembled at the construction-site. The assembling of the whole roof construction took five weeks.

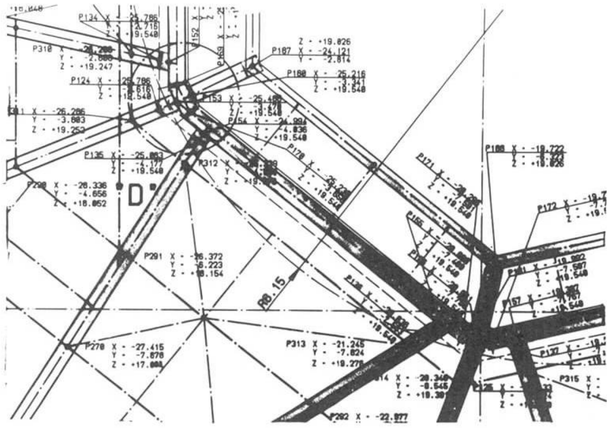

Fig. 4 Section of the Coordinate Plan

4.2 The Shading Pyramid

One of the requirements due to the utilization was the natural lighting respectively the possibility of shading the entire hall. This was reached by arranging the septagonal shading pyramid inside the glass-cupola. The structure weighs about 12 tons and is placed on seven cantilever beams which have a moment-stiff connection to the compression ring. Because of the sensitivee shading lamellas, high deformation criterias had to be met. The area between the shading pyramid and the compression ring is walkable for maintenance purposes.

4.3 The Stage Mechanics

The stage mechanics which bring extremely high loads on to the roof area are arranged on several levels. With the so called "Rolling floors" the transport of the scenery takes place. For that purpose a great number of lifting gear and pulleys are needed. The loading together with the grating structures weighs about 150 tons. Special hanging girder transfers the loading to the main girder of the roof structure.

4.4 The Construction of the Hanging Ceiling

The hanging ceiling is constructed in the middle part - similar to the steel structure - as an inclined stump pyramid, the so called 'Hat-structure', which is then tied to the roof structure. Around the 'Hat-structure', the so called 'Brim-structure' is placed, a plane area with a slight slope to the stage. The gypsum board ceiling is fixed to the base structure with common perforated ties. All areas are walkable for maintenance purposes. The spot-lights for the stage lighting are also placed there.

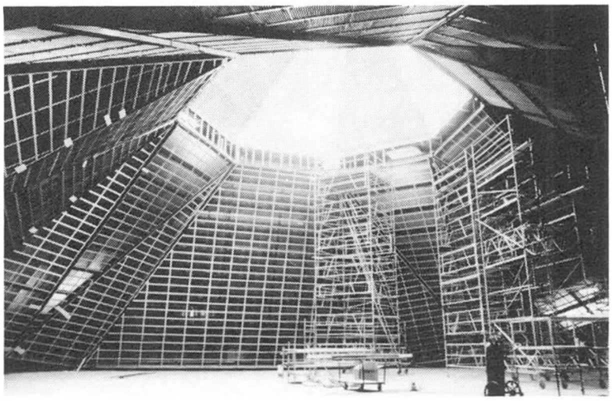

Fig. 5 Base Structure of the Hanging Ceiling

The position of the gypsum board ceiling was located exactly by the architect. It was the basis of three-dimensional modeling of the structure for the CAD-system. In addition we determined analytically with our own geometry programs the axis of the girders, the breakthrough points of the ties and the intersections of planes. Overall more than 1000 three-dimensional coordinates had to be determined. Therefrom a coordinate plan was set up which was the basis of completing the hanging ceiling. Due to the great number of unknown coordinates, three-dimensional CAD work became an important instrument in order to check-out the coordinates.

The assembly of the base structure followed directly the completion of the roof. For that purpose a scaffold was errected at the level of the second gallery.

4.5 The Acoustical Panel Supports

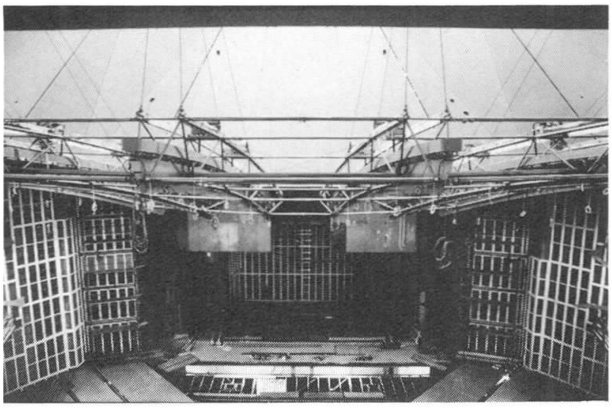

Fig. 6 Acoustic Panel Support

The acoustical panel supports with the hanging sound reflectors are basically to serve acoustical purpose but they are also used as an architectural construction for the hall. Six curved trusses are fixed to one another with cross bars and form a stiff curved shell. In between are 23 sound reflectors which are made of chemically prestressed float-composite-glass sections. They are fixed to the roof with three diagonal steel cables each. To guarantee the stability of the system, all cables have to be under tension and cannot penetrate through the base structure of the hanging ceiling. With these geometric boundary conditions given, a precise planning was necessary with the determination of all fixed and penetrated points.

Due to the high installation loades and the loads from the pulleys, some parts of the structure were made out of aluminium in order to reduce the weight. The trusses were transported into the Centre Hall in one piece by using a window-opening in the second floor. Then they were positioned individually and finally connected with the cross bars.

5. CONCLUDING REMARKS

The many-sided utilization of the "Culture and Convention Centre Liederhalle" placed extremely high demands upon both the planners and the executing companies.

Fig. 7 View of the Centre Hall

Fig. 8 View of the Lobby

High single loads had to be taken up by the roof construction. To avoid ineligible penetrations the whole structure had to be analyzed in a three dimensional system. This was only made possible by consequently using electronic resources. It turned out that perfect technical, economical and time managing planning of this building would have been impossible without using CAD.

In spite of all regards to this technical interesting building, at the end of the structural work one will find out that the given tariff is far from covering the real expenses.

Design of the France-Japan Friendship Monument

Projet du monument de l'amitié franco-japonaise

Entwurf des französisch-japanischen Freundschaftsdenkmals

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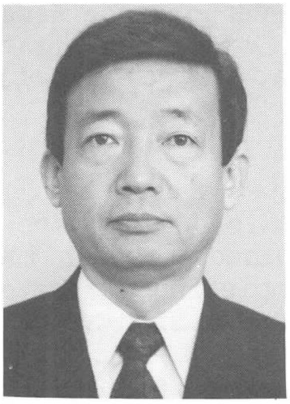


Naoki Uchida, born 1940, received his doctorate from the graduate school of the Univ. of Tokyo where he majored in steel structures. At present, he is the pricipal of the Osaka Engineering Office of Nikken Sekkei Ltd and has been engaged in structural design for more than 30 years.

SUMMARY

A project to construct a Monument as a symbol of friendship between France and Japan is under way in Japan, for completion in 1998. This paper describes the structural design by the Japanese side based on the concept design by French architect Patrick Berger. A 210 m-long, 21 m-width, and 4,2 m-high box-girder steel table, covered with thin bronze plates, will be supported at each end by a jungle-gym-like grid of steel pillars. Prestressed cables will be used to offset deflection caused by the bronze table's weight.

RÉSUMÉ

Un monument symbolisant l'amitié franco-japonaise est en construction au Japon et devrait être achevé en 1998. En se basant sur une conception de l'architecte français Patrick Berger, les ingénieurs japonais ont étudié la structure porteuse. Une poutre-caisson métallique de 210 m de long, 21 m de large et 4,2 m de haut, recouverte de minces plaques de bronze, est supportée à ses extrémités par un treillis complexe de montants métalliques. Des câbles de précontrainte sont utilisés pour compenser les déformations dues au poids des plaques de bronze.

ZUSAMMENFASSUNG

In Japan befindet sich ein Monument zur Symbolisierung der französisch-japanischen Freundschaft im Bau, das 1998 fertiggestellt sein soll. Ausgehend von einem Konzept des französischen Architekten Patrick Berger wurde von japanischer Seite das Tragwerk entworfen. Ein 210 m langer, 21 m breiter und 4.2 m hoher Stahlkasten-Träger, verkleidet mit dünnen Bronzeplatten, wird an beiden Enden von einem dschungelartigen Geflecht aus Stahlpfosten getragen. Vorspannkabel werden zum Ausgleich der Durchbiegungen infolge des Gewichts der Bronzeplatten eingesetzt.

1. OUTLINE OF THE MONUMENT PROJECT

1.1 Background

The Statue of Liberty, symbolizing liberty which was considered as a theme for the mankind in the 20th century, was constructed in the New York Bay as a token of friendship between the United States and France about 100 years ago in 1886. In line with this spirit, the French-Japanese Symbol Association was founded in 1986 aiming at pursuing a joint project for constructing a communication monument, as a symbol of exchange and communication on global scale, which would become indispensable for the global community in the 21st century in Japan, as a token of friendship between France and Japan towards the 21st century.

As a result of the design competition held twice in France, the " Awaji: Garden of Tropics " by French architect Patrick Berger was selected in July 1989.

The Japanese side set up the Japanese Committee for the Japan-France Friendship Monument in December 1989, in order to promote it as a project supported by the nation, for which purpose the Association of Intercultural Communication was set up in May 1993 as an organization entrusted with the construction of this monument, aiming at further promoting the project.

1.2 Progress of design work

The French side presented a conceptual design in January 1990, and the results of the preliminary basic design were presented to the Japanese side in two occasions in June 1990 and November 1992. As a result of technical discussion by both sides, the completion of the preliminary basic design was confirmed, and also that the Japanese side should tackle with the detail design and the construction of the monument by counting with the cooperation from the French side.

For this design, the construction technology study group (5 teams comprising 51 specialists from structural design, wind resistant design, seismic design, materials, and construction planning fields) and the specialist study group (led by Tsutomu Yamane, former President of Honshu - Shikoku Bridge Authority and comprising 15 members chosen from academic, governmental, and design fields) were set up inside the promoting organization on the Japanese side.

At present the Japanese side is working on the detail design, for which the basic design stage has almost been completed, and based on whose results this report is filed.

For the work schedule in the future, the detail design will be completed at the end of 1994 to start the construction of the monument. Completion of the monument is scheduled for the spring of 1998.

2. MONUMENT CONCEPT

2.1 Basic concept

Patrick Berger, who proposed the monument's scheme, aims at realizing a monument embodying the theme of "exchange - communication" as its basic concept. For this reason, the monument is structured to the image of a gate, serving as the starting point of communication.

2.2 Scale of monument and proposed site

The length of the bronze table is so designed as to be compatible with the height of the foundation of the monument (Fig. 1). Size of the element of the monument is :

Bronze table (210 m long, 21 m wide, 4.2 m thick)
A couple of twin pillars (50.4 m high, 8.0 m wide, 12.0 m deep)

The monument will be located on a spot where longitude 135 E, which marks the Japan Standard Time, passes; and, the construction site is located atop a small

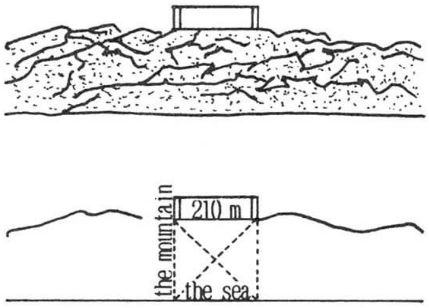


Fig.1 Setting of length and height of the monument

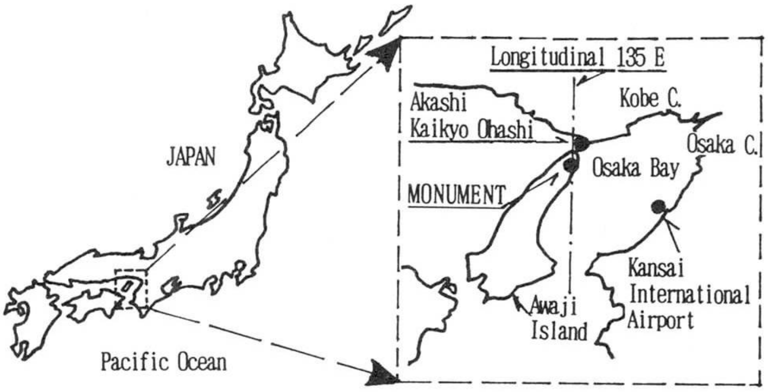


Fig.2 Location of the monument

hill bit higher than 200 m found at about 1.5 km from the coast, inside the Prefectural Awaji Island Park at the northern end of the Awaji Island. And, a superb view open to Osaka Bay will be enjoyed from the site (Fig. 2).

3. STRUCTURAL PRINCIPLE AND ERECTION METHOD

3.1 Structure of bronze table

3.1.1 Structural principle

The external form of the bronze table features an extremely slender structure as the depth of the table beam is 1/50 of its length in the axial direction; while for the transversal direction, it has a rectangular cross section of 1:5 ratio. The steel box-girder type is used to ensure sufficient bending stiffness and torsional stiffness, besides by taking into account the large span length; and, for the single-span steel box-girder style, this bronze table has the world's longest span length. Steel cables for prestressing ("PS cables") are parabolically arranged inside the cross section of the steel box-girder as shown in Fig. 3; thus, the structural principle is to offset the bending moment due to the dead weight of the table with the cancellation moment through the introduction of cable's tensile force, so as not to cause deflection. And high-tensile steel having tensile strength of 7.8 tf/ ㎠ materials are used.

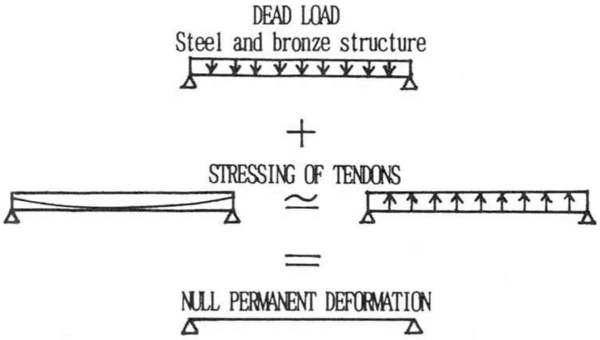


Fig.3 Structural principle of the monument

3.1.2 Structural experiments

Structural experiments were carried out by using 5 speciments(scale: about 1/10) (Fig.4)in order to verify the effect of cancellation moment of the steel box-girder compressed by PS cables, and also to confirm vibration properties of the beam due to the tensioning of PS cables (PS cable tensioning distance: about 17 m). As a result of this test,the cancellation moment effect could be confirmed in all speciments. The tests also revealed that the dynamic characteris-

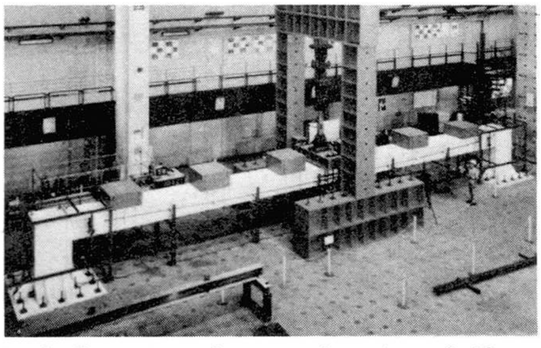


Fig.4 Structural experiments of the box-girder compressed by PS cables

tics of the PS teel beam did not depend on the compressive force of the steel beam caused by the tensioning of PS cables.

3.2 Pillar structure

3.2.1 Structural principle

The system truss structure by using tubular section is widely used in Japan for the roof structure of gymnasiums and assembly halls. The pillar (Fig.5) of the monument is used as a perpendicular support member to sustain the weight of the bronze table,large cross section of the member is required. Also from viewpoint of design, square steel member with solid cross section(150 mm x 150 mm)is used in order to realize a compact cross-sectional area. This cross-sectional form presents a regular square, which is threaded at both ends. A hexagonal sleeve is used at the threaded part to avoid fatigue failure; and, joining is achieved by introducing the compressive force to the sleeve and the tensile force to the threaded part, thus minimizing the alternation of stress at the threaded part.

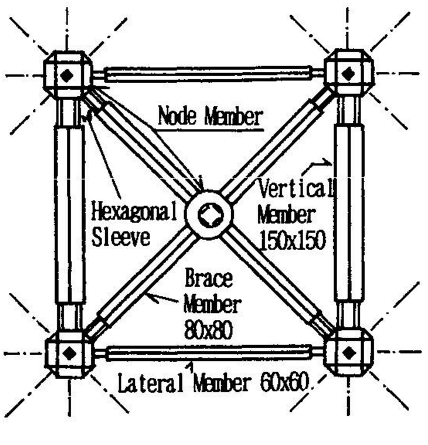


Fig.5 A unit frame of the pillar

3.2.2 Structural experiments

We conducted the manufacturing test and mechanical tests with the structural members, and we also carried out a test for confirming the axial force introduction method by hexagonal sleeve, and the load bearing test.

3.3 Erection method

3.3.1 Lift-up method (Fig. 6)

The bronze table work will be carried out in the following way:

- Pillar materials and bronze table materials will be manufactured at shop. The bronze table materials will be produced by block unit at the shop, and will be carried to the construction site.
- Pillars will be assembled on the base, whose work would have started earlier at the site. The central part of the bronze table will be manufactured at the site as an integral structure on the ground right beneath the proposed place of the table; and, up to the tensioning of cables will be completed.
- The end part of the bronze table will be assembled to the top of the pillar.
- The bronze table assembled on the ground will be elevated and will be joined with the blacket of the bronze table, which is set to the top of the pillars, by using high-strength bolts ("HT bolts").

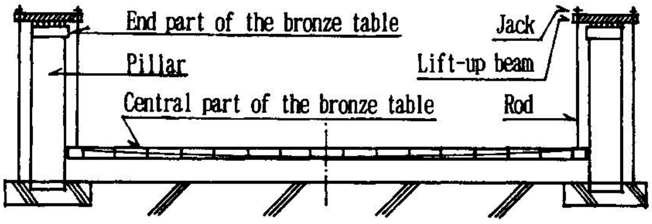


Fig.6 Lift-up method

3.3.2 Joining method

The bronze table should, in principle, be joined by welding before the lifting up. The joining of the bronze table following the lifting up will be made by using HT bolts.

4. DESIGN PRINCIPLES

4.1 Load

The bronze table was designed to the total weight of 5,100 tf, comprising 4,000 tf of steel members, 500 tf of PS cable members, and 600 tf of bronze (21.0 tf/m at the center part of the bronze table). The weight of the pillar is 4,700 tf in total, comprising 3,600 tf of steel members and 1,100 tf of glass. Thus, the total load of the entire monument structure is 9,800 tf.

4.2 Seismic design

Japan is located on the Pan-Pacific Seismic Belt, thus is prone to earthquake, and has experienced a large number of earthquakes of huge scale in the past. For this reason, seismic design is considered as one of the most vital design elements regarding the design of buildings and civil engineering structures.

4.2.1 Seismic design method

The input seismic level used by the earthquake response analysis, which was used for the seismic design of the monument, was set forth as follows:

Level 1 earthquake: Presupposes moderate earthquake, which might possibly occur during the expected durable life of the monument. Maximum velocity of horizontal movement is 20 cm/sec.

Level 2 earthquake: Presupposes severe earthquake, which might occur at least once during the expected durable life of the monument. Maximum velocity of horizontal movement is 40 cm/sec.

And, the following design policy was set forth for the members to allow the use of this monument over a longer period of time as compared with ordinary structures.

Against Level 1 earthquake: The member's stress is to be below the short-term allowable stress.

Against Level 2 earthquake: The member's stress is to be within the elastic arange.

4.3 Wind resistant design

Due to geographic conditions, Japan is under the influence of the prevailing westerlies throughout the year, being the northwesterly seasonal wind prevailing in winter. Typhoons come close to or cross the Japanese archipelago from summer to autumn every year. Wind tunnel tests were conducted in order to grasp the wind-resisting features of the structure.

4.3.1 Wind resistant design method

The design wind velocity was set forth based on the guidline by Architectural Institute of Japan.

Level 1 wind velocity: 61.1 m/sec
Level 2 wind velocity: 68.1 m/sec

Against Level 1 wind velocity: The member's stress is to be below the short-term allowable stress.

Against Level 2 wind velocity: The member's stress is to be within the elastic range.

4.3.2 Design wind loads

Stress analysis was realized for each of the average wind load and the variable wind load, and the results from the both were added together in order to see the total stress. For the average wind load in this case, the coefficient of wind force of the bronze table and the pillar was set based on the results of the static vibration response test. For the variable wind load, the stress analysis was made by providing the displacement, which was sought from the results of the dynamic vibration response test.

4.3.3 Wind tunnel test and wind observation.

As the wind tunnel tests, we conducted the analysis of air flow at site, the site air flow reproduction test, static wind load (three-component force and six-component forces) test, average wind pressure and variable wind pressure measurement, and dynamic vibration response test(Fig. 7). It was found out for the monument that, if wind blows in a direction at a right angle to the bronze table, wind resonance of the bronze table was caused, but destructive vibrations (flattering or galloping) were not caused, and also that such resonance was hard to be generated in the turbulence, but the irregular vibration (buffeting) due to the breathing of wind was prominently caused in the turbulence. An observation tower (70 m tall) was installed at the site, and the wind observation was realized since July of 1992. These wind observation data were reflected in the detail design as basic data for setting forth the design wind speed.

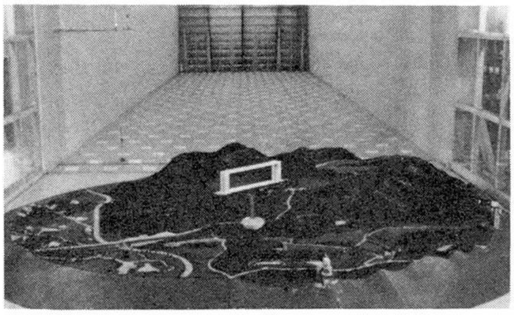
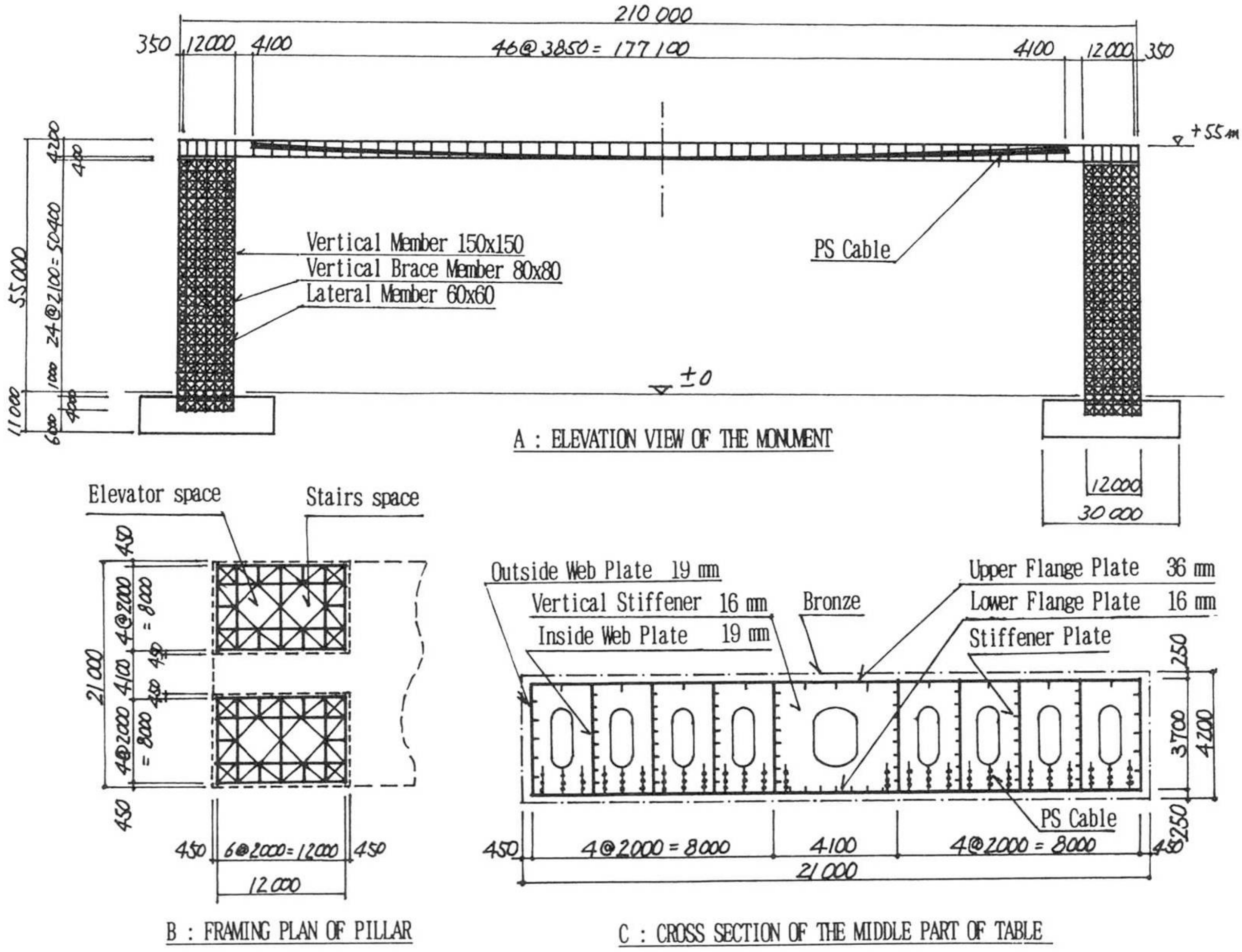


Fig.7 Wind tunnel test (monument and topographic model)

5. STRUCTURAL OUTLINE

A structural outline of the monument is shown below (Fig. 8).



A : ELEVATION VIEW OF THE MONUMENT

B : FRAMING PLAN OF PILLAR

C : CROSS SECTION OF THE MIDDLE PART OF TABLE

Fig. 8 Structural outline of the monument

6. AFTERWORD

The France-Japan Friendship Monument is now in the stage of detail design; and, the above outlines the studies, surveys and design works so far conducted. In addition to those, strength and fatigue tests of pillar materials and the study of flow of forces at the joint between the bronze table and the pillars are currently under way. And, observation of wind and seismic movements at site are also continuously carried out.

Nouveau Ministère de l'économie, des finances et du budget, Paris

Neues Wirtschafts- und Finanzministerium in Paris

New Economy, Finance and Budget Ministry Building in Paris

Bernard LAVERGNE
Ingénieur
Société Dumez
Nanterre, France

B. Lavergne né en 1940 est diplomé de l'Ecole Centrale des Arts et Manufactures en 1963. De 1985 à 1988 il dirigeait et coordonnait les études d'exécution des structures du Nouveau Ministère de l'Economie et des finances, projet d'une grande complexité. Actuellement il est directeur département Bâtiment à la Direction de l'Ingénierie Dumez.

RÉSUMÉ

Cet article décrit la conception d'ensemble et les particularités de cet immeuble-pont de 57,6 m de portée dont la structure principale est en charpente métallique. Les difficultés lors de la conception et de l'exécution de cet ouvrage ont été d'intégrer cette structure dans un bâtiment avec ses contraintes d'encombrement et d'aspect architectural. Pour permettre les variations dimensionnelles en service et dans le cas d'un incendie, et assurer par ailleurs la retenue des piles en tête qui doivent résister à un acte de malveillance, le dispositif mise en oeuvre est fonctionnel et original.

ZUSAMMENFASSUNG

Der Beitrag beschreibt den Gesamtentwurf und Details eines brückenartigen Hauses mit 57,6 m Spannweite in Stahlskelettbauweise. Die Schwierigkeit bestand darin, das Tragwerk in ein Gebäude mit seinen Raumbedürfnissen und architektonischen Gesichtspunkten zu integrieren. Um Längenveränderungen im Normalbetrieb und im Brandfall zu ermöglichen und zugleich die Stützen gegen Sabotageakte abzusichern, wurde zu einer funktionellen und sehr originellen Lösung gegriffen.

SUMMARY

This article describes the general design and particularities of this bridge-building with a span length of 57,6 m and a steel frame skeleton. The difficulty has been to include the structure in a building with its spatial and architectural requirements. In order to permit dimensional changes in service and in case of fire, and also to secure the supporting piers in case of a malicious act, a most functional and original solution has been implemented.

1. LE PROJET MINISTERE DES FINANCES

1.1 Description générale

Cet ensemble immobilier de 180 000 m² hors oeuvre est composé de 4 immeubles sur 3 niveaux de sous sols.L'immeuble principal, le batiment A constitue vu de l'extérieur l'élément fort du projet architectural. Cet immeuble barre de 38 m de haut au dessus de la voirie s'impose comme une muraille, ou plutôt un viaduc de 375 m de long au portes de Paris, lancé entre les berges de la Seine et les voies SNCF de la gare de Lyon. Entre les piles revêtues de pierre et de béton clair poli comme la superstructure, les façades en retrait et en verre foncé s'estompent.

Les voiries pénétrantes : la voie sur berge coté Seine et la rue de Bercy coté SNCF sont enjambées par deux **immeubles-pont de 57,60 m de portée** à chaque extrémité de ce viaduc. C'est le franchisssement coté Seine qui fait l'objet de la description qui suit.

Fig 1: Vue d'avion du BAT A

Fig 2 : Pile en Seine

1.2 Le franchissement coté Seine

Il relie les logements prévus pour les Ministres dans la pile en Seine aux bureaux des Ministres situés entre la 2ème et la 3ème pile.

A l'étage supérieur, un plateau de grande surface sans point porteur intermédiaire est destiné aux réceptions, avec salon d'honneur et salles à manger modulables par cloisons mobiles. Ces locaux prestigieux ont une surface de 16,13 x 57,00 m² et 4,80 m de hauteur sous plafond, et sont entièrement vitrés sur les deux façades.

2. STRUCTURE PRINCIPALE

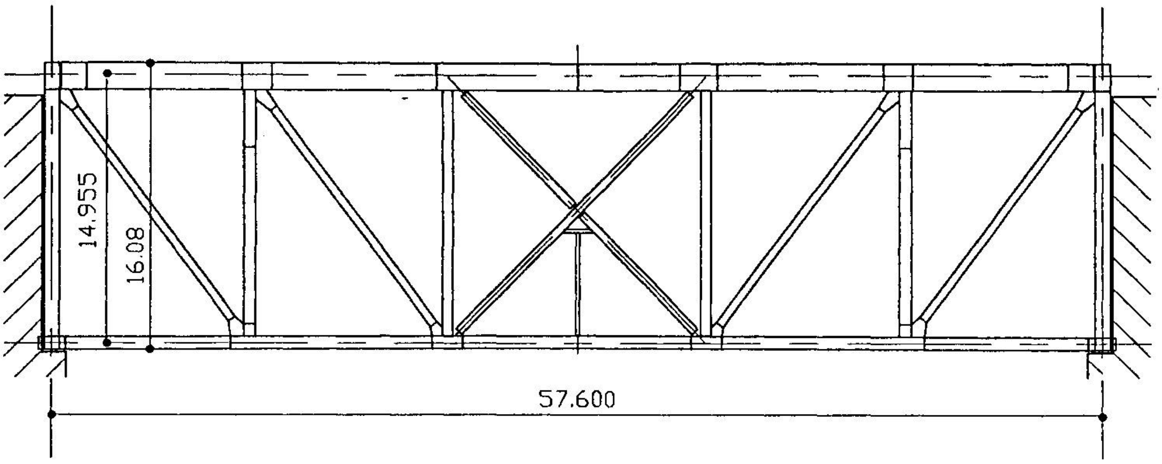
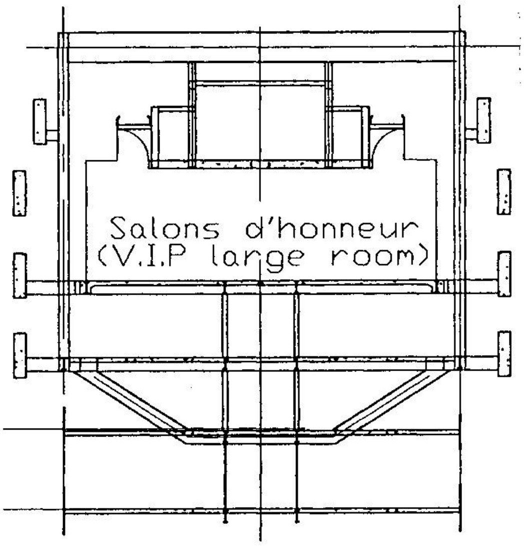


Fig 3: Elévation des poutrestreillis latérales



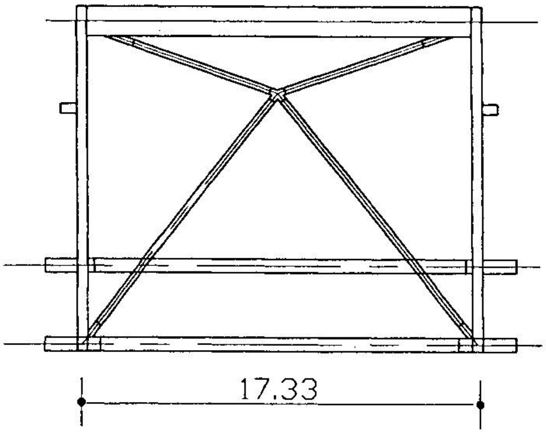


Fig 5 : Palée de contreventement aux extrémités

Fig 4 : Coupe Transversale

L'ossature en charpente métallique est constituée de deux grandes poutres en treillis de 16,00 m de hauteur pour 57,60 m de portée. Elles sont implantées à l'extérieur des façades. Le contreventement transversal est assuré par les 2 palées en croix de St André aux extrémités, un treillis horizontal au niveau des membrures supérieures et un plancher en béton armé au niveau des membrures inférieures formant poutres au vent.

Des diaphragmes transversaux suspendus en partie inférieure supportent les poteaux centraux des planchers, ainsi que les planchers inférieurs suspendus. La couverture et les locaux techniques sont suspendus au treillis horizontal supérieur.

Les façades minérales (pierre et béton clair poli) sont sur consoles, à l'extérieur des grandes poutres

3. INTEGRATION ARCHITECTURALE DE LA CHARPENTE METALLIQUE

Les membrures extérieures de la charpente sont entièrement capotées. Pour obtenir un aspect fini très pur, aucune forme de gousset n'était admise au droit des noeuds.

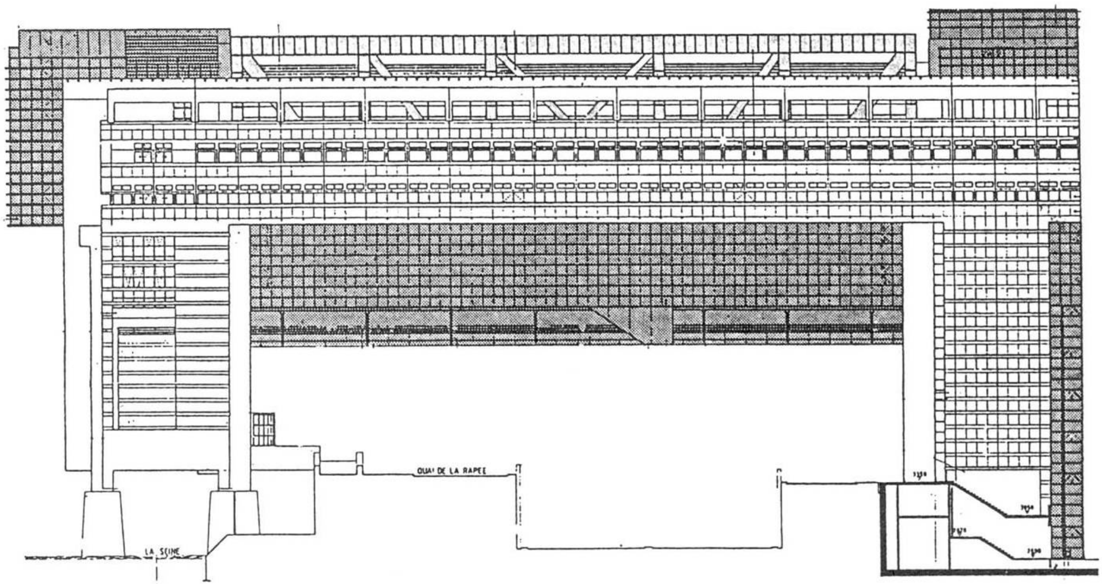


Fig 6 : Façade

La transmission des efforts ne pouvait donc être assurée au droit des noeuds qu'en épaississsant la section des ames. C'est ainsi qu'il fallu assembler des épaisseurs de 70 à 125 mm par soudures double face interpénétrées. Les noeuds ont été réalisés en atelier, les assemblages sur chantier étant reportés dans les sections droites.

Dans les étages pour des impératifs de gabarit sous plafond : 2,70 m, la hauteur des poutres était limitée à 0,75 m plancher compris. Il ne fut pas possible notamment pour les membrures de l'ossature principale de poser les planchers sur les poutres , 0,70 m de hauteur s'avérant être le minimum nécessaire. IL fallu donc liaisonner les planchers sur le coté des poutres avec cornières support et tiges filetées traversantes sur les ames.

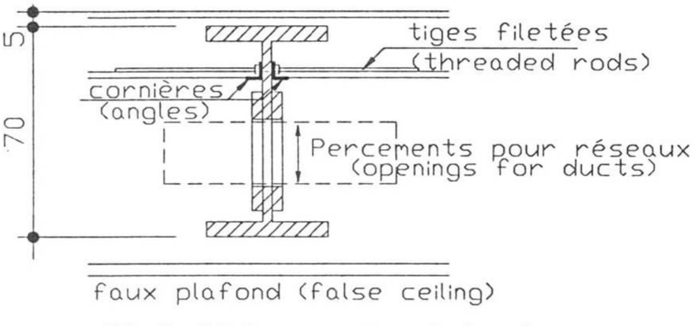


Fig 7 : Liaison poutre et plancher

4. MONTAGE DE LA CHARPENTE

Pour la sécurité, l'ouvrage franchissant les voies sur berges du quai de la Rapée soumises à un trafic intense, un platelage de protection sous l'ensemble de l'ouvrage a préalablement été établli. La charpente a été assemblée sur une plateforme construite sur la pile en Seine, puis poussée au fur et à mesure de l'avancement sur poutres de transfert. Une fois achévée, elle été posée sur ses appuis par vérinage.

Les planchers en béton ont pu être commodémment réalisés sur étaiement reposant sur le platelage.

5. PARTICULARITES DE L'OUVRAGE POUR REPONDRE AU CAHIER DES CHARGES

5.1 Déformations

Pour obtenir la planéité des planchers sous charges permanentes, une contreflèche de 85 mm était prévue au montage de la charpente. Le calcul de la flèche avait été fait selon deux hyppothèses enveloppes :
- charpente seule
- charpente + planchers béton collaborant en prennant en compte la fissuration.

La flèche mesurée après achèvement de l'ouvrage s'est révélée très proche de celle calculée dans l'hyppothèse de la collaboration du béton.

5.2 Variations dimmensionnelles en service et dans le cas d'un incendie

En service pour des variations de rempérature de ± 20° C, la variation de longueur est de ± 12 mm. Au niveau des appuis avec l'action variable des surcharges, l'allongement cumulé peut atteindre 20 mm.

Dans le cas d'un incendie (stabilité au feu de 2H) il a été pris en compte un échauffement moyen de la charpente de 350°C. Pour réduire l'allongement, le franchissement a été recoupé en 2 compartiments par une cloison coupe-feu à mi portée, ce qui a permis de ne considérer l'échauffement que sur la moitié de la longueur. L'allongement ainsi calculé est de l'ordre de 100 mm.C'est cette valeur qui a dimmensionné la largeur des joints de dilatation à chaque extrémité à 50 mm.

Pour éviter une translation de la charpente sous les effets des variations alternées de la température et des surcharges, les déplacements en service ont été limités à ±10 mm au droit de chacun des 4 appuis par un ancrage A (fig 9). Ces ancrages comportent une chape à trou ovalisé pour un jeu de ± 11 mm. Le trou ovalisé a été recentré après mise en oeuvre de la totalité des charges permanentes.

Les axes de couplage ont une résistance calibrée au cisaillement de 900 KN. Ils constituent les fusibles pouvant être rompus lors d'un incendie, limitant ainsi la poussée sur les piles à une valeur prédéterminée prise en compte comme action accidentelle pour le calcul des piles.

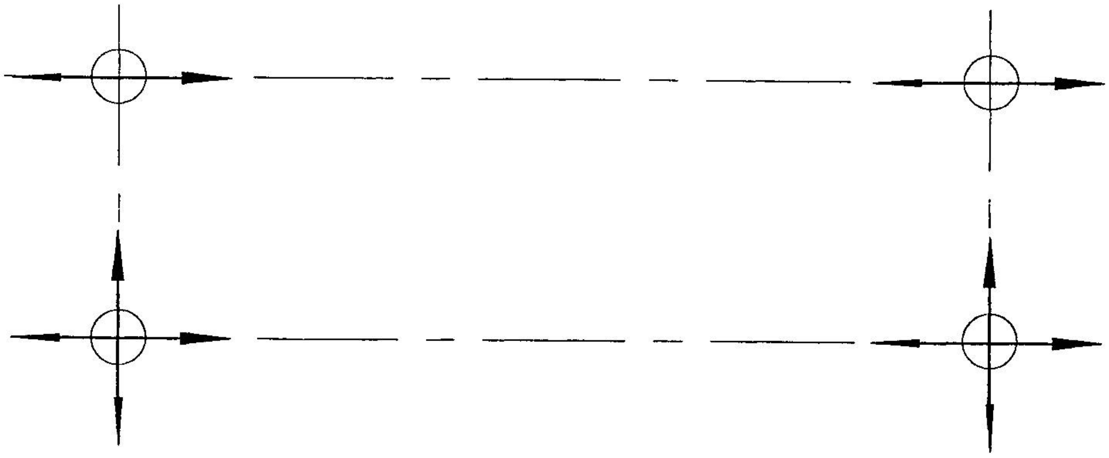


Fig 8 :Vue en plan des appuis glissants

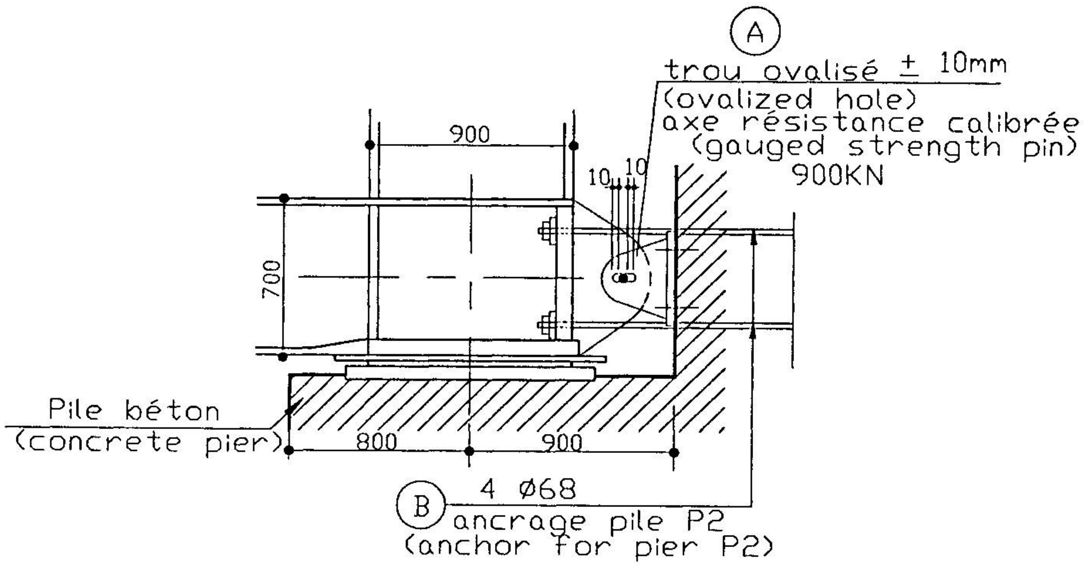


Fig 9 : appareil d'appui et ancrage

5.3 Protection contre les actes de malveillance

Conformément au cahier des charges, les poteaux supports des franchissements, accessibles soit par la route soit par bateau (pile en Seine), ont été calculés sous l'action du souffle de l'explosion d'une charge de 100 Kg de TNT à 2,00 mètres. Si la pile en Seine avec ses gros piliers s'est révélée être autostable dans cette vérification, ce ne fut pas le cas de la pile coté terre recoupée dans son axe par un joint de dilatation. Sa stabilité ne pouvait être vérifiée que dans l'hypothèse d'une butée en tête au niveau des appuis du franchissement. Il fallait donc mobiliser la masse du franchissement pour absorber la réaction horizontale (force vive) de 6000 à 7000 KN. On compléta le dispositif par les ancrages unidirectionnels B(Fig 9).

Le Nouveau Ministère des Finances avec son architecture imposante s'inscrit dans le vaste projet de réaménagement des quartiers Est de Paris. Le transfert en 1988 des bueaux du Ministère qui occupaient antérieurement les bâtiments du Louvre, a permis l'extension et modernisation du Musée du Louvre, autre ouvrage de prestige réalisé par DUMEZ.

Design of a Long-Span Multistory Building above a Railway Station

Projet d'un immeuble élevé enjambant une gare ferroviaire

Entwurf eines weitgespannten Hochhauses über Bahnhofsgleisen

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SUMMARY

The paper reports on the design for a 12-story, long-span building over the tracks of a railway station in the centre of Tokyo. The cores bearing the building loads on either side of the tracks will be supported by diaphragm-wall foundations, and the span between them of approximately 55 meters will support a structure with no footing beams. For the main framework trusses, arches, stay cables, and polygonal suspension plates were considered. Static and dynamic seismic designs were tested The results of the investigation verified that all of these construction methods satisfied criteria relating to factors such as deformation.

RÉSUMÉ

Les auteurs présentent le projet d'un immeuble de 12 étages enjambant une gare au centre de Tokyo. Les deux noyaux du bâtiment s'appuient de part et d'autre de la voie ferrée sur des fondations en caisson, tandis que la construction intermédiaire suspendue franchit la voie d'une seule portée de 55 m. L'article présente l'analyse structurale des charpentes en treillis, des arcs porteurs, des haubans et des structures polygonales suspendues, du point de vue de leur comportement statique et sismique. Tous les systèmes porteurs satisfont aux exigences imposées, entre autres les déformations maximales admissibles.

ZUSAMMENFASSUNG

Es wird von Studien für ein zwölfgeschossiges Gebäude berichtet, das im Zentrum Tokios einen Bahnhof überspannen würde. Die Gebäudekerne werden beidseits der Gleise auf Tragwänden gegründet, währenddem die zwischen ihnen eingehängte Konstruktion mit ca. 55 m Spannweite keine Zwischenstützen aufweist. Für die Tragkonstruktion wurden Fachwerke, Bögen, Schrägseile und polygonale Hängetragwerke auf ihr statisches und erdbebenresistentes Verhalten untersucht. Alle Tragsysteme erfüllten die gestellten Anforderungen u.a. maximal zulässige Deformationen.

1. INTRODUCTION

There are few long-span, multistory buildings in Japan at present, because of the effects of earthquake motion. However, there is increasing desire to make use of the narrow vacant sites that are common beside the groups of tracks close to Tokyo's railway stations. We at the East Japan Railway Company, together with representative construction companies, have studied design methods for a long-span, multistory building which has two core frames on sites on either side of the tracks of a certain station, with no footing beams.

During our studies of this long-span, multistory building, we examined several types of main framework to support the part of the building that bridges the tracks and connects the core frames on either side. These main framework types are super truss, arch, stay cable, and polygonal suspension plates. Details of our investigations are given in this paper.

2. OUTLINE OF STRUCTURE

2.1 Basic Structure and Geology

The building will be approximately 60 meters tall (12 stories), 70 meters deep (across the tracks), and 75 meters long (parallel to the tracks) as could be imagined in Fig. 1. The length of the span over the tracks will be approximately 55 meters, and the width of each core frame will be approximately 7 meters (Fig. 2). The foundations will be a diaphragm wall under each core frame, in diluvial deposits.

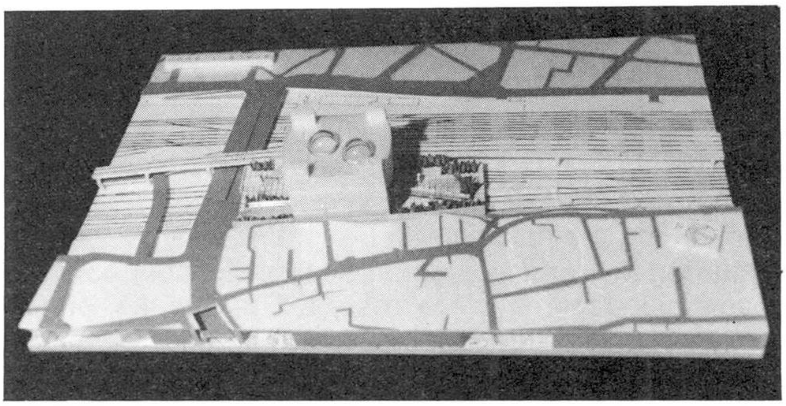


Fig. 1 Aerial photograph of proposed site

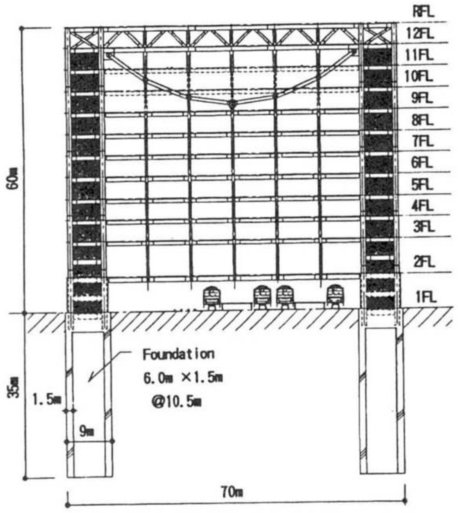
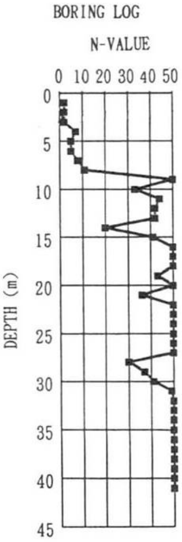


Fig. 2 Section through proposed building

The site that we surveyed is sandy to a depth of 3 meters from ground level, then consists of sand alternating with layers of clay and sandy gravel to a depth of 35 meters below ground level. Two proposals of a sandy layer down to 21 meters ground level or a sandy gravel layer down to 35 meters ground level were considered as the load-bearing subsoil of the foundations under vertical loading, but the biggest effect on horizontal displacement at ground level of the foundations during an earthquake is depth of setting of the foundations rather than changes in stiffness of the foundations, so the depth of foundations was taken to be 35 meters from the ground level.

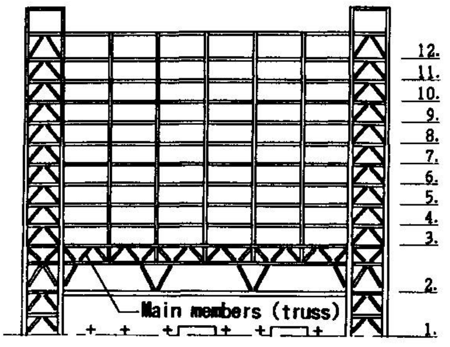


DEPTH (m)	SOIL TYPE	LAYER (m)	NUMBER OF BLOWS (BLOWS/30cm)	MODULUS OF DEFORMATION (MPa)
3.0	Sand	3.0	5	
7.5	Clay	4.5	8	0.306
15.0	Sand	7.5	40	0.275
18.0	Sand & Gravel	3.0	50	0.265
28.0	Sand	10.0	50	0.408
35.0	Clay	7.0	25	0.510
41.5	Sand & Gravel	6.5	50	

Table 1 Results of soil borings

2.2 Framework Characteristics

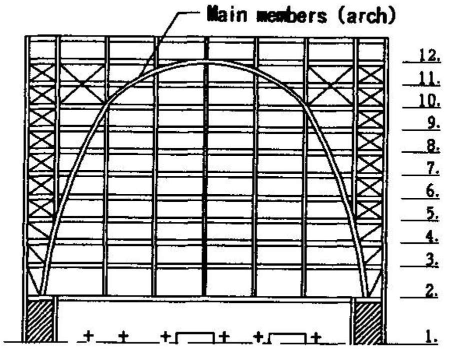
The characteristics of each type of framework that we investigated are described briefly below.



A. Truss framework

Making the first and second floors of super truss frames provides concentrated support for vertical loads. From the third floor upwards, the inner portions have a rigid-frame construction, with earthquake-resisting braces on the core frame portions on either side.

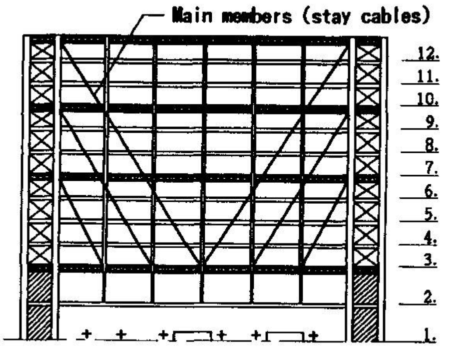
Main columns (1st floor):	Box 1200 × 1200 × 60 × 60 (SM490)
Intermediate columns:	BH 600 × 600 × 36 × 40 (SM490)
Main beams (2nd floor):	2BH 1000 × 500 × 40 × 40 (SM490)
Truss members:	2BH 600 × 500 × (25–32) × (32–60) (SM490)



B. Arch framework

An arch is characterized in bearing both vertical and horizontal loadings. Since arches have a large cross-section and are highly rigid, bracing is provided to resist reverse shear stresses, particularly in the upper floors.

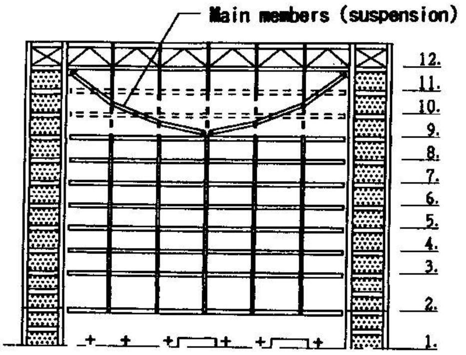
Main columns (1st floor):	B × D = 2000 × 8000 (RC)
Intermediate columns:	Box 900 × 700 × 40 × 40 (SM490B)
Main beams (2nd floor):	BH 1000 × 400 × 19 × (28–40) BH 1000 × 500 × 19 × 40 (SM490B)
Arch members:	I 1500 × 900 × 80 × 80 (SM570Q)



C. Stay-cable framework

This structure is characterized in that the part of the multistory building above the tracks is suspended on cables from the core frames at either side. Beams bear the compressive forces, and the cables together with the core frames acts as effective aseismic elements.

Main columns (1st floor):	B × D = 1500 × 1500 Box 900 × 900 × 65 × 65 (SM490A)
Intermediate columns:	Box 600 × 600 × (16–40) (SM490A)
Main beams (3rd floor):	2BH 800 × 400 × 22 × 36 (SM490A)
Cable members:	2SPWC-367, 283, 301



D. Suspension framework

The suspended members bear only vertical loads in the part of the building above the tracks—they are not intended to bear horizontal loading. Therefore, where the suspension members intersect the beams and columns in the part of the building above the tracks, those beams and columns are paired to allow the suspension members to move freely.

Main columns (1st floor):	Box 1200 × 1000 × 80 × 80 (SM490A)
Intermediate columns:	Box 550 × 550 × 22 – 28 (SM490A) (Floors 2 to 8, 11) 2-Box 250 × 250 × 25 (SM490A) (Floors 9 and 10)
Main beams (2nd floor):	BH 900 × 300 × 19 × 25 (SM490A)
Suspension members:	4PL-750 × 100 (HT80)

Fig. 3 Framework characteristics

3. ANALYSIS PROCEDURE AND CONDITIONS

The flow of the design procedure we followed is shown in Fig. 4.

3.1 Static Analysis

We subjected a two-dimensional frame model of the upper structure coupled with the diaphragm-wall foundations to linear stress analysis, applying vertical loading and static earthquake loading determined from results of preliminary response analysis, and investigated the effects of the sizes of members. We determined subsoil reaction coefficients of the diaphragm-wall foundations by comparing the results of several horizontal loading tests on this type of foundation and results obtained by independent finite element analysis.

3.2 Dynamic Analysis

As the initial step, we substituted the static analysis model into an equivalent spring-mass model for a linear response analysis, and determined that the model satisfied the criteria we had set. Two sets of earthquake wave data were input, El Centro 1940 (N–S) and Taft 1952 (E–W), with the maximum velocity being set to 25 cm/s (level 1).

In the next step, we introduced nonlinear characteristics into the members of the static analysis model and performed an incremental lateral loading analysis at two to three times the static earthquake forces. The resultant restoring force characteristics were obtained for each floor, we then performed nonlinear response analysis with the spring-mass model, and we verified that the design criteria were satisfied. In this case, the maximum velocity of the input earthquake waves was 50 cm/s (level 2).

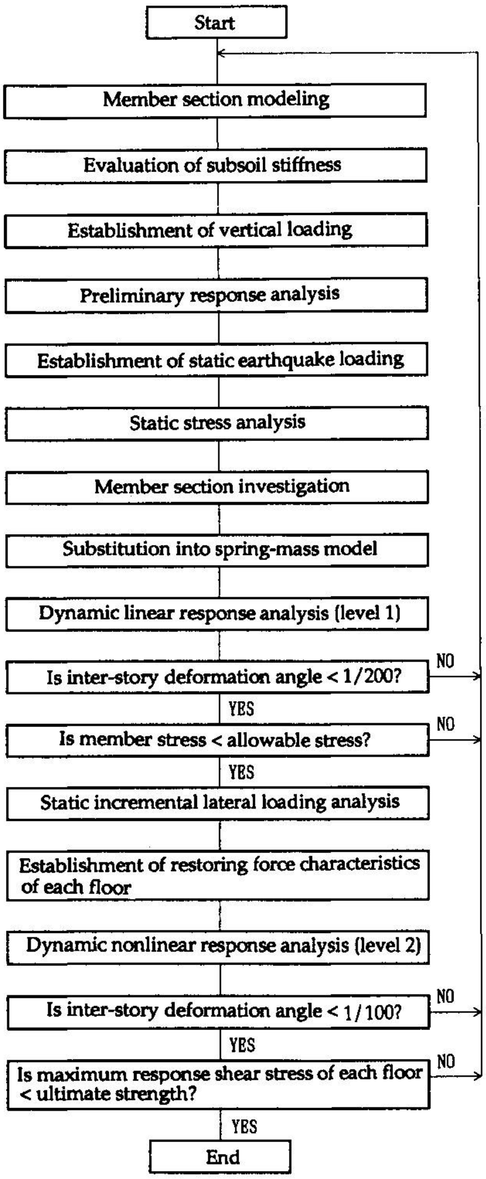


Fig. 4 Design flow

4. RESULTS

The results of the analysis are discussed below.

4.1 Static Analysis (Vertical and Horizontal Loading)

Vertical displacements of the beams under vertical loading at the center of the second floor above the tracks are listed in Table 2.

When the arch and suspension structures were subjected to vertical loading, the members subjected to axial forces resisted the loading, so these displacements were less than with the truss method in which the bending members provide resistance. These values were approximately half or less of the truss method. These values were also less than about 1/800 of the span across the tracks, so would cause no problems. Increasing the stiffness of the structural members is one way of making these displacements smaller that is common to all these methods. For each of these methods, it will be necessary to investigate the handling of displacements further, considering details such as the setting of beam camber during construction and the building sequence.

(cm)

	Truss	Arch	Stay Cable	Suspen-tion
Vertical Displacement	16.7	7.9	*6.5	6.8

* Live Load Only

Table 2 Magnitudes of vertical displacements (2nd floor)

Partial ratios of story shear forces during an earthquake are listed in Table 3. With the truss method, 60% to 70% of the earthquake force is taken by the core portions on ordinary floors. With the arch or stay cable method, their main members pass through even the ordinary floors, so the arch or stay members bear a large proportion of the story shear forces. In the arch method in particular, since the arch members in the upper stories are close to the horizontal, they can bear large shear forces of over 100% so that reverse shear forces can occur in the other parts of the framework. On the other hand, with the suspension method, the core portions and intermediate beams are pin-jointed, so that the suspension members have joints that do not bear any story shear forces, and thus the core portions bear virtually all of the story shear forces.

Static horizontal displacements of buildings of each method under earthquake loading are shown in Fig. 5. We performed analysis on the upper structure coupled with the diaphragm-wall foundation to 35 meters below ground level that all of these construction methods have in common, as described above. Movement of the diaphragm-wall foundation shows a tendency toward roughly rigid-body rotation that is common to all methods. Horizontal displacement of the tops of the foundations was 2.1 to 2.9 cm, and this value was verified to be sensitive to the subsoil reaction coefficient. The lack of footing beams connecting the tops of the foundations has a huge effect on the upper structure and the building's natural period. Since the sizes of most of the members are determined by the stresses they experience during vertical loading, in effect the upper framework becomes extremely strong, and thus the inter-story deformation angle can sufficiently satisfy the condition of no more than 1/200 radians. In the upper framework deformation mode, the characteristics of each method vary with differences in the main member arrangement and the story stiffness distribution.

Story	Truss			Arch			Stay Cable			Suspension		
	A	B	C	A	B	C	A	B	C	A	B	C
12.	30	70	0	25	75	0	-33	28	105	98	2	0
10.	57	43	0	-35	-27	162	25	19	56	75	25	0
8.	65	35	0	-23	20	103	21	19	60	83	17	0
6.	72	28	0	9	32	59	22	15	63	86	14	0
4.	79	21	0	20	39	41	29	13	58	87	13	0
2.	-18	0	118	38	27	35	99	1	0	95	5	0
1.	100	0	0	100	0	0	100	0	0	100	0	0

(*) A: Core frame, B: Intermediate column, C: Main member

(Unit:%)

Table 3 Story shear force partial ratios

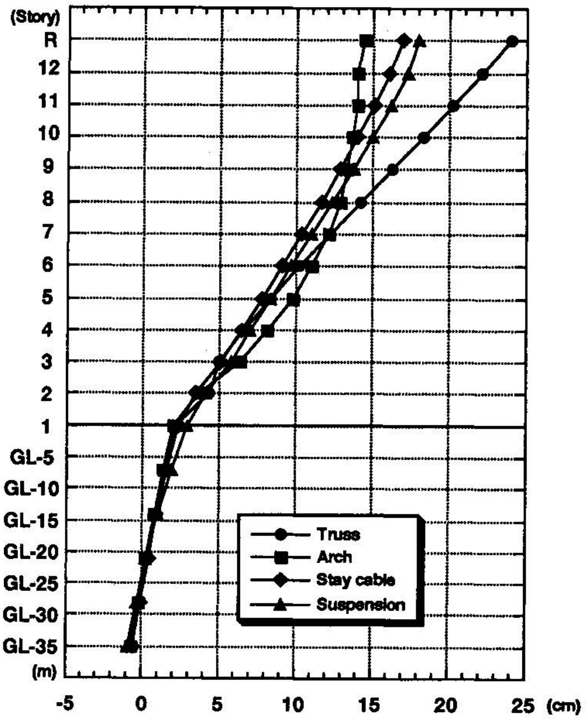


Fig. 5 Static horizontal displacements

4.2 Dynamic Analysis (Levels 1 and 2)

The primary natural periods for each structural method were within the range of 1.4 to 1.5 seconds, as shown in Table 4. In comparison with an ordinary building in the 60-meter-high class, this primary natural period is fairly long, because there are no footing beams. Looking closely at each natural period, it is clear that the arch and stay-cable methods, which impart horizontal stiffness to their members, have a slightly shorter period than the other two methods.

The dynamic analysis was done on a multiple mass model using the equivalent shear springs obtained as a result of static elasto-plastic analysis. In the first-floor columns, subsoil sway springs were considered.

Displacements of the tops of the diaphragm-wall foundations of each of the methods during response to 50 cm/s Taft (E-W) waves are shown in Table 5. There was scattering between the different methods, but displacements were within the range of 3 to 6 cm.

Truss	Arch	Stay Cable	Suspension
1.51	1.44	1.42	1.48

(Unit:sec)

Table 4 Primary natural periods

Distributions of inter-story deformation angle of each of the methods during response to 50 cm/s Taft (E-W) waves are shown in Fig. 6. It was verified that the design criterion of 0.01 radians was satisfied by each structural method. The suspension method tends toward an even distribution with height because the main members have joints that do not contribute to the horizontal stiffness. The other three structural methods exhibit the characteristics that are specific to those methods. For example, a singular point can be seen at the position of the second story in the truss method's case or the third story in the stay-cable method's case, in other words, at the position of that method's main members. With the arch method, a tendency toward decreasing deformation angle can be seen from the third floor upwards, as the horizontal stiffness of the arch increases.

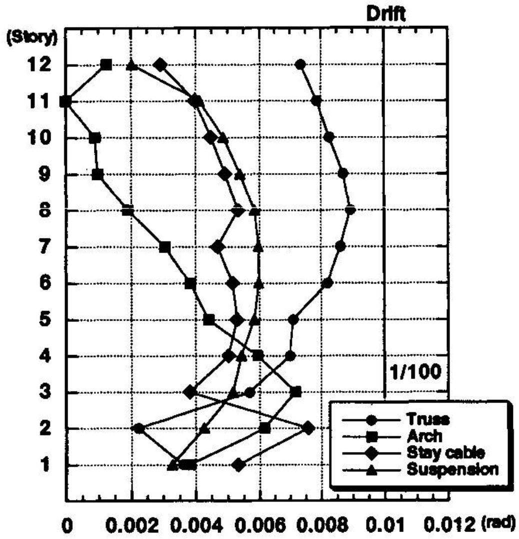


Fig. 6 Inter-story deformation angles

The distribution of story shear coefficients during response to 50 cm/s Taft (E–W) waves is shown in Fig. 7. With the truss method, the shear coefficient rises with floor in the upper framework, but this is due to the way in which the upper stories above the third floor become a rigid-frame structure.

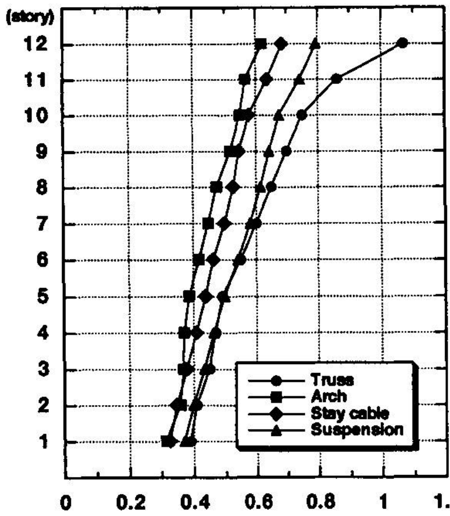


Fig. 7 Seismic story shear coefficients

5. CONCLUSION

Aseismic design in an earthquake-prone country such as Japan necessitates a fair amount of compromise and decision-making in the proposal of a structural framework that suits a certain design concept, when the design takes into consideration factors such as safety, economics, and ease of construction.

This paper has presented the results of our investigations into different structural forms that are aimed at creating a large-span building designed to make effective use of the vacant areas alongside railway tracks. It also clarified that there are some differences in efficiency and functionality of different structural methods, but that they can be implemented.

Truss	Arch	Stay Cable	Suspension
3,96	3,09	3,76	5,50

(Unit:cm)

Table 5 Displacement of diaphragm-wall foundation tops

Such a building would require rather more steelwork than an ordinary rigid-frame structure, but we have determined that it is possible to build a multistory structure in the space over the tracks in order to create a long-span building that does not impede the functions of the railway.

We intend to intensify our investigations in the future, to implement choices and decisions for an even better structural format.

6. ACKNOWLEDGMENTS

The contents of this paper were put together by the "Long-Span Building Design Implementation Investigative Committee" and the author would like to express his sincerest thanks to everyone involved, from committee chairman Jiro TAJIMA and vice-chairman Shigemi MACHIDA to the designers who participated from the construction companies Obayashi, Kajima, Shimizu, Taisei and Tekken ,and JRC.

The Creation of the Roof to the Waterloo International Terminal

Conception de la toiture de la gare internationale de Waterloo

Projektierung des Daches des Internationalen Waterloo-Bahnhofs

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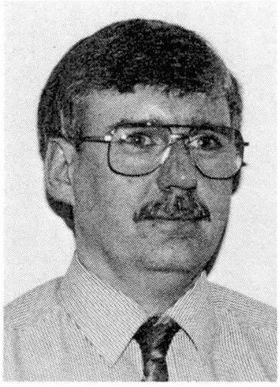

Alan C. Jones, born 1955, received his Civil Engineering degree from Portsmouth Polytechnic, England. He has worked for Anthony Hunt since graduation and has been responsible for the structural design of many notable steel buildings including the Inmos Factory Gwent and the Schlumberger Research Centre Cambridge.

SUMMARY

The train shed roof at Waterloo is a lightweight steel frame spanning up to 48 m and clad in stainless steel with large glazed areas including the whole of the west elevation. The shed is approximately 400 m long and is curved and tapered on plan. This paper describes the design criteria for the roof structure and outlines the analysis, fabrication and erection methods employed in its creation.

RÉSUMÉ

La toiture en sheds de la gare ferroviaire de Waterloo est une charpente métallique légère d'une portée de 48 mètres, revêtue d'un habillage en acier inoxydable et pourvue de grandes surfaces vitrées, y compris la totalité de la façade ouest. La couverture en sheds a une longueur d'environ 400 mètres, est de forme courbe en plan et de largeur variable. L'auteur expose les critères de l'étude structurale de la toiture et fournit les points essentiels des calculs statiques, de la fabrication et des méthodes de montage prévues pour cette réalisation.

ZUSAMMENFASSUNG

Das Shed-Dach über den Zügen im Waterloo-Bahnhof ist ein leichter Stahlrahmen mit bis zu 48 m Spannweite. Die Verkleidung besteht aus Edelstahl mit grossflächiger Verglasung, die auch die Westfront umfasst. Das Shed ist ungefähr 400 m lang, im Grundriss gekrümmt und von veränderlicher Breite. Der Beitrag beschreibt die Bemessungskriterien, die Berechnung, das Fertigungs- und Montageverfahren, die beim Bau verwendet wurden.

1. INTRODUCTION AND BRIEF

1.1 Introduction

This paper sets out to describe the background to the design and construction of the new train shed for Waterloo International, London's gateway to Europe via the Channel Tunnel and the first clear span terminus to be built in the capital since St Pancras in 1868. Trains will run from Waterloo to Paris and Brussels. Each train is 400m long, carries 800 passengers and will travel non-stop from London to Paris in three hours.

Peak passenger flow is 6000 per hour with an anticipated maximum of 15 million passengers per year. There are five dedicated platforms, four for general use and one as a 'stand-by'.

The train shed is, of course, only the tip of the iceberg with, below it, three layers housing departures, arrivals, immigration, car parking and servicing, all designed to airport rather than railway station standards.

This paper deals only with the train shed roof and supports for which we were responsible as consulting engineers.

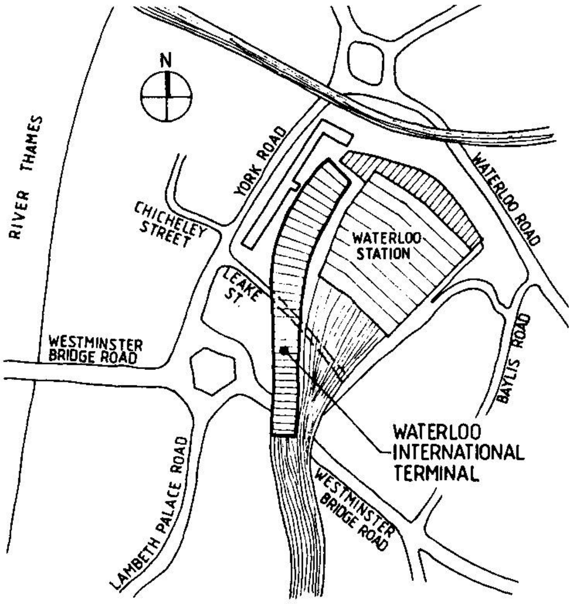

FIG 1. LOCATION PLAN.

1.2 The brief from British Rail

In March 1988 YRM Anthony Hunt Associates were commissioned by British Rail to work with architects Nicholas Grimshaw & Partners on the train shed structure for the new channel tunnel terminal at Waterloo. Sir Alexander Gibb & Partners, who were already acting as transport consultants, were appointed for all the substructure works and Cass Hayward as consultants for the track structure.

The final brief from British Rail was simple in outline although of course quite complicated in detail:

- a clear span structure of approximately 400 metres length unheated and uninsulated. Due to the constricted nature of the site which dictated rather narrow platforms, intermediate columns were considered to be undesirable
- level separation of departures/arrivals with customs hall/immigration areas.
- basement car parking
- alongside vehicle access
- a new link to London Underground
- new high level pedestrian walkways.

Waterloo International is a transport building for the late 20th century linking Britain to Europe. It is a dramatic combination of engineering and architecture using current technology to follow the traditions of the great 19th century British engineers, Brunel at Paddington and Barlow at St Pancras.

2. DESIGN CRITERIA

2.1 Environmental loads

Snow loads were evaluated taking into account the possibility of drifting in the valleys. Uniformly distributed maintenance loads were considered to be within the allowances made for snow but specific point loads were determined at various locations to cater for personal access equipment including a safety harness system on the roof and support for a temporary maintenance cradle below the primary trusses.

Wind loads were assessed using a scale model in the wind tunnel at the University of Bristol and the tests incorporated various stages of development of the surrounding area, including the proposals to redevelop York Road and Elizabeth House.

The structure is in an external location and is therefore exposed to extremes of temperature. The structure was divided into seven independent units each in the region of 50m long with free expansion joints located in the valley sections.

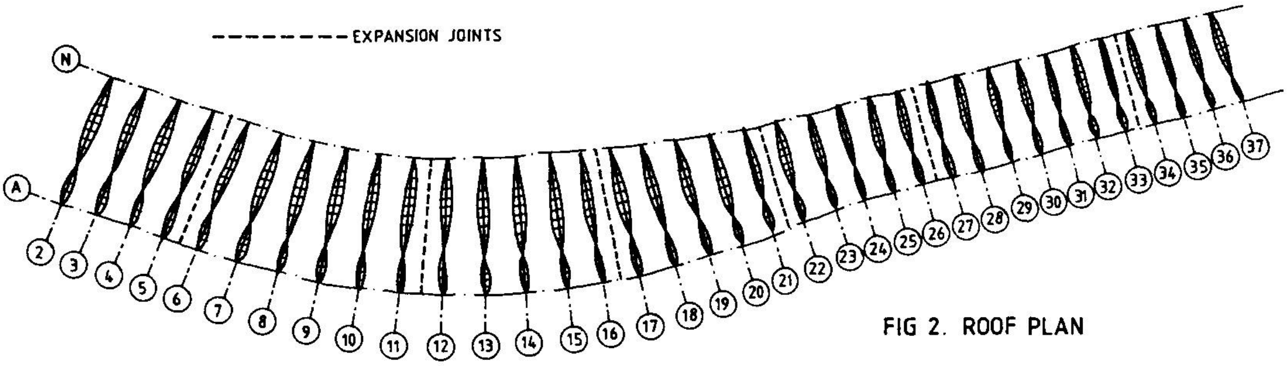

FIG 2. ROOF PLAN

2.2 Foundation Movements

Over its length, the roof structure is supported on many different foundation systems ranging from the new raft foundation system for the main terminal building to the well established mass concrete foundations of the existing brick arch structures of the old station. Consequently, large differential vertical movements occur at several locations. Wherever possible, thermal expansion joints have been located over areas of maximum predicted vertical movements so that both movements can be accommodated at the same time. The roof is also subjected to short term vertical movements created by the passing of trains over the track support structure. This is a more localised effect and is absorbed by the inherent flexibility of the steel frame and cladding.

2.3 Hazard Loads

The movement of trains in close proximity to structure required careful consideration and this was a prime concern at Waterloo. The roof was designed for various extreme loads deemed to simulate an impact from an incoming train. This included the total removal of a single truss support.

3. GEOMETRY

3.1 Plan layout

The plan shape of the building is dictated by the available land around Waterloo. The train shed, which covers the five new platforms and the associated tracks, is 400m long. The 36 roof trusses twist and curve on plan and reduce in width from 48.5m at the northern (town) end to 32.7m at the southern (country) end, to follow the site and limit of the platform widths as passenger flow reduces away from the entrance.

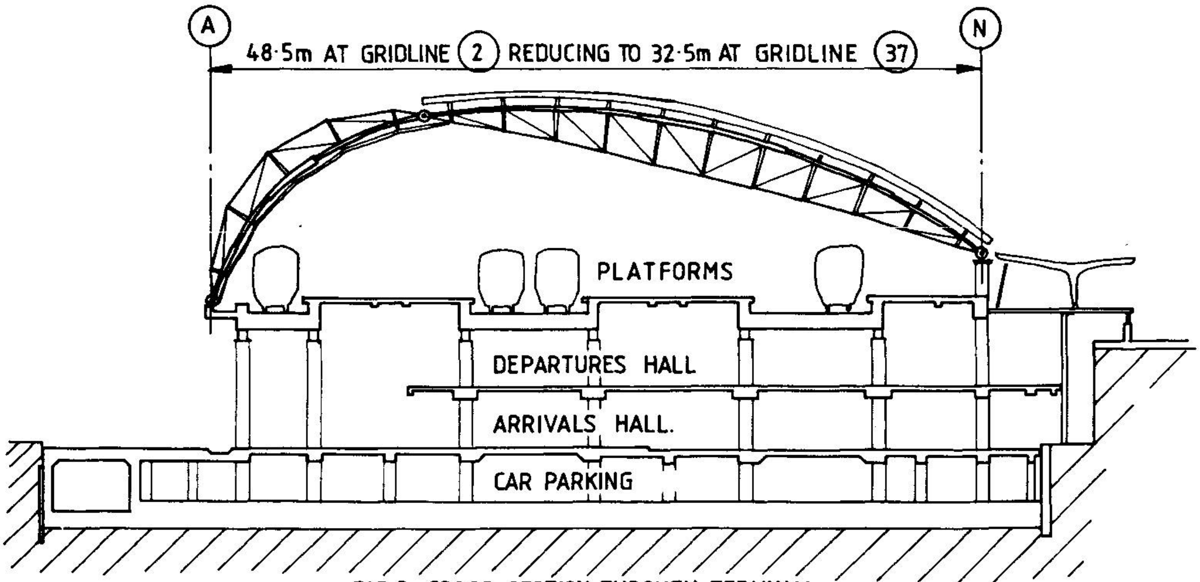

FIG 3. CROSS SECTION THROUGH TERMINAL

3.2 Section

The proximity of the original structure alongside the new roof, and the closeness of Elizabeth House and the access road to the west, set the external building envelope quite tightly. This, combined with an asymmetric platform arrangement, necessitated an external structure with sharply rising cladding above the first railway track. This in turn largely determined the asymmetric arch shape. As a result of this asymmetry, pairs of bowstring trusses are joined off-centre to form a flattened three-pinned arch. The smaller external element is referred to as the minor truss; the longer internal element as the major truss. The bowstring trusses are necessary to accommodate the bending moments created by uneven loading conditions and the building shape. The form has thus developed into a unique structural arrangement that reflects the bending moment diagram and arching forces in response to the natural laws of physics. The result is an efficient and therefore lightweight steel structure which makes good use of the underlying track support structure by utilizing it as a tie between supports.

4. STRUCTURE

4.1 Primary Trusses

The bowstring trusses are joined at the interface between the glazing and decking with a cast steel knuckle and stainless steel pin. A similar connection is provided at the bases, where they sit on the platform structure. The internal truss comprises two telescoping compression booms up to 356mm in diameter with a single 75 dia tension rod. The booms are interspaced with tapered tubes varying from 219mm dia down to 60 dia at the tie rod. Due to the asymmetric arch profile, the external truss is inverted, with a single compression boom 356mm maximum diameter and twin 75 dia outer tie rods.

4.2 Secondary Structure

Single circular hollow section tubes form the secondary structure between trusses. The valley shaped roof can experience considerable out of tolerance loading between bays causing the main prismatic trusses to twist. This is prevented by continuity of the secondary structure provided by tie rods taken from the truss booms to the centre of the longitudinal tubes. On the major trusses tie rods are located both above and below the tubes giving a movement capacity in either direction. However, on the minor trusses only external ties could be incorporated. In this case moments causing the ties to go into compression are resisted by moment connections between the tubes in adjacent bays. The three-dimensional shape of the structure also provides a degree of arch action which helps limit deflections due to imposed loads.

5. ANALYSIS

5.1 Stress and Deflection

The steel frame was analysed using a full three dimensional model consisting of a typical 5 bay roof section. This model was subjected to a comprehensive combination of loads (see section 2) and detailed information on stress levels and deflections was obtained. Stress was checked in accordance with BS 5950: Part 1: 1985 and the deflection results were incorporated into a performance specification which provided the basis of the design for the cladding system.

5.2 Aesthetics

The visual relationship between the structure and cladding and also between individual structural elements was given a high priority among the other design criteria. Computer aided draughting techniques were used to create solid three dimensional models of a typical bay of structure, together with the cladding and all primary connections. This enabled the structural elements to be blended together in the most acceptable way. For instance, the posts in the major truss which pass from the large diameter compression boom to the small diameter tension boom must effect this transition in the most elegant way. It was decided that the most suitable method was to taper the post. This was modelled on the CAD and then alternative sections were investigated, including tapered tubes, box sections and cruciform sections. Examination of the alternatives as they would be viewed from the platform showed the circular section to be the most acceptable.

6. FABRICATION

6.1 Repetition of Detail

Whilst the budget for the roof was set to reflect the prestigious appearance required by the client, careful control of the details was necessary to prevent the cost becoming prohibitive. The setting out of the trusses ensured as many details as possible repeated at each gridline - despite the varying spans. The largest truss was set out to suit its span and height and then each successive truss was reduced according to a scaling factor relating to the span. In this way the geometry at most of the connections remained constant for all trusses. This was important because the principal connections incorporated cast steel nodes which required a new pattern every time the geometry changed. The various truss configurations were rationalised into four different structural types with only two variations on the external diameter of the main boom members. Hence, only two different patterns were required for each main connection

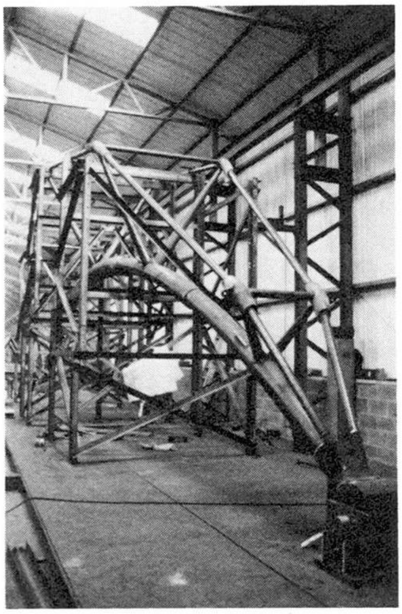

Photo 1. Minor Truss in the fabrication jig.

6.2 Assembly

The primary trusses were assembled in the workshops of Westbury Tubular Structures Ltd, using specially constructed jigs. Comprehensive checks were undertaken of all dimensions using electronic distance measuring equipment and a full regime of non-destructive testing was initiated.

The prismatic form of the trusses made transportation difficult due to the large volume occupied by each unit. It was possible to fabricate and transport the minor trusses as a single piece, however, the major trusses were split into three with site bolted splice connections between them. Each truss was trial assembled in the workshop and surveyed as a complete unit prior to shipping to the paint shop and on to site.

7. ERECTION

7.1 Pre Site Trial Erection

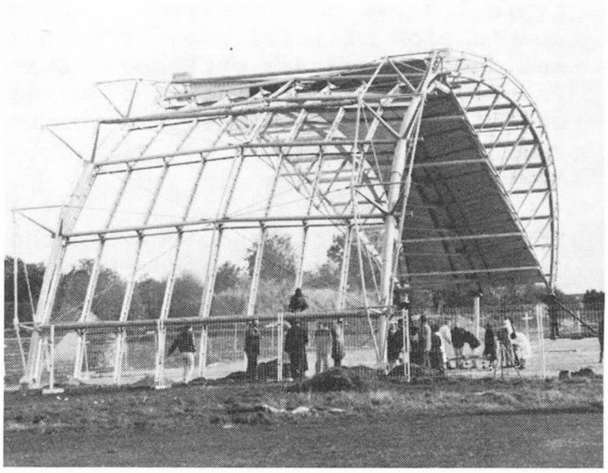
Photo 2. Trial Erection at Wetherby

At a very early stage it was decided in conjunction with the construction managers, Bovis Construction Ltd, to incorporate a full scale trial erection of a single bay into the contract and the area between grids 15 and 16 was selected as being representative of the average complexity of the whole structure. The purpose of the trial was to test the erection procedures within the trade contracts and hone on site activities to give as much overlap as possible. The steel fabricator erected a full bay of steelwork including two primary trusses and associated secondary elements, adjacent to their workshops. This was then clad by Briggs Amasco Ltd who provided the decking and glazing systems. Several minor, but very significant, modifications to the connection design and erection method statements arose from the experiences gained .

7.2 On Site

Erection on site commenced on two fronts, at the north end of the station and half-way down the shed by Leake Street. Each section began with one complete braced bay of steelwork and progressed southward one bay at a time.

The first operation was to position the two base castings onto the track support structure according to the actual dimensions of each truss obtained from the shop surveys. The major truss was then brought to site in three sections and assembled horizontally on cradles resting on the track structure. The main pin connection was made to the base on gridline N and the other end of the truss lifted up into position on the temporary trestles at the central pin. The trestles provided lateral restraint to the truss to prevent any horizontal lateral movement which may have overstressed the base connection. The minor truss was then lifted onto the base casting on gridline A and the pinned connection made. The final operation was to support the internal end of each truss with individual mobile cranes and lower them down together until the forked connection intermeshed and the centre pin could be installed. The temporary trestles continued to provide lateral restraint whilst the next truss was erected and the secondary structure inbetween was assembled.

8. COMPLETION

The roof structure and cladding was completed along with the remainder of the terminal in May 1993, on time and within budget and is due to receive trains from Paris and Brussels in the summer of 1994.

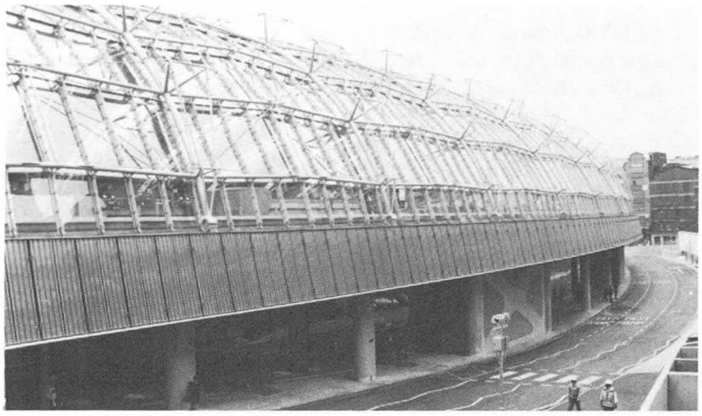
Photo 3. West Elevation of the complete roof

Denver International Airport Tensile Roof Case Study
Toiture en tension de l'aéroport international de Denver
Fallstudie der Abdachung des internationalen Flughafens Denver

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SUMMARY

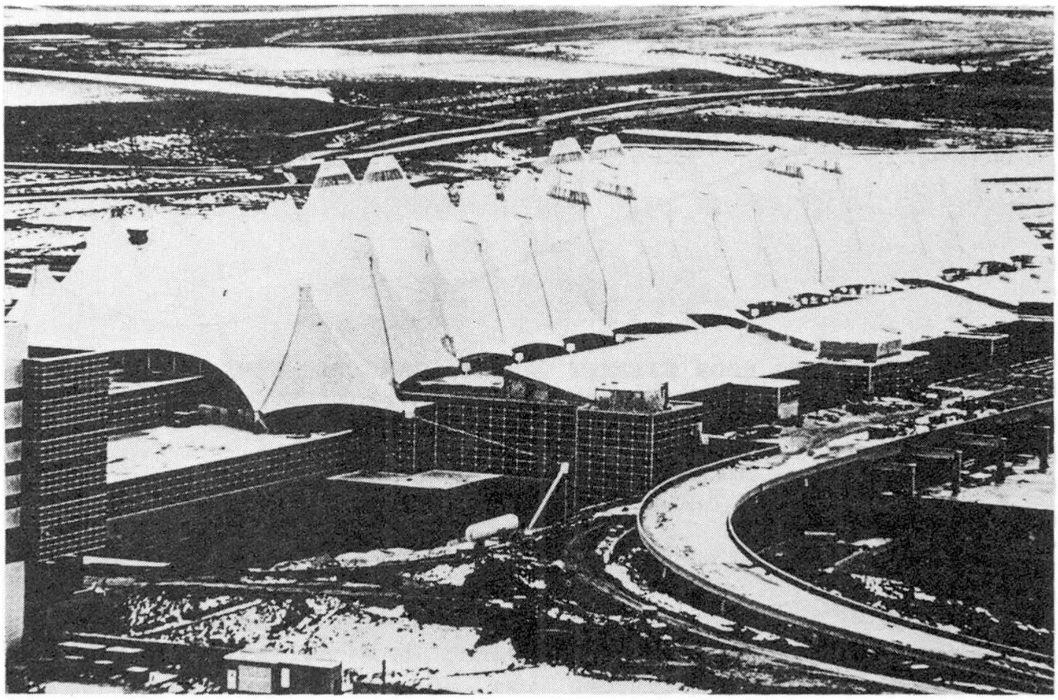
This paper presents the planning, fabrication and construction phases of the tensioned fabric roof system at the new Denver International Airport. It presents the physical modelling and computer modelling required, the fabrication, the installation of the system and some of the associated problems.

RÉSUMÉ

Ce rapport présente les phases de concept, fabrication et construction du système de la toiture en tension de l'aéroport international de Denver. Les modèles physique et informatique, la fabrication, l'installation du système ainsi que d'autres problèmes sont présentés.

ZUSAMMENFASSUNG

Der Bericht enthält die Projektierungs-, Fabrikations- und Errichtungsphasen der Abdachung des internationalen Flughafens Denver. Eingeschlossen sind die notwendigen physikalischen und Rechner unterstützten Modelle, die Herstellung und Installation des Systems sowie andere damit verbundene Probleme.



Aerial View of the Roof

Introduction

The tensile membrane fabric roof structure enclosing the Great Hall area of the New Denver International Airport is truly a milestone project for the tensile structure industry. It unites structural engineering with architecture to produce a magnificent and expansive interior space.

The fabric roof measures approximately 300 by 1000 feet in plan. It is supported by 34 masts of approximately 100 feet in length, has a surface area of about 380,000 square feet, and uses literally miles of structural steel cable. The dramatic peaks and valleys give it a unique shape emulating the Rocky Mountains that are synonymous with Denver and provide a striking backdrop to the new airport's western view.

This paper will provide a case study of the fabrication and construction phases of the project. The topics discussed in detail are the initial planning, the computer modeling, the fabrication, and the installation of the tensile roof system.

Initial Planning

One of the challenges that must be overcome to successfully construct a fabric roof of this magnitude, is determining a safe method to accomplish the installation, in particular, the fabric panels and rigging. Each bay of the structure is comprised of over 20,000 square feet of fabric. The risk of wind damage during fabric lifting is extremely high, if not performed properly. The roof is also vulnerable during the time period when the fabric is partially installed. During this period, the fabric has only partial pre-stress and therefore less inherent stability. It will be subjected to loading conditions that are completely different from the design conditions of the completed structure. To overcome these hazards, extensive planning and analysis requiring both physical and computer modeling techniques are used.

The first step of the installation planning was to construct a working physical model. The physical model is used to qualitatively study the installation and formulate a preliminary plan. In the case of the Great Hall roof, we constructed a 1/8" scale model of half of the structure. The model represented all the major structural components of the fabric roof system and the primary surroundings that would be present during construction.

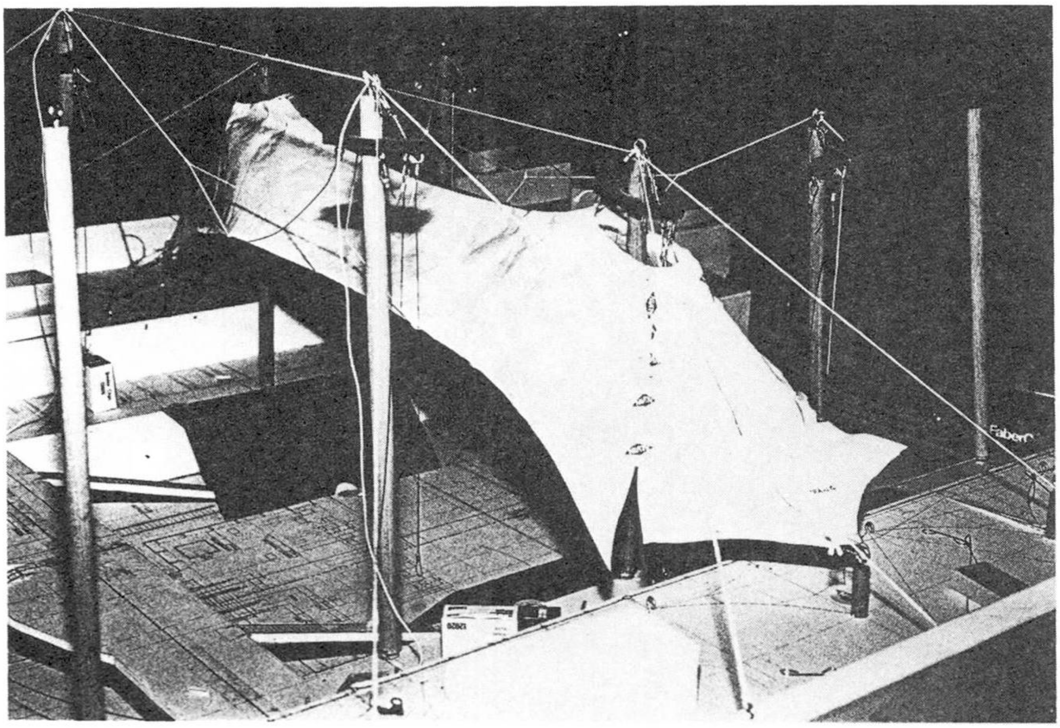


Figure 1: Physical Model. A working physical model is used to develop and test the installation procedure.

Working with scale replicas of the fabric assemblies, we tested out different methods and sequences of fabric packaging, handling, rigging, and hoisting. We worked with the physical model until we had schemes that we believed were physically possible to achieve and could be accomplished safely in the field. Later, the same physical model was sent to the field where it was used on site to help refine procedures and instruct the installation crews.

Computer Modeling

After the qualitative work was completed with the physical model, and a general plan had been established, computer models were built to perform the quantitative structural analysis. Large deflection finite element method analysis software is used for this work. The computer models are required for both the construction planning and the fabrication detailing. Three general types of models are required; overall system models, installation models, and fabric pattern models.

The overall system models are used to represent as much of the entire system as possible, in order to get an understanding of the overall behavior and structural interaction of the system as a whole. In the Great Hall fabric roof, the behavior and equilibrium of the various components are all inter-related. The system model is used to determine the geometrical configuration and pre-stress forces that will work in equilibrium together to produce the desired architectural and structural performance.

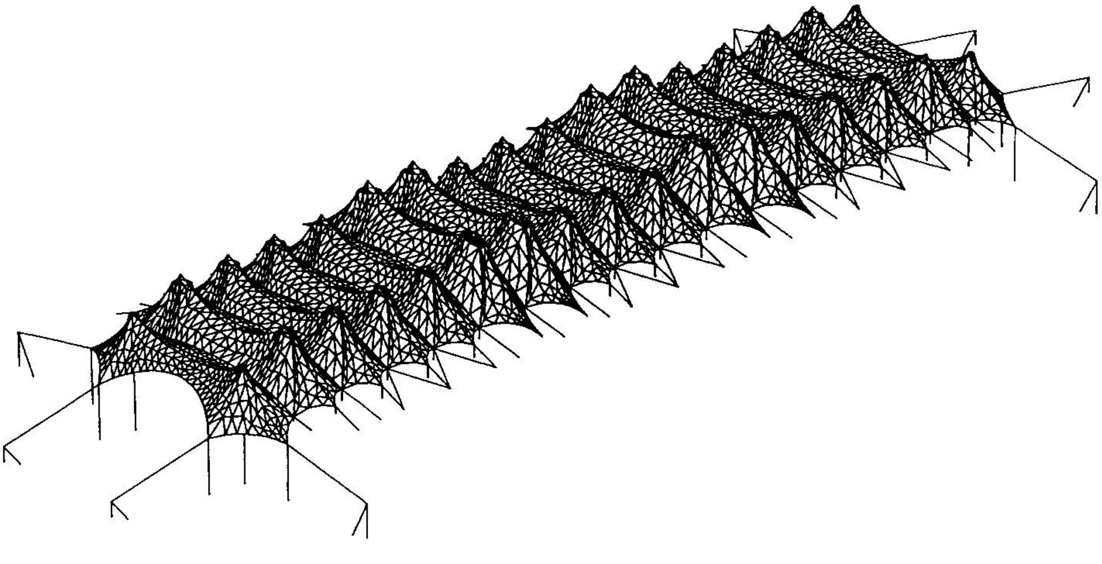


Figure 2: System Model. A "coarse mesh" system computer model is used to quantify overall behavior.

To make the installation models, a portion of the system model appropriate to represent a particular stage of construction is used. The installation rigging and temporary guying systems that will be present are added to the model. The pre-stress forces and geometry are modified to better represent the real conditions. The installation models are used to design the installation rigging and check the permanent roof components during the different construction phases. It is not unusual to uncover problems that were not possible to determine during the design phase when the final installation sequence was not known.

The pattern models are used to produce a very precise representation of the final geometry of the membrane and cables. These models will be used to produce the fabric cutting patterns and final cable fabrication lengths. A different pattern model was built for each bay of the Denver Airport. The pre-stress forces and boundary geometry established through work with the system models are used in the input data to these models. A much "finer mesh" is used to better represent the actual geometry. The software used to generate the pre-stressed equilibrium shape of the membrane also pulls the node lines (later to become seam lines) onto geodesic curves (ie. shortest path curves) along the membrane surface. This insures optimal seam locations both from a fabrication and aesthetic perspective.

Fabrication

The patterns are produced on the computer by laying sections of the model down into 2-D. The pattern data is then transferred to the fabrication shop electronically where a wide-area plotter plots the templates full-scale on paper. A typical template is 12-feet wide

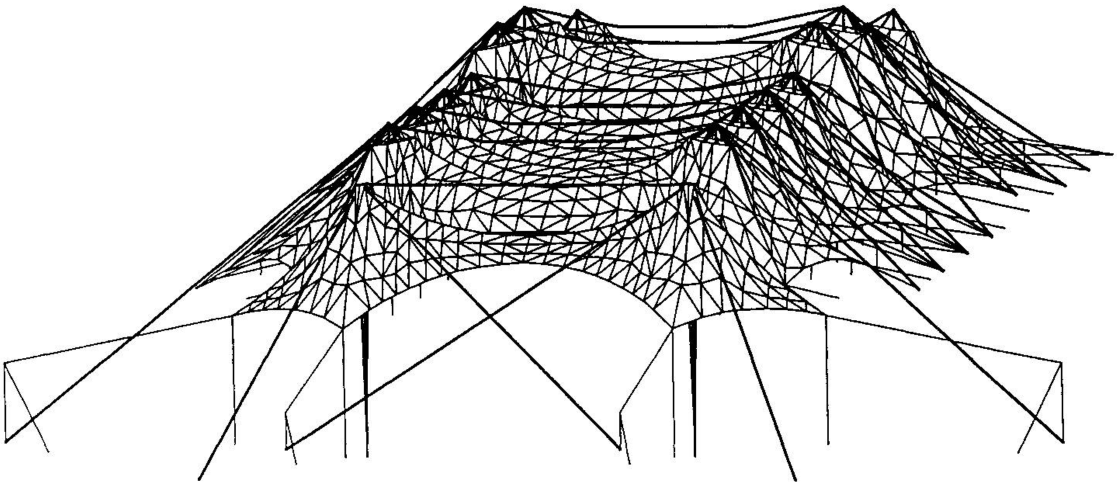


Figure 3: Installation Model. An installation computer model is used to analyze and design the temporary rigging and partially installed roof.

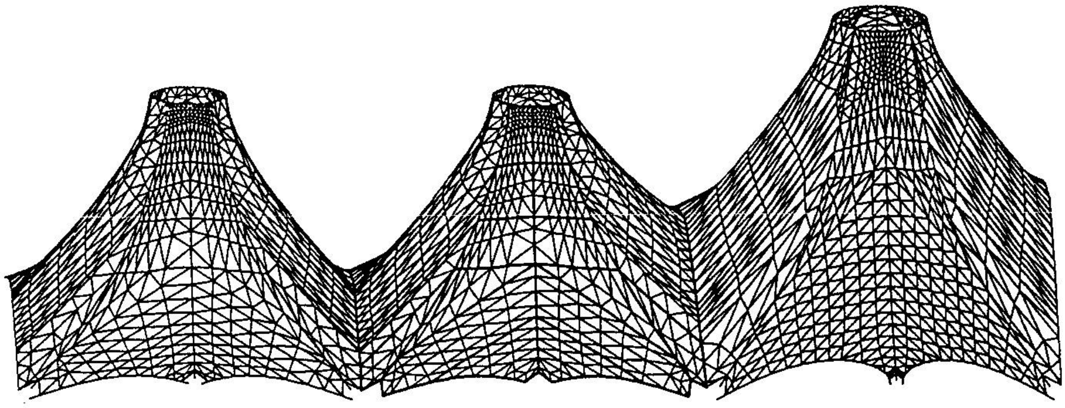


Figure 4: Pattern Model. "Fine mesh" computer models are used to generate the precise shape for patterning the fabric. The mesh lines (later to become seams) are established along geodesic curves on the surface. An example of these is shown with heavy lines in the center bay.

to match the fabric roll width, and up to 100-feet long. Fabric panels are cut from the templates and heat-welded together in the shop to form "assemblies". Each bay of the Great Hall roof consisted of 4 fabric assemblies. Each fabric assembly was individually rolled or folded and then packaged for shipment to the site.

Installation

At the time installation of the roof systems began, the concrete structure of the Great Hall building was complete up to the 5th level. Level 5 is the floor level for the primary Great Hall space. It was used during installation as a staging area and work surface for both men and equipment. Designed for live loads as much as 250 PSF, it was able to support up to 40-ton cranes, provided load-distributing mats were used.

The masts were delivered to site in one piece. Top weldments, rigging, and miscellaneous hardware were attached while the masts were on the ground. The masts were then erected using conventional boom cranes located outside of the building. In the completed structure, the masts are stabilized by the fabric roof system and associated cables that are located within the shape of the membrane surface. They have no external guy cables and therefore must be allowed to pivot on spherical bearings at their bases. Temporary guy cables were required to stabilize the system during installation. As there was no place to position guy cables that would not interfere with fabric installation later, temporary mast top extensions were bolted to the masts to provide a place to attach the guying system.

The guying system and partially-erected fabric subjected the masts to loading conditions and bending moments in the upper sections that the masts would not be able to carry. The problem was analyzed and solved using the installation computer models discussed earlier. The solution used was to add temporary stay cables to work in conjunction with the truss rings (similar to the stay cables on a boat mast) and remove the bending movement in the masts.

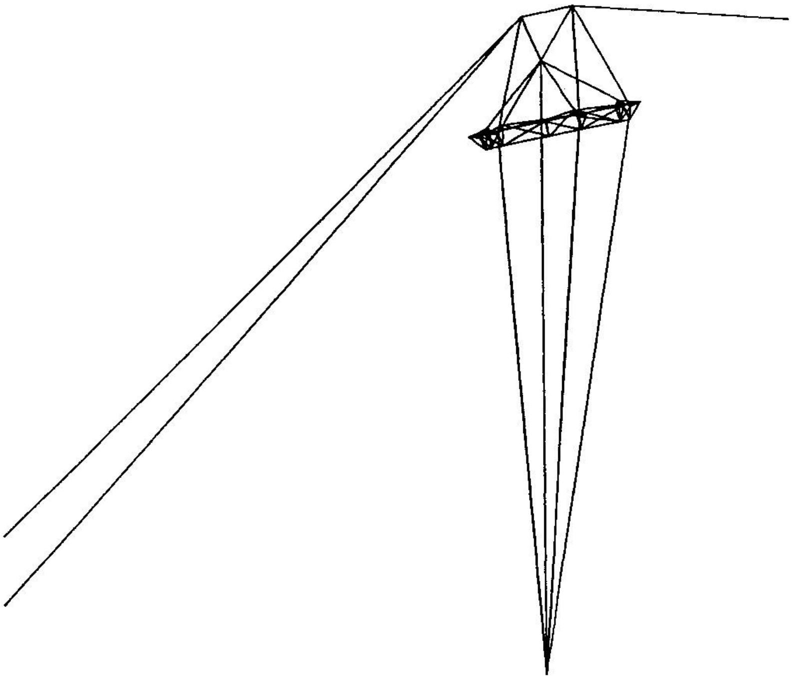


Figure 5: Stay Cable Rigging. A computer model is used to design the temporary stay cable rigging and mast extensions.

The truss rings were delivered to the site in two pieces, set around the mast bases (at the Level 6 elevation), and then welded together. The mast top units, skylights, and mechanical equipment were then assembled on the rings. Hoisting of all the rings (two at a time) was accomplished with a large drum hoist secured in one location on Level 5. The drum hoist cables traveled through a series of sheave blocks and fairleads up to Level 6, over to the appropriate mast, up the mast, and into a block and tackle system to produce the required mechanical advantage. Using this system, the rings together with their mast top units were sequentially hoisted in pairs.

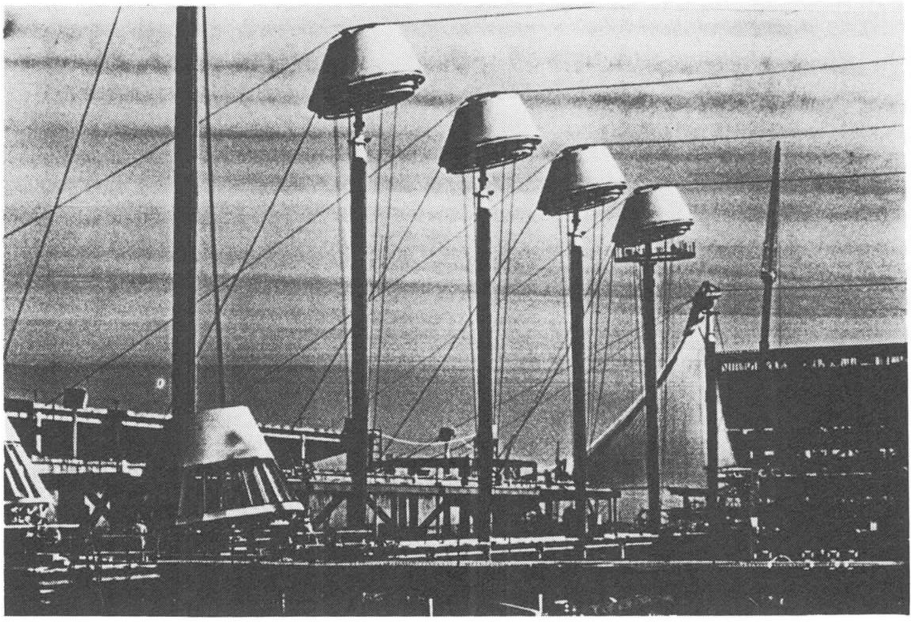


Figure 6: Mast and Truss Ring Installation. The masts are guyed with external temporary cables. The truss rings are assembled around the mast bottoms and then winched up into position.

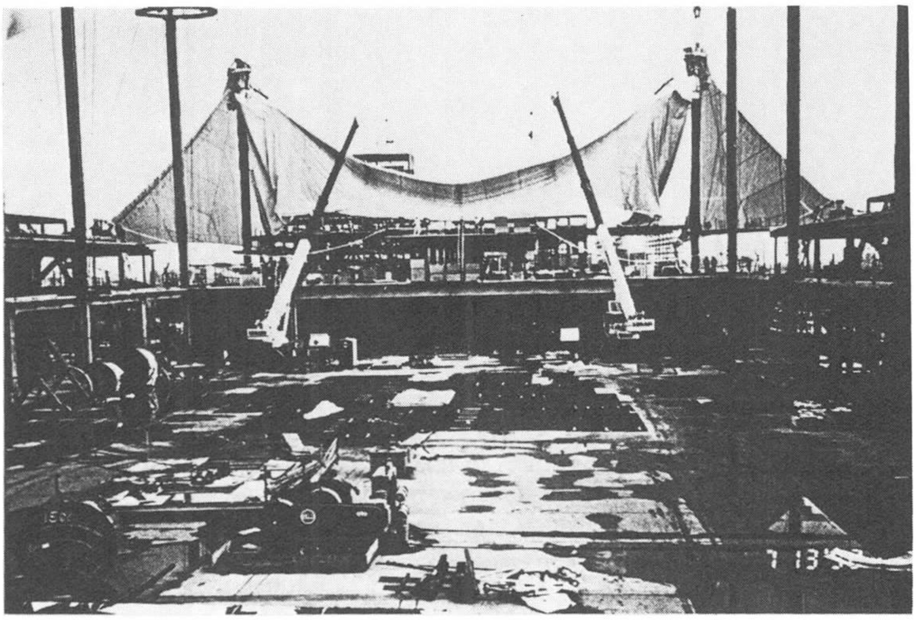


Figure 7: Fabric Hoisting. The fabric was lifted with a drum hoist secured on the level 5 slab (seen in the foreground). Two hydraulic cranes were used to assist.

As the ring assembly and hoisting proceeded, the outer fabric installation began. The fabric assemblies were unrolled on the Level 5 slab, and installed one bay at a time. The perimeter clamping hardware and cables (ridge, valley, etc.) were attached to the fabric while down on the slab. The fabric was positioned such that the two halves of a bay rested together, one on the top of the other prior to hoisting. The primary hoisting was performed using the same winch that was used to hoist the truss rings. The hoist cables were attached to each end of the bay's ridge cable which were lifted towards the rings. Two hydraulic cranes (positioned on the Level 5 slab) were also used to assist. As the ridge cable was lifted, the fabric bay went with it. Once the ridge cable was pinned, the fabric bay was spread open and attached at the valleys to the neighboring bays.

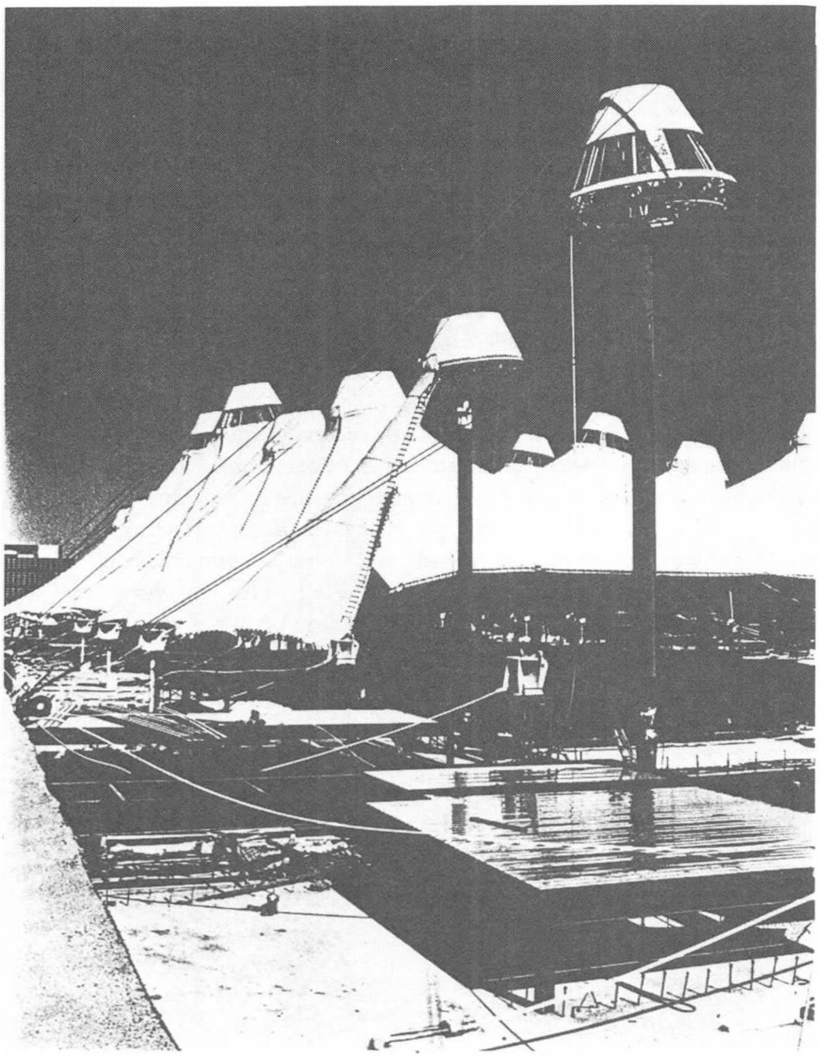


Figure 8: Partially Installed Roof

Following completion of the outer fabric the clerestory framing was installed. This work was erected from the inside of the building using hydraulic cranes situated on the Level 5 slab. An air-inflated expansion joint was installed to close the space between the outer fabric and the rigid clerestory framing.

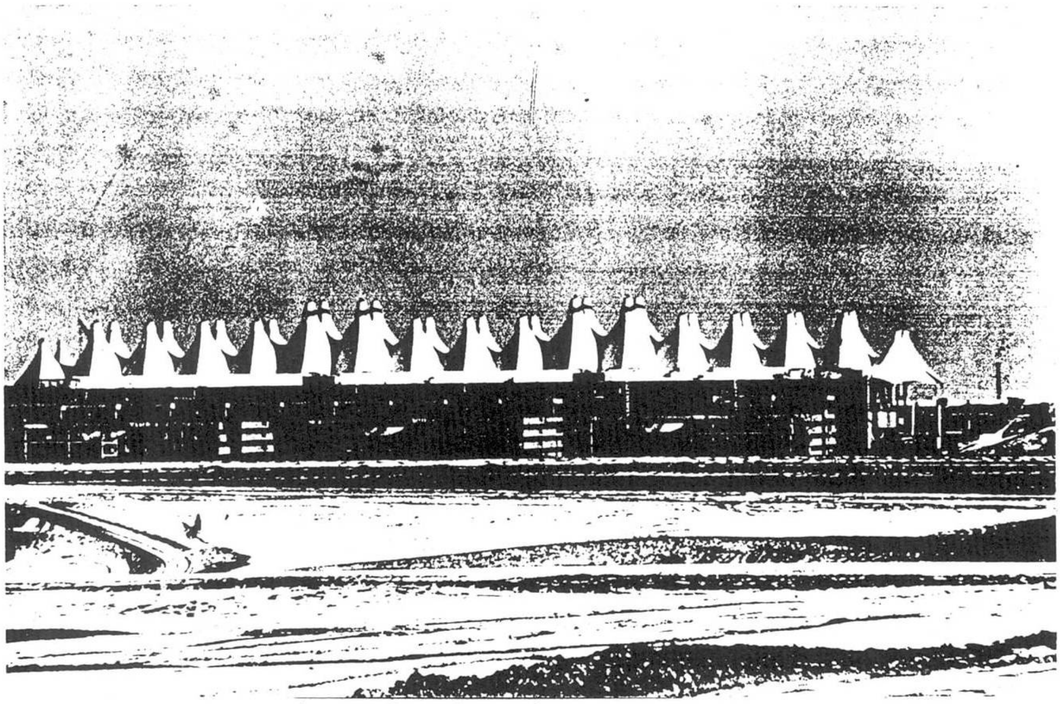


Figure 9: Elevation of the Installed Roof

The liner was installed after the clerestory glazing was complete, and an interior space was protected from the weather. It was erected sequentially in much the same manner as the outer fabric. However, being much lighter and protected against the wind, small electric winches were used instead of the large drum hoist. A temporary dust barrier was installed with the liner to minimize dust accumulation on the fabric that would be produced by the finishing trades to follow.

The fabrication and construction of the roof system took the efforts of more than 300 people, over a time period of approximately three years. The fabric roof will become a landmark to the City and County of Denver, recognized worldwide for its unique architecture and the magnificent space it creates.

POSTERS

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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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David Phillips obtained his degree in Civil Engineering at the Univ. of Surrey, Guildford in 1974. Since then he has worked for John Laing Construction on many projects, principally in the Midlands area. From 1982 he has been regional engineer for Laing Midland.

Alan CARNEY
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Alan Carney gained his initial experience in the aerospace manufacturing industries. Since 1989 he has been the Engineering Manager initially for the International Convention Centre and, on completion, of the National Indoor Arena, responsible for both the ICC and NIA.

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.

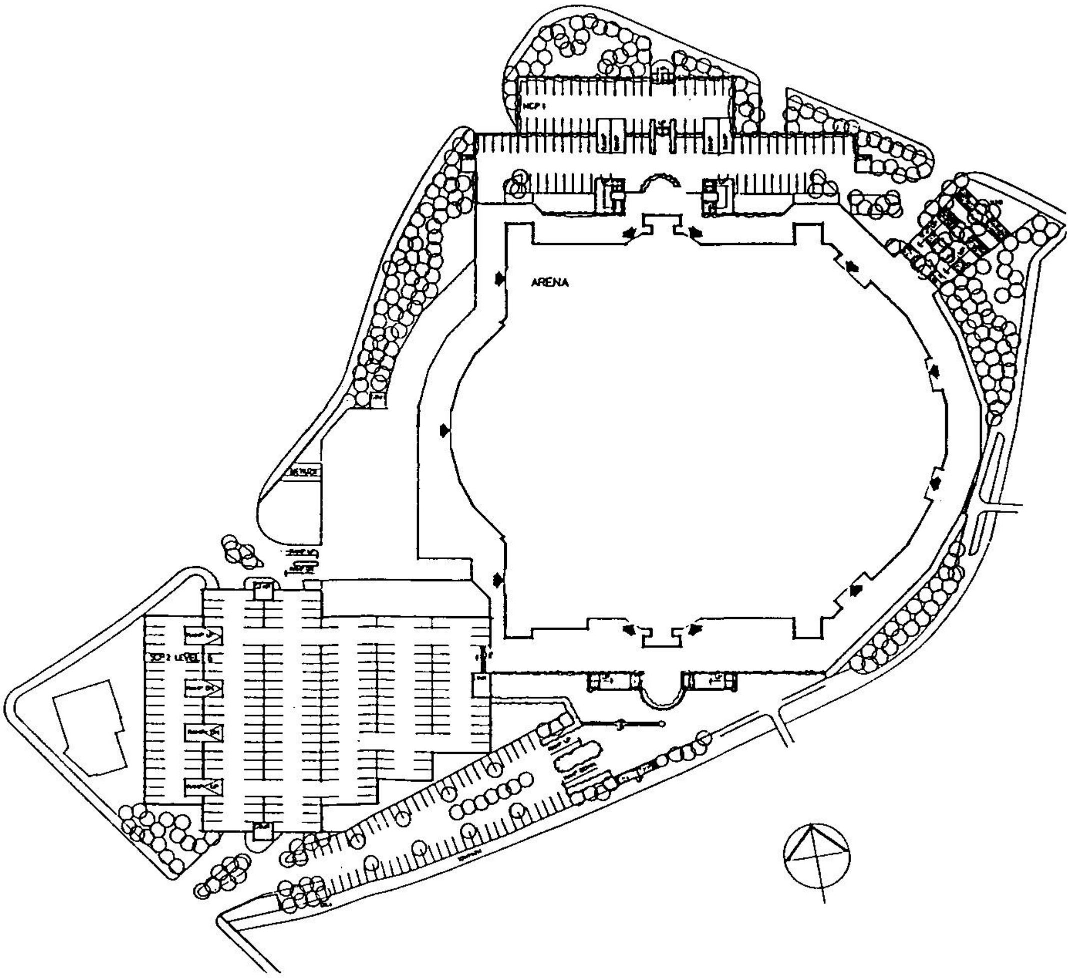

Fig.1 Plan of the Arena and car parks

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.

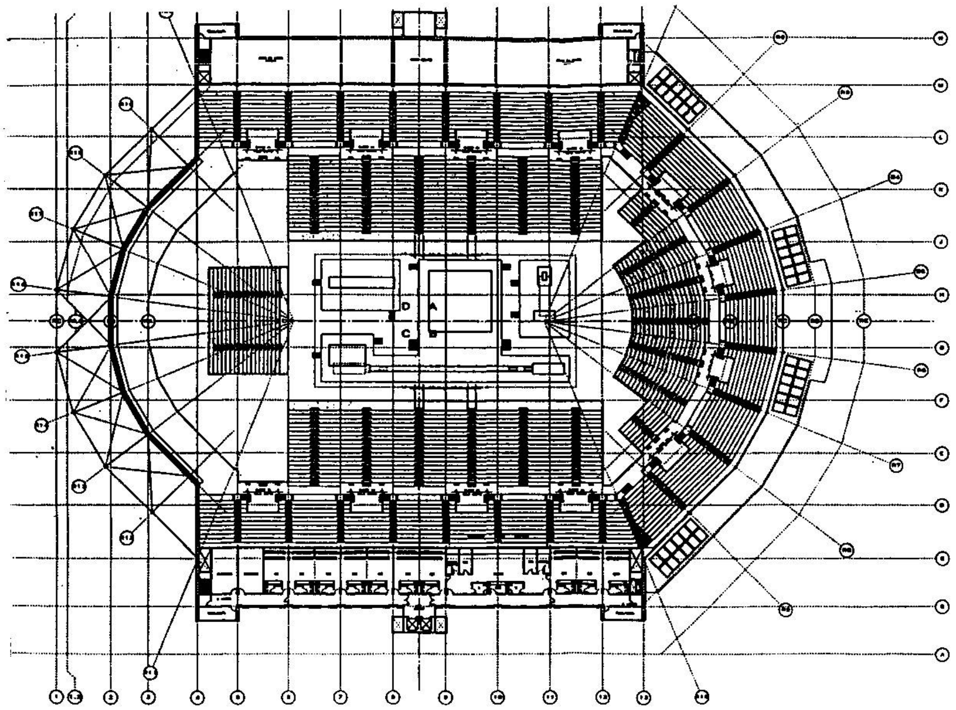

Fig.2 Plan of Arena showing tiered seating

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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Ian Liddell, born in 1938, was educated at St Johns College, Cambridge and Imperial College, London.

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.

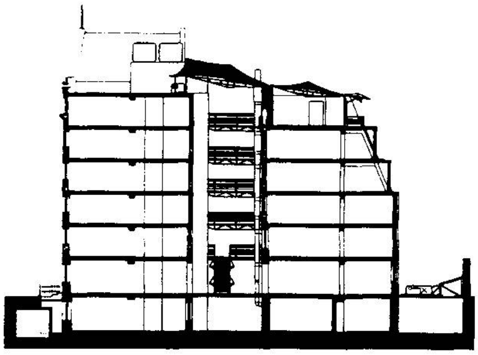

Figure 1 : Cross Section

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/m^2. Under wind these stresses will increase to about 600 kg/m^2, 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).

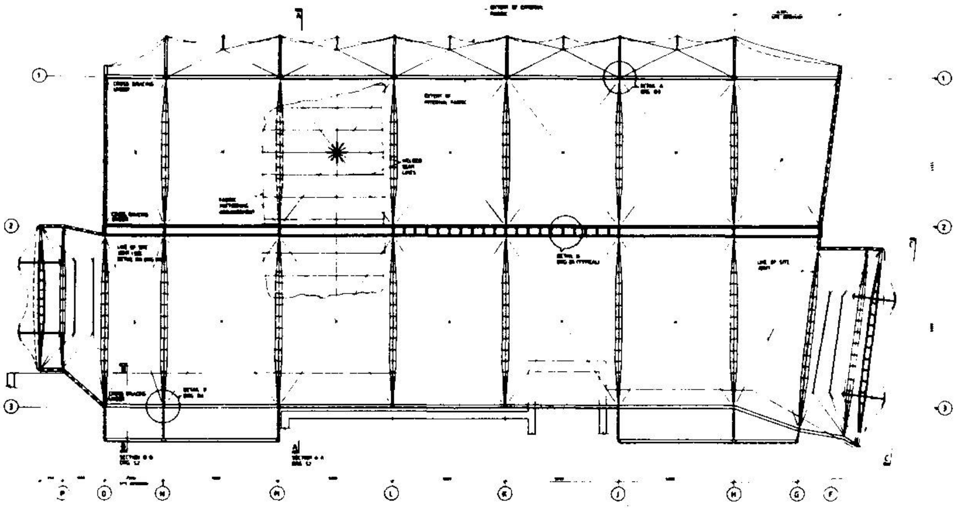

Figure 2 : Roof plan of Atrium

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.

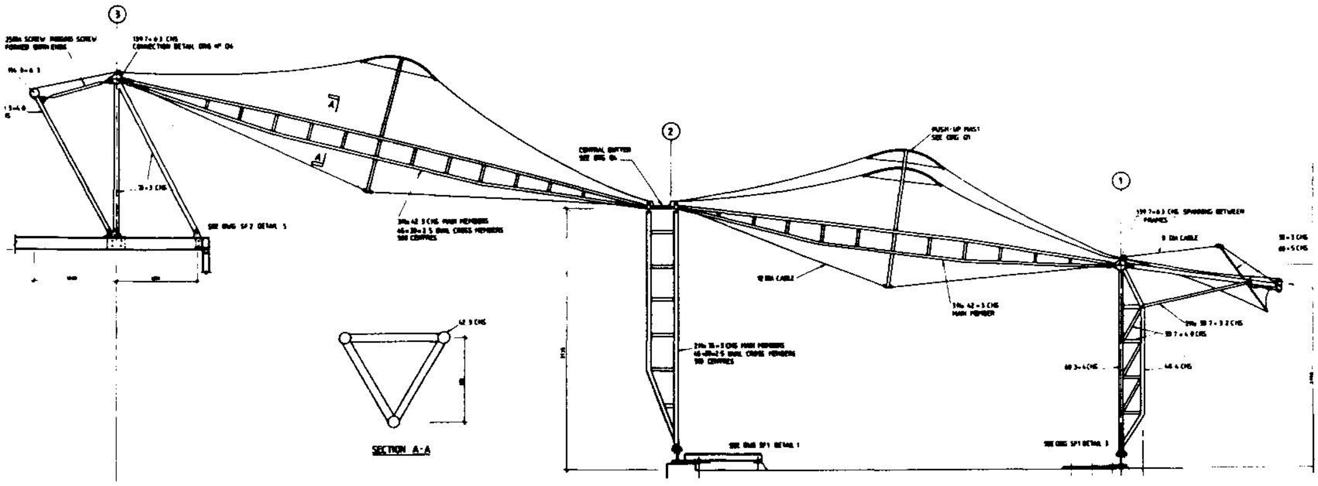

Figure 3 : Section through Atrium roof

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.

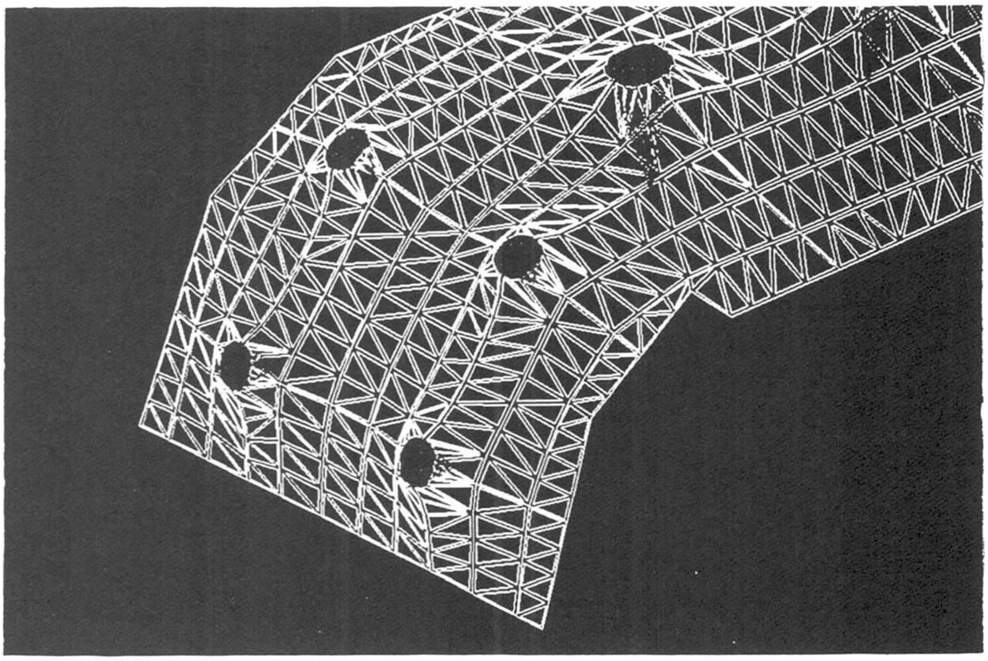

Figure 4 : Computer analysis of roof form

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.

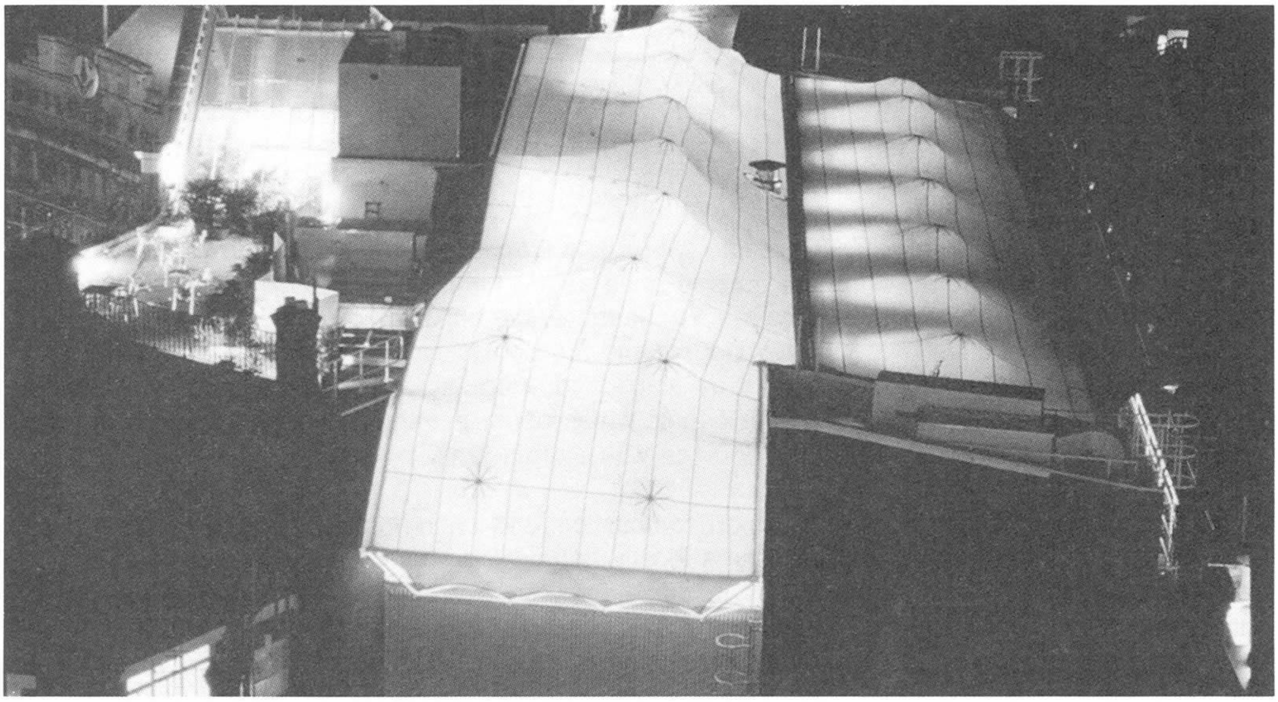

Figure 5 : Atrium at night

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.

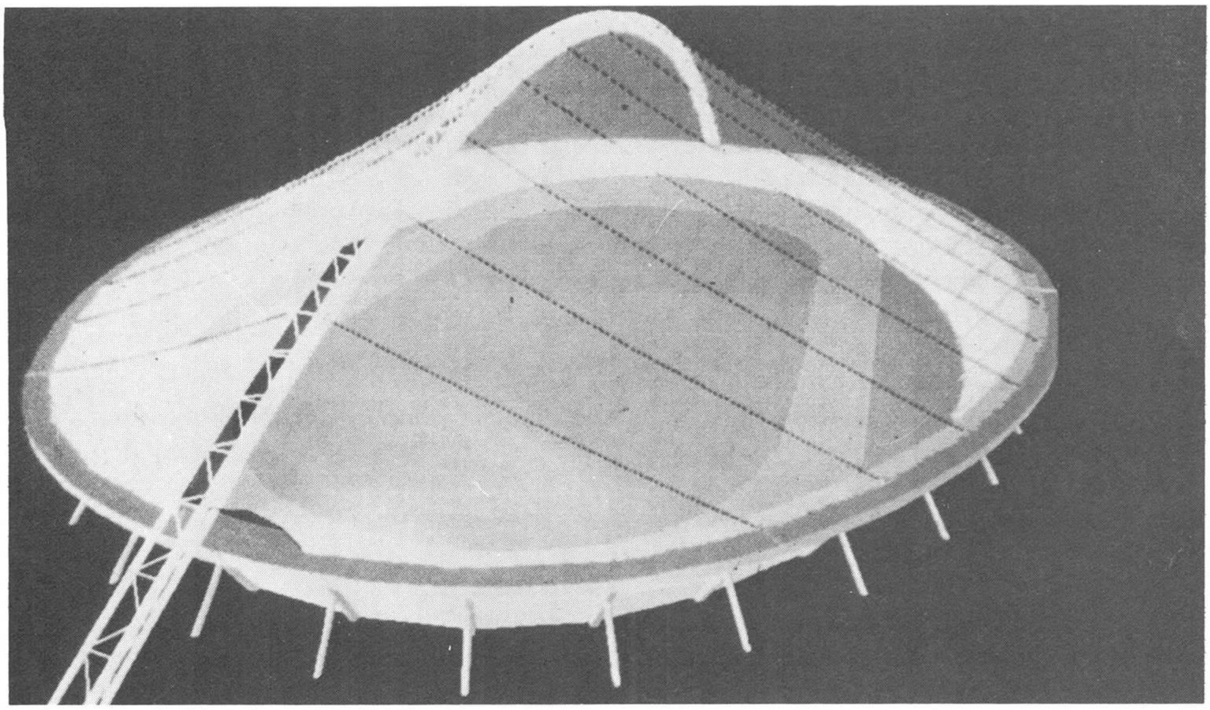

Fig. 1. Computer model of Velodrome

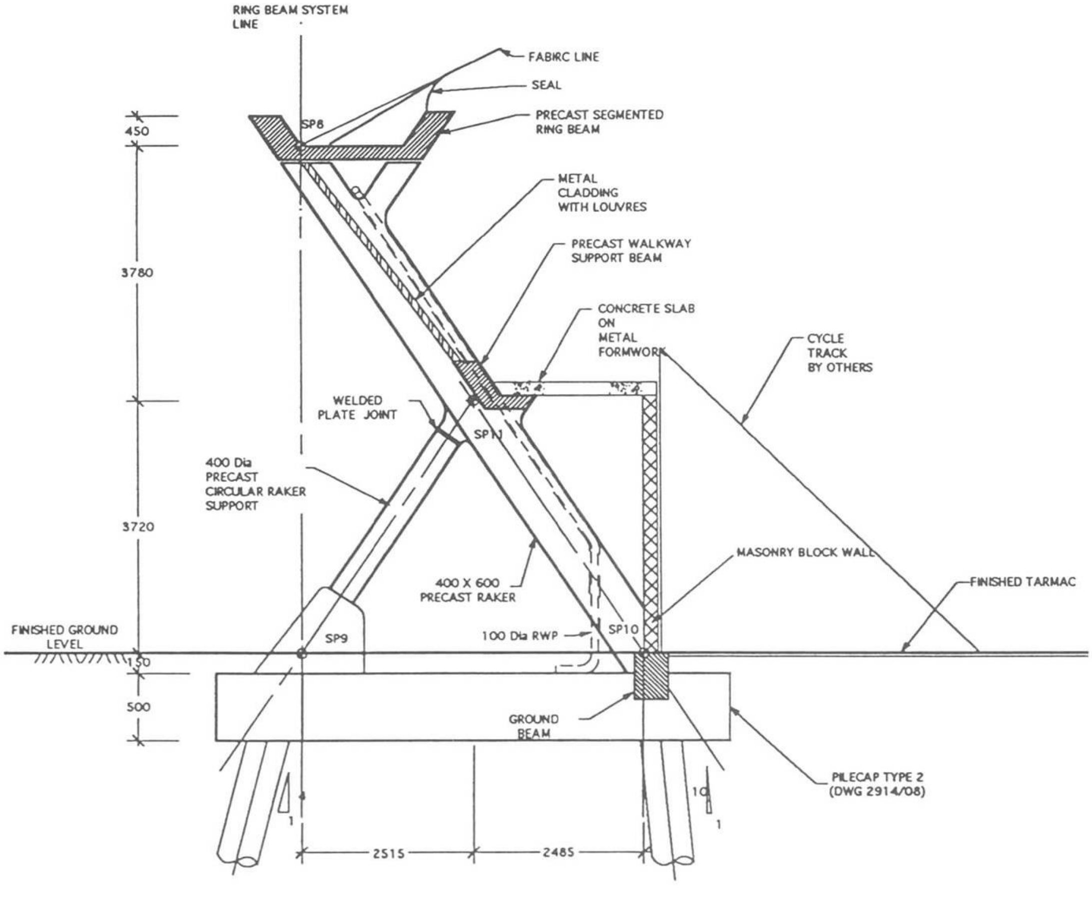

Fig. 2. Structural section

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.

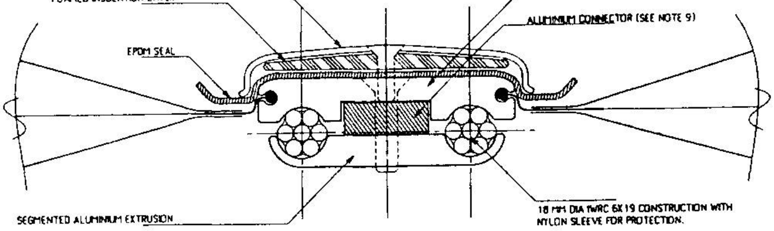

Fig. 3. Flexible extrusion connection detail

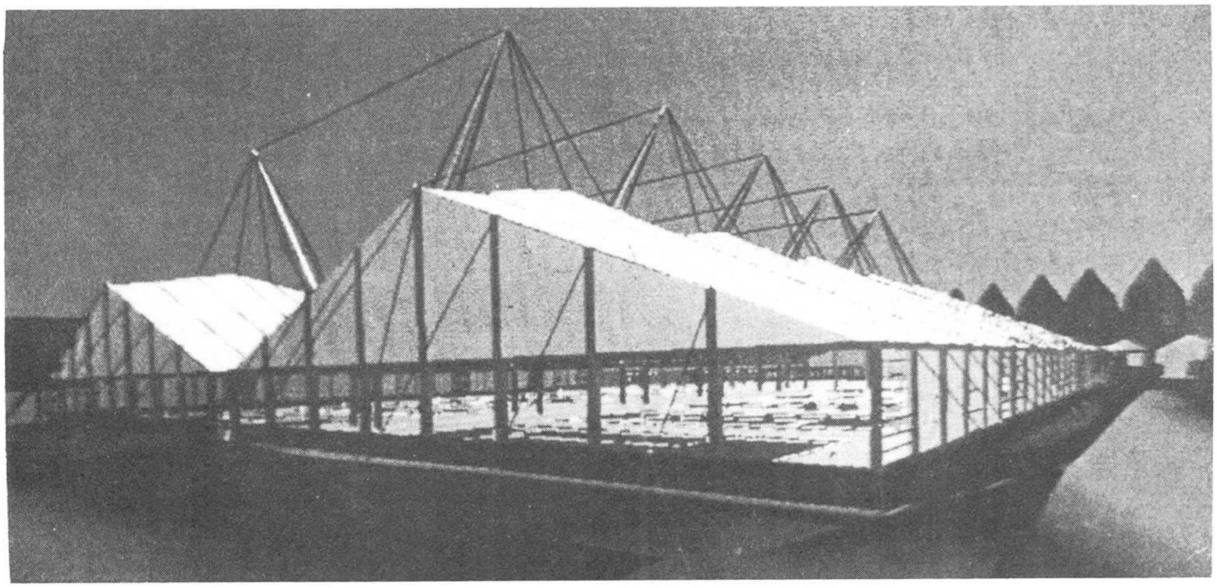

Fig. 4. Computer modelling of tennis halls

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.

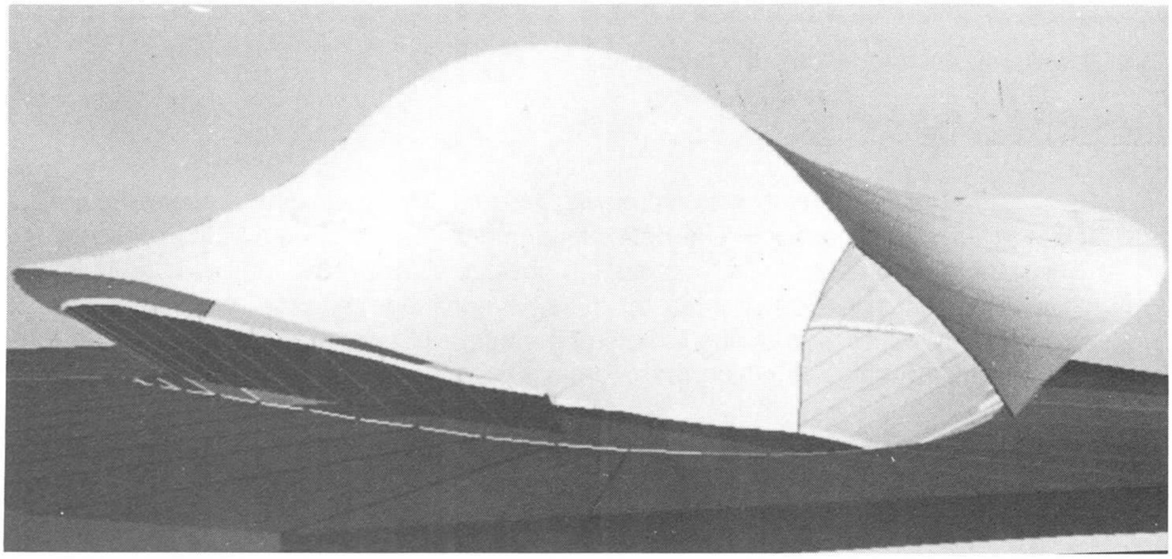

Fig. 5. Computer model of Stadium Roof with fabric closed

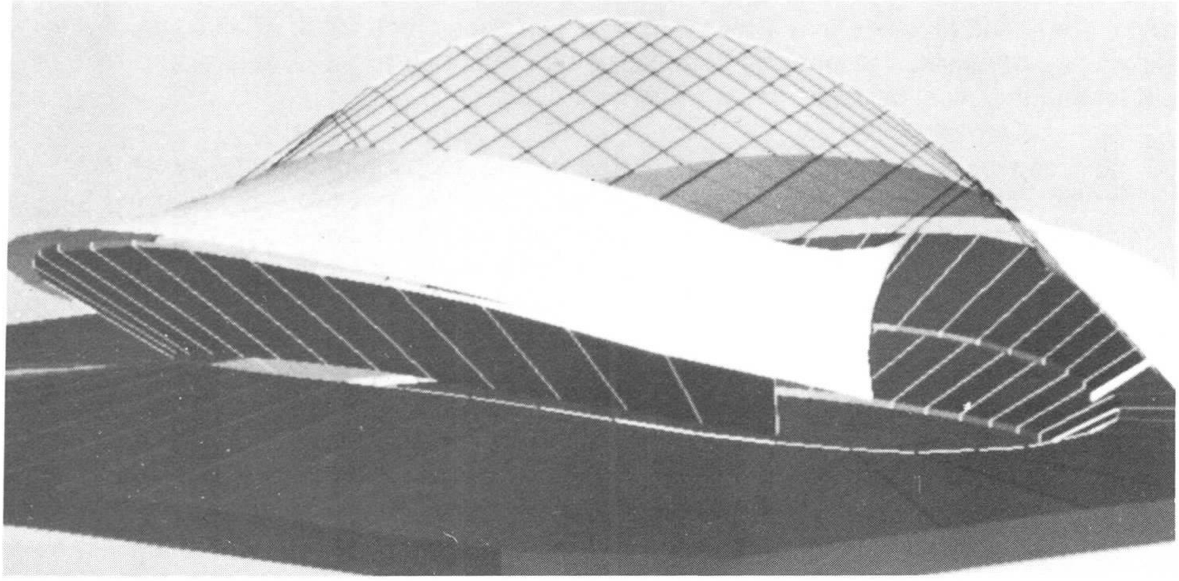

Fig. 6. Computer model of Stadium Roof with fabric retracted

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.

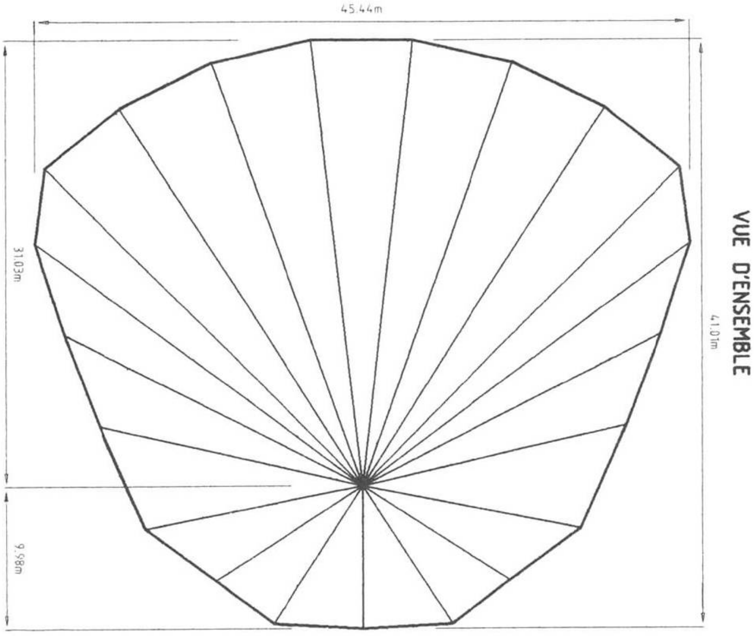
Le volume intérieur de la toiture devait, quant à lui, être aisément accessible notamment pour assurer la surveillance et l'entretien des nombreuses techniques contenues dans ce volume et en particulier pour pouvoir assurer la maintenance de l'éclairage de l'hémicycle.

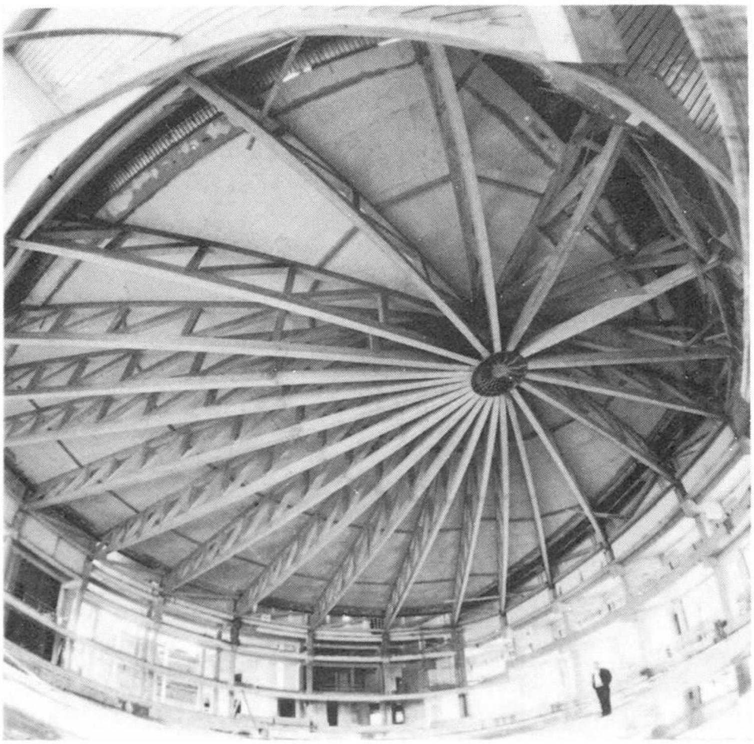
1.2 Impositions structurelles

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.

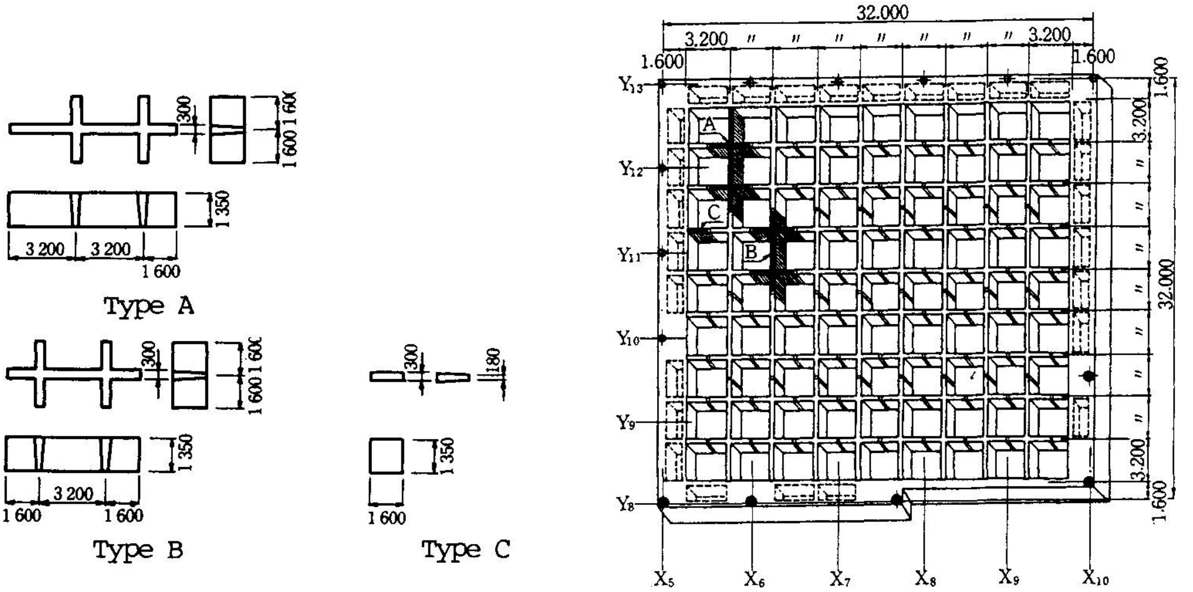


Fig. 1 Types of Pre-casted members and plan of the roof

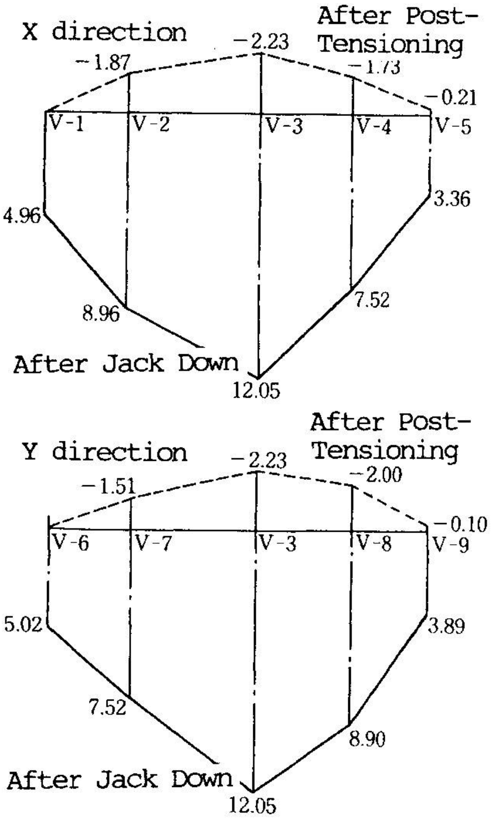


Fig. 3 Deformation of the long span girder

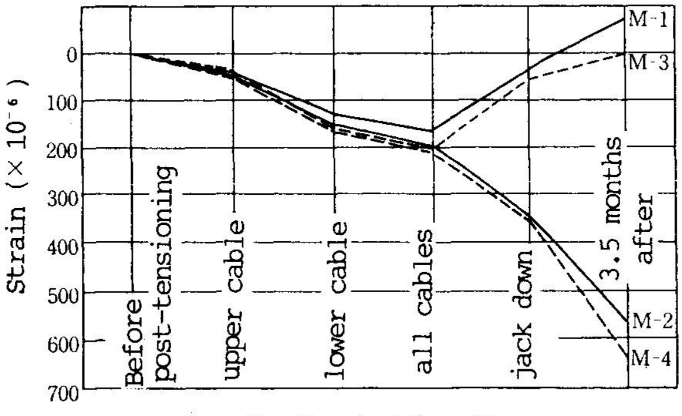


Fig. 2 Strain Time-Story

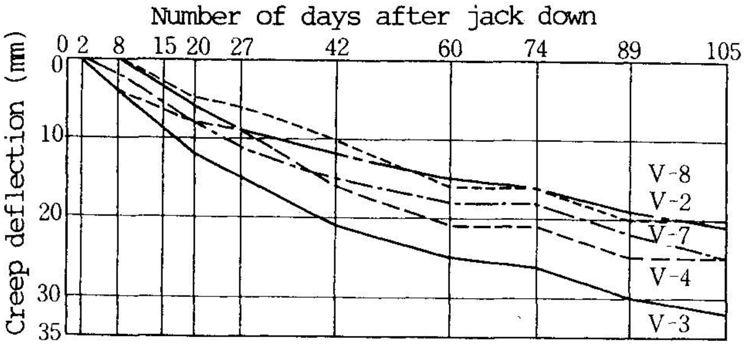


Fig. 4 Creep deflection after jack down

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

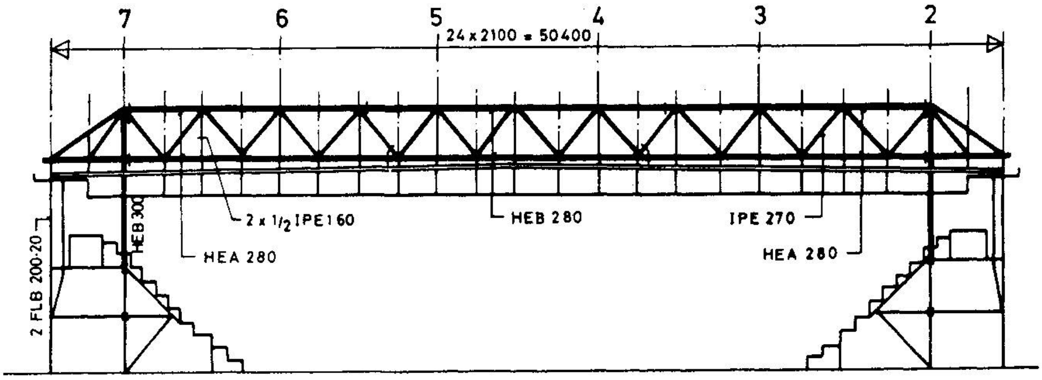
1. DESCRIPTION OF THE BUILDING

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.

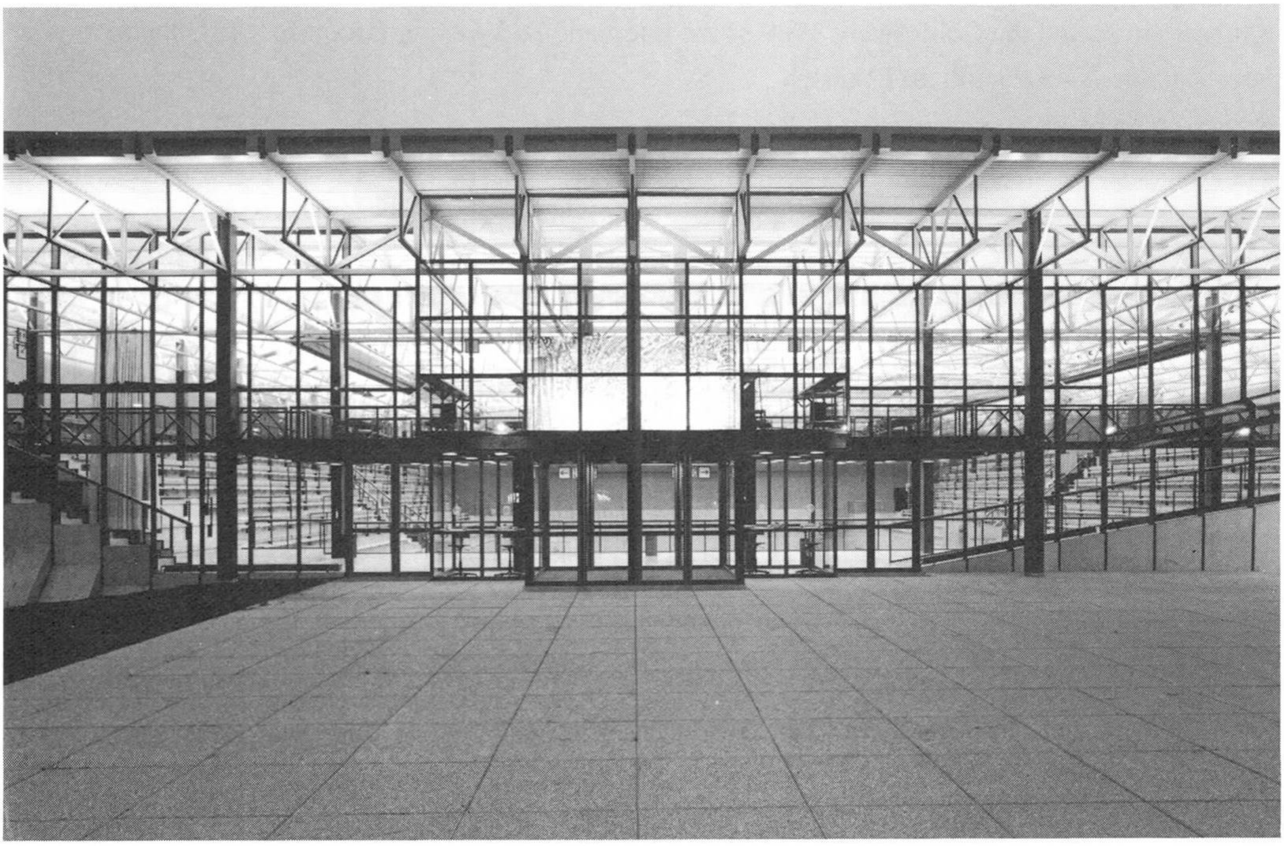
4. ROOFING AND INSULATION

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

Steven LUKE
Civil and Structural Eng.
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.

Fig. 1 - Aerial View Feb 1993

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 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].

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.

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.

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.

Reference

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

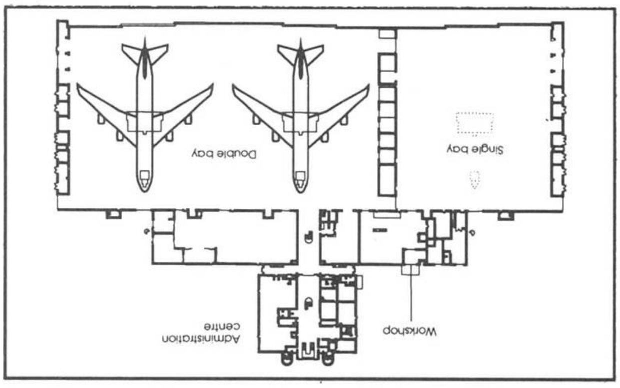


Fig. 2 - Plan

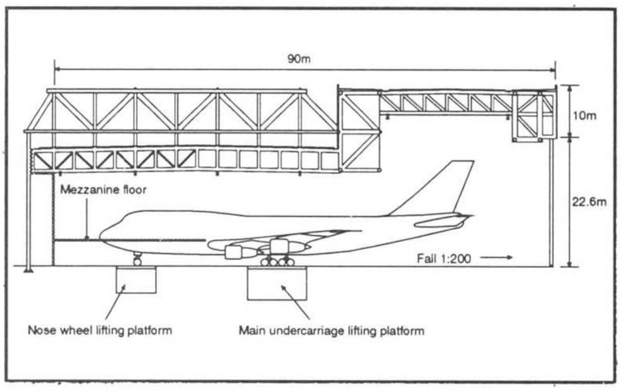


Fig. 3 - Hangar Section

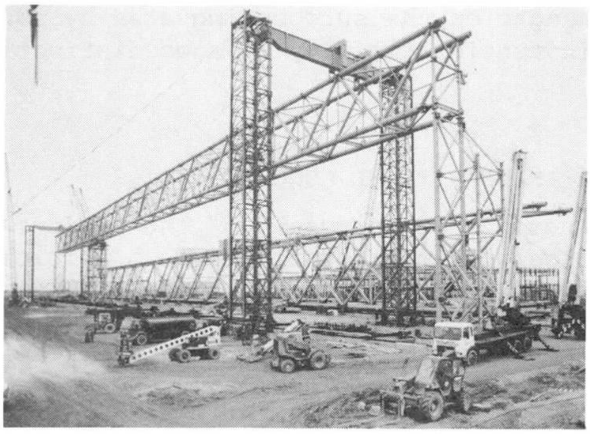


Fig.4 - Door Girder Lift Jan 93

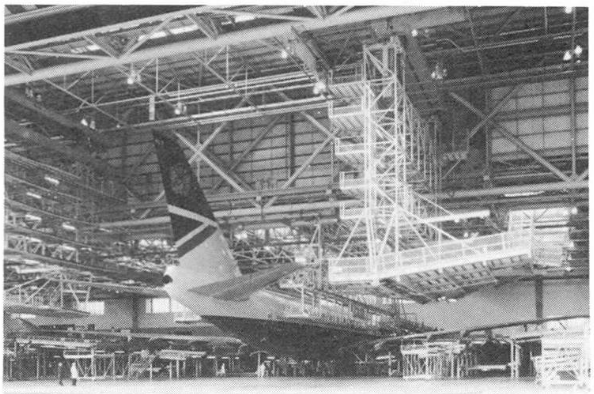


Fig. 5 - Access Docking

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).

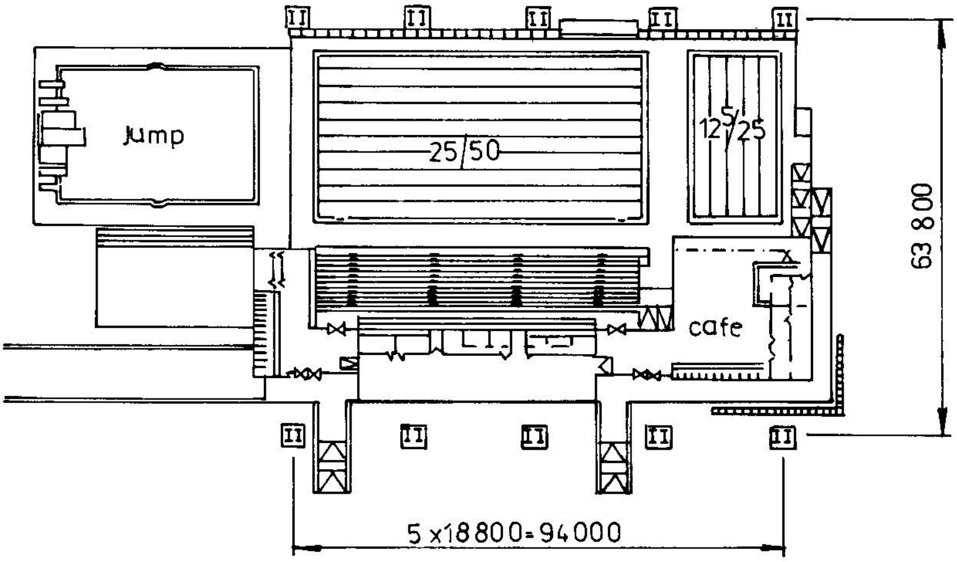

Fig. 1 Plan of the swimming complex

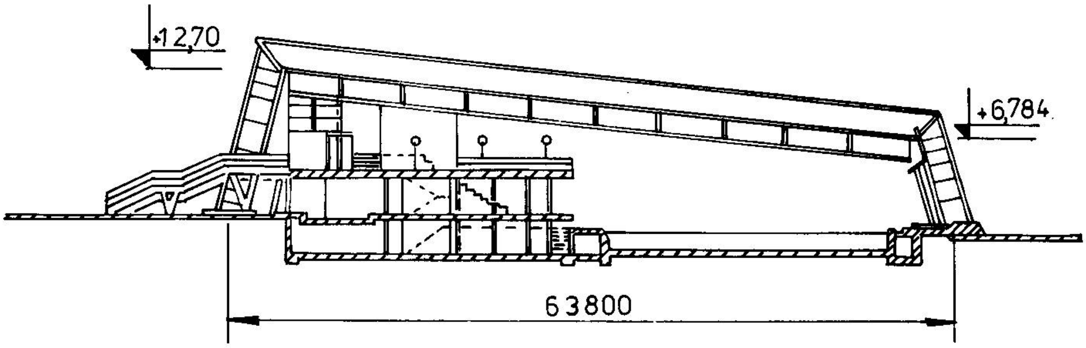

Fig. 2 Cross section