THE ARUP JOURNAL

SUMMER 1991

THE ARUP JOURNAL

Vol.26 No.2
Summer 1991

Published by
Ove Arup Partnership
13 Fitzroy Street,
London W1P 6BQ

Editor:
David J. Brown

Art Editor:
Desmond Wyeth FCSD

Deputy Editor:
Caroline Lucas

Editorial Assistant:
Gabrielle Scott

Front cover: Ponds Forge: steelwork detail (Photo: Jane Richardson)

Back cover: Lobby perspective, Cerritos Arts Center, California (courtesy of the architects)

Ponds Forge International Sports Centre

Architect: FaulknerBrowns*

Mike Brown, Martin Butterworth, Paul Kay, Paul Stevenson

Introduction

The city of Sheffield (population 560 000) is a former industrial empire built on quality steel. In the late 1970s and '80s this industry collapsed and, apart from the massive unemployment this created, the lower Don Valley was left as a corridor of closed steelworks stretching out from the city centre. In an effort to revive the fortunes of the city and as part of its regeneration, the City Council decided to apply to host the XVIth Universiade 1991, the World Student Games. One of the aims of these events is that the host city be encouraged to build new facilities, and so give long-term benefit to itself and its community.

The new Ponds Forge International Sports Centre is one of several facilities Sheffield is providing for the Games. It has been designed as a focal point for a variety of events — swimming, diving, water polo and volley-ball. Elsewhere, other buildings include an outdoor multi-purpose stadium, an indoor multi-purpose arena and a water polo/leisure complex. Existing buildings, such as the Lyceum Theatre (see *The Arup Journal*, Spring 1991), are being restored to house a cultural festival, and the 1960s Hyde Park flats refurbished to accommodate the 4000 competitors.

Competition, brief, and concept

In November 1987, FaulknerBrowns* were invited to take part in a limited competition for the design of the new sports and leisure complex. Later it was decided that it should also incorporate an international diving pool. The design had to comply fully with the rules set down by FINA (Fédération internationale de natation amateur), to provide the first international swimming and diving complex to be built in Britain for 20 years.

The building is located on a prominent city centre site, Ponds Forge — acknowledged as the gateway to Sheffield — which had been occupied by industrial premises for almost 200 years. A new linear park will be created to extend from the railway station across the landscaped roundabout at Park Square and then through to Cutlers Wharf. The River Sheaf is visible in places, but culverted under Ponds Forge. Within the tight confines of the site, the design attempts to explore and articulate the complex relationships between international sports competition, leisure pursuits and tourist attractions, whilst relating it all to the needs of the city community. In the long term it is intended that a light rail system known as 'Supertram' will complete the public transport facilities to the building, running along the frontage of Commercial Street.

Ponds Forge consists of three interlocking volumes of opposing curved roof forms above and below a public terrace, which links several pedestrian routes to and across the site. These volumes become a backdrop for the circular portico of the entrance, known as the Rotunda, which forms the focal point of the corner site overlooking Park Square. The Rotunda is the meeting place and distributor to the other facilities, and can also be used for exhibitions and displays. At the lower level are interconnecting changing rooms leading to the main competition pool hall, the leisure pool and the multi-purpose sports hall. The design concept of what are effectively three buildings provides an ideal location for movement joints and subsequently assisted with packaging the works into manageable contracts.

The site

Its principal former use was the manufacture of iron and steel using two natural resources close to the site — water from the River Sheaf and coal from a 2m thick seam at a depth of around 25m. The site was overlain by fill of varying composition which included industrial contaminants, foundations of previous buildings and remnants of the industrial past. The infilled Ponds Dam occupied a significant area, fed by a number of water courses. The sloping rockhead varied between 4m and 7m in depth and mine workings were known to exist.

The first site activity was to establish the extent of these workings, their depth and their ability to sustain the load from the buildings. The condition of the overlain sandstone and mudstone and depth of the workings necessitated grouting of the abandoned mining seams over a considerable area.

1. Sheffield city centre with Ponds Forge International Sports Centre in foreground.

2. Location plan.

3. Pool hall elevation.

4. Central core elevation.

The main pool hall

The focus of the complex is the 90m × 60m pool hall. The main competition pool is 50m × 25m × 2m-3m deep with 2500 spectator seats. The width will allow 10 lanes of 2.5m, providing superb conditions for international swimming. Flexibility of use is provided by power-operated floating floors, one at each end of the pool, which can be lowered or tilted by hydraulic rams to provide a wide range of water depths. Two mobile pool dividers allow the international long course to be sub-divided into two × 25m short courses, water polo pitches, synchronized swimming areas, or three separate pools. The two areas which have floating floors can also allow a shallow-water teaching environment. The independent diving pool measures 25m × 16.5m × 5.8m deep and has the most comprehensive range and number of boards in Great Britain.

Leisure complex

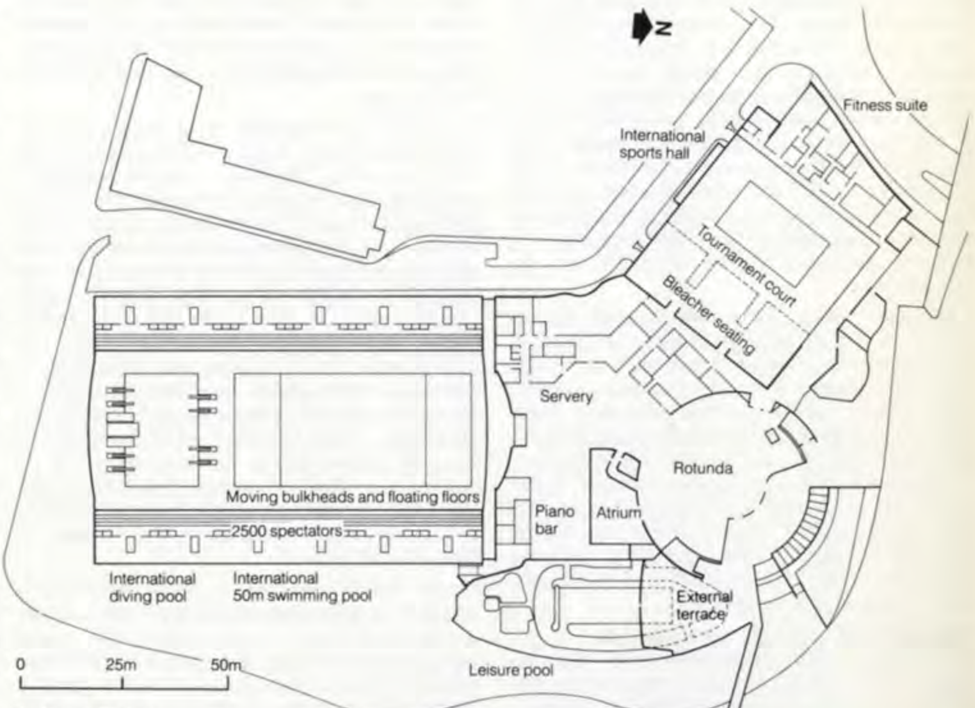
This contains 650m^2 of leisure waters, comprising a 25m warm-up pool and many fun features — a 100m slow river ride, two 80m flumes, a spa grotto and children's play area, plus a wave machine in the 25m pool. Associated facilities include a health suite with sauna, steam room, spa pool, and fitness suite. The leisure complex also houses a cafeteria, bar, restaurant and function suite, and an electronic museum is planned as a further visitor attraction.

Sports hall

The third element of the complex provides a multi-purpose space, 40m × 38m × 9m-12.5m high, able to cater for a wide range of multi-use activities, with power-operated, retractable seating for 1200. Planned uses embrace international volley ball, tennis, badminton, basketball, table tennis, gymnastics and indoor bowls. Community facilities including a crèche and shops are adjacent, as well as changing facilities with a fitness gym and offices. Below the sports hall, two levels provide parking for 170 cars.

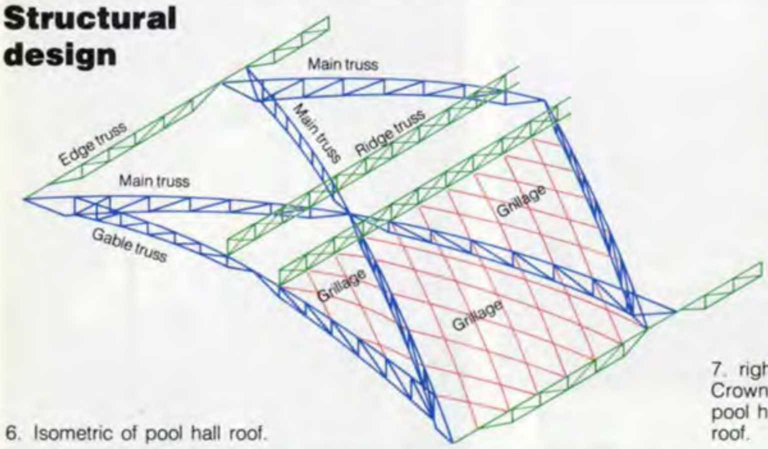
Programme

The brief required completion of the building for the summer of 1990, so that a period of acquaintance and use could be gained before the Games in July 1991. With the appointment of the design team in late 1987, it was decided that the only way to achieve completion on time was to adopt a management contract where the design and construction could take place as parallel activities with contracts let as a series of packages. The management contractor was on site by March 1988, only four months after the appointment of the design team, giving a 31-month construction period. An analysis of activities, together with the tight site conditions, revealed that construction of the main pool hall roof was critical. Early roof erection would release the main work areas, unhindered by scaffolding, formwork or fixed cranage.



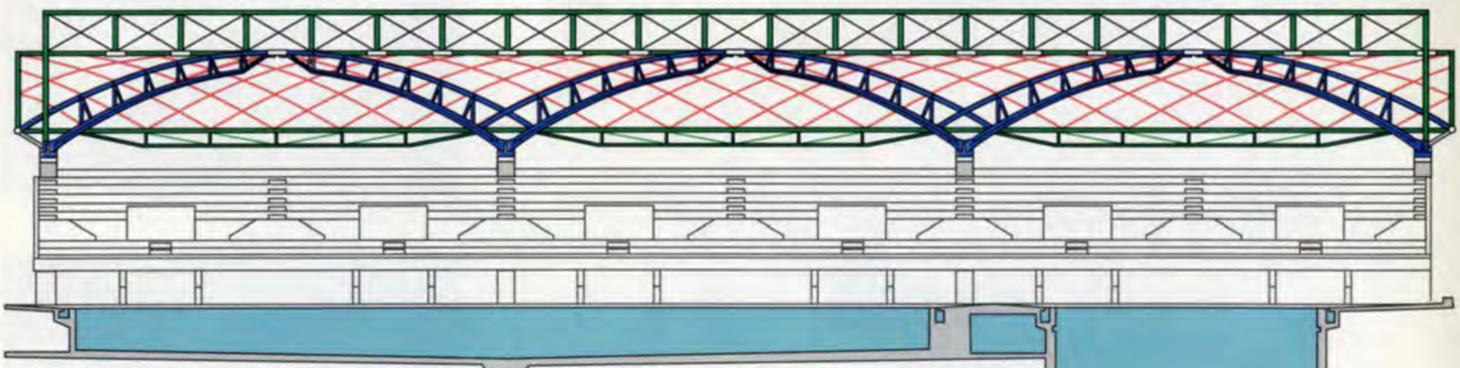
5. Level 2 plan.

Structural design



6. Isometric of pool hall roof.

7. right: Crown of pool hall roof.



8. Pool hall section.

Pool hall

The original scheme which won the design competition adopted an external four-masted system for the roof but, as the design developed, misgivings arose about piercing through the roof membrane to support the steelwork trusses, with the resulting cold bridging and waterproof detailing considerations. Then the diving pool was added and the masted structure finally abandoned as the supports could not be accommodated on the site. The concept changed to a vaulted roof design with cross arches, developed as an 84m long tubular steel barrel vault, forming a three-pinned arch over the 54m clear span. The roof contains 400 tonnes of structural steelwork. The main roof elements are tapered triangular trusses spanning diagonally between four concrete supports on each side, with gable trusses

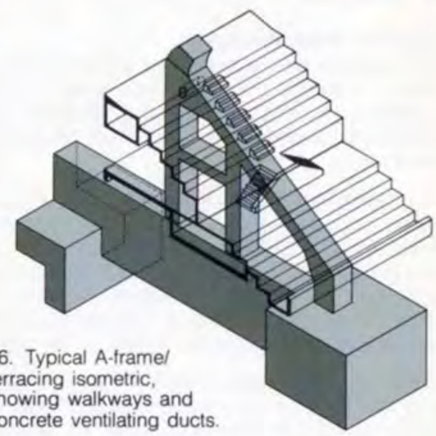
9

10△ 11▽ 12▽

14△ 15▽

9. Forging.
10. Main trusses being lowered into casting.
11. 12. Eaves casting supported on temporary steelwork.
13. Steelwork roof supported at the crown on temporary towers.
14. Gable end casting.
15. Ridge truss detail.

13▽



16. Typical A-frame/ terracing isometric, showing walkways and concrete ventilating ducts.

spanning directly between the end supports to complete the roof. The concrete supports are A-frames with kentledges founded on bedrock.

Thrusts of the roof are tied longitudinally along the top of the seating and those across the building are resisted by the A-frame.

The roof trusses are welded to cast steel pinned nodes at the crown and eaves of the roof; cast steel was selected due to the complex geometry and the large forces to be transmitted, while the form was chosen to provide direct load paths, to give a castable shape, and to minimize the weight. The material had an equivalent strength to the grade 50 steel used in the rest of the structure. The tube to casting connection was refined to ensure that the parts were thin enough to reduce heat loss in the body of the casting, and castellated sockets detailed so that sufficient weld length was provided. A cup and cap arrangement at these sockets allowed the trusses to be lowered directly into position with adequate erection tolerances.

Interwoven between the trusses are the diagonal curved infill grillages of tubular steel section, which provide a simple support system for the roof claddings and link the trusses together, giving the same structural rigidity as a shell roof. Edge and ridge trusses complete the support for the grillage. The whole roof is covered with an insulated sandwich-type construction, clad with aluminium panels for acoustic absorbence.

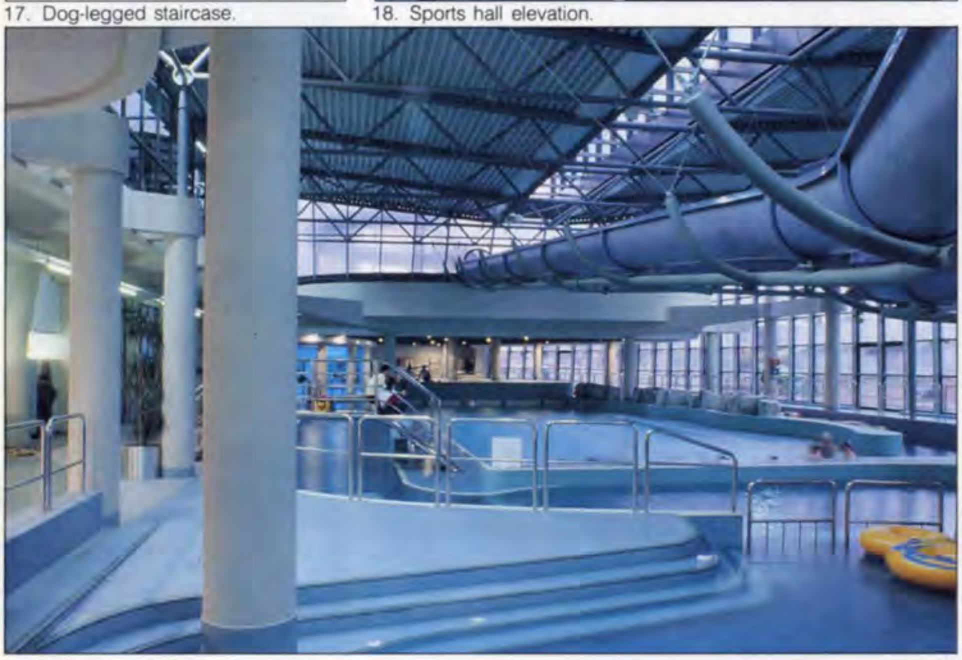
As already noted, the erection of the steel roof was the key to the programme and this placed the A-frame supports and other foundations on the critical path. Extensive former coal mine workings, 25m-30m beneath the site, had to be grouted as a preliminary contract, with particular attention paid to early release of areas occupied by the massive support foundations.

For erection of the roof, five trestle towers were built to support the five cast steel nodes which run down the centre line of the pool, and other trestling erected around the A-frame piers to cradle both the nodes and the trusses. The preformed trusses were lowered into the sockets of the nodes and welded together. Fixing the diagonal infill grillages followed, and finally the temporary supports were removed and the roof allowed to settle into shape on its eight bearing points. Despite the complex nature of the roof, the planned three-month erection sequence was completed without mishap, allowing construction of the main pool to follow.

The concrete terracing is supported by the main and secondary A-frame concrete supports which also incorporate walkways and integral service ducts. The pool was designed in watertight concrete to *BS8007* with no waterproof membrane, and a slip plane provided beneath the ground-bearing base. No movement joints were incorporated, due to problems on previous pools with tile failures. The pool construction sequence was very tight, employing procedures similar to a factory control process with small bays, strict limitations on casting sequence, and well-prepared construction joints with a *Hydrotite* water seal. Pool tolerances were also extremely tight and the pool was made oversize to allow for these, and temperature and shrinkage movement. For competition use, the bulkheads will be used to make the final adjustments.

17. Dog-legged staircase.

18. Sports hall elevation.



19. Leisure pool.

Central core design

The complex central core area consists essentially of a leisure pool, a central area accommodating plantrooms, changing facilities, refreshment areas, and the main Rotunda entrance.

The hydraulics of the leisure pool were initially tested at Salford University and the structure designed on similar principles to the main pool. Due to problems of filled ground in the area, the pool was suspended on supports taken to bedrock. At ground level a full-height curved glazed wall runs along the length of the pool.

The design philosophy required a slender support structure for the roof and glazed wall: a series of light trusses supported by slender columns shaped at the head to give a pencil-like effect. Lateral stability is provided by bracing within the plane of the roof, tied to the adjacent concrete structure.

The central area consists of three storeys and a basement, the latter designed to *BS8110* and tanked. The structure is of flat slab construction supported by columns with flared heads; this allowed clear service runs and gave a practical solution to varying ceiling heights. Delicate tied trusses form the roof and a feature is the large dog-legged staircase leading to the restaurant and bar areas.

The Rotunda provides the main entrance to the site at concourse level; its structure is reinforced concrete slabs and beams to concourse level, with double-height steel columns above supporting beams centred on radial lines. The pitched roof slopes to the centre and is supported by the series of columns on a circular grid, with wind trusses provided in the plane of the roof and running across the front of the structure. These also provide restraint at the head of the large glazed wall forming the main entrance elevation.

20. Sports hall bleacher seating.

Sports hall design

The architectural concept called for the sports hall roof structure to give a column-free area of 48m × 38m × 12.5m clearance at the highest point. The roof spans 40m and consists of five inverted triangular space trusses, 2m deep, supported on curved edge beams which are then carried by universal columns at 8m centres. To support the roof cladding, secondary UB beams are provided, curved in profile. The members of the main trusses were made up of rolled hollow sections, which minimized jointing difficulties at complex node points, facilitated steelwork fabrication, and provided consistent external member dimensions by varying the section wall thickness to suit design loadings. Stability is achieved by vertical bracing within the wall construction, while horizontal bracing is provided within the plane of the roof on all four sides.

The structure below the games hall level is of in situ concrete, one-way spanning, flat slabs supported on shallow beams. An 8m × 4.9m column grid was adopted to facilitate car parking.

Environmental services

Outline

Ponds Forge demanded a wide range of environmental conditions and services to cater for its diversity of activities. A services input to the design team was established during the competition stage, and philosophies for servicing the building agreed at this time. These included the highly topical concept of wet and dry ducts to the pools, the many advantages of which were highlighted during the competition presentation. Good design team consultation allowed early resolution of the fundamental services requirements for plantrooms, main distribution routes and services voids. Integration with the structure, detail and finishes was a high priority, and a great deal of time was spent ensuring this during design and construction. The final outcome made all the effort worthwhile.

Energy use in swimming pools is always a high profile consideration. This led to a specific development of the Oasys THERM program to be able to model swimming pools and determine their energy consumption, utilizing a variety of systems and heat recovery methods.

Corrosion problems were addressed from very early days with Arup Research & Development assisting in the selection of materials, finishes, and protective coatings. The mechanical and electrical specifications went to such lengths as to detail fittings, fixings, finishes, paint types — even down to the use of brass nuts, bolts, and screws.

Plantrooms

Some 3600m^2 of plant area has been provided throughout the complex for the services installations. The 2000m^2 main basement plantroom houses most of the central equipment; a 900m^2 water treatment plantroom is also located under the raised seating of the main pool hall. Three other air handling plantrooms are strategically sited elsewhere, simplifying distribution routes.

21. District heating heat exchangers.

22. Secondary system pump sets.

District heating

The primary heat source is a city-wide district heating scheme which obtains its heat input from the city waste incineration plant some 2km away. Sheffield were very keen to promote this 'green heat' scheme and many of the city centre public buildings are now connected to it. Two 1900kW plate heat exchangers remove heat from the district heating mains to supply the building. The district heating supply temperature is compensated and varies from 110°C to 85°C depending upon external temperature.

Main pool hall

The ventilation system is based on the wet and dry duct principle. Concrete supply air and extract ducts have been integrated into the structure, resulting in a visually duct-free pool hall space, leaving the roof structure in full view.

Supply air is input along the longitudinal edges of the space through slots designed to throw the warm dry air over the whole of the pool roof. Total supply air provides three air changes per hour. Return air at poolside level suppressed the high humidity zone to the lower levels (Fig. A).

A major development is that the wet duct has become a dry return air duct, with the pool water and air completely separate (Fig. B).

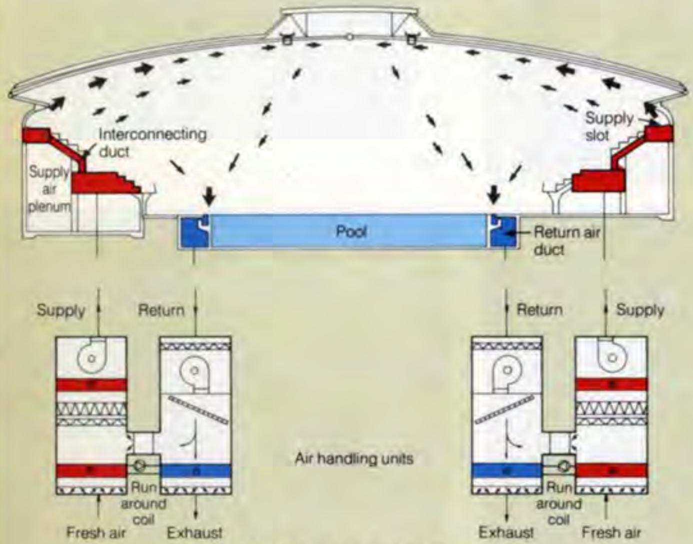
Smoke tests confirmed that the throw of supply air over the underside of the roof met all design expectations. A check on the humidity levels at pool side and in the return air duct showed no increase, thus proving that the new method of separation of pool water and air is successful.

Energy use is minimized by modulating the recirculation of the pool hall air and the use of a run-around coil heat recovery device on the fresh air inlet/exhaust air outlet sections of the air-handling units. Control is simple: sequence control of the run-around coil and main heating coil for temperature control, modulation of the fresh air/recirculated pool air for humidity control.

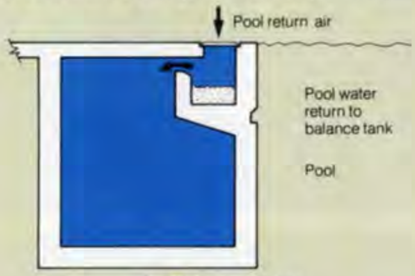
The lighting installation was required to cater for a wide variety of levels to meet the intended usage of the pool hall. Many discussions were held with the architects, sports bodies (FINA, Fédération internationale volley ball) and TV broadcasting companies to ensure that the stringent lighting and aesthetic requirements were met. Lux levels of 300, 500, and 1000 were provided for club, national, and international competition.

For televised events an illuminance of 1400 lux at pool level in the vertical plane facing the cameras is achieved.

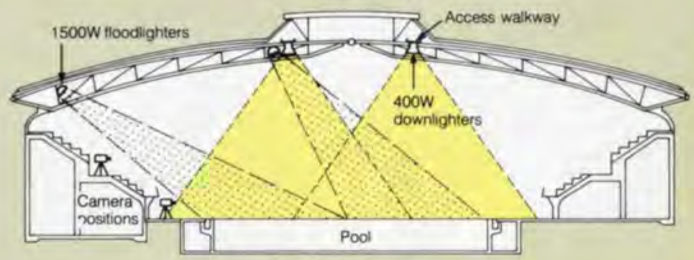
The luminaires comprise 400W metal halide downlighters and 1500W metal halide floodlights. The former are centrally positioned and provide the two lower light levels. Phasing in the floodlights provides the higher light levels including the all-important vertical plane illumination (Fig. C).



A. Pool hall ventilation.



B. Return air duct.



C. Pool hall lighting.

HV and LV distribution

The main incoming supply to the complex is provided by the Yorkshire Electricity Board, with a supply capability of 3.1MVA. The maximum demand is in the region of 2.9MVA, which will occur during the Games. From the HV consumers' intake panel there are two HV radial circuits which feed three 1250kVA dry cast resin transformers. The LV distribution network comprises two main switchboards feeding out to 23 switchrooms throughout the complex.

Fire protection

Due to the central core area of the building exceeding the compartmentation requirements, fire protection came under the close scrutinity of building control and the Fire Officer, who had a large influence on the extent of the systems provided.

The fire alarm and detection system provides comprehensive coverage to the whole complex, with manual break glass units and automatic smoke/heat detectors all linked to a central control panel in the Main Control Room. The building has a two-stage, two-phase evacuation system, with the alert and evacuate messages to each phase of the building being broadcast automatically by the PA system. Sprinklers for life safety are installed in high risk areas together with dedicated mechanical and natural smoke ventilation systems.

Public address

The public address system is split into nine zones. Facilities include eight multi-cassette decks, a pre-recorded message player and a master switching panel enabling any message or music to be broadcast to any combination of areas. Special features include:

- Audio induction loop for the hard of hearing in main pool hall, sports hall and function suites
- Sound reinforcement system within the main entrance, sports hall, leisure pool and main pool, allowing local use of the PA system for general commentary, underwater speakers, aerobics classes, etc.
- Integration with the drowning alarm system within the leisure pool and main pool
- Integration with the fire alarm system to enable alert and evacuate messages to be broadcast to all areas.

Arup Acoustics assisted with the development of the PA system, particularly in counteracting the problems of reverberation times within the main halls and Forum.

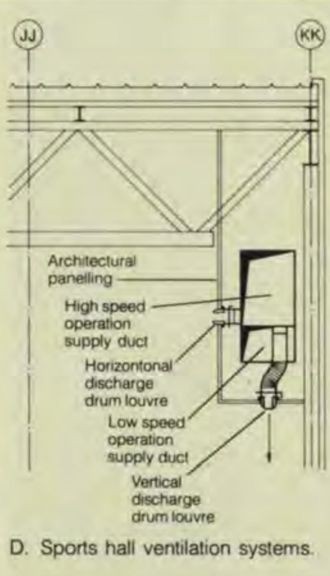
Sports hall

Important factors considered in the design of the sports hall ventilation system were the wide range of occupancy of up to 1800 spectators during events, low air velocities across the playing arena, maintaining the required unobstructed playing zone (40m × 40m × 12.5m), and the height of the hall.

Computer analysis showed that the requirement for cooling was marginal and would only be required for a very small part of the year. Therefore, mechanical ventilation only was provided. Two-speed fans were utilized which cater for the varying occupancy and allow energy saving in normal use: low speed providing minimum ventilation and heating for normal use (1.75 air changes per hour), high speed for events to provide free cooling (5.0 air changes per hour).

A 'duct within a duct' type distribution was adopted to cater for the low and high speed system, ensuring that correct air jet velocities were maintained from the inlet drum louvres. Low speed uses one section of the duct and 50% of the vertical discharge drum louvres. High speed uses both sections of the duct and all drum louvres including 10 horizontal louvres aimed over the spectator area (Fig. D).

Lighting is very similar to the main pool hall, comprising 400W and 1500W metal halide down and flood lighters, providing 300, 500, 750, 1000 lux levels and 1400 lux in the vertical plane for broadcasting.



D. Sports hall ventilation systems.

Energy management system

A project of this size and complexity demanded a very flexible controls system which could easily be adapted in use. A JEL *Jelstar II* system is installed, its main function being to control the HVAC plant and equipment. Many other facilities are linked, such as fire alarms, pool filtration monitoring, smoke control, lighting control, sprinkler systems monitoring, energy consumption, and blocked drainage sensors. Lighting control is via contactor switching on a simple time clock basis generated from the controls system.

23△

26

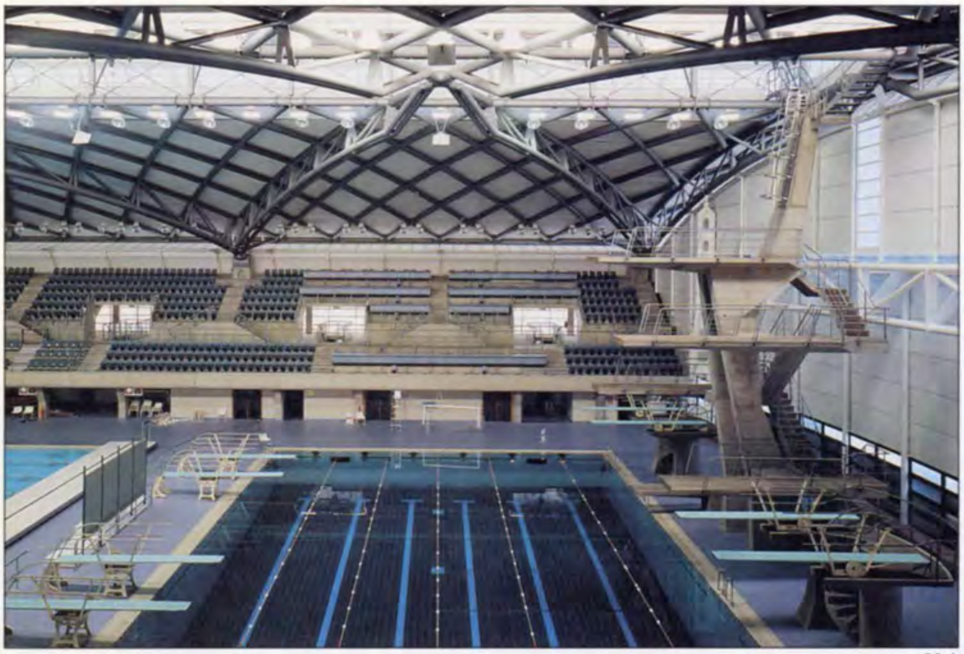
24△

25▽

27△

28△

29▽



30△

Conclusion

Already the competition pools are being heralded as world class. Prior to the Games, Ponds Forge hosted the European Diving Cup and World Swimming Cup. Three world records have been broken since the centre was first used at Easter 1991, an unprecedented event in Great Britain. HRH The Princess Royal opened the complex officially on 17 April 1991, and joined a capacity audience of enthusiastic parents and children at a Sheffield schools swimming gala. The omens all suggest both the complex and the Games will be a resounding success.

Credits

Client:
Sheffield for Health

Architect:
FaulknerBrowns*

Structural engineers:
Ove Arup & Partners Newcastle

Mechanical & electrical engineers:
Ove Arup & Partners Manchester

Quantity surveyors:
Gleeds

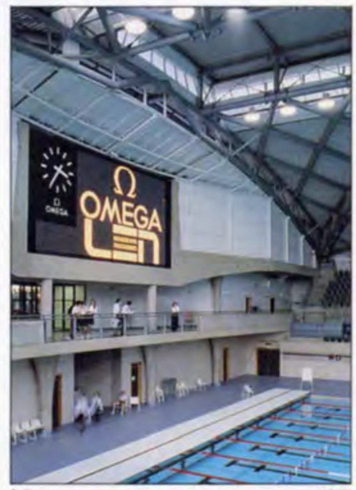
Management contractor:
Mowlem Management Ltd.

23. 28. 29. Rotunda entrance.
24. Leisure pool elevation.
25. Atrium.
26. Steelwork detail: leisure pool.
27. Pool hall elevation.
30. Diving pool.
31. 34. Pool hall.
32. Diving boards.
33. Scoreboard.

Photo credits:
1, 4, 18, 32: Jane Richardson
3: John Holt
7, 12, 13, 19, 20, 23, 25,26, 28, 30, 31, 33, 34: Peter Mackinven
11: Harry Sowden
14: R.J. Davis, Thomas Telford Ltd.
15: John Gregory
17, 24, 27, 29: Martin Butterworth
21, 22: T.O'Malley

31△

32△



33△

34▽

New Guesten Hall

Architect: Associated Architects

Gary Marshall

Background

In *The Arup Journal*, March 1969, John Martin wrote about the inception of the Avoncroft Museum of Buildings at Bromsgrove, Worcs., and in particular the 14th century Guesten Hall roof which it was then hoped would soon be re-erected over a purpose-built structure at Avoncroft. Now, 22 years later, and after languishing for 17 years under a temporary asbestos roof, the old timber trusses are at last in position over a new building.

Design concept

To echo the original Guesten Hall in Worcester, the Avoncroft building is to be a visitor centre, and will form the entrance to the Museum. The design parameters were dictated by the required use, by the need to display the roof adequately, and by the structural demands of the roof trusses themselves. Possible full-width entries at each end and inclined viewing panels necessitated independent side walls; the gable end framing offered several options but had to provide full width support against vertical loads and wind forces. The weight of the roof is considerable and the collar tie works as a compression strut rather than a true tie, providing the feet of the truss are restrained. In this form, the trusses perform satisfactorily, but if allowed to spread, the timbers are over-stressed and the joints suspect.

It was also considered desirable to use modern technology within the new building as a counterpoint to the old structure, whilst not detracting from it.

Design solution

The side walls, each 890mm thick and 6m high, were constructed of post-tensioned diaphragm brickwork. Utilizing some 46 *Dywidag* bars in each wall, post-tensioning proved to be a cost-effective solution. The bars were stressed to 75% of their final load, set eccentrically to counterbalance the roof thrust, before the bare roof was erected. The final stressing was carried out before the boarding and tiling.

The gable end framing is of traditional timber construction above eaves level, but is supported by a triangular section truss. This combines the provision of a wind girder with framing for viewing panels and gives a clear span between side walls. The truss is formed of timber members joined at nodal connections, its clean lines harmonizing well with the exterior of the building, and it provides an interesting comparison with the traditional trusses on view within. The trusses were manufactured by Mero to our design, but using their proprietary steel-to-timber connections. All glazing and doors below the truss may be removed in the future, depending on the growth of the Museum.

Conclusion

The New Guesten Hall was officially opened on 16 November 1990, marking the beginning of a new chapter in a long history. Ongoing is the extension to one side of the Hall, which will be built when funds permit; sponsors are being invited for the decorative corbels to the main trusses.

Credits
Client:
Avoncroft Museum of Buildings
Architect:
Associated Architects
Structural engineers:
Ove Arup & Partners Nottingham

Key data

Roof
Span of trusses: 10.56m
Truss centres: 2.43m
Number of bays: 8
Truss reactions: 44kN vertical; 30kN horizontal
Main rafter: 450mm × 180mm oak

Walls
Length: 24.18m
Thickness: 0.89m
Height: 6.45m
Prestressing: 46 28mm diameter *Gewi* bars giving 575T per wall

Hong Kong Park Aviary

Ian Robinson
Brian Forster

1.

Introduction

Ove Arup & Partners Hong Kong, in collaboration with the London Lightweight Structures Group, were appointed in 1988 as consultant structural and geotechnical engineers for the design of the Aviary, conceived as one of the major attractions at the new Hong Kong Park.

Although the Territory has relatively large areas of park, most are outside the residential and commercial zones, due to the demand for building space. Hong Kong Park, a joint development by the Urban Council and the Royal Hong Kong Jockey Club, is one effort to redress the balance. It is situated in the Central District of Hong Kong Island between Cotton Tree Drive and Kennedy Road, formerly the site of the Victoria Barracks. The Park will cover 100 000m^2 and include the Aviary, a greenhouse, a t'ai chi garden, lakes, a children's play area, and a visual arts centre, all surrounded by some of Hong Kong's most impressive buildings.

The concept

The client's original concept for the Aviary was indicative only: a 3000m^2 enclosure within which birds could fly freely in a sub-tropical environment. To achieve this the site was chosen carefully, a steep-sided valley, 62m wide at its broadest point, which twists in plan and whose sides fall at differing rates. It descends 12m through the 75m of the Aviary's length. A natural stormwater course passes through the valley, which is densely wooded with a variety of mature species, the tallest being approximately 13m. The client required most of those trees within the area of the new Aviary to be retained.

Preliminary studies by a Hong Kong-based architect proposed a conventional mast and mesh enclosure, typical of other aviaries throughout the world, with the public passing through on elevated walkways just below tree canopy level. The disadvantage with this, however, was that the construction would have necessitated the removal of many of the trees.

A new approach was therefore required.

The Aviary had to be seen as a landscape element and to this end its general form and shape were influenced by the site's natural landform. By spanning clear across with arches, the valley's shape in plan and section would be reflected by the structure. A surface of wire mesh suspended between the arches would create the enclosure. The retention of the mature trees, and a requirement that the perimeter-enclosing concrete kerb should follow the constantly changing ground level by little more than 0.5m, were now possible.

The mesh aperture size chosen by the client was particularly small, 12mm × 12mm compared to a more usual 60mm × 100mm. This size is not, as one would expect, to prevent birds escaping, but to stop rodent infestation, found to be a major problem in another Hong Kong aviary. This choice became the largest single influence on the scale of forces applied to the structure and foundations.

The design

Arups were given freedom to develop the whole concept of the enclosure and carried this out using a 1/100 scale topographic model. It seemed important that the perception of scale within the valley should continue to be by reference to the trees, so there are no columns. The use of simple, slender arches stops the structure and enclosure competing for attention.

Three major arches were used, stabilized by doubly-curved cable net surfaces stressed and anchored at points round the site boundary at ground level. The relative stiffnesses of arch and cable net influence the size of bending moments and axial forces which develop within the arches. The tubes comprising the arches had diameters of 560mm, 403mm and 323mm, giving span/depth ratios of 110. The grade 50C steel used was formed to smooth radii by heat induction bending. A fourth arch was later introduced at the head of the valley to accommodate the entrance lock. All the arches are supported by hand-dug caissons bearing on weathered granite. At the base of the three major arches, spherical bearings accommodate the rotations that can occur under wind loading and during construction.

The cable nets are built up from pairs of 14mm or 16mm diameter stainless steel cables running in two directions forming equal length quadrilaterals, each of which is uniquely skewed. Within each net the node-to-node length is a constant 2.4m, important for fitting the mesh to the cable net. The mesh wire themselves form a net of equal length quadrilaterals which are progressively skewed to conform to the cable net, made possible by using a woven mesh.

The cable pairs in both directions are clamped together at their crossover points by stainless steel castings with shaped grooves, to maintain the 2.4m dimension and provide structural loadbearing capacity between upper and lower cable pairs. One central bolt holds the two cable pairs together, permitting rotation during erection and in service.

Attachment of each cable pair to an arch is by a split casting incorporating a central eye bolt with loose cross pin and a radial spherical plain bearing. This pin enables equalization of load in the two cables.

The ends of each cable pair passing across the valley are equipped with tubular rigging screws to allow prestressing. This is required to ensure that no cables become slack as a result of movement during mesh installation or whilst in service. A cross-head links the two cables to a single eye-bar and via a bearing onto the circular baseplate. Holding down bolts to the baseplates are cast into concrete piers projecting from a continuous boundary wall, which are anchored into the bedrock by minipiles.

2.

2. Anchorage of cables passing across valley with rigging screws for prestressing.
3. Multiple cable anchorage.

4. Showing cable cross-over clamp, cable connection to arch and hanger bar system which connects cable net to mesh panels.

The mesh envelope was built up from individual strips of woven crimped stainless steel wire mesh, suspended below the cable nets and arches by a series of hanger bars which follow the longitudinal edges of the mesh panels. D-shackles connect the mesh to the hangers via reinforcing edge strips welded to the mesh panel, and are of a size to maintain a maximum gap of 12mm.

The structure was analyzed with FABLON, a non-linear space frame analysis program, on a full arch and cable •net model having 1200 nodes and 1800 elements. Stability analyses were also performed using FABLON. Wind forces were created using specially written subroutines allowing automatically for wind direction, surface inclination, drag coefficient, and distinguishing between windward and leeward positions.

The calculation of surface geomery was made using FABCAB, a non-linear tension structure program. One primary object was to control the skew angle changes within the cable net cells, another to define the precise points where the last cable elements cross the concrete and arch boundaries. The specific geometry of the boundary concrete wall was derived from land survey drawings using MOSS.

Erection

The valley depth, and restrictions on the removal of trees, initially appeared to rule out the use of scaffolding, except for supporting the arches during fabrication. At tender it was therefore envisaged that once the arches were in place and at least one cable present between arches, a system of pulleys could be used to pass the remaining cables across the valley and between arches. The mesh would then be erected similarly. In Hong Kong, scaffolding has a different meaning from many other places. Bamboo, light, strong, and flexible, is used without any planking and erected at great speed. This being the case, the contractor chose to scaffold the entire site, producing what appeared to be a mould for the ensuing Aviary envelope, whilst still allowing work at the valley floor to continue during erection.

5. Computer-generated image of aviary.

9.

The arches arrived in sections up to 6m long, and were butt-welded in position. At the same time the cables were being cut to predetermined lengths in matched pairs and the relevant half of the cross-over clamps and rigging screws fixed in position. These cables were then connected to the arches and each other in a slack condition. The cables had to be prestressed to ensure they would not go slack due to movements during mesh installation, and also in service. This was done cyclically, initially in three stages, tensioning the cables to approximately 15kN per pair by reducing in length those passing across the valley through the use of rigging screws swaged to their ends. Throughout this process the arches, although propped initially, were free to rotate; hence careful monitoring of arch location was essential. The simplest method used a series of plumbobs providing essential information, followed by a survey of points on the arches themselves.

With tensioning complete, sample cable forces were measured to check the accuracy of the analytical predictions. Minor adjustments corrected any unacceptable force distribution or out-of-tolerance of arch location.

The mesh installation also required tensioning to prevent 'flutter' at everyday wind speeds and to create a taut, smooth appearance. This necessitated two processes. The first was to pull the mesh taut with an applied tension of approximately 1kN/m at the boundary, and then clamp to the boundary wall. This took most of the sag out of the mesh panels. A slightly greater tension was required, and calculated destressing of some cables by increasing their length was used. The effect of this operation was to raise the cable net and hence the mesh. As the latter is clamped at the boundary wall and of fixed length, increased tension resulted.

The completed structure

Within the completed aviary there is a sense of space and calm. The intention was to design an unassertive structure, and although the largest arch rises 30m above the bottom of the valley, the aviary does not dominate the landscape.

After extensive planting, the site will be allowed to mature prior to the introduction of 150 species of birds indigenous to South East Asia's subtropical forests. Once the birds have become acclimatized, the Aviary will open to the public some time early in 1992.

Credits

Client:
Royal Hong Kong Jockey Club

Structural and geotechnical engineers:
Ove Arup & Partners

Architect:
Wong Tung & Partners Ltd.

Main contractor:
Dragages et Travaux Publics

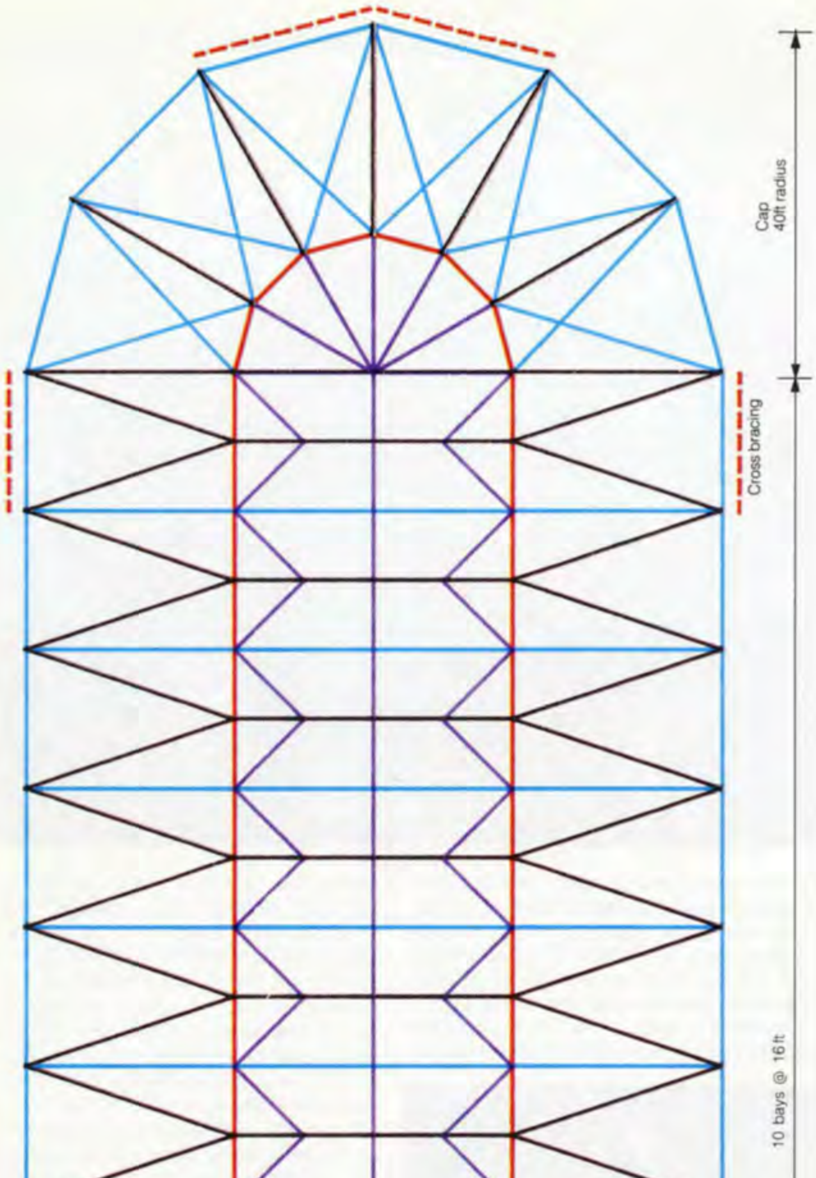
Erection contractor:
Kent Ho Engineering Works

Photos:
1-3, 6-10: Bird Wong.

Weatherstone Riding Ring

Architect:
Cooper Robertson & Partners

Guy Nordenson
Liam O'Hanlon



Introduction

Weatherstone Riding Ring is an indoor facility at a private client's country estate in Northern Connecticut. The building is 80ft × 200ft[1] in overall area with the roof rising from 16ft at the eaves to 40ft at the ridge. The ring itself forms an 80ft × 160ft rectangle plus a 40ft radius semicircle at one end.

Adjacent are stables for 12 horses and living facilities for the manager and groom.

The architect, Jaquelin Robertson of Cooper Robertson & Partners, sought a structure for the roof that was of a uniform density, rather than consisting of parallel trusses, arches, or portals. His preferred material was wood. In addition it had to accommodate a 4ft high clerestory all around, thus creating a 32ft wide gable about the ridge separated by the clerestory from a surrounding 24ft sloping apron.

The design of the structure was undertaken by the New York office of Ove Arup & Partners; the outcome of our deliberations with the architect consisted of a series of wood and steel trusses spanning the rectangular portion of the plan and a pair of half cones (apron and gable) at the end 'cap' (Fig. 1), organized on an 8ft module in all directions. Columns are double this height and the same distance, 16ft, apart, with the truss rising a further 16ft over the 24ft apron and 4ft to the ridge from the top of the clerestory (Figs. 2 & 3).

Trusses

The trusses are of a single two-directional geometry that has been, in effect, unfolded to obtain a partly three-dimensional system. The main structural action is nevertheless primarily in one direction, across the span. As shown in Fig. 2, the compression elements, of generally paired, glue-laminated timber, rise from the columns at an angle to meet the horizontal cross-members midway between the columns, directly beneath the clerestory. Pairs of tension rods extend directly between the columns. They are pulled up by single rods that come down 8ft from the compression member joint. These form a pattern of Vs along the length of the ring, under the clerestory. The ridge is then supported by saw-horse/A-frames over the horizontal cross-members that split at these members to rest on the tension rod joints. This last joint was nicknamed the 'stirrup' (Figs. 4 & 5). Finally, a series of light wires was extended from the joint of the A-frame and cross-member to hold up the double tension rods at mid-span.

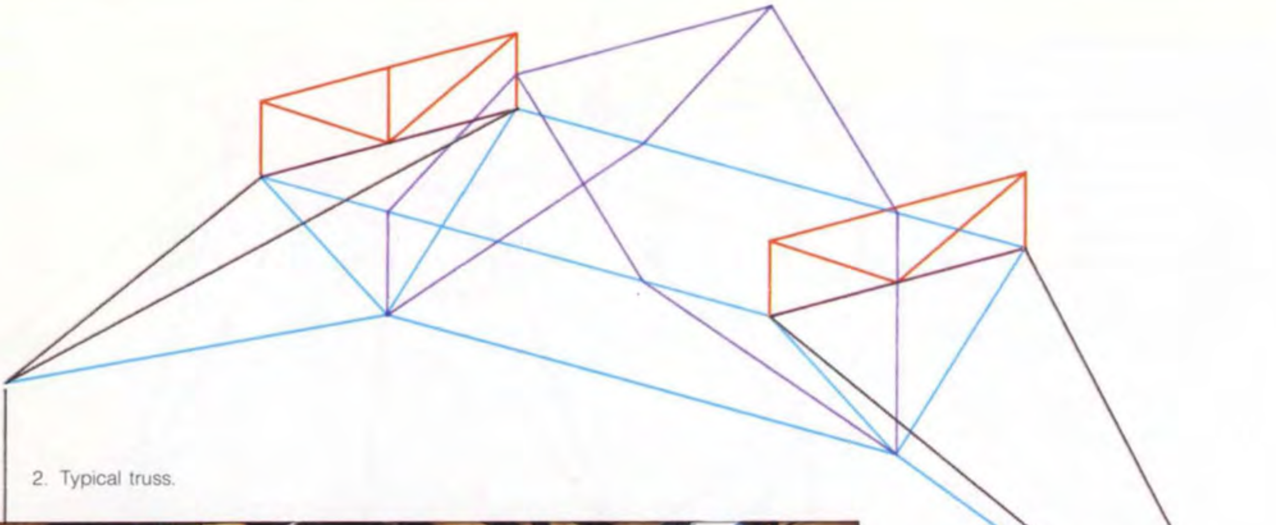
The connections are steel plates worked into the rather complex geometry by being hidden between the twin glulam members, their presence, extent and function revealed only by the bolt pattern (Fig. 6). Six joints are included in each typical truss, of which there are 10 in all.

Within the clerestory a truss of glue laminated timber and 3.5in diameter steel tubes serves as 'bridging' if the live loading were unbalanced, or if one of the trusses suffered damage (Fig. 7).

(1) As this project was designed in US units, these have been retained throughout.

Colour key for all diagrams
- Glulam
- Wire, steel rod or pipe members
- Clerestory or stiffening truss
- Gross bracing (rod)
- Ridge framing and diagonals (glulam)

1. Plan of roof structure.



2. Typical truss.

3△

4▽

5▷

3. Interior view looking toward front entry.

4. Tension chord and diagonal 'stirrup' connection.

5. Close-up of 'stirrup' connection.

6. Typical connection at top of columns.

7. Exterior view during construction with main and clerestory trusses in place.

6▽

7▽

The cap

To maintain the structure's uniformity, it was important that the scale and density of that part of the roof around the cap be constant. To achieve this the cap was treated as a framed half-dome relying on a tension ring of double tie-rods along the eave, and a compression ring consisting of the clerestory truss and an 8ft deep, lightly braced, glulam stiffening truss (Fig. 8). The latter was required to allow for unbalanced live loading that would cause bending in the compression ring. The top portion of the cap, above the clerestory, is a simple arrangement of ribs radiating from the end of the ridge towards the top of the clerestory (Fig. 9).

The effect of this approach was to allow the use of ribs similar in size to the compression members of the main trusses, and eliminate the need for a large truss across the end of the cap. The results were large tension and compression loads delivered to the eave and clerestory. On the eaves these are resisted by the tie rod bracing between the columns. The compression load to the clerestory truss also finds its way to the rod bracing via the connection to the compression glulam members beneath the apron. The tendency of the two apron portions to splay apart (in plan) as a result of these force couples is resisted by a tie between the columns embedded in the wall at the end opposite the cap.

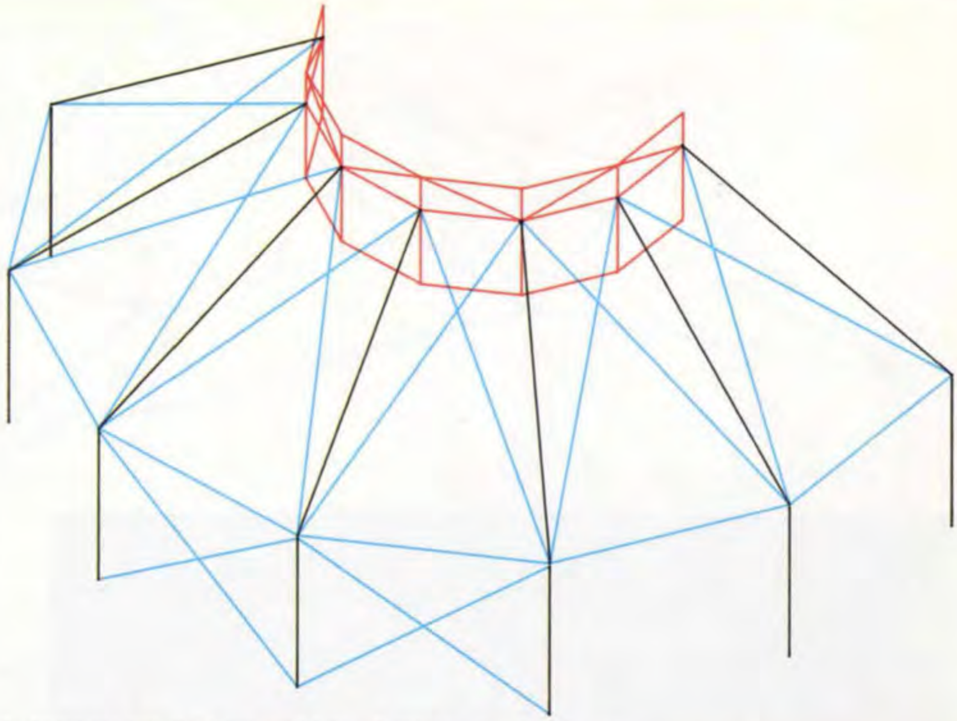
Stability

Careful evaluation of all extreme load conditions, including 100 year wind and severe unbalanced snow load cases, revealed that the tensions in the rods would not be overcome. Overall stability is provided by six bays of rod cross-bracing, two on each side and two at the cap (Fig. 1). The timber decking overlaid with ⅜in plywood and the compression members (forming a truss in plan) together act as a diaphragm to distribute the lateral loads to the wind bracing. The clerestory truss ties the gable roof to the apron diaphragm.

Under the effect of unbalanced snow loads or cross winds the main trusses will deflect, causing bending in the horizontal cross compression member (since the truss is not fully triangulated the light wires will relax). To resist this bending, a horizontal 'flitch' plate is inserted as an extension of the A frame/cross member joint.

Decking

The roof surface consists of tongue and groove laminate decking 3½in thick, overlaid with plywood, exposed on the underside. The spans are up to 16ft on the apron. The decking served a useful role, acting as a deep 'horizontal' beam along the aprons. This contributed to the resistance of the structure to vertical loads by distributing these to the end wall and cap bracing, and thereby serving as a kind of abutment for the trusses to arch.



8. Computer plot of the cap, and below 9. Interior view of end cap.

Conclusion

All the members and connections were fabricated off-site to the required dimensions. Tolerances were quite small. The structure was erected off a full scaffolding and was completed in 10 weeks. All the bolt holes were pre-drilled.

A sequenced pretensioning schedule was prepared in conjunction with the glulam fabricator and the contractor to obtain the final geometry after the shoring was removed. The geometry was achieved to within ¼in tolerance all round.

Credits

Client:
Private client

Architect:
Cooper Robertson & Partners

Structural engineer:
Ove Arup & Partners

Services consultant:
John L. Altieri Consulting Engineers, Norwalk, Connecticut

Main contractor:
Herbert Construction, New York

Glulam fabricator:
Unadilla Laminated Products, Unadilla, New York

Photos:
Courtesy the architects

10. Weatherstone Riding Ring, stables and living facilities.

Cerritos Arts Center, California

Architect: Barton Myers Associates

Chris Jofeh

1

Introduction

The Arts Center is the principal component in the 120-acre masterplanned development of Cerritos city centre. Currently under construction, with completion due at the end of 1991, it will provide a wide variety of performance areas, community meeting rooms and ancillary spaces.

Cerritos, a forward-thinking Southern California city with a population of 60 000, could neither afford nor support three separate theatres and an exhibition area. An early study by Theatre Projects Consultants had identified a need for three different spaces, suitable for music, drama and trade shows. This study showed how, using Northampton's Derngate Centre[1] as a model, the city could afford to construct a multi-purpose hall that would satisfy its cultural ambitions.

When it opens in 1992, Cerritos will have a facility that is unique in North America. Its technical innovations will make it one of the most sophisticated theatre spaces on the continent.

The timetable

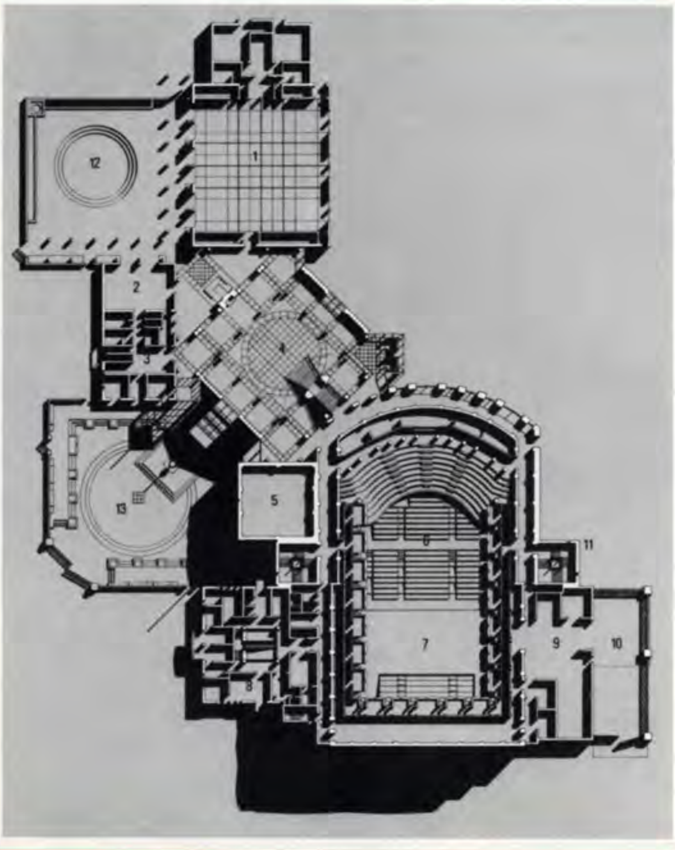
As is common in the USA, the design period was split into three distinct phases, with costings and formal presentations to the client at the end of each. Before a job goes out to bid (tender), it is subjected to a formal review process known as plan check. The Building and Safety Division of Los Angeles County Engineer-Facilities carried out this work on behalf of Cerritos, carefully reviewing the design drawings and specifications for code compliance.

Schematic design	March to Nov. 1987
Design development	Dec. 1987 to March 1988
Construction documents	March 1988 to Jan. 1989
Plan check Phase 1	Completed June 1989
Plan check Phase 2	Completed Jan. 1990
Bid	May 1989
Construction began	Sept. 1989
Flytower topped out	Jan. 1991
Auditorium topping out ceremony	May 1991

2. Plan showing the six major buildings.

Key

1. Large meeting room
2. Small meeting room
3. Offices
4. Lobby
5. Plant and services
6. Auditorium
7. Stage
8. Dressing rooms
9. Scenery store
10. Loading
11. Stairs
12. Poets' garden
13. Entry court



The design

As well as the Arts Center, the site will eventually contain an office park, an hotel, and a 75-acre regional shopping centre.

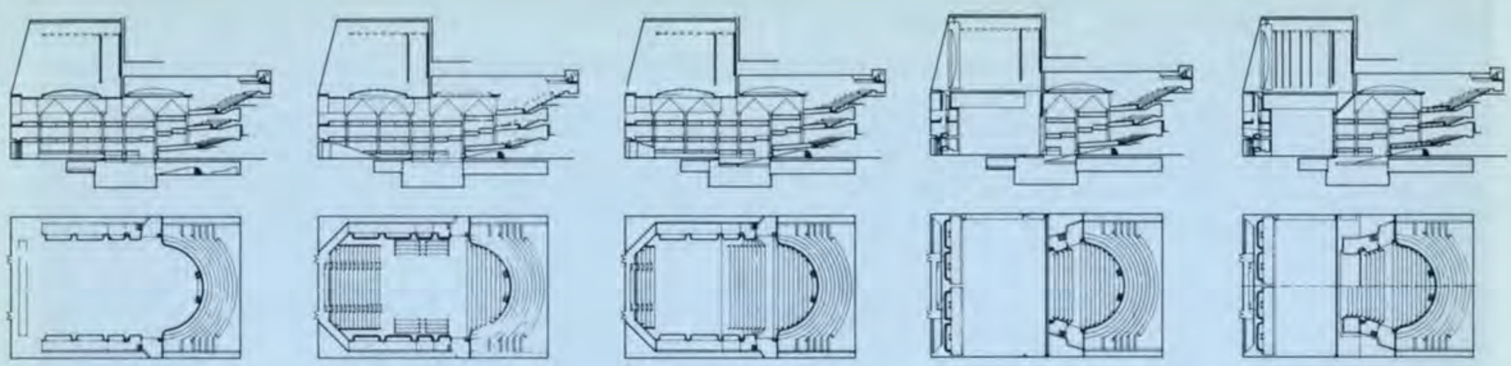
An urban context was thus missing, apart from some undifferentiated office blocks and an ocean of car parking. This was in contrast to architect Barton Myers' previous and highly successful theatre in Portland, Oregon, which occupied a very dense city centre site, sandwiched between two buildings listed on the National Register of Historical Buildings.

Thus the first challenge to the architects was to create a sense of place. This they did by arranging the Arts Center as a series of pavilions with gardens and courtyards. The benign climate allows designers this freedom, and the forms, materials and details used are rooted in the traditions of Southern California regionalism. Barton Myers had several images in mind as the massing studies progressed: the clustering of buildings in a Mediterranean village; Bertram Goodhue's flamboyant designs for the 1915 Panama-California Exposition in San Diego, now known as Balboa Park; and the spirited shapes, volumes, roofs and shadows of Los Angeles' chateau buildings.

A particular problem was how to deal architecturally, in a parking lot, with the large mass of the flytower. The solution was to cascade the buildings' roofs and towers down to garden walls, thus mediating to a pedestrian scale; the garden walls also insulate the pedestrians within from the cars without. The courtyards thus formed are well-suited for outdoor receptions. Each major building component has pyramid-shaped roofs and many also support tall flagpoles. There is much rooftop lighting and the Center, which is visible from the nearby freeway, will have a beacon quality (Fig. 1).

The architects believe very strongly that place itself is an important part of theatre-going, and have deliberately striven for a cheerful, welcoming, carnival-like building. To this end, much attention has been given to the external cladding. As well as a creamy white stucco, there is much banded stonework, in some ways reminiscent of James Stirling's Staatsgalerie in Stuttgart. Polished red granite and French limestone are used. The roofs are clad in highly coloured and patterned ceramic tiles, computer-designed specially for the project by April Greiman.

The six major buildings (Fig. 2) are the auditorium and sidestage; the actors' block; the lobby and box office tower; the large meeting room; the office block and small meeting room; and the mechanical tower.



A. Banquet, 6390ft^2 B. Arena, 1824 seats C. Concert, 1719 seats D. Drama, 972 seats E. Lyric, 1416 seats

3. The auditorium: showing the five basic configurations.

The auditorium

The heart of the project is the auditorium. It offers unparalleled flexibility, with five basic configurations (Fig. 3) transforming the auditorium's seating arrangement, sight lines and acoustics by the use of moveable seating towers, lifts, two proscenium lines (and two fire curtains), a flown acoustical concert ceiling above the stage, and a device known as the flipper.

The arena and concert configurations allow for centripetal viewing of a central stage. The lyric format is designed for opera and large-scale musical productions. The drama format is similar to the lyric, but uses the forward proscenium, a forestage (the orchestra lift), and closes the top balcony. The flat floor configuration houses community events, banquets, and trade shows.

Architecturally, Barton Myers had a choice when designing the interior of the auditorium: to celebrate the technology of the moving parts or to make each configuration look permanent.

He chose the latter.

At its largest, in concert configuration, the shoebox shape brings to mind the Musikvereinsaal in Vienna. The seating towers form traditional boxes from which one can see and be seen. The box fronts help define the volume of the room, which is richly finished in light ash, a darker stained ash, cherry and olive green painted steel. The seating towers are constructed of tubular steel frames and lightweight concrete on metal deck floors.

A challenge to the architects lay in the contrast between the proscenium configurations (drama and lyric) and the audience-in-the-round or flat floor configurations (concert, arena and banquet). In the former there are two distinct volumes, the auditorium and the stagehouse or flytower. In the arena and banquet configurations the occupants should feel that they are in one room. This is achieved in two ways. The first is by stepping the seating tower boxes (Fig. 4). This follows closely the rake of the two balconies so that there is no abrupt transition in the side seating as the audience wraps around from the edges of the balconies to the sides of the stage.

The ceiling treatment is the second device used to give the feeling of one room. Suspended within the flytower is a flown concert ceiling made of three large steel and laminated timber honeycomb panels. When not needed they are rotated to lie in vertical planes and hoisted electrically to the underside of the gridiron, an open steel platform high above the stage. For the concert, arena and banquet configurations, the panels are lowered into place, rotated and locked together at the same level as the lighting bridges and ceiling in the auditorium. In addition to its visual effect, the ceiling closes off the absorptive soft goods in the stage house, maximizing reverberation. Fig. 5 shows the flipper, an unusual and maybe unique item of theatrical hardware. An integral part of the fire separation between stage and auditorium, it is attached to the front proscenium and rotates on pivots to three positions: downwards (drama mode) creates a proscenium arch; angled (lyric mode) forms a second proscenium arch further upstage; whilst horizontal (banquet, arena and concert modes) links the concert ceiling to the auditorium ceiling. The flipper weighs approximately 27 000lb and is moved to its three preset working positions by an electric motor — horizontal to vertical in about two minutes. Tracking side panels suspended from the flying system adjust the proscenium width between lyric (45ft wide) and drama (35ft wide) formats.

Four lifts are used to reconfigure the floor, forming as needed a forestage (drama mode), an orchestra pit, or an extension to the orchestra level seating area. The lifts also support seating wagons which may be lowered to a storage area beneath the stage. A fifth lift at the rear of the orchestra level is used to exchange seating for an in-house sound mixing location.

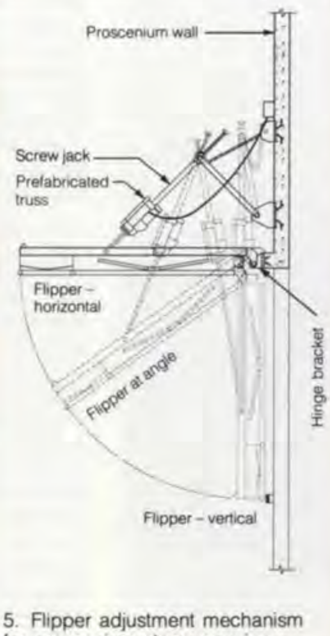
The seating wagons move on air-powered castors, as do the four clusters of seating towers: the auditorium side towers, the side stage towers, the proscenium towers and the rear stage tower. Microprocessors co-ordinate the movements of lifts and seating towers so that, for example, the side stage towers do not tumble into the orchestra pit.

The auditorium side towers (Fig. 6) are used in two positions, parallel with the auditorium walls in concert mode and rotated 15° inwards in lyric or drama mode. Movement is by electric winches. Hydraulic rams with steel pipe arms located between the backs of the towers and the side walls of the auditorium provide seismic restraint. The architectural transition is achieved by folding capitals.

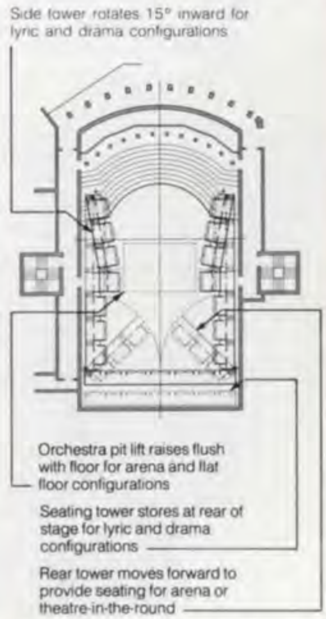
When the auditorium seating towers (150 000lb each) are rotated away from the side walls, hinged walkways known as jet ways link audience access doors to the rear of the seating towers. Raising, lowering and tracking of the jet ways is electrically powered, with an interlock system to prevent deployment at undesirable times.

The proscenium (43 000lb) and side stage (140 000lb) seating towers are moved manually or by electric tow tractor on air castors between floor lock down positions (two for the side stage towers and four for the proscenium towers), which provide seismic restraint.

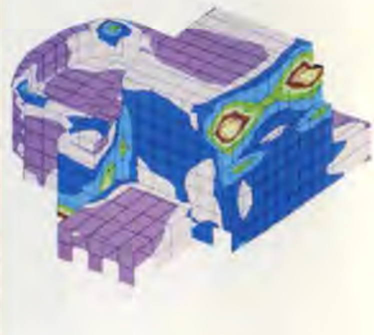
4. Stepped seating tower boxes.



5. Flipper adjustment mechanism for proscenium stage opening.



6. Seating tower movement patterns.



7. Analysis of dynamic model.

The rear stage seating tower (225 000lb) is much shallower front to back than the other towers, and requires stabilization even when in transit. It is moved by an electric winch that hauls cables running beneath the stage floor. From each tower base eight T-bar guides travel in floor slots and lock into position when it is stationary. Power, lighting and sound are delivered by cables supported by a cantilever arm pivoting from the rear wall.

Acoustically, the auditorium is transformed by moving parts that change its volume and the reflectivity of its surfaces. The moveable concert ceiling has already been mentioned; a free-standing orchestra shell may also be used to reflect sound towards the audience. Provided within the auditorium are acoustic banners and curtains which are intended to adjust the reverberation to accommodate a variety of uses and occupancies. Electrically operated from a control panel in the sound room, the banners retract vertically into pockets beneath the ceiling line. One set of curtains retracts horizontally into pockets within the roof void, whilst curtains at the rear of each box withdraw into the columns between boxes.

Curved concave surfaces such as balcony fronts are broken down to a finer scale whose convex curvature and hard wooden surfaces help scatter sound and thus avoid unwanted focussing.

The boxes have three principal planes: the front, which visually defines the volume of the auditorium; the back, a perforated metal screen acoustically transparent at the upper boxes but reflective near the stage to support the musicians; and the rear of the tower, which is similarly treated. Low frequency sound passes through the screens, bounces off the side walls of the auditorium, and re-enters the room. High frequency sound is reflected and scattered by the wooden panels on the box fronts. To preserve low frequency energy, the walls and roof of the auditorium and flytower are massive and reflective, either reinforced concrete or reinforced solid-grouted masonry. This posed major structural challenges both for design and erection.

All public bathrooms are in their own seismically and acoustically separated box, with a floating slab on a sandbed, at ground level adjacent to the curved rear wall of the auditorium. All associated pipe and ductwork is vibration-isolated. All major plant is separately housed in the mechanical tower, seismically and vibration-separated from the auditorium structure.

The auditorium contains sophisticated sound and lighting systems: theatre sound; technical intercom; paging and show relay; video; a master antenna television system; control units containing communications facilities for stage management and technical usage; a hard-of-hearing system; and a portable sound system. The theatre is one of only two in California designed to be used for major recording sessions. Stringent criteria were applied to the design and detailing of the air distribution system. More than 750 dimmers are used to control the lighting.

The lobby

Deliberately set off-axis, the lobby is much more than a place of assembly and dispersal. Its three upper floors overlook an internal courtyard which can be used as a performance space. Leading up to circulation spaces behind the auditorium balconies is a monumental staircase of cast in situ concrete, the set piece of the lobby, whose roof is just a little lower than that of the auditorium. Attached to one corner of the lobby is the 60ft tall steel and glass box office tower.

Cost cutting reduced the lobby space, but it is still generous when compared with those of, for example, New York's Broadway Theatres. The mild Southern California climate allows the gardens and courtyards to be used as extensions of the lobby. From outside it can be approached in opposite directions: by car or by foot via a walled entry court from the parking lot.

The large meeting room

A tall, single-storey, column-free area, this can accommodate up to 400 for banquets. A grid of tracks for operable partitions allows it to be divided into as many as five individual, sound-isolated meeting rooms. The room is theatrically equipped with lighting and sound systems, rigging trusses, self-climbing chain hoists, draperies, a portable stage platform and a portable dance floor. A telescopic seating unit provides seats for 160. The room can be used as a rehearsal space while the main auditorium is being used, as well as by small theatre groups or musical ensembles. Being close to the hotel, it can serve as an extra conference area. A two-storey wing houses a kitchen and some bathrooms.

Office block and small meeting room

This small, two-storey building for the Center's administrative staff opens onto a walled garden which it shares with the large meeting room.

The mechanical tower

This is a five-storey, square building housing chillers, boilers, fans, cooling towers and air-handlers. The concept of a completely separate, mechanically dedicated structure springs from the need for acoustical isolation and the architectural desire to avoid a single monolith. Problems of air intake, exhaust, access to equipment and acoustical separation are all solved by the presence of the mechanical tower. Its mass, taller than everything except the auditorium and flytower, and topped by four small pyramid roofs and a flagpole, is an essential part of the cascade of roofs and towers and shadows that help so large a theatre sit comfortably in its surroundings.

Engineering

In California matters are arranged a little differently from the UK. Firstly, most of Southern California is in Seismic Zone 4 as defined by the Uniform Building Code. This specifies substantial lateral forces for which buildings are to be designed. Secondly, the geotechnical investigation and the selection of the type and capacity of the foundations are the responsibility of independent geotechnical engineers.

Foundations

The soils investigation by Moore and Taber, geotechnical engineers, revealed silts, clays and sands, with a design ground water level only 4ft below the orchestra slab on grade. The recommended foundations for all buildings were driven prestressed precast concrete piles 14in square, varying from 20ft to 60ft in length. The geotechnical engineer designed a permanent pumped dewatering system for the site, but nevertheless the piles beneath the deep orchestra pits were designed to cope with full hydrostatic uplift minus the weight of the pits, in case the dewatering pumps failed.

Seismic design concept

The Center is arranged as a series of five seismically separated structures. As a result, each building apart from the auditorium has a regular layout of its lateral system, and could therefore be designed using the static lateral force procedures of the Uniform Building Code. The separations also enabled each building to have a lateral system compatible with its function. For example, the mechanical tower uses a steel ductile moment frame which allows air flow through the exterior screen wall to the air-handling units, and permits easy installation or removal of large equipment; the auditorium shear walls provide lateral strength and stiffness at the same time as acoustical mass.

Auditorium seismic design

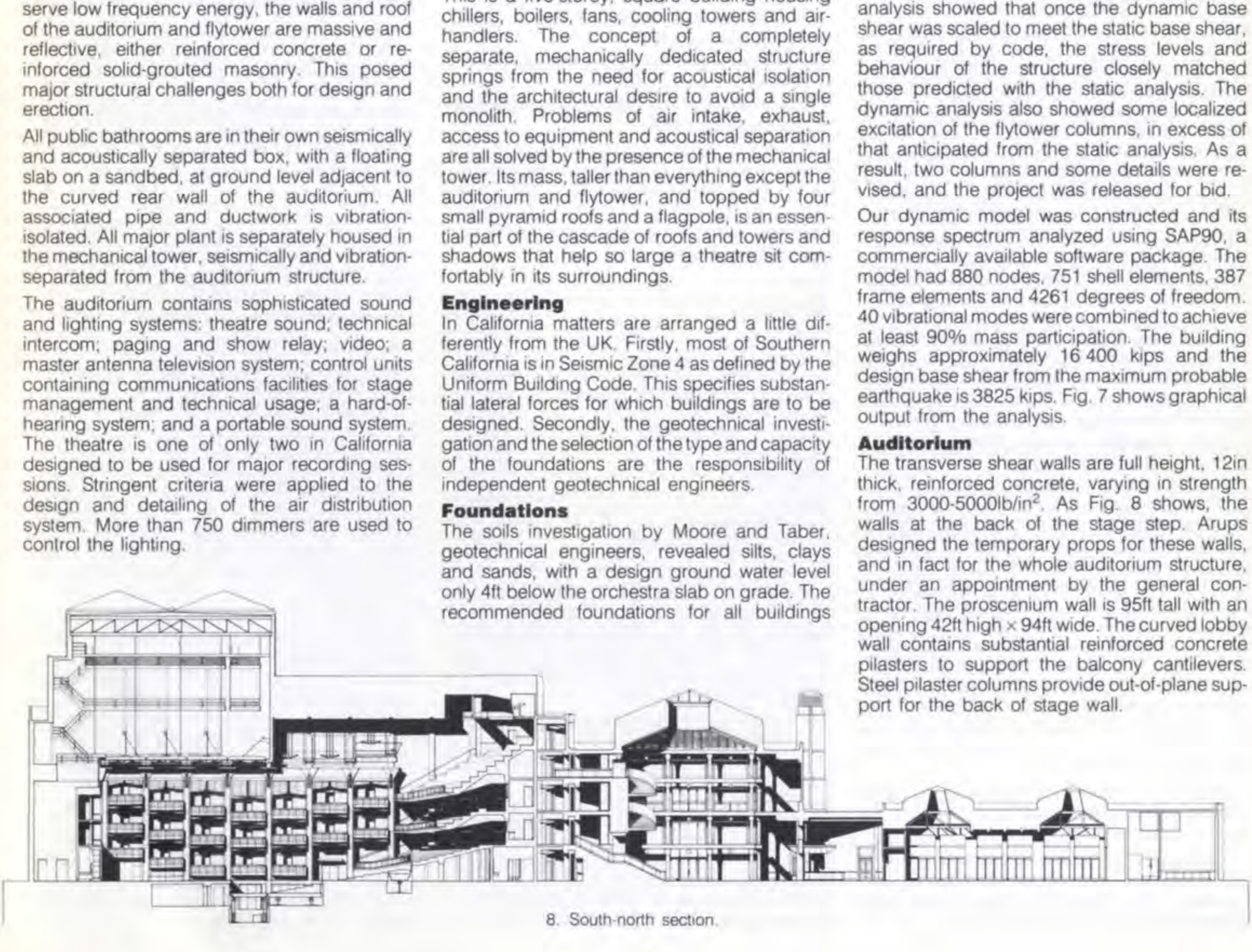
Prior to the 1988 Uniform Building Code, the requirement for dynamic analysis was determined by the building official who had jurisdiction. For this project it was agreed during the schematic design phase that, due to the building's box-like nature, it could be designed using equivalent static lateral force procedures and the most stringent K-factor, 1.33. The building could then be designed for an assumed distribution of mass.

When the building was submitted for plan check a year later, the 1988 Uniform Building Code had just been issued, and under its provision a dynamic analysis was required for the auditorium building. The owner then authorized Arups to carry this out, to verify the adequacy of the original static analysis design. Our analysis showed that once the dynamic base shear was scaled to meet the static base shear, as required by code, the stress levels and behaviour of the structure closely matched those predicted with the static analysis. The dynamic analysis also showed some localized excitation of the flytower columns, in excess of that anticipated from the static analysis. As a result, two columns and some details were revised, and the project was released for bid.

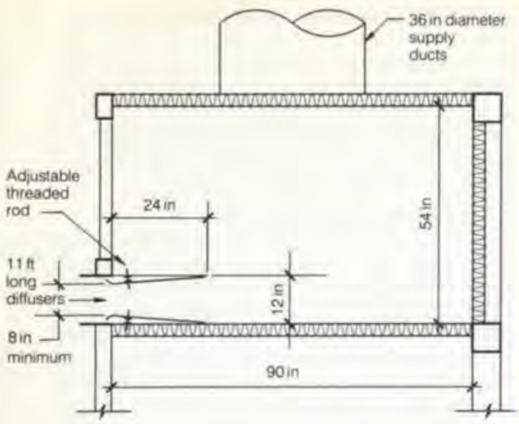
Our dynamic model was constructed and its response spectrum analyzed using SAP90, a commercially available software package. The model had 880 nodes, 751 shell elements, 387 frame elements and 4261 degrees of freedom. 40 vibrational modes were combined to achieve at least 90% mass participation. The building weighs approximately 16 400 kips and the design base shear from the maximum probable earthquake is 3825 kips. Fig. 7 shows graphical output from the analysis.

Auditorium

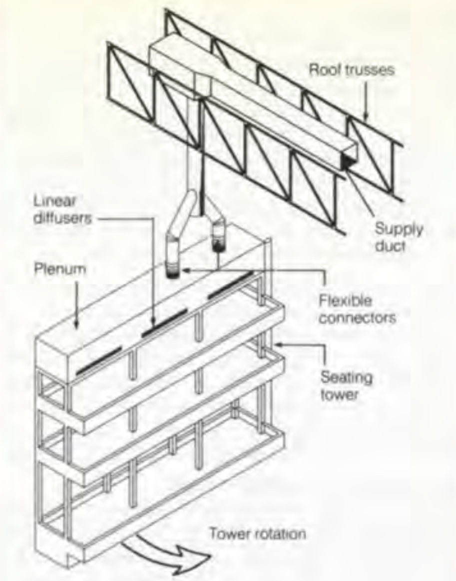
The transverse shear walls are full height, 12in thick, reinforced concrete, varying in strength from 3000-5000lb/in^2. As Fig. 8 shows, the walls at the back of the stage step. Arups designed the temporary props for these walls, and in fact for the whole auditorium structure, under an appointment by the general contractor. The proscenium wall is 95ft tall with an opening 42ft high × 94ft wide. The curved lobby wall contains substantial reinforced concrete pilasters to support the balcony cantilevers. Steel pilaster columns provide out-of-plane support for the back of stage wall.



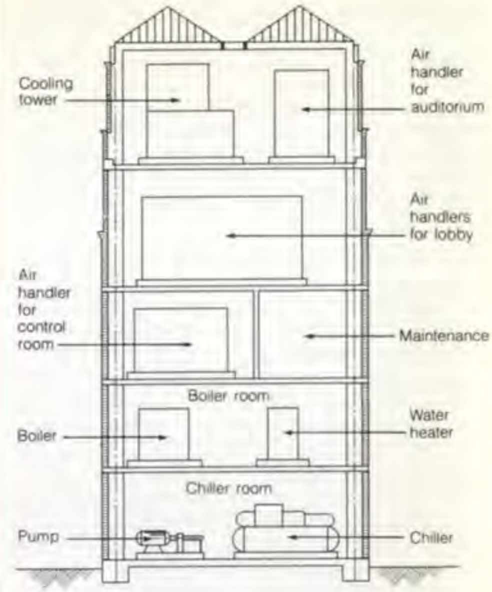
8. South-north section.



9. Vertical section through auditorium seating tower plenum and diffuser.



10. Air supply to auditorium seating towers.



11. Mechanical tower showing equipment on the five different levels.

The longitudinal auditorium walls began life as reinforced concrete with steel pilasters but were cost-driven to reinforced masonry with embedded steel pilasters for out-of-plane support of the wall and gravity support of the main steel roof trusses. Erection of a composite masonry and steel wall has been a challenge for the general contractor and his subcontractors.

The three warped and curved balconies are supported by deep radial, sloping, tapering, curved beams that cantilever from the rear wall pilasters. The balconies are cast as dished slabs onto which steps are later cast, in order to achieve the best control of final balcony elevations for the seating sight lines.

The auditorium and flytower roofs are supported by steel N-trusses. The flat flytower roof, of hardrock concrete on metal deck, has 6ft deep trusses spanning from proscenium to rear stage wall. Suspended from these trusses are the gridiron, three levels of catwalks, and the loft blocks. The headsteel spans 72ft, supporting 191 kips horizontally and 231 kips vertically. Most of its load comes from the pulleys over which the scenery counterweight cables run.

The vaulted auditorium roof, of lightweight concrete on metal deck, is carried by 12ft deep trusses spanning from sidewall column to sidewall column. Their bottom chords support 4in thick precast dense concrete ceiling panels. These trusses also support the lighting bridges and architectural ceiling, whose shape followed both acoustical and lighting considerations. Steel pipes and angles and wooden panels are used in both bridges and ceiling panels.

Lobby, meeting rooms, office block, and mechanical tower

The lobby is a steel-framed structure, at its lower levels engaging the auditorium, office building and large meeting room. Above, it is approximately 80ft square. Its perimeter moment frame is designed as a backup to the monumental concrete stair which acts as a very stiff cantilevering shear wall — thus attracting most of the lateral load in an earthquake. A two-way steel truss carries the roof. The moment-framed 60ft × 10ft × 10ft structure of the box office tower was fabricated in one piece off-site. It has a specially detailed sliding connection that permits earthquake-induced movement independent of the lobby in one direction while linking them perpendicularly.

The 80ft-span steel roof trusses of the large meeting room are supported by perimeter shear walls of reinforced masonry.

The office block, like the large meeting room, has reinforced masonry shear walls. Its suspended floor and roof are steel-framed, with lightweight concrete on metal deck.

The 28ft square × 72ft tall mechanical tower has lightweight concrete on metal deck floors supported by a steel frame. The lateral system is a perimeter ductile moment frame. It is clad with a screen wall of reinforced masonry panels made of square blocks with openings in them.

Mechanical systems

The special features are:

- The stringent acoustical design criteria
- The flexible ductwork for the moveable seating towers
- The purpose-built supply plenum diffusers in the auditorium, stage and large meeting room (Fig. 9)
- Compliance with California's strict Title 24 energy requirements
- The seismic anchorage and restraint needed to deal with Seismic Zone 4
- The structurally separated mechanical tower
- The stage sprinklers.

The acoustical performance criteria were specified by Kirkegaard Associates in terms of room criteria (RC) and preferred noise criteria (PNC) and are listed in Table 1. Both RC and PNC are sets of curves on graphs of sound pressure level versus octave band.

Table 1

Performance space	*Acoustical criterion*
Audience and stage areas	PNC-15
Control rooms: audio	PNC-20
Control room lighting and projection	RC-25
Orchestra pit	PNC-15
Underseat storage areas	PNC-20
Dressing rooms	RC-30
Lobby areas	RC-25
Meeting room	PNC-20
Kitchen	RC-30
Administrative areas	RC-35
Spaces opening onto stage and into house	PNC-20

Kirkegaard recommended, for acoustically sensitive spaces, low velocity, low pressure air distribution from remotely-located air handling equipment. Constant volume systems were preferred over variable volume. Ducts are lined and provided with silencers.

The lowest level of the mechanical tower houses two centrifugal chillers, each with a cooling capacity of 273 tons, specially modified to use ASHRAE *Standard R-123* refrigerant. This is a chlorofluorocarbon, but with a hydrogen atom replacing one of the fluorine or chlorine atoms in its structure to make it less stable, so it will break down in the lower atmosphere and not get to the stratosphere. Chilled water is supplied at 44°F and returns at 56°F. The chillers are served by two centrifugal counterflow cooling towers at the top of the mechanical tower. Water enters at 95°F and leaves at 85°F.

Hot water is supplied by two natural gas-fired boilers. Supply temperature is 185°F and return temperature 165°F.

The auditorium is served by a constant volume, single-zone system with supply and return fans. Air is at high level from specially designed sidewall plena diffusers and extracted low down the auditorium sides and beneath the balconies.

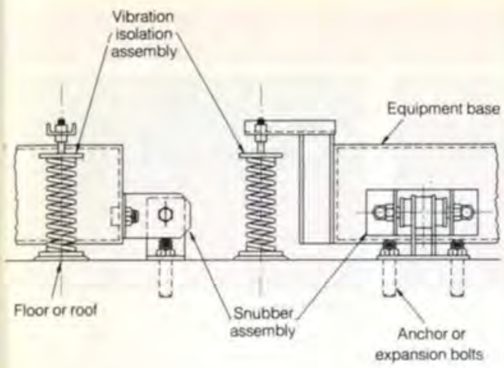
Large supply and return ducts link the air handlers in the mechanical tower with the roof space above the auditorium ceiling, where ductwork is carefully integrated with roof trusses.

Each group of supply and extract terminations is fed from the main air-handling units via a separate, self-balancing, ductwork system with no dampers. All auditorium air termination devices are specially designed to provide the required air distribution with minimum generated noise. Fig. 10 shows the folding and extending ductwork for the auditorium seating towers.

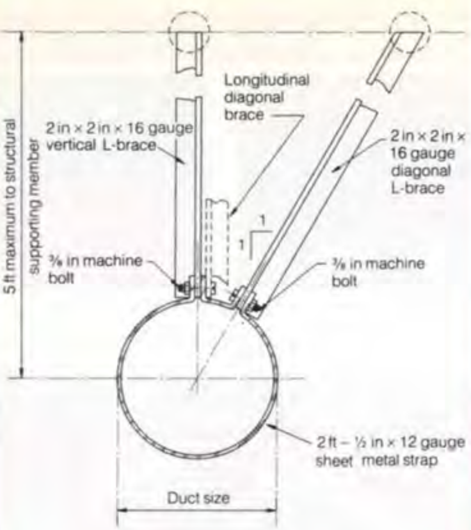
The lobby also uses a constant volume, single-zone system with supply and return fans. The meeting rooms use a multizone, constant volume system with supply and return fans. The office building uses a variable volume system. The mechanical tower is shown in Fig. 11.

Title 24 of The California Administrative Code contains limits on the energy consumption of buildings and rules for calculation. This was not written with theatres in mind, and so interpretation is a matter of negotiation between the designers and the plan checkers. It sets minimum standards for wall and roof insulation, for window insulation and shading coefficients, and for equipment efficiencies. Arups calculated the annual energy consumption of the buildings using industry-developed energy simulation software approved by the California Energy Commission.

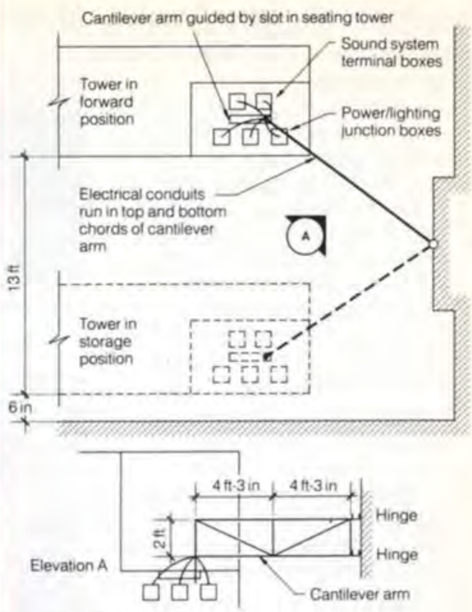
To deal with possible earthquakes, all mechanical equipment is seismically anchored. Some can simply be bolted down. Other equipment, supported on vibration isolation, is restrained by devices that limit its travel when the ground shakes (Fig. 12). Seismic anchorage of pipes and ducts is largely but not entirely covered by guidelines published by SMACNA (Sheet Metal and Air Conditioning Contractors National Association). Fig. 13 shows a typical duct seismic restraint, the design of which is generally the responsibility of the mechanical subcontractor. At the seismic separations between buildings, special connections are provided to allow substantial relative movement caused by earthquakes.



12. Vibration-isolated equipment restrained by snubbers.



13. Typical bracing for 28 in diameter and 36 in diameter ducts.



14. Flexible conduits for power, sound and lighting to towers.

In California, detailed design of sprinklers is carried out by specially licensed contractors, working to the designers' performance specification. Fire suppression and smoke extraction of the stagehouse received a lot of attention from the City Fire Marshall. The agreed scheme has gridiron level deluge sprinklers, low level, high velocity horizontal throw sprinklers on opposite sides of the stage, smoke extract vents in the flytower roof, and forced smoke extraction via low level ductwork when the concert ceiling is in place.

Electrical systems

The Arts Center receives a 4000A, 277/480V three-phase, four-wire service from the Southern California Edison Company. The main electrical room is at the lowest level of the mechanical tower, acoustically isolated from the auditorium. From the main switchboard, power is distributed via transformers to two distribution switchboards at 120/208V. The first switchboard is in the main electrical room, the second on an isolated slab in the dimmer room of the theatre. The main circuit breakers are rated at 1600A and 3000A respectively. Flexible connections limit noise transmission from the transformers. Panelboard locations were carefully chosen to meet the differing needs of architect and electrical engineer.

As with the mechanical systems, it was necessary for the electrical design to show compliance with the energy conservation requirements of Title 24. Based on work by the Illumination Engineering Society, Title 24 relates allowable power consumption to functional categories and is very task-oriented. Theatrical lighting exists as much for art as for the carrying out of tasks, and Title 24 thus is difficult to apply to theatres, with interpretation again negotiated by designers and plan checkers. A major collaborative effort by Arups as electrical engineers and Theatre Projects as work, house and production lighting designers was required to iterate the design towards compliance.

Power, sound and lighting for the moveable seating towers are supplied either via flexible conduits (Fig. 14) or by unplugging, moving the towers, and plugging in again.

Some theatrical productions generate smoke on stage, and to prevent that from activating the smoke detectors at high level over the stage, there is a bypass switch, operated by the stage manager, which shuts down the detectors for four hours.

Emergency power is provided by a 400kW generator.

Lighting design

Theatre Projects and Barton Myers developed the lighting concepts for the main public spaces, whilst Arups incorporated the lighting within the total electrical design of the Arts Center. Many of the public areas incorporate timers as part of their lighting controls.

In the theatre, lighting falls into four categories: house, concert, stage and work lighting; all controlled via 766 dimmer channels, all with full memory, from the main console at the rear and from local panels around the stage and auditorium. The large meeting room has 60 control channels for 192 dimmers.

Conclusion

Construction is now well under way (Figs. 15, 16, 17). When complete, Cerritos will have a world class, multi-use facility to mark another stage in the city's development. 35 years ago Cerritos was best known for its dairy farms: today it is for its regional shopping centre and auto sales. Soon it will be a centre of excellence for the performing arts.

Reference

(1) REID, F. Municipal multi-form: Derngate Centre Building Study. *Architects' Journal*, *179*(10), pp.49-64, 7 March 1984.

Credits

Client:
City of Cerritos Redevelopment Agency
City Project Manager:
Kurt Swanson
Theatre manager:
Phill Lipman
Architect:
Barton Myers Associates
Structural, mechanical, electrical and public health engineers:
Ove Arup & Partners California
Theatre consultant:
Theatre Projects Consultants
Acoustician:
Kirkegaard & Associates
Landscape architect:
Burton and Spitz
Graphic/interior designer:
April Greiman
Cost consultant:
Vermeulen and Logg
General contractor:
SAE/Continental Heller Construction
Project management consultants:
Stegeman and Kastner
Photos:
15, 16: Barton Myers Associates
17: Otto Jensen
Drawings
1-4, 8: courtesy the architects
5, 6, 9-14: Ove Arup & Partners

As this project was designed in US units, these have been retained here.

1 kip = 1000 lbf ≈ 4.45 kN

15. 16. 17. Cerritos Arts Centre under construction.

15.

16.

17.

DESIGN FOR HAZARD: FIRE

Introduction

Two of the papers already published in this series have dealt with wind and earthquake design. Both are specializations which have grown out of the basic language of structural engineering and as such needed only limited introduction. Fire engineering is a little different: an embryonic discipline beginning to have an increasing impact on building design, drawing on a very broad base of engineering disciplines.

In order to illustrate the state-of-the-art position, we have chosen to discuss projects we have been involved with which fall outside the normal categorization of buildings and occupied structures as envisaged in the Building Regulations.

Transport terminal buildings, with potentially long escape routes in large volume spaces, demand an understanding of fire development and smoke spread in order to keep people safe as they move away from any fire incident before it is fought.

Offshore structures, particularly in relation to the safe use of combustible materials, require an understanding of how to restrict fire growth and keep operatives safe until a means of rescue arrives, possibly after the fire is extinguished.

These situations need quantifiable understanding of the physics of fire applied to the structure, service systems and functional planning of the buildings and installations involved. They also require an understanding of human reactions, pedestrian traffic engineering, and the pre-planned response of the emergency services in order to complete equations of safety for personnel and (where significant) property.

The essence is to quantify and make sound engineering judgements about the particular objectives of designing against the effects of fire. It may be surprising to realise that many of the fire safety-related aspects of the building regulations are subjective and do not necessarily have an engineered basis. As we continue to develop our discipline, especially in areas outside the scope of current regulations as described below, so we will be better able to influence on a rational basis the design of all buildings to make them safer.

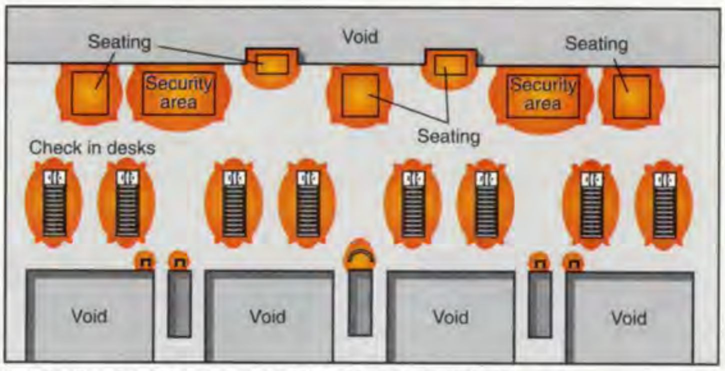
Transport terminals

We have contributed fire safety advice to a number of projects concerned with transport terminals. These buildings have features in common, and the problems which have to be tackled illustrate how an integrated approach may be used to develop a soundly based strategy. A number of calculation techniques come into their own, both simple and sophisticated, and covering a range of technical disciplines.

In order to ensure the convenience of large numbers of people using a terminal building, it is frequently necessary to have very large public spaces. This removes the usual protection of fire-resisting compartment walls to limit fire and smoke spread. In the event of fire, people may have to move long distances across possibly unfamiliar territory in order to reach a place of safety. In international terminal buildings the escape procedures may be complicated by the need to maintain the landside/airside boundaries, and the problems of communicating in foreign languages. However, on the positive side, transport terminals are designed specifically to promote a smooth flow of people. There tend to be open spaces for circulation which are largely free of combustible material. In addition there may be a high ceiling over all or much of the terminal which can act as a smoke reservoir.

Smoke spread as escape starts — Further spread with time — Last person out

1. Stansted Airport: combined smoke spread and people movement modelling.



2. A simplified plan of Kansai airport check-in concourse: shaded areas indicate heat flux >20 kWm² assuming a fully developed fire in each area.

It is the task of the fire safety engineer to exploit the positive features of this kind of building and to propose additional fire protection measures which will ensure that people are at no more risk in the proposed building than in one that can comply with the letter of the regulations.

Stansted Airport

The recently opened new terminal building at Stansted Airport (see *The Arup Journal*, Spring 1990, pp.7-15) provides a good example of how a range of techniques may be brought together to form a coherent fire safety strategy. It was decided that people should be protected from the effects of a major fire as they made their escape, by fitting all the areas of high fire load (for example shops) with a local roof to carry sprinklers and a smoke extraction system. The required extract volume for smoke was calculated using standard techniques based on the volume of air entrained into a hot plume, and assuming that all the heat from the sprinklered fire goes into the hot smoke layer. The use of this conservative approach ensured that smoke would be contained within a limited area, even without fire-resisting walls. This was termed the Cabin Concept.

Outside the Cabins, an unsprinklered fire might occur in seating or in passengers' baggage. The ceiling of the building is some 12m above the floor and it was argued that this provides a reservoir to keep smoke above head height in the event of a fire in the circulation or seating areas.

To study this in detail, a computational fluid dynamics (CFD) analysis was carried out to assess smoke flow across the ceiling. This technique divides the three-dimensional space under consideration into a large number of cells. The mass, momentum, heat and turbulence conservation equations are solved for each cell for each short time step. By considering a fire suddenly starting at a given point in the building it is possible to study the flow of smoke away from that point as a function of time.

Calculations were additionally carried out to assess how people might evacuate the space, based on measured walking speeds as they moved through an airport baggage reclaim area. The average speeds, around 1 m/sec, were in accordance with those quoted from a number of studies elsewhere. The flow of people through the building exit doorways, as a function of time after escape started, was estimated by assigning the measured distribution of speeds to the maximum population within the building.

By combining the smoke modelling with the people flow modelling, it was possible to demonstrate that people would be able to move freely over the escape period with large margins of safety, with use made of graphic display techniques to visualize the movement of people compared to the flow of smoke. Fig. 1 shows how people moving towards an exit might appear to an observer close to the fire after two minutes.

This was almost certainly the first time that a CFD analysis had been used in demonstrating fire safety in a building, and with current advances in computing speed and graphic output this is a technique which has enormous potential in the future.

Kansai Airport

A similar set of problems was tackled in developing the fire safety strategy for Kansai Airport, to be built in Osaka Bay in Japan. The issues are related to the fact that this is a huge uncompartmented space within which people might be moving for many minutes whilst a fire was detected, confirmed and evacuation took place. The fire safety strategy depends on the Cabin Concept and exploits the high roof space once again as an effective smoke reservoir.

Attitudes of the regulatory authorities vary, when presented with a fire safety strategy based on engineering calculations rather than on regulations. The UK Building Regulations have a flexibility which allows this kind of approach, though paradoxically this does not always make the path of the negotiations smoother. In Japan, where the regulations are less flexible, the building control authorities have the option to set up a committee of experts if they feel that they are not in a position to judge the schemes presented. Whilst this introduces an element of delay into the proceedings, the prospect of negotiating on an engineering level with this committee has definite advantages.

The Ministry of Construction in Japan recently published a unique integrated guide to fire protection. This addresses a wide range of issues and provides calculation techniques for the sorts of studies that are relevant. The guide proposes design fires, sets alternative safety standards for escape routes, deals with smoke flow, suggests how fire resistance of steel may be calculated, and so on.

The document is comprehensive, drawing on research work from all over the world and quoting all relevant equations and sources. It points the way to future developments in regulatory matters which will free the designer to adopt a range of approaches to fire safety. It should be said that the document has no legal status in Japan, but it was applied in developing the fire safety arguments for Kansai International Airport, and its value in this was noted with interest by the authorities, and formed the basis for discussions with the expert committee.

In the negotiations, the committee expressed concern that fire spread might occur across the circulation spaces and suggested that the absence of compartment walls required some other kind of additional protection. Estimates were made of the radiation from possible flames in a fully developed fire in every combustible area, assuming that the sprinkler system had failed. It was shown that each area was sufficiently far from the next for the incident radiation on fresh fuel to be too low (<20 kW/m^2) to cause ignition. This became known as the Island Concept (see Fig. 2). It was additionally shown that even if passengers departed without their baggage, fire spread across the space via the abandoned baggage would not occur.

Fire resistance

Large compartment sizes and long escape distances are not the only issues which need to be addressed in terminal buildings. Architectural concerns are often of essential significance in major public buildings. It is frequently desirable, for example, to maintain as a feature exposed structural steelwork. It is then necessary to consider the thermal response of the steel under fire conditions and to assess how that affects the structural response of the building as a whole. At Kansai Airport calculations on the response of parts of the exposed steel structure to fire conditions were combined with structural calculations to show that structural collapse was not a safety problem. Similar calculations were carried out for the parasol roof structure designed for the extension to Marseilles Airport. In both cases, the limited amount of combustible material in the building, together with the reserve strength in the structure, allowed arguments to be made for leaving the steel bare.

Offshore fire engineering

Our first involvement in offshore fire engineering work arose indirectly from the Piper Alpha disaster, which occurred on the night of 6 July 1988 when an explosion on the platform resulted in a fire that escalated to involve the main oil export line. 20 minutes after the initial incident a second explosion in a gas pipeline led to a massive intensification in the fire. Of the 165 fatalities 79 occurred inside the accommodation unit where personnel had mustered.

The report from the public inquiry, headed by Lord Cullen, took over two years to complete and was eventually published in November 1990. In the interim period offshore operators questioned their safety procedures and problems were highlighted, particularly in relation to the use of living accommodation as a place of safety.

Temporary safe refuge: Heather 'A' PLQ

It was during this time that the Unocal Corporation approached us through the Aberdeen office with the brief to investigate their existing Personnel Living Quarters (PLQ) on the Heather Alpha platform. The aim was to protect personnel who had mustered inside the PLQ for the duration of any likely fire. Escape could then be effected after the fire if necessary. Extra process safety equipment was to be provided which would reduce the risk of secondary fires occurring.

The PLQ was a three-level 'Armadillo' unit built up of repetitive modular sections which were effectively cells with plywood and timber framed walls, floors and roof. The plywood panels were skinned internally with plasterboard and externally with GRP. The side of the PLQ facing the oil and gas process equipment, shielded by a 6mm steel plate cover, had a fire resistance of 60 minutes when measured against the standard furnace test used to simulate a cellulosic fire. It was not designed to withstand a hydrocarbon fire, which generates higher temperatures over a shorter time period. Thermal shock and greater heat fluxes had to be accounted for when assessing passive fire protection.

The timber structure was clearly the main problem for fires outside and inside the PLQ. It was concluded that this should be fully protected from involvement in any likely fire. From experiment it was found that the timber, with a fire retardant treatment, started to char and give off smoke at a temperature of approximately 250°C. This was chosen to be the maximum temperature that the wood could reach before it was assumed to become involved in the fire.

Consequently, fire scenarios for the inside and outside of the accommodation unit were developed, with estimates made of the likely severity and duration of each of the design fires. For internal fires it was found that the plasterboard would prevent the timber from reaching its critical temperature of 250°C. No major internal structural changes were necessary and only upgrading works to make good services, doors and surface finishes were recommended.

For external fires the story was different. These included potential fires from process equipment, crashed helicopters, refuelling tanks and from diesel generator storage tanks. Flame sizes and shapes were estimated, taking into account the effects of wind, and used to calculate heat fluxes onto the accommodation unit. The worst fire considered had flames impinging directly onto the PLQ with a temperature of 1000°C and a heat flux of 150 kW/m^2. Fig. 3 shows some of the scenarios considered. Even at a relatively low calculated heat flux the plywood-GRP interface would exceed the critical temperature of 250°C. Therefore it was necessary to insulate the wood.

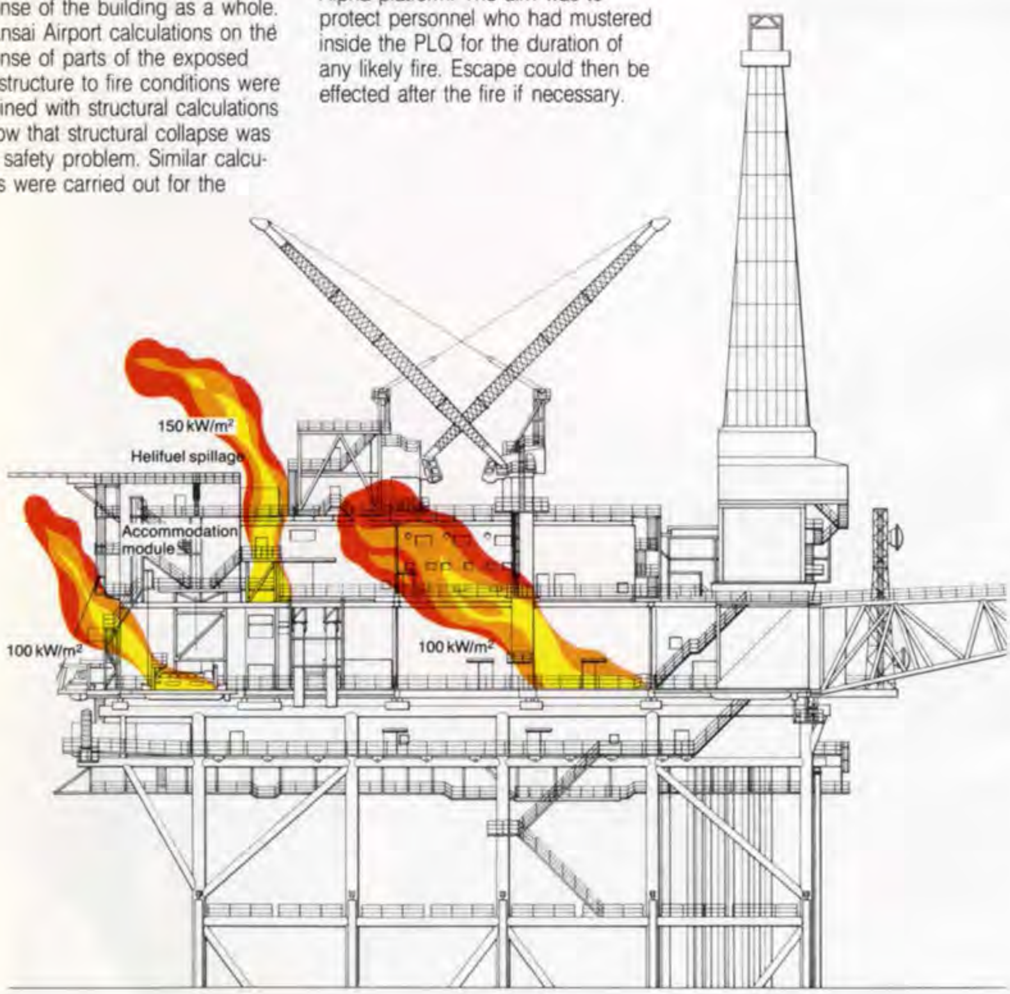
A 20mm sprayed-on application of intumescent to the outer skin with suitable fixing details was evaluated as the most efficient solution to protect the accommodation unit. The weight and thickness of this material made it the most desirable and it had been tested against hydrocarbon fires. Thus, the existing structure was maintained and a solution to upgrade the PLQ to protect against all credible fires was established.

Through negotiations with Lloyds Register of Shipping, both the internal and external protection systems were proved with full-scale panel tests. All PLQ upgrade work has now been successfully completed.

After we had carried out this work and other fire engineering studies on platforms in the southern North Sea, the report by Lord Cullen into the Piper Alpha Disaster was published. In a wide-ranging review of the offshore industry's safety measures, Cullen recommended that existing fire regulations should be replaced by 'goal setting' standards. His aim was to introduce engineered solutions to fire hazards by means of 'scenario-based design' so that an integrated approach to safety can be developed. We in the fire engineering team adopt this philosophy in all our projects both onshore and offshore.

Summary

This article has only touched on a few of the large number of major projects to which fire engineering techniques have been successfully applied in recent years. More and better tools are becoming available to us all the time. These, when coupled with more flexible attitudes on the part of regulatory authorities, may permit us to have an even greater influence on building designs of the future.



3. Some offshore fire scenarios.

Printed in England by Reedprint Limited, Windsor, Berkshire