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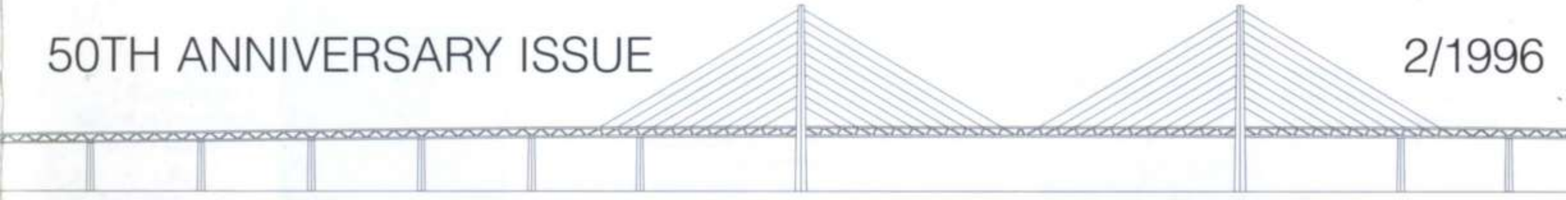

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Front cover:
Johannesburg Stadium
(Photo: James Burland)

Back cover:
Brisbane Convention and Exhibition Centre
(Photo: Patrick Bingham Hall)

Øresund Link image: Nigel Whale

Editorial

This is the second of two special numbers of *The Arup Journal* to celebrate the 50th anniversary of the founding of the firm. Issue 1/1996 was devoted to recent projects in Britain and Ireland, in both of which countries Ove Arup started his consulting engineering practice simultaneously in 1946. Over the ensuing half-century, the work of the organisation he created has expanded geographically and diversified into new fields of activity to an extent beyond anything he could have envisaged at the outset. This second celebratory *Arup Journal* includes recent projects in five continents - North America, Europe, Asia, Africa, and Australasia. The work described and illustrated here covers many different disciplines, ranging from 'traditional' structural engineering in widely varied forms to civil engineering works on the most massive scale; from the creation of energy-efficient internal building environments to the sophisticated analysis of a single structural detail to help withstand earthquakes; and from the architectural design and engineering of a new stadium celebrating a reborn nation to planning a visionary bridge link between two Scandinavian countries.

Sydney Opera House, conceived nearly 40 years ago, is still arguably Arups' most famous project, and its success has stimulated competition between other Australian cities to provide comparably spectacular public facilities. The latest of these landmark buildings is the Brisbane Convention and Exhibition Centre, with its vast and innovative roof designed by Arups' Australian practice. On the other side of the world, Ove Arup & Partners' New York office was responsible for a very different structural solution to an equally dissimilar public building, the museum of invention known as 'Inventure Place' in Akron, Ohio.

On the West Coast, the Los Angeles office has undertaken a number of seismically-related commissions. Behind the historic fabric of UCLA's Powell Library, discreet structural strengthening measures were carried out, of which the 1995 Northridge earthquake provided an unscheduled test. And at Duarte, also in California, new buildings for the City of Hope National Medical Center will incorporate a structural steel connection specifically designed to resist major ground movements.

Arups have now carried out building projects in most of the countries of mainland Europe. Examples in France and Germany are included here: the civil and building engineering design of new premises at Strasbourg to house one of the most important of European institutions, and the façade structure and environmental design for a private office building in Frankfurt.

Increasingly, different parts of Arups worldwide collaborate on projects. This can range from provision of a single, crucial piece of specialist advice, to the deployment of many disciplines from centres of excellence round the globe. Arup Botswana's design of a new shell roof for the country's National Assembly, adjacent to the existing facility designed by the firm over 30 years ago, benefited from input by the Arup engineer involved in the original building. At the other end of the scale, the Shajiao C Power Station in mainland China, the new Johannesburg Athletics Stadium in South Africa, and the design of the Øresund bridge link between Denmark and Sweden each demonstrates how Arups can bring to bear a global resource of creative engineering and design appropriate to major projects that could hardly be more varied.

Brisbane Convention and Exhibition Centre

Ian Ainsworth Tristram Carfrae Bill Short

Introduction

The new Convention and Exhibition Centre (BCEC) in Brisbane, Queensland, was opened to public acclaim in June 1995. The building is vast - over 400m long, up to 150m wide, and 32m tall - but its most individual feature is the roofscape, which breaks down the bulk and brings the edges of the building down within reach of its mainly pedestrian visitors. This endows a human scale and gives the sense of craftsmanship and personal input of which the late Peter Rice spoke so eloquently in his 1992 RIBA Gold Medal speech[1], and in his book[2].

The roof is also innovative in its structural action. Following the use of cable or rod-stayed roofs in the '80s, there has been a trend towards steel lattice grid shells, most of them using one-way curvature, usually as a simple barrel vault. The Brisbane roofs break new ground by using two-way, anticlastic curvature, which results in a shell with little propensity to buckle, which can therefore be extremely delicate.

BCEC is even more remarkable given that procurement was by competitive design-and-construct tender with a very short programme. Great credit must be given to the client and design team for their courage to embark upon such a project with such time and budget constraints.

Outline description

There are five large halls, each 72m x 72m in plan with 14m clear ceiling heights. Four are dedicated exhibition halls; the other (the Great Hall) can be used in exhibition, convention, theatre or banquet modes. It incorporates over 3000 seats, both fixed and raiseable in tiers; the latter can be hoisted to roof level out of the way for exhibition or for banquet modes. The halls are augmented by a $2200m^2$ ballroom; outdoor exhibition space; a series of meeting rooms from $30m^2$ to $1000m^2$; associated support facilities including a central kitchen that can cope with meals for up to 8000; and basement parking for 1600 cars. All halls are directly accessible to vehicular traffic via an elevated service road. The total floor area exceeds $110\,000m^2$, which makes the centre the largest building project ever undertaken in Queensland.

Tender

The Queensland Government identified some years ago that the State's share of Australia's convention market was in decline due to the capital's lack of international-standard facilities. It was decided that a new exhibition and convention centre should be built and in late 1992 tenders were called for its design and construction. Arups were invited to join the team led by Leighton Contractors with Philip Cox Richardson Rayner in association with Peddle Thorp as architects.

The required area of exhibition and conference halls only just fitted within the surrounding roads, so the basic plan was reasonably obvious - the row of five 72m square spaces with 14m clear headroom, interconnected by enormous doors allowing for independent or combined use. The halls have entry foyers for the public on one side and service access for heavy goods vehicles on the other. Essentially it was a big shed, which could to all functional intents and purposes have been much the same as an aircraft hangar or high bay warehouse.

All Asia Pacific exhibition centres compete for the same market, so each strives for some positive visual identification or public appeal. The main opportunity for differentiation is in roof design. The typical warehouse or hangar-style deep truss is cheapest, but for a small increase in structural cost many other systems become feasible. The choice is largely aesthetic but must also consider the site and its ground conditions and boundary constraints; cable-stayed designs, for example, may not be appropriate if permanent tension anchorages cannot easily be constructed, nor if there is insufficient room for back stays.

The actual choice of roof system therefore comes from an intimate collaboration between the architect and structural engineer, both responding to the particular site. For this project the design team decided on a roof shape recalling some of the free-form characteristics of the fabric structures that Brisbane people fondly remember from World Expo '88, held on the same site. Inter-State rivalry made it essential that the BCEC's roof could not be confused with the Darling Harbour Exhibition Centre in Sydney, also designed by Cox and Arups in the mid-'80s.

Coupled with this desire for a striking design was an absolute value-for-money requirement, and avoidance of any cost premium or risk associated with innovation. These potentially conflicting objectives were brought together ▶

2 Above: Exhibitions halls concourse.

1 Top: BCEC in its city context at night.

3. View towards north east with Brisbane River and city centre beyond. From right to left: principal elements are the main foyer, the Great Hall, and exhibition halls 1-4. In front is the concourse, with the loading dock behind halls 2, 3, and 4. To the right of the loading dock is the railway plaza structure supporting the twin roofs of the ballroom behind hall 1 and the flat-roofed main meeting room and pre-function terrace behind the Great Hall. The complex occupies two city blocks with the dividing street passing under hall 2 and car parking on both sides under halls 1, 3, and 4.

4. Main foyer entrance

▶ with the proposal to use doubly-curved steel grid shells as the primary roof structure, arranged to form a modulated roofscape which echoed the distant hills to the south.

To benchmark the proposed roof against more conventional ones, pyramidal and gabled truss alternatives were designed. The comparison demonstrated that the lightness and visual appeal of the preferred design more than compensated for the additional complexity and marginal increase in cost.

A further advantage of the thin shell was its ability to accommodate very tall exhibits either side of the diagonal truss. The decision to proceed with these complex shells, based on a few weeks of hurried work, set the course for the future. If Government accepted the tender, the design team would have to provide the promised building for the offered price within the allocated programme.

After detailed evaluation and assessment of bids from five short-listed tenderers, the Government awarded the contract to Leighton Contractors in February 1993 .

The special roof form is said to have had a major influence on this outcome. Upon contract award the team had four weeks to design the footings with a further eight weeks before the roof steelwork went to tender.

5. Entrance to Hall 1.

Substructure

The site is just south of Brisbane's Central Business District in the west corner of the South Bank redevelopment area. Bounded by Glenelg, Merivale and Melbourne Streets and the main New South Wales to Queensland railway, it was once part of the South Brisbane Railway goods yards. Geotechnically it is highly variable, due to the nearby Brisbane River and the presence of buried creek channels.

The soil profile comprises variable strength rock (Brisbane schist), overlain by soft clays and sand up to 22m deep, with a high water table. The site is subject to periodic river and local flooding.

The high water table and variable depth to rock forced a combination of foundation types. Driven precast concrete piles were used over most of the site, with short bored reinforced concrete piers and conventional pad and strip footings used where rock levels permitted. Where possible, basement extents and column locations were selected to minimise temporary shoring, in particular by the existing railway embankment. Foundations were also arranged to minimise clashes with known buried obstructions from now-demolished buildings.

Conventional block and reinforced concrete retaining walls were used except at the site's south east corner, where the platform was cut very close to the inter-State rail lines. Here, soil nails were used through the relatively unstable ash-rich embankment.

Service towers

An advantage of the chosen roof type is that it is a fully-resolved, simply-supported structure only requiring primary supports in the four corners of each hall, allowing all the service elements to be concentrated into towers containing concrete shafts that stabilise the building much like the central service core of a high-rise building. They also break up BCEC's enormous length, define the individual halls, provide a rectilinear contrast to the undulating roofscape, and house plantrooms and other facilities. The towers vary in size, standing up to 28m above ground, or 24m above exhibition level. Whilst lower levels are of in situ concrete, above the roof supports they are largely steel-framed with composite floor slabs supported on profiled steel decking. To achieve the desired sculptured appearance, the entire structure of each is clad with precast concrete panels.

Each tower was designed to be built independently of the hall roofs and to provide lateral restraint to concourse and loading dock roofs so that these would be independent of the hall roofs. This articulation provided maximum flexibility for detailed services co-ordination in the towers to be finalised whilst the roofs were being erected.

Floor slabs

All ground slabs are of conventional reinforced concrete, isolated from columns and designed for the variable support conditions offered by the existing subgrade.

6 Left: From street level, the service towers clearly define the halls. The main foyer is on the right.

7 Right: From a distance the roof provides the principal visual impact.

Many schemes were considered for the main suspended slabs, including reinforced or prestressed concrete flat slabs and band beams, composite steel construction, and precast concrete. The cheapest were reinforced concrete flat slabs, varying in thickness from 220mm in suspended car park areas to 350mm for the most heavily loaded exhibition areas (20kPa). At first, prestressed flat slab options seemed competitive, but the plan extent of areas between expansion joints - typically 72m x 50m - and the need to connect slabs rigidly to the relatively stiff roof support shafts at the ends of the 72m sections ruled them out, due to their higher long-term axial movements and resulting restrictions on pour sequences. Added benefits of the selected scheme, with its thicker slabs, included being able to accommodate exhibition service pits without soffit steps, and flexibility for many changes in position of service penetrations and set-downs before, during, and after construction.

Slab types in the upper levels of the building varied, with reinforced concrete beam/slab, prestressed concrete band beams and transfer beams, and steel-framed composite floor construction all used as appropriate to construction access and usage of the areas involved.

Railway plaza structure

The ballroom was added to the brief after the initial design and construct competition. This was to be built over the adjacent railway, thus also providing a connection to the existing South Bank redevelopment to the north. To support this extension, a 200m-wide bridge structure - the plaza - was designed, and built over the existing suburban and inter-State railway lines near the main building. Whilst railway authorities initially indicated that intermediate support to the plaza structure could be provided by central piers between tracks over part of its length, the final design comprised a total of 89 precast, prestressed bridge girders, each weighing 42 tonnes and spanning the full 38m across the railway corridor. The girders are the longest possible of their type, and required the largest available casting bed to be extended. These enormous beams had to be thoroughly investigated for stability during lift, as well as in their final configuration.

By eliminating intermediate supports, and putting column lines well away from the tracks, restrictions on working hours and methods dictated by track proximity were reduced. As a result, piers and headstocks were readily constructed during normal working hours and the prefabricated bridge girders erected during regular track close-down periods, three to five girders per night. A 200 tonne crawler crane was used - and removed during the day to allow the railway to resume.

The girders act compositely with a cast in situ reinforced concrete deck, the whole assemblage supporting major superstructure elements including the 2200m², 15m high ballroom, pre-function terrace, pedestrian plaza and a major meeting room. The closeness of the rail tracks below posed a potential acoustic problem for the ballroom and meeting room, so the deck structure was completely isolated from headstock supports via bearings. In addition, a floating floor in the ballroom and meeting room area was provided, and isolated in turn from the deck structure via a further series of bearings.

Hall roofs

Each roof is supported by a structure comprising six elements:

- The doubly-curved hyperbolic paraboloid shells, cut and folded about one diagonal, which form most of the roof surface.
- A bowstring truss, triangular in section, which spans 100m diagonally across the hall to support the edges of the shells where they were cut and folded.
- A shear frame comprising the perimeter members plus the truss bottom chord acting as a massive diagonal brace across the 72m x 72m square.
- Two overhead cables which run diagonally across the roof perpendicular to the bowstring truss and connect it to the opposite corners of the roof. This forms the most direct method of preventing the truss rotating about its bottom chord under out-of-balance loading.
- Slender tubular columns along the front and back of the roof to lightly support the delicate shell edges.
- Trusses spanning from side to side between the shafts to provide unimpeded communication between adjacent halls; they both support the roof edge and also hang the massive openable doors. They have an open V-section to allow each roof to move relatively independently under thermal loads.

This combination of elements creates a fully resolved roof structure which delivers 90% of the applied loads to the two support shafts at each end of the diagonal truss. The structure is clad with simple open channel purlins supporting profiled metal roof sheeting over fibreglass insulation. The perforated metal ceiling is placed underneath the purlins but above the supporting structure. The opportunity was taken to increase the roof's visual impact by running cladding down and up the inclined faces of the truss to open a huge diagonal gash in the roof surface. Following the slightly cluttered result of the underside of the roof at Darling Harbour Exhibition Centre, efforts were made to make Brisbane tidier. The air ducts were confined to bulkheads around the roof perimeter, or within the envelope of the spine truss. Only the light fittings were directly hung from the shell, and lifting points for exhibition loads of up to 3 tonnes were provided at each node point.

9 Below: View along bowstring truss of typical exhibition hall. The main air supply ducts are run from the service shafts through the truss and around perimeter to give both efficient distribution and an uncluttered ceiling.

8 Left: 38m long precast, prestressed concrete girder being lifted into place above the inter-State railway. Note the steel bracing required to maintain stability of the girder whilst being lifted.

10. Typical exhibition hall.

Great Hall roof

The Great Hall roof, although identical to the other four in external appearance, required a more complex design to support the seating for 3000 (which can be raised up by chain blocks attached to the roof), and full-height subdividing operable doors. Also, its cladding contained a mass layer to provide better acoustic performance. As a result, the design gravity load for the Great Hall roof was twice that for the other halls.

Despite this dramatic increase, the basic shell and bowstring truss were retained, but the system was strengthened with an additional grid of tension/compression members at eaves level. This allowed the arch and catenary members to act independently as tied arches and propped catenaries, which removes load from the truss and delivers it instead to the short columns around the perimeter of the hall. It also provides a grid of ceiling members to reduce the apparent ceiling height to proportions appropriate to the halls when subdivided.

Roof loads

Special loading criteria had to be considered in the design of the roof structures. Imposed loads had to be large enough to cater for the most ambitious exhibitions, such as a 747 jet hung from the roofs. The Great Hall roof also required additional strength for the raiseable floors, additional subdividing walls, and a multiplicity of roof-mounted catwalks, lighting grids, and other services.

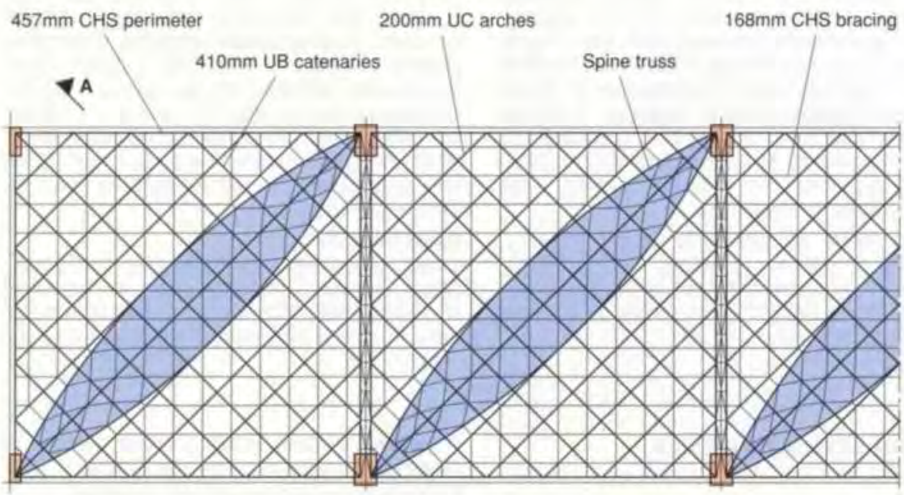
The wind loading on the building was of special concern due to Brisbane's near cyclonic winds. The very light shell roofs are subject to substantial net uplift forces, the extent of which were quantified by a wind tunnel test which incorporated time and spatial averaging to generate realistic estimates of aggregate loads in preselected balanced and unbalanced configurations. The testing revealed substantially higher uplifts than the values initially derived from the Australian Wind Code and required intensive redesign, as the results became available only shortly before fabrication was due to commence.

A variety of analytical techniques were used in the design of the hall roofs. Initial non-linear analyses using Arups' Fablon program were carried out to check buckling under uniform and out-of-balance load cases. Once the non-linear effects were quantified, detailed linear analysis of single and multi-hall models was used to design individual members with appropriate load magnification to allow for the non-linear effects. Finally, the approximate ultimate collapse behaviour of the roof was determined by Fablon using all its facilities for elasto-plastic behaviour together with elastic buckling. This gave the designers a measure of additional comfort.

As part of a fixed price, design and construct contract, it was vital to keep the cost of the roof steelwork as close as possible to the initial tender estimates. If some elements were found to require strengthening, other elements had to be reduced by an equal amount. The calculations for the roof were independently reviewed by Arups' Perth office as part of the project's quality management. This review determined that every member was being used to at least 90% of its capacity. The result is a roof in which the shell elements weigh a mere 25kg/m^2, and the total weight, including the bowstring truss and perimeter frame, is only 45kg/m^2.

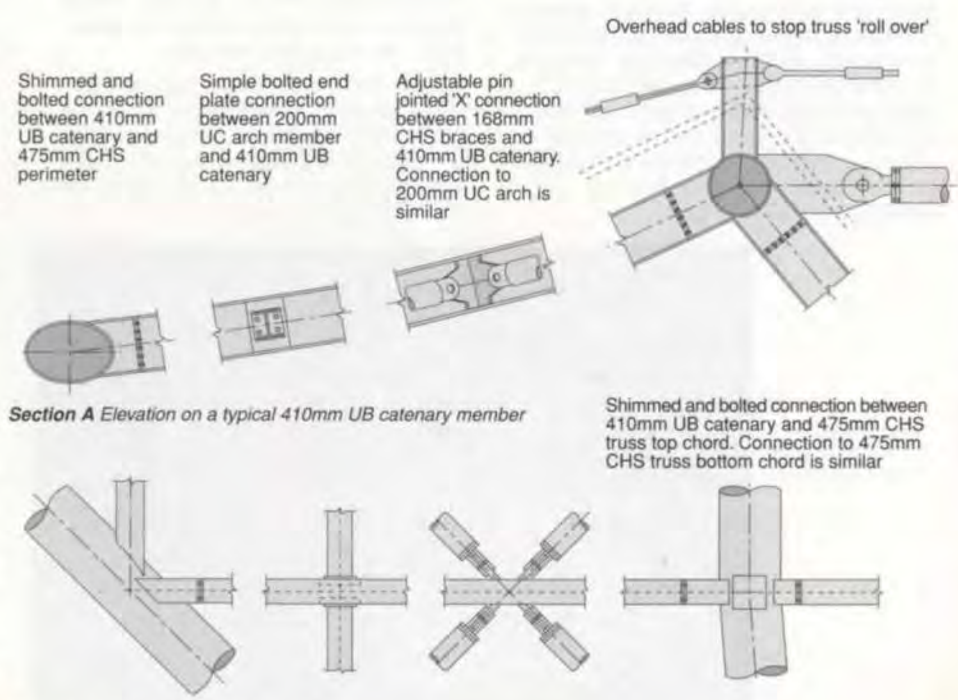
Roof steel sections

The preliminary design was intended to utilise circular hollow sections (CHS) throughout the roof. However, to simplify detailing and allow the steelwork to be more competitively tendered, the tubes were changed to conventional universal hot rolled sections.



11 Above: Part roof structural plan.

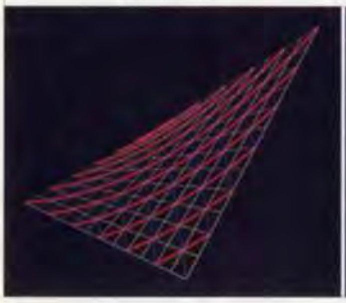
12 Below: Roof steelwork details.



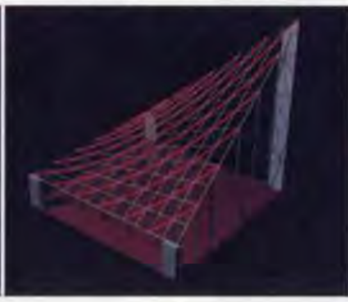
Roof geometry

The preferred geometric form for the individual hall roofs was the hyperbolic paraboloid, a pure membrane form evocative of the fabric structures at Expo '88. A hypar is the surface formed when ruling straight lines between edges of a square in which one corner has been raised above the level of the other three. At approximately 45° to these straight line generators (grey on A) are the lines of principal curvature which are equal and opposite (blue and red). To develop sufficient curvature to be structurally efficient over the 72m x 72m hall, one corner would have to rise 60m relative to the others (B). This would have given an unnecessarily large internal volume, and massive, expensive perimeter walls. Instead, the surface was cut along an inclined diagonal plane and folded down about the cut diagonal (C). The result was the same efficient shell but now with uniform, minimum height walls and a rise of only 10m to the centre of the cut edge. The halves are joined together by a triangular section bowstring spine truss spanning 100m diagonally across the hall and supporting the half-shells along their cut edge. The whole is trimmed by a perimeter frame and supported on a combination of concrete shafts in the four corners and perimeter columns (D).

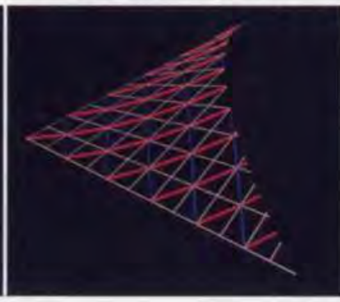
A.
Hyperbolic paraboloid shell.



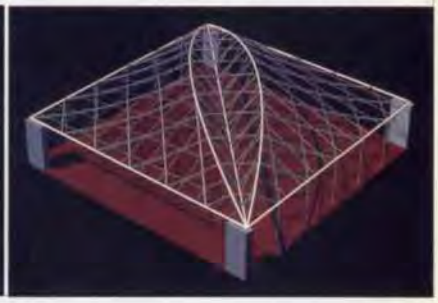
B.
72m x 72m efficient hypar rising up 60m in one corner.



C.
Same hypar cut along an inclined diagonal plane.



D.
Two cut shells with bowstring truss and perimeter frame.



The catenaries, which are continuously curved and support straight purlins, were changed to a 410mm deep universal beam (UB) section, and 200m deep universal columns (UC) were used for the arches, which are straight between node points and simply bolted to the catenaries. To control the effective length under compression of these I-shaped sections, the bracing (168mm diameter CHS) was shifted from conventional cross-bracing to an offset diamond pattern, which supports the members about their weak axis at mid-span.

It also tidies up the detailing by giving a straight crossover junction at the joint between the catenary and arch members, and a dissociated X-form where the brace members come in. The alternative would have had eight members all converging on one point in space. The truss chords and perimeter members are 457mm diameter CHS, to clarify the connections with members arriving at various positions. During detailed analysis, it became apparent that the shell performance was significantly affected by the stiffness of the perimeter shear frame. To stiffen the latter cost-effectively, the perimeter members were filled with concrete and the truss lower chord was augmented by 12 reinforcing bars of 50mm diameter.

Roof details

It was recognised from the outset that appropriate detailing was the key to the roof steelwork. Whilst this can be seen inside the halls, it is not intended to compete visually with the exhibits, so simple neat details which aided construction were devised. Most of the connections are simple bolted end plate connections within the depth of the sections. Shims are provided to take up length tolerance and the end plates are sufficiently flexible to absorb alignment tolerances by local plate flexure. To both provide this flexibility and minimise use of material, all end plates were designed using yield line techniques, which were also employed to minimise the internal stiffening of tubular sections where the main roof members try to punch into the relatively thin walls of the perimeter tubes. The diamond bracing members were tubes, terminating in classic pin connections. As these members carried relatively light loads, the investment in a more articulate detail was worthwhile in terms of the clarity provided at an otherwise awkward skew junction. One end of each brace had a shimming adjustment made possible by the use of high strength cap screws.

Each corner of the roof was supported on the reinforced concrete service shafts by elastomeric PTFE/stainless steel bearings. One of these was fixed in location, two were completely free to slide, and the fourth was guided in one direction only. This arrangement gives complete freedom for the roof to expand and contract under thermal loads.

Foyer roof

The wave form foyer roof is approximately 45m wide and up to 90m long. The supporting structure consists of curved UB sections supported on columns within the foyer, trussed mullions on the entry glass line, and a single 'tree' by the main entry stairs. A tubular steel truss supports the leading edge of the roof spanning 32m from the tree to the adjacent service tower.

The roof is braced at lift core locations, and separated from surrounding structures by perimeter expansion joints. The foyer roof is clad with lightweight curved profiled steel sheeting similar to the hall roofs.

Concourse and loading dock roofs

These appear as projections of the hall roof geometry - an extension of the hyperbolic paraboloid shape. In them the primary structure follows the straight lines which define the surface. The twisted surface is therefore utterly conventional in its structural action, using simple steel beams, purlins and bracing.

Roof structural action

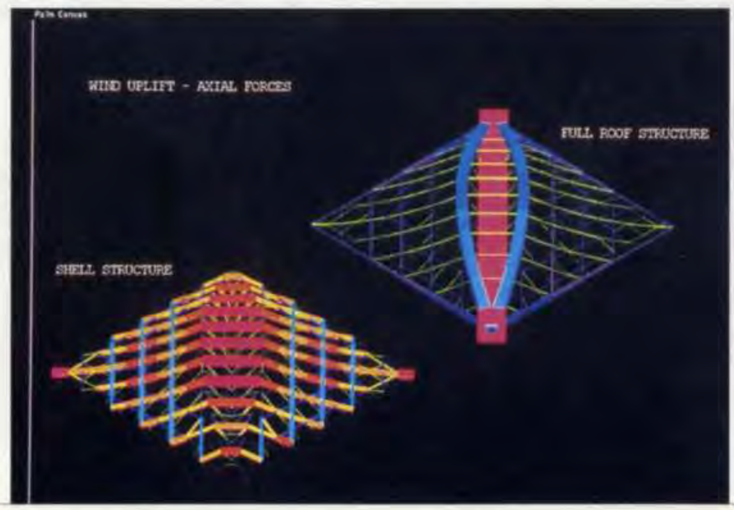
The roof shell action is accomplished by using tension and compression members running along the lines of principal curvature. Under overall downward loading, the curved 'catenary' members running down from truss to perimeter act in tension, and the faceted 'arch' members at right angles to them are in compression. Under wind uplift, the compression members act in tension and vice versa. The structural action is completed by bracing which controls the shell's buckling behaviour and also redistributes patch wind loads.

At the shell perimeter, both the horizontal forces perpendicular to the edge and the vertical forces from the catenary and arch members cancel out, leaving only a force in the direction of the perimeter itself.

This is resisted by the roof perimeter, braced diagonally by the bottom chord of the spine truss, acting as a shear frame (E).

The net result of these primary actions is to deliver almost all the applied load to the truss and thence to the concrete shafts at each end of it. The perimeter columns could thus be very slender and elegant, as they only support about 10% of the total roof load.

E.
Axial forces in roof structure under uniform wind uplift.



Erection of shell roofs

F. End of bowstring truss showing 12 Y50 reinforcing bars threaded through bottom chord.

The erection sequence and method were carefully thought out and calculated in advance, and then executed rapidly on site. A simple sequence of steps was followed to erect a typical roof shell:

• The bowstring spine truss, complete with Y50 reinforcing bars in the bottom chord, was assembled at slab level into four pieces (F). These were then lifted onto temporary supports and joined with a predetermined pre-camber.

• The pre-cambered V-truss between halls was erected, together with the tubular perimeter members and supporting columns (G).

• Overhead cables were installed to prevent the spine truss from rolling over. The truss could then be de-propped and no further temporary works were required.

• The catenary members were assembled at ground level into continuous arcs up to 40m long, and draped progressively from the spine truss, from the centre outwards. Each catenary was shimmed equally at each end to obtain a pre-determined sag at its midpoint for that particular member in the erection sequence. For the entire roof, pre-erection analysis was performed, in which the erection method was reversed and members removed sequentially so that the correct starting point for each and every catenary member could be determined.

G. Bowstring truss stabilised by overhead cables with perimeter frame and V-truss.

H. 410mm UB catenaries draped from truss to perimeter. 200mm UC arch members follow behind to brace perimeter.

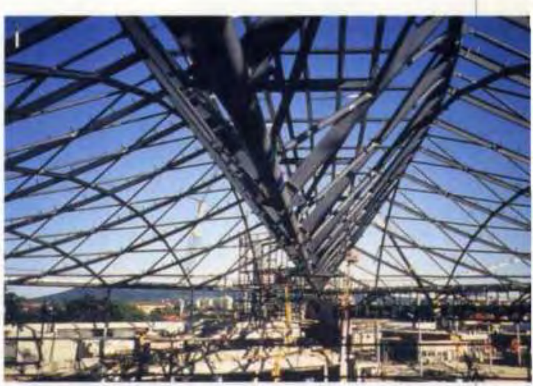
This simple level measurement was the only piece of surveying and geometric control required during the entire roof erection operation (H).

• After the erection of a catenary, the line of members comprising a complete arch was installed, starting with the shortest. The perimeter member was thus continuously braced against lateral deflection throughout erection, without any need for lateral propping. The arch members were fabricated slightly short and shimmed just to fit the actual gap on erection.

• After all catenary and arch members were erected, tubular bracing members were installed, using their built-in adjustment to fit the available gaps. At this stage the initially pre-cambered roof had descended to its theoretical geometry (I).

• The bottom chord of the bowstring truss was lightly post-tensioned using the Y50 reinforcing bars, and the perimeter members filled with concrete. This slightly pre-cambered and stiffened the roof, so that under the cladding's self-weight it once again descended to its correct position (J).

I



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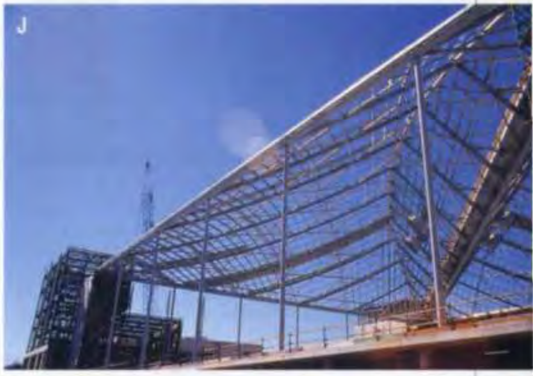


K

• Finally, the purlins, roof sheeting, and ceiling were installed (K).

13.
The Great Hall in concert mode. All the tiered seating can be raised to the roof and/or the space can be subdivided into four parts by 12m high, acoustic, operable doors, to give immense flexibility of use.

The beams are supported adjacent to hall perimeters, and cantilever out over the concourse glazing line and loading dock roof support trusses respectively.

Simple UB rafters are used together with diamond pattern tubular bracing to match hall roof detailing.

Ballroom roof

The form of the two roofs to the 2200m^2 ballroom is somewhat similar to those of the main halls, but on a smaller scale and with a single ridge line each. With a span of roughly half the main halls, a two-way spanning structure primarily dependent on bending is feasible, with no need to develop more complex shell action. As for the foyer roofs, straight UBs are set in both directions along the straight line generators of the hypar surface, meeting at a simple tubular ridge member arching diagonally from one corner to the other. The common edge shared by the two modules is supported by a tubular steel truss which supports a subdividing operable wall. The roof line extends out to form awnings around three sides of the ballroom.

Civil and transportation engineering

Whilst largely overshadowed by the spectacular building works, civil and transportation engineering played important roles in the success of the project.

The poor ground conditions in combination with a congested site required careful design and supervision of excavation, drainage, and pavement works.

The combination of a very large car park, proximity of major roads, and the competing demands of service vehicles and a bus interchange, created major challenges and opportunities for resolution by the transport engineers.

Construction

In true fast-track fashion, foundations were designed and constructed well in advance of the design of BCEC'S upper levels. Physical separation of major structural components via expansion joints - for example, isolation of the hall roofs from surrounding roofs - played a large role in allowing the building to be readily divided into manageable packages for design, tender, sub-contract, and construction purposes.

Conclusion

BCEC is undoubtedly regarded as one of Brisbane's most valuable assets. For its $170M cost it is expected to inject $800M into the Queensland economy in the first 10 years of operation, boosting the tourism, hospitality and entertainment industries, and opening up a whole range of business and investment possibilities in the city. Further, as a landmark image BCEC will portray Brisbane to the rest of the world as a modern, thriving community with an identity of its own.

It has already received the inaugural BHP Steel Award for Architecture, the biennial National Merit Award for Structures from the Association of Consulting Engineers Australia, and joined Sydney Opera House and Sydney Football Stadium among other Arup projects in receiving a Special Award from the UK Institution of Structural Engineers. It has been selected to be the venue for the Royal Australian Institute of Architects National Awards ceremony - for which it is unfortunately not eligible until next year.

Projects like this clearly demonstrate that Australia has the ability to design and construct products of the highest quality, allowing it to take its place as a respected member of the world's building industry.

Programme

November 1992
Pre-tender design

March 1993
Design and construct contract awarded

March 1993
Earthworks tendered

April 1993
Earthworks commenced,
foundations tendered

June 1993
Hall roof steelwork tendered

August 1993
Towers and other roof steelwork tendered

September 1993
Hall roof steelwork subcontract awarded

November 1993
Towers and other roof steelwork awarded

December 1993
Railway plaza girders erection complete

January 1994
Hall roof steelwork erection commenced

May 1994
Concrete works completed

May 1994
Ballroom roof completed

June 1994
First hall roof completed

April 1995
Partial handover
of completed building to operator

June 1995
Official opening of the centre.

References

(1) RICE, P. RIBA Royal Gold Medal Speech 1992. *The Arup Journal, 27*(4), pp.20-23, 1992.

(2) RICE, P. An engineer imagines. Artemis, 1994.

Credits

Client:
QBuild Project Services as agent for Queensland Government

Project manager:
Leighton Contractors Pty Ltd

Architect:
Philip Cox Richardson Rayner & Partners Pty Ltd in association with Peddle Thorp Architects

Consulting structural, civil, geotechnical and transportation engineers:
Ove Arup & Partners Australia Ian Ainsworth, Paul Callum, Christine Capitanio, Tom Dawes, Frank Gargano, Stewart Hobbs, Chau Luu, Tom Mulvaney, Gary Robertson, Barry Robson, Andrea Ryan, Bill Short, Robert Thompson, Warwick Walbran (structure)

Tristram Carfrae, Angus Johnson, Alan Shuttleworth, Mark Trueman (roof structure)

Peter Burnton, Robert Donnan (civil structures)

Kirsten Bell, Frank Vromans (civil)

Paul Wallis (geotechnical)

Trecy Boyd, Andrew Douglas, Graeme Krisanski (transport)

Building services engineers:
Norman Disney & Young

Contractor:
Leighton Contractors Pty Ltd

Hall roof steelwork sub-contractor:
Evans Deakin Industries

Tower and other steelwork sub-contractor:
Beenleigh Steel Fabrications

Illustrations:
1 - 6, 9, 10, 13, 14: Patrick Bingham Hall
7: Michael Rayner 8: Leighton Contractors
11, 12: Alan Shuttleworth/Trevor Slydel
A, B, C, D: Tristram Carfrae E: Mark Trueman
F- K: Ian Ainsworth,
Robert Donnan, Andrea Ryan

14. The ballroom set out for a banquet.

Inventure Place, Akron, Ohio, USA

Raymond Crane Caroline Fitzgerald

1. General view from south west.

Introduction

The United States National Inventors Hall of Fame (NIHF) is an organisation dedicated to the individuals who conceived the great technological advances which the US fosters through its patent system.

Founded in 1973 by the National Council of Patent Law Associations, the NIHF has a two-fold mission: to celebrate the contributions of significant inventors, and to foster an interest in science in its visitors. It was originally housed within the facilities of the US Patent and Trademark Office in Alexandria, Virginia (near Washington DC), but outgrew this home during the 1980s. The NIHF conducted a search across the United States for the ideal place to build a new Hall for the 21st century and found it in Akron, Ohio - once an industrial stronghold dubbed 'the tire capital of the world'. James Stewart Polshek, an Akron hometown boy and now a prominent New York City architect, was commissioned to design 'Inventure Place', as the NIHF'S new home is called. His response was a Hall designed to serve not only as a container for what is inside it but as a celebration of innovation through its own building fabric.

Outline of the project

In 1989, Polshek's firm invited Ove Arup & Partners' New York office to collaborate as structural engineer for Inventure Place. The commission was an important affirmation for the then new office - established that year as a structural engineering-only practice. (By 1996 Arups in New York has grown to 75+ employees providing a range of multi-disciplinary services.) Together, the two firms developed a bold architectural and structural response to the programme requirements, setting the pace for Akron's downtown redevelopment. Fund raising ensued and a new Director of the NIHF was brought aboard to spearhead the effort. Building work finally commenced in the fall of 1993.

The new purpose-built structure (Fig.1) allowed the NIHF to give physical form to display concepts appropriate for the subject. Rising from a wide plaza, a dramatic stainless steel sail encloses the 31m (100ft) high Inventors Hall of Fame, home to exhibits recounting the achievements of the country's great creative minds - beginning with the very first person honoured as a member (or inductee), Thomas Edison.

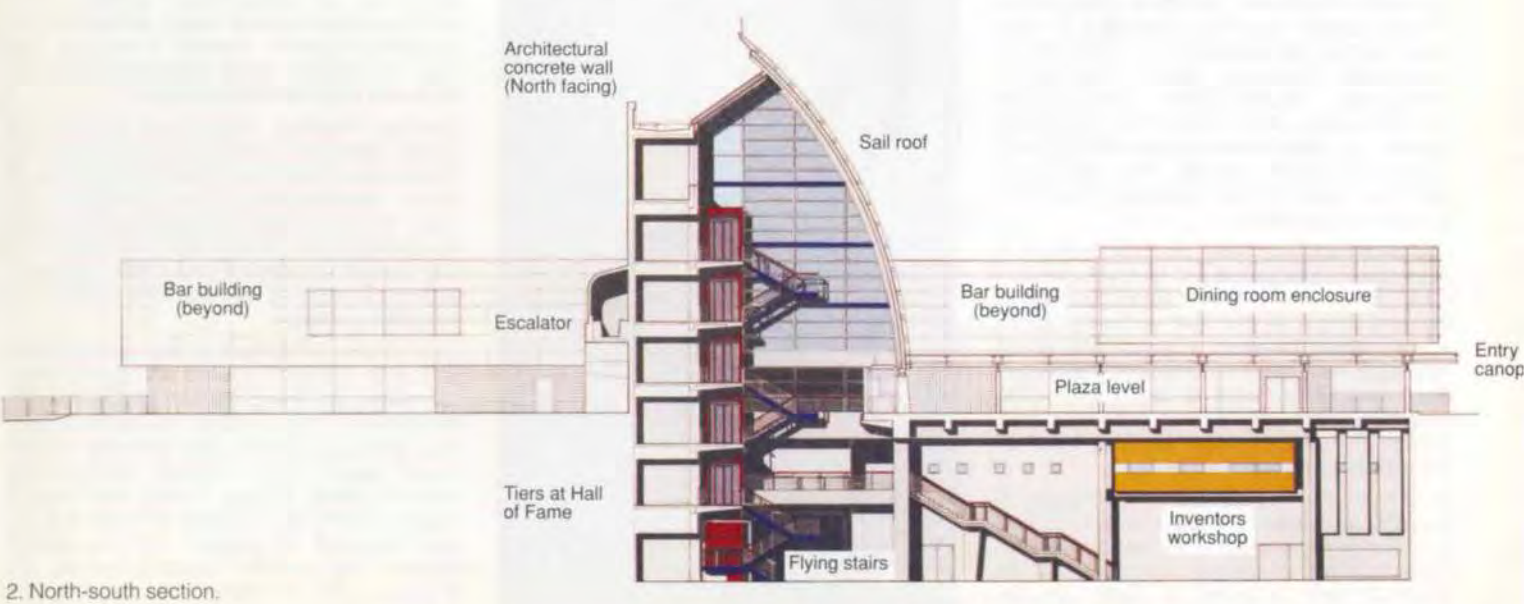
Below the plaza, the Inventors Workshop is a 10m (33ft) high space wedged into the site to create a 'playground for the mind' (Fig.2). An adjacent Bar Building stretches 80m (260ft) long and houses support functions, including classrooms, dining facilities, a museum shop, mechanical plant, and administrative offices. A 24m (80ft) tall mast rises from the plaza to announce the entire complex, doubling as a signage tower.

The Inventors Workshop and below-grade works

The Inventors Workshop is a busy 1850m^2 (20 000ft^2) room full of exhibits. These are well-described by *Architecture Magazine* as 'resembling unfinished Saturday-afternoon projects in someone's garage'.

In creating the appropriate environment for such a workshop, the design team created a space which in itself celebrates the art of putting things together. Architectural concrete clearly exposes the structural system, and mechanical, electrical, and plumbing systems hang bare (Fig.3): the building's works stand proudly for all to see, to understand and to gain inspiration from, while at the same time generating the casual 'garage-like' atmosphere. President Bill Clinton, on a visit to the site in October 1994, proclaimed that his 15-year-old daughter Chelsea would 'have a great time here'.

3. Inventors Workshop.



2. North-south section.

4.
West elevation of Hall of Fame at night. The upper end of the exterior elevator is on the left.

Carving the Inventors Workshop and the below-grade levels of the Hall of Fame and Bar Building into the site posed some engineering and construction challenges. The site itself is approximately 85m (280ft) x 60m (200ft) on plan, sloping down from west to east, and surrounded on its south and west sides by sidewalks and roadways. Where open-cut excavation was possible, on the east side, self-standing retaining walls with large counterforts supported on continuous concrete footings were used. At the south and west sides of the Inventors Workshop, open-cut excavation was not possible, due to the proximity of the basement walls to the busy streets and sidewalks at the plaza level. The engineered solution to this challenge was to design basement walls to span vertically propped by the plaza structure and the foundations up to 11m (36ft) below grade. The foundation contractor designed a temporary shoring system allowing excavation to take place without any disturbance to the traffic at street level. The shoring system consisted of contiguous auger-drilled cast-in-place concrete piles which were then supported laterally by steel waler beams at approximately 2.75m (9ft) centres. The latter were then tied back into the subgrade via drilled and grouted anchors.

The plaza level concrete structure acts as a diaphragm to prop the top of the basement walls. To accommodate a large opening in the diaphragm at the Hall of Fame atrium space, Arups designed a deep beam spanning east-west between adjacent diaphragm slabs and shear walls at plaza level. The beam, 450 mm (18 in) deep and more than 1.50 m (5 ft) wide in plan, was disguised as the handicapped access ramp.

All of this work was carried out during one of the most severe winters of this century. Today it is a thrill to see the completed Inventors Workshop full of school children (and an architect and engineer or two!) playing, exploring and discovering new ideas.

Hall of Fame

The Hall of Fame stands as a centrepiece to the new NIHF and as a tribute to technology. The atrium within its stainless steel arc is punctuated at each of the five levels by tiers of exhibits which are in turn connected by 'flying stairs'.

After passing through the entry and ticketing area, the visitor to Inventure Place is most likely to ride up the exterior escalator, in its enclosure clinging to the north side of the Hall of Fame, to its top level. The escalator trusses are supported off architectural concrete brackets which cantilever from the concrete exterior wall. Disembarking from the escalator, the visitor begins to tour through the lives and inventions of NIHF inductees. The backdrop is an exposed architectural concrete wall forming the north exterior elevation (Fig.4). Exhibits are displayed on a series of architectural concrete tiers formed by 3m (10ft) long cantilever beams with 3.65m (12ft) long backspans (Fig.6). The latter are supported by columns at 6m (20ft) centres and the north exterior wall.

5. 'Flying stairs'.

To achieve the high level of finish required of architectural concrete, plastic-coated *Finn-Form* (13-ply) plywood was used for the formwork, together with stainless steel form ties. Tie placement and the locations for reveals were meticulously worked out by the design team, which necessitated close co-ordination with the placement of construction joints. In addition, the use of 'form-saver' rebars allowed the contractor to pour sections of the vertical structure where floors and/or beams would be framing in without having to perforate the costly form boards with starter bars.

After the forms are stripped, a cast-in coupler accepts threaded dowels. The architectural concrete mix consisted of white cement, sand and aggregate along with a colouring agent. After the forms were struck, the surfaces were acid-washed to help create a uniform appearance.

To move vertically between the tiers the visitor has the option of riding in an all glass-enclosed elevator or using the 'flying stairs'. Neither route is for the acrophobic! Unlike other double cantilever stairs which rely on the stringers to support the treads at each end and then cantilever between the floor levels, the stairs at Inventure Place adopt a more adventurous approach. Each stair consists of a 300mm (12in) diameter steel pipe section along its centre, or spine (Fig.5). The pipes are then supported at the tier levels by steel fixings cast into the ends of the cantilevered tier beams.

The stair treads consist of steel plate bent into a closed triangular section. One end is then welded onto the pipe spine section. The intermediate landing platforms also comprise bent steel plate sections connected to and strapped across the central spine, and cantilevered from it. Arups worked very closely with Polshek's office to ensure that the structure of this stair was clearly and elegantly expressed; each stair arrived on site fully assembled, and the ends of the pipe spine were then field-welded to the cast-in connection plates. The railings and other finishes were constructed later.

To confirm the feasibility of this triple cantilever scheme, Arups in New York sent their structural analysis model to the London office for an independent check which confirmed that the stair would perform within accepted dynamic criteria. A run up and down the finished stairs convinced the architects and engineers of the same.

Architect Polshek notes that the Hall of Fame's dramatic form was inspired in part by his boyhood trips to an old dirigible hangar in Akron. Recalling the large curved forms of these structures, the 'sail' structure encloses the atrium space south of the tiers. To set the sail, Arups developed the basic structural system, formed by curved W14 sections at 6m (20ft) centres.

Each of the ribs arrived on site in two pieces and were spliced together with full-penetration welds. At the top, the sections are fixed to the roof of the Hall of Fame, and at the bottom they are supported by steel pins set dramatically apart from tapered architectural concrete piers. Closely spaced infill beams support interior and exterior finishes and are also designed to support hanging exhibit pieces. The exterior cladding panels are finished in 316 stainless steel and form the striking surface visible from all around.

At each end of the atrium space, a combination of clear and fritted glazing is supported by custom-designed steel trusses, the latter consisting of two standard pipe sections connected via a perforated steel plate. Each of these 'dumb-bell' sections varies in length to fit the space formed by the curved roof form.

The Mast

The southwest corner of the entry plaza is marked by the Mast structure, which in addition to supporting signs, lighting and a flag-pole, helps define the boundaries of the Inventure Place site. Echoing the spirit of the Inventors Workshop below, the designers assembled the mast from simple, off-the-shelf components. The main structure is a conventional radio transmission tower of vertical pipe sections and solid rod diagonal bracing. The curved members supporting the signs are stainless steel pipe sections bent and connected into a 'wishbone' configuration. Detailing the connection of the wishbones to the tower was challenging, in that the tower supports the pipes, but the tower lateral movement from wind loading coul occur in several directions.

7. Entry canopy.

6. Hall of Fame interior.

Entry canopy

Inventure Place encourages the visitor to use his or her mind from the very start, by using an unmarked entrance. Inventors-for-a-day will find the entrance situated at the east end of the plaza, parallel to the Bar Building and graced by a 30.5m (100ft) long steel entrance canopy structure, which provides shelter for visitors entering Inventure Place from the south end of the site (Fig.7). The exposed steel structure is supported at 6m (20ft) centres by pipe columns which cantilever vertically from the plaza concrete framing. At the tops of these columns, custom-fabricated T-sections cantilever 1.5m (5ft) and 2.75m (9ft) from either side of the column centrelines. Unbalanced snow and wind loading creates large bending moments in the structure. A continuous pipe spine beam connects the tops of the columns together and supports additional T-sections at 3m (10ft) centres. Infill framing supports metal panel cladding. The connections to the tops of the columns were detailed with bolts to allow for field erection.

Conclusion

Inventors selected for induction into the National Inventors Hall of Fame are honoured each year. In July 1995 the inductions coincided with the four-day grand opening of Inventure Place, which event, dubbed 'Inventure '95', drew 25 000 followers. In addition to the NIHF Induction Ceremony where seven new inductees were honoured for their inventions, the festivities included a 'Family Invent' attracting adults and children with hands-on creativity stations set up by community groups and entertainers, and a fireworks and laser display where spectators filled the plaza and surrounding streets. Today Inventure Place, with its daring exposed structure, proudly serves its mission to honour past inventors while working to light the spark of invention within its visitors.

Credits

Client:
National Inventors Hall of Fame
Architect:
Polshek and Partners, Architects
Structural engineer:
Ove Arup & Partners Raymond Crane, Caroline Fitzgerald, Daniel Brodkin
Mechanical engineer:
Byers Engineering
General contractor:
Welty Building Corporation
Illustrations
1, 3-7: Jeff Goldberg, Esto
2: Polshek and Partners/Nigel Whale

Schwedlerstrasse, Frankfurt am Main, Germany

Brian Cody
David Lewis
Constant van Aerschot

1. The Wintergarden.

Introduction

When the advertising company J W Thompson learned that the lease on its Frankfurt premises was to expire in July 1995, it approached the architects Schneider + Schumacher to find an appropriate site and design a building to suit their needs. The brief was simple: 5000m^2 of office space in which team spirit could flourish. Arups were called in at the beginning of 1993 to help the architects develop various approaches, which led to a final design that included a large enclosed glazed space called the 'Wintergarden' on the north side of the building, its façade covering some 1600m^2. Arup GmbH was appointed in November 1993 for the design of the façade's exposed steelwork structure and for the entire building services.

Planning approval

In Germany, planning and building control approval can often be quite difficult to obtain. In this instance the open plan offices and the associated building height void formed by the 66m x 20m x 2.9m Wintergarden required fire authorities' approval.

Arup Fire in London came in at the start of the project to assess the situation and confirm that no fire protection would be required for the steel. Much investigation was also carried out for the façade glazing, approval finally being obtained after a study and review by the glass technology department of Darmstadt University.

Design

The architect's aim was to minimise the structure and thus maximise the transparency of the façade, and Arups designed - with the aid of their GSA program - a hanging structure plus structural glazing to achieve this. The building itself is a straightforward in situ concrete structure (designed by a German consultant), with 300mm reinforced concrete slabs spanning 7.5m and two central cores containing the staircases and services shafts. Six storeys high, 66m long, and 18m wide, it has a floor-to-floor height of 3.35m. The glass panels for the façade therefore had to be the same height, and the design team chose a width of 1.8m.

Wintergarden energy study

In the architect's original proposals, both the Wintergarden's façade and the partition walls between it and the offices were to be single-glazed, the intention being for this to form the thermal equivalent of a double-glazed façade. However, the thermal performance is approximately 22% less effective than a façade with a glass U-value equal to 1.7W/m^{2}K, as used on the building's south side. The client's requirement that the minimum temperature in the Wintergarden be 10°C further exacerbated the problem. This meant that the Wintergarden would need to be heated, the energy demand thereby increasing to a value some 67% greater than an equivalent building without a Wintergarden. In principle, the building user was gaining a circulation space in the form of a Wintergarden with a minimum internal temperature in winter of 10°C.

One of the first issues with which Arups was concerned was how to achieve greater integration of the Wintergarden into the total building energy concept as a climatic buffer zone, while meeting the client's 10°C minimum internal temperature requirement.

To double-glaze the office/Wintergarden partition walls would merely have shifted the heating energy input's point of entry from the offices to the Wintergarden, with no effect on the overall heating energy demand.

Though if the external façade was double-glazed however, the minimum temperature requirement could be fulfilled without directly heating the Wintergarden, and the heating energy demand would be reduced to some 29% of an equivalent building without a Wintergarden.

This option was recommended to the client and the proposed design adjusted accordingly. The slight improvement in energy performance achieved by double-glazing the partition walls as well was not worth the extra costs involved.

Wintergarden steel structure

On the sixth floor, angled steel columns pick up the 14mm diameter vertical hanging cable onto which the single glass panels are fixed via a clamping fixing detail. At the top a tension tie, anchored in the roof slab of the sixth floor, transmits the horizontal forces back to the concrete slab. Vertical steel columns support the edge of the concrete slab. At the corners of every glass panel, horizontal props transmit wind forces back to the concrete structure, bridging the 2.9m space through the Wintergarden. Again, the structure had to be as fine as possible, and stainless steel circular hollow sections (CHS), 42.4mm in diameter, were designed. Triangulated horizontal props at the end provide the overall stability of the façade.

'Spider' detail

Due to the architect's design requirements and the final weight of each glass panel, a specially designed glass connection ('spider') was needed. High strength steel (690N/mm^2) was chosen to minimise thickness. This helped with the production of the spiders, as they could be cut from 10mm plate and cold-formed to their final shape.

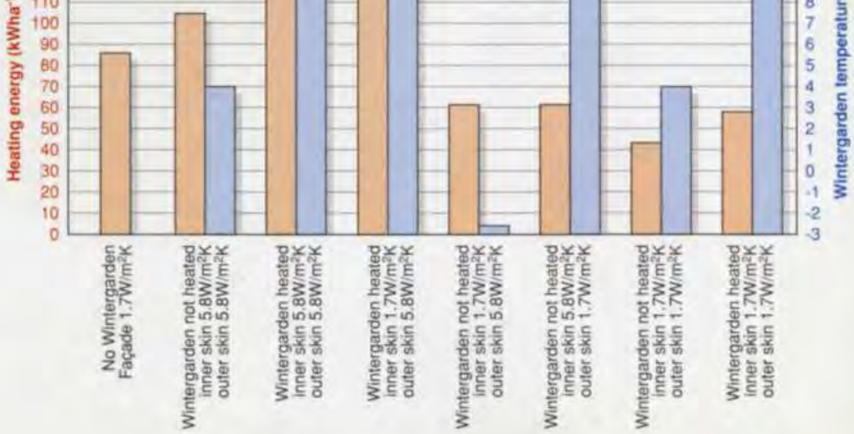
Clamping detail

Two half-shells clamp the cable which provides vertical support to the glass panels.

Testing was essential to determine the clamping force and surface requirement, since the support relies on friction against the cable for its safe use.

Glass design and testing

The large size of the glass panels produces high stresses. As the allowable stresses of clear glass are higher than for tinted glass,



2. Wintergarden heating energy study.

the former was adopted, and clear glass also gave the transparency the architects required. Double-glazed safety glass and partially prestressed glass for overhead use were designed, and the following panels with different thicknesses were used:

Type, location	Outer glass (mm)	Gap (mm)	Inner glass (mm)	Panel size (m)	No. of bolts
Overhead	15	16	12	2.90 x 1.8	6
Front panels	15	20	6	3.35 x 1.8	4
Edge panels	19	16	6	3.35 x 1.8	4
Side panels	19	16	6	1.67 x 2.9	4

The gap between the inner and outer glass panels varies in order to keep the outer dimension identical. The largest glass panel is the top one, having a 400m cantilever to the top edge but keeping the same bolt arrangement.

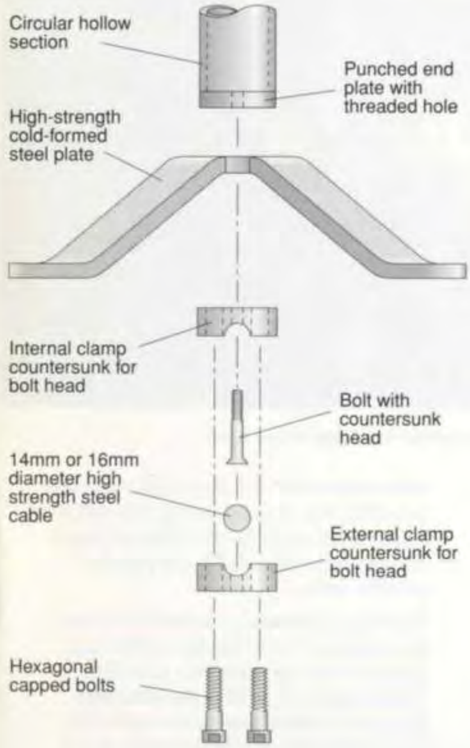
Pilkingtons in England manufactured and supplied the glass, and following normal German practice, an independent engineer checked the calculations. In this instance he was unable to give his approval, so Arups carried out a finite element analysis, modelling the glass panel and its support under the full wind load (in comparison with the British code where only 50% of the wind load has to be taken). This appeared to be difficult, as the Pilkington design relies mostly on data provided by extensive tests. However, the calculated deflections were found to be in order of 70mm, or span/48. As there was no existing structure behind the glass capable of damaging the surfaces, Arups were able to convince the checking engineer that no criteria limiting the deflections should be taken. The stresses in the glass only had to be limited to 50 N/mm².

A visit was arranged to the glass factory in Bavaria of Flachglas, a German subsidiary of Pilkingtons. Here a 15mm glass panel 3.0 x 1.5m was 'tested': six people stood on it, causing measured deflections of some 130mm. As the glass did not fail, this constituted a further step towards approval.

Erection

After the concrete structure was completed (apart from the roof slab), the steelwork was brought to site for erection. The edge and angled columns were installed, together with the tension tie and its steel plate which had to be cast into the roof slab. After the latter was poured, the vertical cables were put in place and pretensioned to equal the self-weight of ▶

5 Right: Interior of the Wintergarden.



3 Above: Exploded view of 'spider' connection.

4. Close-up of 'spider'.

6. Structural detailing at the sixth floor, showing tension tie.

7. Testing the clamping detail.

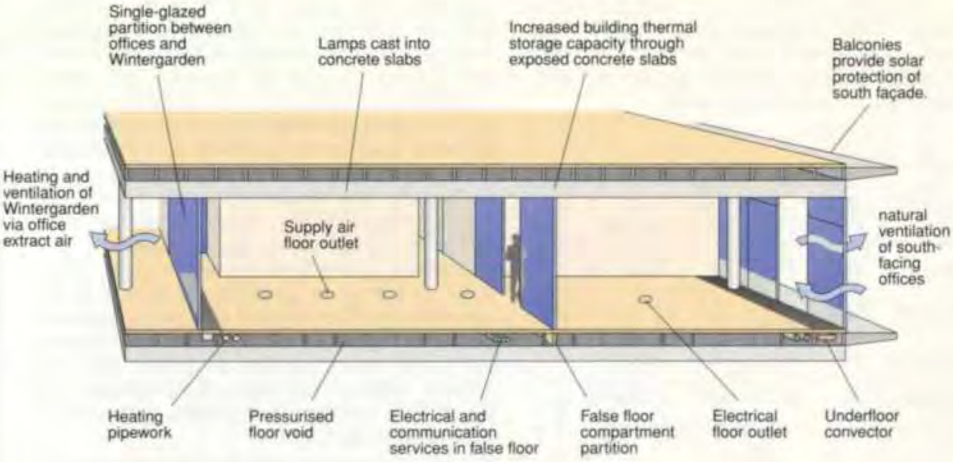
Environmental design

The building is effectively divided into three distinct zones with different environmental criteria and methods of treatment.

The north-facing Wintergarden, as an unheated buffer zone with its minimum internal temperature of c10°C in winter, contains lifts, secondary staircases, and circulation areas. It is heated and ventilated in winter via the office extract air and can be naturally ventilated in summer by opening vents in its lower and upper areas.

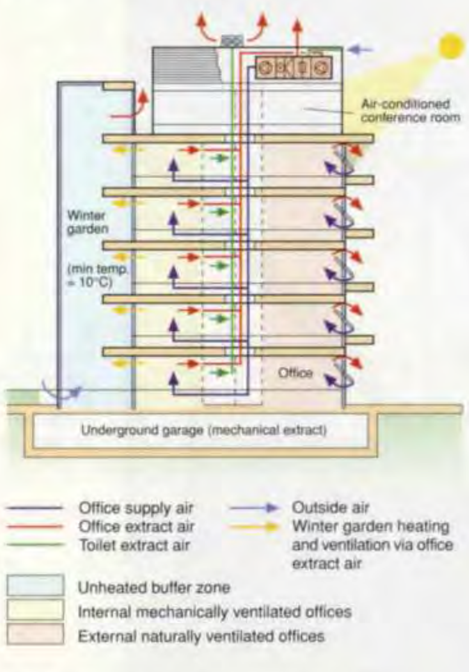
On the south side of the building, cellular offices are located, heated in winter to 20°C and naturally ventilated by openable windows. Overhanging balconies (average depth 900mm) give solar shading in summer, supplemented by heat-reflecting glass and an internal shading device. Exposed concrete slabs increase the room's thermal storage capacity.

The third environmental zone of the building consists of internal open-plan offices arranged between the south-facing external offices and the north-facing Wintergarden. Here, mechanical ventilation is provided, operating 24 hours a day during hot weather. This quasi-displacement ventilation system has extract at high level and supply air floor outlets, supplied via a pressurised floor void. During hot weather, the building is purged at night with cooler outside air from the mechanical ventilation system, so that radiant cooling via the cooler slab temperatures is provided during the day.



B. Perspective of office interior.

C. Office interior, showing single-glazed partition and supply air floor outlets.



A. Cross-section showing three environmental zones.

The client's express wish was that the building be non air-conditioned, largely for cost reasons. Nevertheless, through the use of building thermal storage, good solar shading, and night-time forced ventilation, a comfortable environment can be provided in the rooms without resorting to air-conditioning.

Should a future tenant desire air-conditioning it could be retrofitted. The ventilation and heating systems have been sized, so that the entire building can be mechanically ventilated if required; this would be achieved by replacing the air-handling plants in the rooftop plantrooms with larger units.

All ducts and pipes in the vertical shafts were sized to accommodate the higher flow rates, so that retrofit work on the office floors could be kept to a minimum. Space for an air-cooled chiller on the roof and for chilled water pipework in the shafts has also been provided. Full air-conditioning with individual room control could be provided by gravity cooling units integrated in the internal partition walls.

Heating is provided by underfloor convectors installed in the false floor, which are supplied with hot water from a modular natural gas-fired boiler in the basement plantroom. Heating pipework runs in the floor void, which also accommodates the electrical and communication services. Lamps are cast into the concrete slabs.

Other areas in the building include a conference room on the fifth floor, which has its own dedicated air-conditioning system, mechanically ventilated toilets and tea kitchens, and a mechanically-ventilated underground garage.

► the glass panels, so as to eliminate vertical movements during the next erection phases. The horizontal props were adjusted together with the spider detail and clamped into position, individual surveys being needed to determine the exact location of each prop.

The roof panels were then put in position, followed by the first row of vertical glass panels starting from the top. The pretensioning in the vertical cables was then released by the equivalent weight of one row of glass panels before the second row of was installed, and so on until the last row. Eventually, the sealant was applied between the glass panels.

Conclusion

The project started on site in April 1994, and was completed and handed over to the client by November 1995 (a four-month extension to the old building's lease having been granted). It was completed within its budget of Dm20M, of which some 12.5% was the cost of the façade. It has recently received the following awards:

- *Förderpreis Bundes Deutscher Architekten* (Award of German Architects Association).
- *Vorbildlicher Bau der Architektenkammer Hessen* (Award of the Architect's Association of the Land of Hessen as 'Exemplary construction').

Credits

Client:
Michael Loulakis

Architect:
Schneider+ Schumacher

Structural (façade) and services engineers:
Arup GmbH Dusseldorf
Constant van Aerschot, Brian Cody, Kiran Curtis, David Lewis

Structural engineers (building):
Philip Holzmann

Project management:
Hans Pfefferkorn

Main contractor:
Wayss & Freytag

Façade sub-contractor:
Magnus Müller

Glazing sub-contractor:
Pilkington/Flachglas

Illustrations:
2, 3, A, B: Trevor Slydel
1, 6, C: Waltraud Krase
4, 5: H G Esch
7: Magnus Müller

1.
The 'head', with the Commission Room (left) and Court Room (right) flanking the entrance hall.

The European Court of Human Rights, Strasbourg

Colin Jackson
Andrew McDowell
Mick White

Background

The Council of Europe came into being when representatives of the 10 original member states signed its Statute on 5 May 1949 in London. Among its stated aims are the protection of human rights and fundamental freedoms, and to this end the European Convention for the Protection of Human Rights and Fundamental Freedoms was signed in 1950. This led to the European Commission of Human Rights being set up in 1953, and the European Court of Human Rights (ECHR) in 1959.

The Commission considers applications from member governments or individuals that a violation of the Convention has taken place, and attempts to reach a friendly settlement. Failure to do so may result in the case being referred to the Court.

By the mid-'80s the original Court next to the Council of Europe's HQ in Strasbourg clearly was no longer adequate; by the end of 1987 there were 21 member countries, and over 13 400 human rights applications had been lodged. A new 2.4ha site nearby, on the banks of the River Ill and close to Strasbourg's other European buildings, the Palace of Europe and the European Parliament, was donated by the City, and a scheme prepared by the City's architects. Dissatisfaction with this, thought to come principally from President Mitterand, led to a competition between five European architects. Richard Rogers Partnership won in October 1989, with Ove Arup & Partners as engineers.

Architectural concept

The building occupies about half the site, using the full length of the river frontage. The plan form closely follows a bend in the river to give a waterfront elevation, a Strasbourg tradition that features strongly in the medieval town. The rest of the site is landscaped and planted.

The principal functions are expressed clearly in the massing. The 'head' of the building includes the circular Court Room (facing the park) and Commission Room (along the river), and between them a circular glazed entrance hall leading to the public and press amenities. Two office wings for the Court and the Commission, similarly facing park and river, reduce in height with distance from the head, and are separated by an open, landscaped courtyard and cascade.

The head and offices are separated by the Chambers, several large double storey-height meeting rooms. To meet the budget of some FF10 000/m^2 the offices had to be low cost, with an economical structure and natural ventilation.

Recognising that the ECHR would evolve, the brief included provisions for future expansion. The design of the office structure and services plant allowed for lengthening the office wings, whilst preserving the stepped elevation, or adding a storey at the Chambers end, giving a 10% increase in floor area.

The possibility of an additional circular courtroom in the landscaped area east of the Court and Commission drums also figured in the architect's planning.

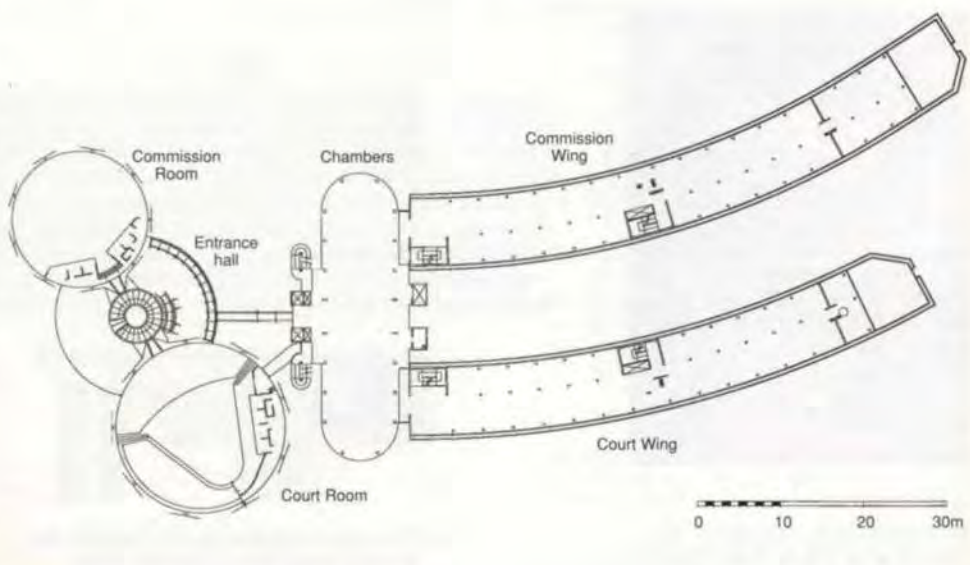
Procurement

Under the terms of the contract between the client and the professional team, Arups provided geotechnical, structural, mechanical, electrical and public health engineering services up to contractor appointment.Thereafter a local *Bureau d'Etude Technique* was responsible for approving the contractor's detailed design and overseeing the site phases. All documents were produced in French, and all text on drawings in French and English. The procurement procedure was to let separate contracts for some 33 packages; the first batch, including ground improvement, reinforced concrete work, and structural steelwork, were tendered in May 1991.

Construction began with ground improvement in September 1991, and by December 1991 foundation construction was well under way. At this point, events in the wider world took a hand, with dramatic impact on the project. In the two years since design team appointment, momentous political changes had taken place in Eastern Europe.

The Iron Curtain was beginning to lift. In November 1989 the Berlin Wall came down and a piece was obtained for the entrance

Text continued on page 19 ▶



2. Second floor plan, following client's revisions.

ECHR Building services

Concepts

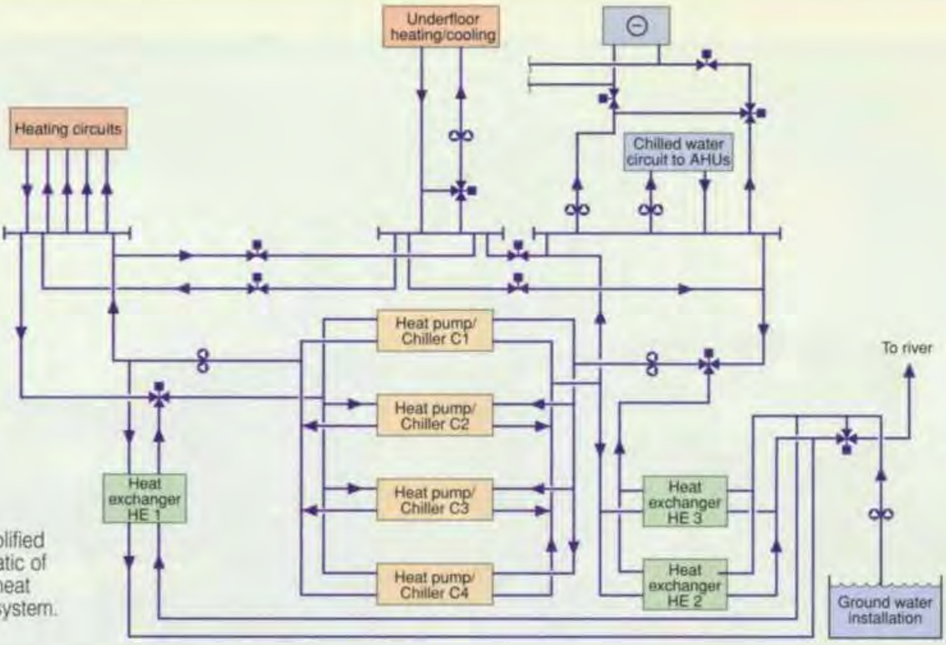
The design team took very different approaches towards servicing the new building's head and offices. The office Wings have a passive environment with minimal intervention from mechanical or cooling systems.
The courts, meeting rooms and lobbies of the head, with their more demanding acoustic and technical requirements, are highly serviced, fully air-conditioned areas.
The simpler systems in the office wings allowed more resources to be spent on the prestigious head.

Energy sources

The building uses an underground aquifer for both heating and heat rejection, as required. Several buildings in Strasbourg use it as an energy source and there is a strict limit on the quantity of water that can be removed. A temperature contour map of the aquifer showed where water could be abstracted, away from the influence of other buildings' discharges. To avoid warming the aquifer further, it was decided not to discharge 'used' water back into it, but rather to the adjacent River Ill. Discharge flow rates have to be varied to ensure the recommended temperature difference between the river and the discharge is not exceeded.

The client's decision for an all-electric building conformed to France's energy policy; the extensive nuclear power programme means that electricity is abundant and relatively cheap. Arups produced a comparative analysis of a conventional gas boiler/electric chiller system and an electric heat pump system, but something of a prejudice against gas was found in a country where electricity is the norm and burning a fossil fuel is seen as dangerous and polluting. The benefit of one set of machines providing both heating and cooling was also considered significant.

To make the heat pump system as energy-efficient as possible, a sophisticated scheme was devised to cycle the heat from the cooling to the heating side (Fig A). For example, if parts of the head require cooling at the same time as the offices need heating, heat rejected from the chillers or recovered from the extract air is transferred to the office fan coil units' heating circuit. This means that water is only taken from the aquifer when there is a net heat demand or surplus from the building as a whole.



A. Simplified schematic of chiller/heat pump system.

The building receives two high voltage services at 13.2kV from Electricité de Strasbourg. The two services supply a six-section HV switchboard which in turn feeds two low voltage substations, each comprising two transformers and a twin-section 400V low voltage distribution switchboard. All the HV equipment is rated for 20kV use with the transformer having dual voltage primary taps at 20 and 13.2kV. For emergency power (confined to supporting the head building, the offices only being supplied with emergency lighting) two 1500 kVA diesel generators are provided. Aluminium conductors are used throughout; the original design was in copper but the French sub-consultants advised that aluminium would give substantial cost saving.

The offices

The Wings have an automatic system of motorised, retractable blinds for protection against solar heat and glare. These are external to the building and an integral part of the façade architecture; views to outside are considered important so they are lowered only when necessary.
The control system monitors weather conditions and external light levels, and controls vertical movement and slat angle of blinds on the four façades separately. On any individual façade all blinds move together for architectural reasons. Initially there were some conflicts amongst occupants with windows on the same façade, as blinds can descend when parts of a façade are in shade. Performance and acceptability of the blind system has been improved by fine tuning control parameters.

The offices are naturally ventilated by large openable windows. The structure and concrete ceilings are exposed on the inside, to gain benefit from the thermal mass. Temperature conditions in most of the offices are reported as acceptable, even through the exceptional summer of 1995. Interestingly, the only floor where temperatures have moved outside the calculated temperature range is the lightweight structured fifth floor - an 'add-on' when the building was extended during the design stage. Additional retractable shading has now been added, but the benefits of exposed thermal mass have been clearly demonstrated. The offices are heated with fan coil units mounted on the wall in slim-line casings. Radiators were not used because of the low temperature of heating water available from the heat pump. The fan coils also provide a degree of upgrade potential. Should the client wish to provide summer cooling, chilled water could be passed through the (dry) coils.

The head

The triple-height glass entrance hall forms a dramatic entrance, with high level walkways to the courtrooms and a basement well to reach the library, press room, and meeting rooms below. To achieve the desired architectural effect and create maximum transparency, the entrance hall is single-glazed. This, combined with a winter design outside temperature of -15°C, created a challenge for the mechanical designers.

B.
Bullet-proof screen, fan coil unit, and 'pepperpot' air diffuser in entrance hall.

CFD

Computational Fluid Dynamics (CFD) analysis illustrated graphically the extent of thermal problems in the entrance hall. It allowed possible solutions to be tested, and ventilation schemes for the drums and library to be checked. A two-dimensional slice through the cylindrical entrance hall (C) was assumed to give a reasonable indication of the dynamics of the space. Perimeter convector heaters at low and mid level (the low level heaters being fan assisted) were modelled and adopted into the scheme, together with deflectors to inhibit downdraughts and anti-stratification fans on the high level walkways.

C. CFD section of entrance hall and basement well, showing temperature contours in winter.

D.
Natural lighting in the Commission Room.

Dynamic thermal modelling and CFD analysis was used to assist development of the scheme.

During summer, the low-level ventilation strategy in the entrance hall tends to encourage temperature stratification, and hence excessive temperatures on the high-level walkways where dignitaries enter the courts. To overcome this, a local micro-climate is established around the walkways using low-speed/low-noise fans mounted in holes in the walkway to pull cooler air from below and disrupt the stratification.

The Court and Commission Rooms are air-conditioned using displacement ventilation. This makes use of their internal height to give temperature stratification, thus reducing plant capacity and allowing savings on energy use and plantroom space. The high supply air temperature associated with floor supply systems maximises the benefit of free-cooling on the full fresh air plant.

Conditioned air enters the space from the insulated 'saucer' of the drum via floor-mounted circular twist outlets, with extract from several exhaust points concealed behind the perforated acoustic panelling.

The ductwork is integrated with the structural steelwork in the drum's double skin. Air-handling plantrooms are located in enclosures above the interpreters' booths within the drums themselves, so noise and vibration control was crucial, given the NR25 criteria set for the courtrooms.

Each of the meeting rooms and courtrooms in the head is served by five interpreters' booths. An infra-red communication system would have provided the required flexibility and performance, but for security reasons a hard-wired system was adopted. The temperature and humidity in each booth has to be closely controlled to meet ISO standard. A special low-noise fan coil unit was developed to fit in the booths' floor void.

The Court and Commission drums are predominantly daylit: an angular skylight in the roof of each allows natural light to give the ambient light requirements for more than 70% of the working year. The geometry of these skylights permits direct sunlight to fall across the walls of the drums, increasing the feeling of contact with outside. The passage of time is apparent in the shifting patterns of sunlight on the walls as the day proceeds. However, the geometry stops solar rays striking the Rooms' occupants directly, as this would cause glare and thermal discomfort. In each drum low level windows allow the eye to rest at infinity. Automatic internal perforated blinds control the brightness through these openings during summer.

E.
Interpreters' booths in the Commission Room. The perforated acoustic panel conceals the air handling plantroom.

Light surface finishes are used on the walls and roofs of the Court and Commission Rooms to enhance natural light levels by inter-reflection, and to fill in shadows cast by the roof structure. They create a light and airy atmosphere, with the particular colour and variability of natural light apparent throughout the working year.

A sub-basement 'race track' of 2m high, concentric builderswork ducts serve the head plantrooms with preheated and filtered fresh air, exhaust air and water services. This approach minimised horizontal distribution on the upper levels where distribution space is limited.

Extension

Though the client's decision to increase the scale of the building necessitated much detailed redesign, it was possible to carry through the building services concepts already developed without significant change.

3.
The Commission Wing, Chambers, and Commission Room facing the River Ill.

Text continued from page 17

hall (a local increase in loading allowance for the floor slab). The effects of these changes were difficult for the client body to predict. Many of the new democracies had applied to join the Council of Europe, Hungary becoming a member in November 1990 and Poland in November 1991. The client decided to revise the brief, with a 40% increase in floor area from 20 000m^2 to 28 000m^2. The changes included larger Court and Commission drums, an additional storey on the Chambers and offices, and lengthening of the office wings beyond what was originally envisaged. The consequences are described below. The wisdom of these changes has been borne out by events. In February 1995 Latvia became the 34th member and seven further applications, including Croatia and Macedonia from the former Yugoslavia, were under consideration. The foundation stone was laid by President Mitterand in May 1992 and the same month the Queen planted a tree on the site during a visit to Strasbourg to address the European Parliament.

Structure

The design was to current French codes. An application to the *Bureau de Contrôle, Socotec*, to use draft *Eurocode EC2* for reinforced concrete column design was turned down, much to the client's embarrassment. One of the stated aims of the Council of Europe is to 'show that Europeans now live in a framework that goes beyond the nation state'; this particular framework was to be designed to the French national standard. In addition to dead, live, snow and wind loads, the structure had to be checked for the effects of earthquakes, since Strasbourg lies in a zone of weak seismicity according to the French earthquake code *PS69*, which reports that Alsace has suffered five intensity 7 events since 1021. In 1728 a commemorative inscription on Strasbourg Cathedral was damaged, and the events of September and November 1802 kept the local stonemasons busy for six weeks.

Another potential hazard is the proximity of the River Ill and the site's vulnerability to flooding. The *'Service de la Navigation'* monitors water levels in boreholes throughout Strasbourg, and provided Arups with high water levels for given return periods. A 1 in 200 year flood level of 136.8m NGF was adopted for the design and waterproofing of the structure. This was taken as the lowest slab level, and internal waterproof render of lift pits and underground service ducts is designed to resist hydrostatic pressures. The render does, however, extend higher than this to above the level of the external ground.

Foundations

Site investigations in 1988 and 1990 revealed a thin layer of made ground overlying Rhenish sand and gravel to a proven depth of 15m, with a water table of about 1m below existing ground level. When the river is in spate its level rises by up to 1m, the former Rue de la Wantzenau - now part of the site - acting as a dyke to prevent flooding.

Pad footings, a raft, and piles were all viable foundation options. However, site investigation revealed random loose pockets of sand, one of which near the underside of a pad footing could lead to unacceptably large differential settlements. This option was therefore only possible if preceded by a campaign of ground improvement. Vibrocompaction over an area 3m beyond the building footprint and 8m deep was proposed. At Socotec's insistence this was extended to 8m beyond the footprint, but it still proved the cheapest solution.

The level adopted for the surface of the Rez de Jardin slab was a 1 in 200 year flood level, which meant that the underside of the pad footings would be above the normal water table and only excavations for lift pits and the head service duct would require dewatering.

When the decision came to increase the building's size, vibrocompaction was complete, foundation construction under way for the offices, and excavation for the head service duct started. Both offices and head grew so that the edge of the treated ground was closer than 8m from the edge of the building. Indeed, for the head, the footprint extended to the edge of the treated ground. Using Arups' VDISP program to model both treated and untreated ground, a settlement analysis showed that although greater settlements were predicted for the large drum foundation, differential settlements were acceptable and extending the treated zone was unnecessary.

Head

The two drums of the Court and Commission Rooms were originally 26m and 22m in diameter, formed by a series of steel wall and roof trusses, joined at base and eaves level by trussed ring beams. At the base of the walls a series of V-columns are supported on what appear as shallow concrete bowls or saucers, the tops forming the floors of the Rooms. The bowls are each in turn supported on three reinforced concrete columns.

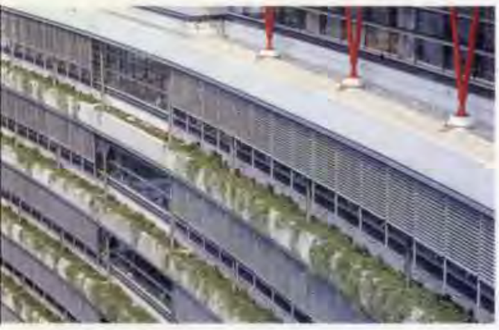
A series of steel radial cantilever trusses was considered appropriate for the bowls, with precast concrete panels on the soffit. The architect was keen that the precast concrete was not just cladding to the steel frame but formed part of the structural system.

4.
Circular mezzanine above the main entrance.

5.
The main entrance, with mezzanine, and stairway down to the basement 'well'.

6.
Seating for Judges in the Court Room, viewed from the main entrance.

8.
Planters and
externally shaded glazing on the Court Wing.



A prestressed solution was considered feasible, but one which would be limited to specialist designers and contractors.

Between the Court and Commission Rooms, the glazed entrance hall contains a circular 'donut' mezzanine floor, suspended from the drum structures. This was designed in exposed tubular steel, with steel stairs and scenic lifts from the entrance level.

A major consideration in the mezzanine floor design was its dynamic response to footfalls, and dynamic analyses were carried out with the PAFEC suite of programs to ascertain the effect of different structural arrangements and member sizes on the natural frequency. Once a structure with a natural frequency of at least 5Hz was found, further detailed analyses were carried out using the in-house program MULTIQ to look at the response at various points on the floor to a series of routes a person walking across it might take. Further dynamic analyses with Arups' GDSPEC program of a combined model of the entrance hall roof, mezzanine floor, and the two drums evaluated their interaction under seismic loading, to meet the requirements of *PS69*.

The 1991 revisions increased the diameters of the Court and Commission Rooms to 32m and 26m respectively. (The architect described the change as 'simply enlarging them on the photocopier'.) The enlarged sections and elevations, however, preserved the original vertical dimension between the drum floor levels and the top of the three supporting columns, requiring a 20% increase in the cantilever length with increased loading at the tip, but no increase in depth.

7. The library.

Chambers

The chambers are isolated from the head and the offices by movement joints. The structure is in reinforced concrete with floor slabs supported on beams spanning 13m across the rooms, which are in turn supported off columns with a high quality visible finish. Reinforced concrete cores give lateral stability. Mounted on top of the cores are glazed, exposed steel structures supporting lift motor rooms. The 1991 revisions introduced an additional storey: a steel roof structure supported on steel V-columns outside the façade.

Offices

The architect wanted the clear horizontal bands created by the glazing and external planters uninterrupted by a vertical joint, which would have to be of 40mm minimum width to comply with *PS69*. Moreover, such a joint would look distinctly odd owing to the differing lengths of each storey. It was thus decided to design the offices without movement joints for temperature variations and shrinkage, and impose this constraint upon the contractor. Consequently, the cores would not have reinforced concrete walls running lengthwise, lateral stability coming from a slab-column moment frame in this direction, with concrete core walls for lateral loads across the building. The French reinforced concrete code, *BAEL83*, requires specific calculations for these effects where joints at a maximum spacing (for eastern France) of 35m are not provided. The Commission Wing is 110m long. Design for these effects was greatly influenced by the lowest storey, used for archive storage, having a floor-to-floor height of 5.5m, compared with 3.3m for a typical floor, and, although the architect wanted a single column size for all the offices, a different size was permissible in the archive. A 3D skeletal computer model of the structure was analysed using GSA for in-service temperature variations and shrinkage, and the resulting forces used for member justification, appropriately combined with those due to dead, live, wind and seismic loading. The resulting movements were given to the architect for inclusion in the design requirements for the cladding and partition head details. Limits on the lengths of slab that could be cast before substantial completion of the level above was included in the specification as an added precaution.

The floor plate consists of a 200mm thick slab supported by 350mm deep beams spanning transversely with an offset internal column, and was constructed by the popular and cheap *prédalle* building method, using precast concrete downstands and thin slab soffit units with the remainder in situ. The columns have an as-cast high quality finish and are located inside the offices, the outer ones centred 400mm from the façade's inside face and the inner to one side of the central corridor.

The architect was keen to minimise the size of these columns, but his target of 400mm required over 5% reinforcement, the maximum allowed to *BAEL83*. Socotec refused to sanction a higher percentage if couplers were used in place of laps. A design spreadsheet was therefore developed which evaluated slenderness moments more rigorously than the code, including the moment from horizontal movement of the tip. The method was approved by Socotec, and led to a 400mm diameter column being adopted for all the offices.

The 1991 revisions lengthened each storey and introduced an additional one, whilst preserving the stepped elevations. The extra storey has a steel roof supported on a central reinforced concrete column and steel V-columns outside the façade.

Contractors' changes

The various package contractors were responsible for detailed design of various elements of work. The most radical changes to Arups' design were proposed by the *gros-oeuvre* contractor, whose package included the Court and Commission bowl structures, and the reinforced concrete offices and chambers.

For the bowls, he proposed in situ post-tensioned prestressed concrete, confident that the architect's intended high quality exposed finish could be achieved. This meant that instead of inspecting numerous precast panels with the possibility of rejecting and replacing those with an unacceptable surface finish, the architect was faced with some 800m^2 and 500m^2 of in situ concrete being revealed on removal of the formwork for the two bowls.

For the offices the contractor proposed using the core closest to the centre of each wing for longitudinal stability. For the longer Commission Wing this meant 65m from core to end of building, which still exceeded the code limits. A movement joint was proposed to coincide with a reinforced concrete fire compartment wall that halved this length.

The architect accepted this, with the proviso that the external planter boxes and façade be detailed to absorb longitudinal movements in several narrow joints, thus avoiding the appearance of a wide vertical joint on the face of the building. The longest continuous length of office thus became 78m with a stability core close to the centre. The end sections, 32m long at ground floor, were stabilised longitudinally by frame action.

Conclusion

The ECHR was completed at a cost of FF361M (c.FF12 500/m^2), and officially opened by President Mitterand in June 1995. The offices were occupied in late 1993 and the Court used from late 1994. The final cost equates to less than FF11 000/m^2 at October 1987 prices, the original base - only 10% higher than the original budget despite the extensive late revisions.

Credits

Client:
The Council of Europe

Architect:
Richard Rogers Partnership

Civil, structural, and building services engineers:
Ove Arup & Partners Andrew Chan, Lesley Graham, Richard Hough, Colin Jackson, Neil McClelland, Sean McGinn, Craig McQueen, Jane Peel-Cross, Mick White (structural)

Hilary Caton, Richard Gargaro, John Gautrey, Simon Hancock, Lidia Johnson, Denise Kee, Andrew McDowell, Geoff Powell, Mahadev Raman, Andy Sedgwick, Tom Smith, Steve Walker (services)

Bernard Lemius, Alain Marcetteau (geotechnical)

Local Bureau d'Etude Technique:
OTE

Local architect:
Claude Bucher

Economiste:
Thorne Wheatley

Vibroflotation contractor:
Keller

Gros-oeuvre contractor:
Urban (part of Campenon Bernard)

Structural steelwork contractor:
Viry

Mechanical contractor:
IMHOFF

Electrical contractor:
Spie Trindel

Building Management System:
Staefa

Illustrations:
2: Lesley Graham
1, 3,4, 6-8, B, D, E: Peter Mackinven
5: Paul Raftery A: Trevor Slydel
C: Richard Gargaro

Shajiao C Power Station, Guangdong Province, People's Republic of China

Rick Higson

1. Left to right: the two 350MW units and 210m chimney of Shajiao B, and three 660MW units and 240m chimney of Shajiao C;

The Background

The successful completion in April 1987 of the 2 x 350MW Shajiao B power plant in Guangdong Province marked a significant point in the development of Hopewell Holdings' interest in the Asian power market. The project - effectively the world's groundbreaker for build-operate-transfer (BOT) power developments - was completed in record time, and has subsequently operated successfully with station availability of over 90% (compared to a market norm of 70%), contributing to early repayment of loan capital.

Consolidated Electrical Power Asia (CEPA) was formed as a Hopewell subsidiary on the back of the confidence generated by this success, to take advantage of the increasing power demand market in the rapidly developing South East Asian region.

In Guangdong, industrial output growth of 24% pa has led to a substantial increase in base load electricity requirements, and currently the annual per capita consumption is around 700kWh, compared to a norm in the industrialised West of from 5000kWh (UK) to 11 200kWh (USA).

Shajiao C is a 3 x 660MW coal-fired station, the largest fossil-fired plant in the PRC and CEPA's largest project to date. Developed and financed on a BOT basis, at approximately $US1.9bn it was a significant step up in scale from previous projects. When operational, the three units will contribute a further 25% to Guangdong's current generating capacity of 8276MW.

The Partners

The plant was developed by a joint venture of the state-owned Guangdong General Power Corporation (GGPC) and Hopewell Energy Ltd, a CEPA subsidiary. The joint venture company, Guangdong Guanghope Power Company, was formed under a 20-year cooperation agreement, after which ownership and operation of the plant would transfer to the state-owned partner.

The company awarded the turnkey contract to a consortium comprising GEC Alsthom of the UK, ABB Combustion Engineering Systems (USA), and CEPA Slipform of Hong Kong. Of the heavy power-related plant (E&M) works, GEC-A designed and built the turbine island, cooling water, coal, ash, and balance of plant systems, whilst ABB-CE undertook the boiler island and electrostatic precipitators. CEPAS were responsible for the civil works. GECA were overall project managers, with technical leadership, and co-ordination.

Following their successful relationship on Shajiao B, Arups were appointed by CEPAS as designer for all geotechnical, civil, structural, maritime, architectural, and building services engineering. Arups also provided a site liaison team for the duration of the civil works.

Programme

The commercial success of BOT power projects demands both high levels of plant availability, and generating capacity to be brought on line as rapidly as possible. Together with early completion bonuses, this creates an environment in which traditional lead times and construction programmes must be radically reviewed.

Initially, Shajiao C was to have two 660MW units, with the contract starting in April 1992. Power stations of this scale would traditionally take some four to five years to bring on line, but commercial operation of the first unit at Shajiao C was required in three years. Although this was already an ambitious target, the working programme was developed on the basis of bettering this by three months, and in August 1992 (to stretch the challenge further) the contract was extended to include a third 660MW unit.

The site

Shajiao C stands next to the Shajiao B plant on the Pearl River estuary, 80km from Hong Kong. Some 35% of the 55ha site was

2. Location of Shajiao site.

Shajiao B jetty access arm left foreground, Shajiao C jetty access arm extreme right.(photograph taken from Shajiao C jetty)

reclaimed early in the contract through filling the shallow margins of the river to 5m above mean sea level. The ground conditions onshore and offshore in the region are typically colluvial and alluvial fill, overlying a graduation of completely to highly to moderately decomposed granite (MDG). The depth to the MDG varies greatly, from 2m - 40m, dipping north west to south east, and the stratification of overlying fill material is equally variable.

The power block, which contains most of the heavily-loaded structures, is on the better original ground in the north west of the site, 600m from the river. The reclaimed area, where estimated long-term consolidation settlements of up to 1m had to be taken into account, is principally used for the coal stockpile, balance of plant such as water treatment, and ancillary facilities including warehouses and storage.

Offshore the Pearl River gains depth gradually, so the associated structures had to extend 1.5km from the shore to achieve sufficient depth for vessel berthing and cooling water intake. As on land, the sub-marine ground conditions vary greatly, with the difference that there are extensive clay and marine mud pockets.

Civil works concepts

It is reasonably safe to say that the form of Shajiao C's civil works is and will remain unique in the power world. The designs were developed in liaison with a client willing to be innovative, and challenge accepted norms. However, they were based on sound principles derived from the dictates of the contract programme, the financing, and availability of local resources and skills.

The key drivers were:

- Much of the financing in Chinese currency (RMB) had to be expended in the PRC.
- Import duties of up to 70% made imported goods expensive.
- Export duties on re-usable plant and materials like sheet piling were also prohibitive.
- There was limited local availability of land-based construction plant.
- Marine plant was available up to a certain scale but large-scale plant was limited - 'available at a price'.
- Labour was readily available and cheap, but:
- local construction technology, labour skill, and workmanship were relatively unsophisticated.
- Concrete components were on hand and cheap, and:
- reinforcement was available and cheap to bend and fix, but:
- local formwork was of poor quality, and:
- structural steelwork was less available and cost-effective.

Prior to commencing the contract, the client decided to maximise the use of reinforced concrete and slipforming to optimise the use of available local resources and skills, whilst supplementing them with plant and equipment technology.

Slipforming used CEPAS' extensive plant resources and experience and, with appropriately developed concepts, allowed primary structures to be built rapidly. As a result, 40 structures like the boiler support and coal conveyor towers that would normally have been in steelwork and provided by the E&M contractor (GECA or ABB-CE), were slipformed and included in the civil works. The latter at Shajiao C thus grew to some 24% of the capital cost, compared to the usual 15-18%.

Plant data

Contract value: $US1.9bn

Site area: 55ha

Generating capacity: 3 x 660MW

Power plant:

- 3 x pulverised coal-fired recirculation boilers
- 3 x single reheat steam turbines
- 3 x 19kV hydrogen-cooled generators

Fuel consumption: 3M tonnes coal/year

Coolant: 66m^3/sec from the Pearl River

Station systems

A power station comprises several integrated systems. The primary electricity generating and transmission facilities of the power block are supported by extensive ancillary facilities to deliver coal, cooling water (CW), demineralised water, and hydrogen, and to discharge and treat effluent gases, ash, and water.

There are some 70 individual and unique structures within these systems, each developed in line with the concept principles. In this paper three principal components of the power station - the boiler, the CW system and the coal unloading jetty - are described to demonstrate the development process and the principles of the design and construction.

The boiler

Traditional support structure

The Shajiao C boilers are controlled circulation, radiant reheat, steam generators. Boilers are generally top hung, for two main reasons: firstly, the 50m high furnace walls are structurally slender and only supported efficiently in tension; secondly, with the main high pressure pipes extending from the roof of the furnace, it is easier to accommodate expansion of the walls downwards.

Such units are usually supported by traditionally-braced structural steel frames around 70-80m high and 50m x 60m on plan. The boiler and ancillaries have a total mass of about 10 000 tonnes and are typically suspended from a grillage of plate girders supported off six primary columns.

Erecting the steam drum and furnace walls are critical path elements of the work, but cannot commence until the support structure is in place. For a braced steel structure this usually means all vertical and horizontal bracing and diaphragms.

Shajiao C concept

CEPA, with Arups and ABB-CE, decided to develop a slipformed vertical boiler support structure. This would shorten construction time and bring boiler erection forward, because the lead-in times associated with structural steel fabrication and shipping to the PRC would be avoided. The concept also satisfied many of the construction principles previously identified, but the potential to reduce overall boiler erection time by up to three months was the main advantage.

The structural concept was somewhat limited by the need to develop it around a standard boiler configuration, and the scheme that emerged provided the six primary support columns braced by a series of C- and L-shaped shear walls, typically 700mm thick, 70m high, and designed as vertical cantilevers. Their form also had to allow for passage and support of the air ducts, pipework and boiler access platforms. Penetrations up to 8m x 13m were required for the primary air ducts, and secondary structural support steelwork involved numerous pockets and approximately 900 embedment plates. As the slipformed scheme developed, the 60m high structure supporting the coal bunkers and the structure to the turbine hall enclosure were included. However, because of the significant differences in height, plan area, and applied loading, the turbine hall superstructure was designed to be structurally independent.

Boiler design

The boiler structures at Shajiao are designed for two principal environmental loads: seismic, (the region is classified as seismic Zone 2a to the UBC), and typhoon wind loading, based on a gusts of 44-63 m/sec (0m-70m height).

The E&M plant loads are principally 10 000 tonnes of top-supported dead and imposed loads, and lateral wind and seismic loads applied through the boiler guides at six discrete levels.

The complete boiler structure was simply modelled to identify principal load distribution, which resulted in four levels of horizontal bracing being introduced to reduce the slenderness of the heavily-loaded free ends of

3.
Initial stages of boiler units 1&2 slipform at August, 1993. To the right and towards the rear, the CW pumphouse excavation has been completed. In front, slipforming the chimney is beginning.

4.
All three units at July, 1994, with administration building under way (extreme right), condensers being installed in unit 3 adjacent, and turbine hall roof being erected (distant, left).

the shear walls. These were located at levels consistent with the E&M floor structures. Specific elements were then analysed using finite element techniques. The walls where penetrations for the air ducts occurred were areas of particular concern, as stress concentrations were significant.

Throughout, the design was reviewed to ensure that the details developed were consistent with slipforming techniques, ease of reinforcement, and concrete placement, as well as tolerant to construction inaccuracies because of the number of interfaces with the E&M plant.

Construction

The boiler structure foundations were commenced in late October 1992. Each unit was founded on 70 hand-dug caissons, 1.2m-2m in diameter, between 15m and 40m long, bearing and socketed into the MDG rock. Progress on hand-dug caissons does not usually exceed 1m per day, limited by the curing time of the caisson shaft lining. However, with each caisson concurrently worked on by teams of three labourers, progress can compete with more sophisticated mechanised techniques, and the cost-effectiveness of the process is difficult to better with rates for labour as little as 15RMB (£1) per day. The substructure to Unit 1 - a grillage of 2.5m deep tie beams and pile caps - was slipformed in March 1993 and the superstructure slipform began the following month. Each unit was slipformed in two sections and contained 10 000m^3 of concrete. The lower wall sections, made relatively complex by the penetrations and embedments, resulted in an overall progress rate of about 110mm per hour.

Boiler erection commenced in August 1993 with the primary steelwork and it was here that the benefits of being able to delay erection of non-critical secondary elements was realised.

Another distinct advantage of the open section shear wall structure was that the clear vertical and horizontal access enabled erection of large pre-assembled elements, such as the furnace bottom, which would not have been possible with braced structural frames.

The full programme and erection benefits were not realised during construction of Unit 1, but the learning curve benefited Units 2 and 3. The concerted opinion of those involved was that overall time savings of c6-8 weeks were recognised.

Project materials data

Total concrete (inc. mass): 500 000m^3

Reinforced concrete: 350 000m^3

Slipformed concrete: 120 000m^3

Reinforcement: 45 000 tonnes

Structural steelwork: 3800 tonnes

Precast piles: 350 @ 450mm diameter: 8000m total length

Hand-dug caissons:
465 @ 1.2 - 2.8m diameter
14 000m total length
Steel tube piles: 213 @ 1000mm diameter: total 2950 tonnes

5.
Slipforming unit 1 boiler and turbine hall, November 1993.

6.
The station from the landside.

The cooling water system

Cooling water from the Pearl River has to be extracted at $66m^3/sec$ ($5.7Mm^3/day$), delivered to the condensers located under the low pressure cylinders of the turbine generator, and then discharged back into the river, warmed by 7-9°C, in a location that avoids recirculation.

Water is extracted at a submerged intake some 1.3km offshore and delivered through three culverts at atmospheric pressure to the forebay of the pumphouse. Here six vertical pumps push water at 1.2 bar pressure through six inlet pressure culverts to the condensers and then six outlet pressure culverts to the sealweir. The latter creates a closed system whereby siphonic action helps drive the water, resulting in more efficient pump operation. From the sealweir, water is discharged through an open channel to the river, downstream of the intake.

Intake culvert concept

The recirculation study and bathymetry defined the 1.3km offshore distance for the intake head, which resulted in a total length from intake to pumphouse of approximately 1.8km. The maximum allowable pressure loss in the intake culverts was designed as 0.9m water head, which resulted in three culverts of 4.2m x 4.2m cross-section. Offshore, they had to be placed in water up to 12m deep with currents rising to 2m/sec. Culverts are often precast, but in line with the guiding principles it was decided to develop a slipformed reinforced concrete solution.

The culverts were restricted to a maximum lift weight of 200 tonnes, as there are plenty of floating cranes in the Pearl Delta with this capacity. Larger cranes are less numerous and as a result often carry cash premiums to ensure 'reliable' availability.

Culverts with wall thicknesses of 300mm were therefore slipformed in 15m lengths, typically in groups of four.

Construction

The vertical construction presented a few problems. The units had to be slipformed off the rubber seal that would form the compression joint when installed. In order to lift and turn the units and transport them to the barge prior to placement, a mobile gantry crane system was devised.

A pair of sleeves were cast in opposite walls at a level just above the centroid, so that an axle could be inserted through after slipforming. This was then used for lifting the unit by the primary gantry hoist, a nominal end lift by a secondary hoist being sufficient to rotate the unit. To lift and place units offshore, a frame was specially designed to connect to threaded lifting eyes cast into the corners of the culverts.

Installation

Offshore, marine deposits had to be removed by grab and suction dredgers along the length of the culvert formation, which was then filled with rock to make a suitable base. Rock was unloaded from barges by labourers and then divers - working in zero visibility - placed levelling stone by hand against screed rails.

Bearing slabs, to restrict differential movement at the compression joint, were then placed to support the culvert ends, and the units positioned by marine crane working away from the onshore/offshore interface.

The cofferdam

A tricky element of construction occurs at the interface between onshore and offshore works, ie in the CW intake and outfall systems. A cofferdam scheme was developed using sheet piling to enable a shoreline excavation some 100m long, 17m wide, and 14m deep. Constructed in reclaimed ground, a system of wellpoint dewatering and sheet-piled cut-off walls was also installed. To avoid duplicating the works for intake and outfall, the design was developed to sit the outfall channel on top of the intake culverts.

This also significantly reduced the onshore excavation works.

Pumphouse concept

The pumphouse receives and delivers $66m^3/sec$ of water and is designed and carefully modelled to ensure efficient flow to the six pumps as well as to deal with surge flow should the pumps trip out. The client's initial idea was to slipform the structure and minimise temporary works and excavation, which at 18m deep in primarily reclaimed material was a significant task that could sterilise a large area of the site. Sheet piling and diaphragm walls were not favoured on cost and plant availability grounds, and the concept was developed to construct the entire perimeter wall, some 67m in diameter, as a 'self-sinking caisson'. These are used for some bridge foundations but the scale of this operation would have been unique. Details for the cutting shoe, and the construction sequence of concurrent slipforming, excavation, and sinking, were developed. With these techniques, a circle was the most appropriate structure for the pumphouse. Ground investigation data from the site showed, however, that the rock head level dipped some 10m across the width of the pumphouse. This would have made the sinking operation difficult to control, and increase the chance of the caisson breaking its back. The scheme was therefore dropped.

Construction

Eventually the form of the pumphouse was retained, but with traditional excavation methods using extensive wellpoint dewatering to control the natural water table at c1m-2m below ground level. In the event, the excavation exposed two existing rubble seawalls, one of them previously unknown, which would have further jeopardised the caisson scheme.

The 16m high pumphouse walls and pump chambers were slipformed in six sections from a 1.8m thick base principally designed for hydrostatic loads. The modelling of the internal walls and chambers required for hydraulic performance created complicated slipforming profiles, but careful co-ordination and detailing resulted in an extremely effective construction operation.

7.
CW pumphouse external wall complete, pump chambers in progress, November, 1993

8.
CW intake and outfall cofferdam under construction, March, 1994.

Inlet/outlet pressure culvert concept

Upstream of the pumphouse through the condensers and to the sealweir, the culverts delivering the cooling water work at an operating pressure of 1.2 bar but must be designed for a surge pressure of 4 bar ($400kN/m^2$).

The two principal considerations in their design are the joint detailing to cater for operating pressures and potential settlement, and the lifetime performance under aggressive scour conditions resulting from sea water and sediment travelling at 2.3m/sec.

Traditionally, cement-lined pipes or reinforced concrete culverts are used. Steel pipes were not favoured as import duties made them economically unattractive, so reinforced concrete was chosen. Precasting and slipforming were both considered, but the joint detailing was complex and it was eventually decided to develop an in situ scheme using travelling forms.

To control cracking and to allow the culverts to articulate under differential settlement, joints are at approximately 5m centres. In situ construction enabled a detail to be developed for the joints using traditional cast-in rubber waterbar which is readily available in China. The joint filler had previously been found to be prone to erosion or removal under operating conditions and a cast-in capping strip was specifically developed with Fosroc to ensure durability and long-term protection of the water bar.

Cooling water system

Water flow: 66 m^3/sec

Intake culverts
Offshore culverts: Total length 5.4km
Slipformed: three 4.2m x 4.2m culverts

Pressure culverts
In situ reinforced concrete: six 2.2m x 2.2m
Total length: 3.2km

Design pressure for surge: 4 bar ($400kN/m^2$)

9 Below:
CW pumphouse: pressure culverts from pumps under construction, June, 1994.
10 Inset left:
Inside CW pumphouse forebay prior to flooding, with screens at entrances to pump chambers.

11.
Culvert and caisson precasting yard and slipforming yard, May, 1994.

The coal unloading jetty

Under full load Shajiao C consumes 8100 tonnes of coal per day, and whilst the stockpile holds 40 days' supply, under normal operation the station must receive four or five coal vessels per week. The jetty is designed to receive and unload 50 000DWT vessels holding around 12 500 tonnes of coal using two grab unloaders and one continuous unloader, each able to extract 1500 tonnes/hour.

Coal is transferred by conveyors via a series of transfer towers which weigh, screen, and crush the coal prior to discharge to either the stockpile or directly to the bunkers. 50 000DWT vessels draw about 12m and need 14m water depth at berth. The relatively shallow margins of the Pearl River forced the unloading jetty to be located 1500m from the shore. It is linked to land by an access arm which carries the conveyor, services, and an access road.

Jetty concept

To moor 50 000DWT vessels and support the unloaders and conveyors, the jetty had to be 25m wide and 300m long, although this is extended by a mooring dolphin. The deck level was set partly as a function of the unloader operation and also maritime considerations of wave loading and tidal ranges, which in the Pearl River estuary is typically -0.9m to +1.3m PRD (Pearl River Datum) - although extreme conditions can push it up to +2.87m PRD. It was decided to set the deck level at +5.0m PRD. This was 2m lower than Shajiao B and as a result, with a wave height of 2.9m, wave loading was a more significant consideration.

Shajiao B's jetty has an in situ reinforced concrete deck on driven steel tubular piles but, though that had been a successful operation, there were few local Chinese subcontractors capable of undertaking the work. This lack of available competitive tenderers led to various concepts being considered including slipformed caissons.

Eventually, based on experience with Shajiao B and success in negotiating an acceptable price with the same contractor, a piled scheme was selected. The detail design was developed but shortly before piling began the steel tubes that had been available were required for use on Hopewell's Superhighway project in Guangdong Province. The rapidly agreed alternative was to develop a slipformed caisson scheme based on a number of principles:

- As there was no slipway, caissons were to be lifted into position, not floated.
- The unit weight was to be about 300 tonnes to suit available marine cranage.
- Deck construction was to be precast as far as possible to avoid excessive in situ work over water.
- Repetition and simplicity of precast work were essential.
- The structure was to be tolerant of construction inaccuracies likely in placement of caissons offshore.

Jetty design

The design had to cope with seismic, berthing, wave, thermal, differential settlement, and live loading including the unloaders, each of which weighs some 1200 tonnes.

It was based on 16.5m high, 3m diameter, caisson shafts slipformed from 9m square bases. 31 pairs of units were placed at 9.75m centres to support transverse and longitudinal beams approximately 2.3m square in section. A corbel was cast on the top of each caisson to allow substantial tolerance in their locations and adequate bearing for the precast units prior to in situ stitching. The corbels also allowed vertical lifting eyes to be cast through to enable connection of the lifting frame.

The deck was designed with only two principal units for each slab, each transverse beam, and each longitudinal beam, which expedited the precasting. The beam units were designed to simply bear onto shims placed on the caissons and, prior to in situ stitching, provided dead load and some propping action during preloading. Once this was completed the units, which had been free to articulate, could be jacked to level. Then the nodes and deck, designed with 'loose fit' reinforcement, were reinforced and concreted.

12.
Diver placing levelling stone for caissons and culverts.

14.
Placing jetty caissons: those in front of the crane to the right are temporarily stored, hence the angle.

13.
Slipformed jetty caissons awaiting placement, March, 1994.

Construction

Placing the caissons and preloading were key to the scheme's success, which had to produce a robust end product with relatively close tolerances for successful operation of the unloaders. Before construction began, the formation was dredged and 3-5m of unsuitable material removed to a formation of medium dense sands and gravels. To avoid this silting up, rockfill was placed immediately to the underside of the caisson base and dynamically compacted in 1.5m layers by 5 tonne drop weights. As with the culvert formation, levelling stone was hand-placed by divers, but the difficulties of quality controlling this led to the development of a preloading scheme to prove the works. Once the caissons had been accurately placed and levelled - only possible at slack tide - they were sand-filled and the precast deck units placed dry. Each caisson was to be preloaded to its maximum working load by 800 tonnes for up to three days or until nominal settlements were recorded. Because of the difficulty and time involved in handling kentledge offshore and the desire to apply controlled concentric loading, a system was developed using water tanks. These - 16m high and 8m in diameter - were placed on a precast table onto a seating block accurately located on the caisson core. Anchoring the precast table to the deck units gave temporary stability. The tanks themselves, which could be pumped full in less than an hour, were more stable under typhoon conditions when full.

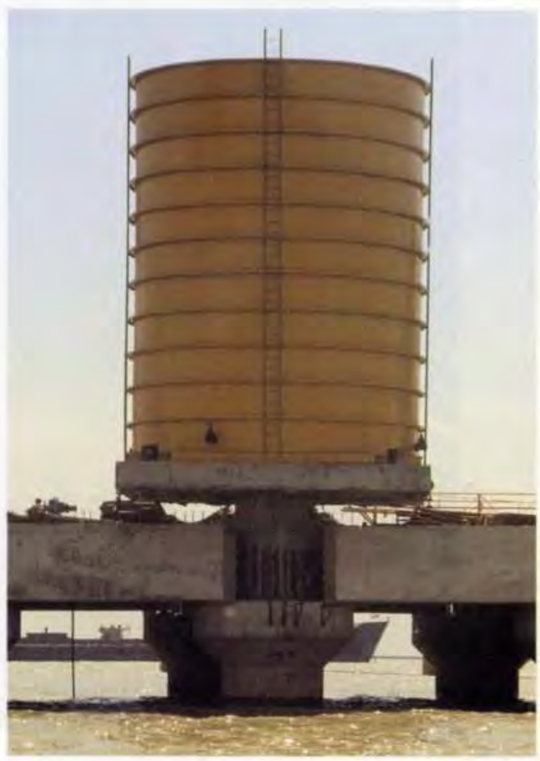
The preloading was applied by filling the tanks in a controlled loading cycle, monitoring level and verticality at all stages. When the full cycle of load had been applied and held with recorded settlements less than 0.05mm/hour, preloading ceased but monitoring continued during preloading of adjacent caissons close enough to be of influence. The deck construction followed the preloading operation two clear bays behind to avoid 'prestressing' the structure through settlements occurring after concreting.

In the event the jetty caisson installation and deck construction proceeded relatively smoothly. The formation proved to be adequately prepared, and predicted settlements of 30-35mm under full preload were typically reflected on site. This assisted progress on the deck and topside construction, as no caissons required re-levelling and few precast units needed jacking to level.

From start of dredging to erection of the coal unloaders, building the jetty took about 13 months. During construction, the steel piles that had been diverted off-site became available again and were eventually used for the 1.5km access arm. This had also been redesigned as a caisson scheme, but was presenting particular problems in terms of extensive dredging in poor ground.

15.
Attaching lifting frame to jetty caisson.

16.
Placing caisson preloading tank, and in situ deck construction, August, 1994.



17.
Preloading tank on precast deck.

Conclusion

Arups' Shajiao C project team in Hong Kong was supported on numerous fronts by other Arup resources and experience world-wide: all in all, c300 engineers and technicians worked on the project. Contributions ranged from short and long-term tours by individuals to Hong Kong and the site, to the design of packages of work which were significant projects in their own right. Notable amongst these were London (Industrial projects) for the chimney and coal unloading jetty; Cardiff office for the turbine hall roof design and site-wide fabrication drawings; Nottingham office for the workshops, warehouses and ancillary buildings; Sydney the jetty access arm, water treatment, ash silos, coal transfer towers; and the London Detailing Group. Personal contributions came no greater than from those individuals who spent time on a site where the living and working activity and environment were always interesting, though not a little challenging.

The civil construction works was substantially completed in May 1995. All three 660MW units were synchronised ahead of schedule and in May 1996 the reliability trials were completed. Full commercial operation commenced by the contractual completion date of June 1996.

Credits

Client:
Guangdong Guanghope Power Company

E & M contractors:
GEC Alsthom
ABB Combustion Engineering

Civil works contractors:
CEPA Slipform Power System Ltd
(Previously Slipform Engineering Ltd

Civil, structural, geotechnical, building services, maritime and architectural consultants:
Ove Arup and Partners

Hong Kong core team:
David Chan, Jeremy Chatwin, Keith Chong, Robin Forster, Adrian Fox, HW Fung, Rick Higson, Judy Ho, Ian Jones, Mark Jones, William Lam, Wilson Mau, John Powell, Grant Robertson, Ian Webb (civil/structural)

Paul Chan, Mark Green, Johny Ho, Andy Lam, Anthony Lam, Brian Littlechild, Joanna O'Brien, Glen Plumbridge, Terrence Yip, Stanley Yuen (geotechnics)

Alisdair Bamford, Danny Cornish, Paul Dunne, Kieran Flynn, Andrew Harrison, Peter Samain, Kelvin Tang, Patrick Yu (building services)

WC Chan, OY Kwan, David Lai, KY Lai, CW Lam, WM Lee, Martino Mak, TC Ngai, Lioni Ramos, KL To, SW Wong, Tim Wong, Reman Yick (draughting)

Lara Tang, Teresa Yung (administration)

UK
Ian Brookes, John Hamilton, Andrew Horton, Eve Jardine-Young, Bob Lea, Donald MacMillan, Chris McCormack, Daniel Osafo, Noel Tomnay, John Tyrrell, Paul Westaway (London - industrial)

Ed Forwood, Peter Hartigan, Allan MacInnes, John Senior (London - civil/structural)

Ranjit Basu, Les Davey, Bill Dineen, John Jackson, Derek Robinson, Kelvin Ward, Colin Wright (London - detailing)

Mike Hastings, Andy Keelin (London - building services)

Graham Gedge (London - AR&D)

Howard Corp, Justine Garbutt, Eddie King, Peter Monkley (Cardiff - draughting)

Steve Cliff, Roger Pickwick, John Read (Nottingham - civil/structural)

Stuart Hunter (Edinburgh - structural)

David Lancaster (Bristol - civil/structural)

Brian Rogers, Mel West (Birmingham - detailing)

Brian Raine (Perth - civil/structural)

John McDonald (Glasgow - detailing)

Australia
Ian Hooper (Brisbane - civil/structural)

Mike Cooke, Graham Curnick, Angus Johnson, Nasrin Nowparvar, Bob O'Hea, Umesh Rajakarian, Ernest Tang, Bill Thomas (Sydney - structural)

Paul Anderson, Adam Blatchford, Tony Carrol, John Collins, Ian Crowe, Paul Hanley, Charlie Houston, Trevor McBride, Steve Pennel, Alan Shuttleworth, Paul White (Sydney - draughting)

Space precludes mention of many others in Arups who also contributed.

Illustrations:
1, 6, 18: CEPAS

2: Denis Kirtley

3, 4, 5, 7, 8, 10, 11, 12, 13, 14, 15, 16, 17, 19: David Lancaster

9: John Powell

18.

19. The complete jetty, inset 18. shows turbine hall essentially complete, October, 1995.

Saving a landmark: California style

Catherine Wells

1. Powell Library exterior, fronting onto Dickson Plaza.

The seismically vulnerable Powell Library

When the University of California at Los Angeles carried out a seismic study of their campus, many of the buildings were found to be at severe risk in an earthquake. Amongst them, the Powell Library was of primary concern.

Built in 1927, it was one of the four original buildings around a green commons area, and remained at the heart of the campus as it grew into one of California's largest. The building also stayed at its architectural heart with an ornate Romanesque style of domes and roofs - and it was the varying levels and the heavy brickwork, as well as a non-ductile concrete frame, which placed it at the top of the campus's seismic risk list.

UCLA invited proposals for a feasibility study for a seismic upgrade to the building, and in turn the architects Moore Ruble Yudell came to Arups for assistance. Their joint study identified many deficiencies in the building's architectural layout, as well as major seismic and fire safety problems from a 1960s precast annexe at the rear (containing the near-obsolete mechanical equipment), not to mention the non-ductile frames and unreinforced brickwork.

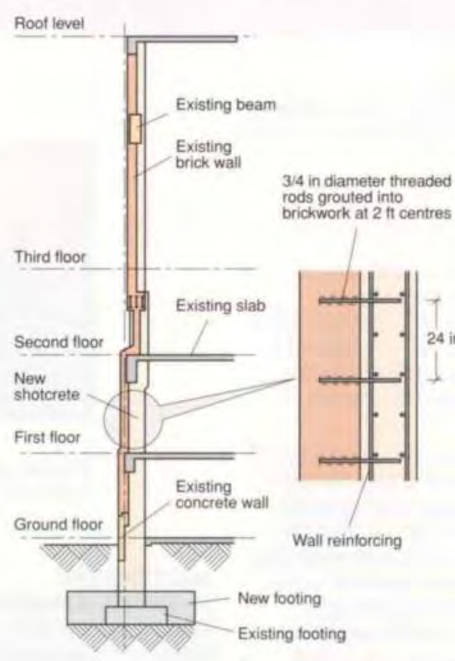
The solution was to demolish and replace the '60s structure. This could give the Library an appropriately styled formal entrance to the rest of the campus, and house a new elevator, mechanical units and vertical risers. Seismically, new reinforced concrete shear walls could be ductilely detailed and tied into all the varying levels of the building. Removing interior finishes for shotcrete strengthening to the inner face of the exterior brickwork would allow services to be replaced or upgraded, as well as permitting redesign of some of the office spaces. The project as schemed went into the long process of state approval and release of funding.

Then on October 17, 1989, the Loma Prieta Earthquake shook the San Francisco area, collapsing freeways and buildings. Funding for the Library was approved and the project moved ahead.

Structural design criteria

UCLA's campus architect has developed an innovative program for seismically upgrading the older buildings by adopting a performance standard for the structural work. This approach to analysis, unlike a code-based design, looks at actual anticipated building behaviour based on the real member material properties. These are input in a computer model of the structure and subjected to a representation of the earthquake vibrations expected at the site, based on its specific ground conditions. By understanding the building's potential weaknesses in an earthquake, attention can be focused on remedying these deficiencies, without the expense and impact of trying to modify an older building to comply fully with current codes. UCLA chose to adopt a life safety standard of performance; this aims at avoiding injury to people in or around the building during a major earthquake, but acknowledges that structural repair may be needed afterwards.

Through this approach, UCLA has been able to seismically upgrade many of their older buildings, a process which might otherwise have been cost-prohibitive.



4. Typical wall section.

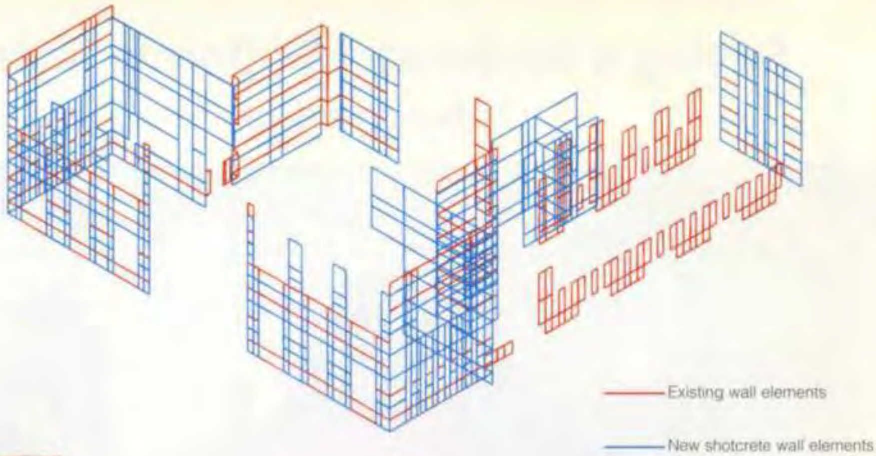
2. Reference Room painted cast plaster ceiling: pre-earthquake.

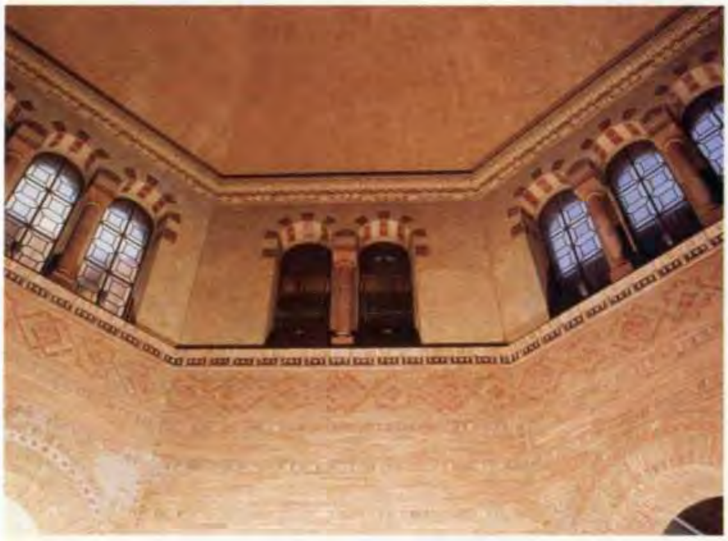
3 Right: The Central Reading Room, looking into the Reference Room.

Lateral system analysis

Although the existing brickwork was in excellent condition, no reinforcement meant that the walls could be subject to sudden brittle failure in an earthquake. Thus a major objective of the upgrade was to ensure adequate wall capacity with proper reinforcement to carry the seismic loads. This made it essential to understand the shear capacity of the existing walls and then add appropriate new walls and shotcrete strengthening.

5.
Computer model from seismic analysis, showing degradation of existing brick strength, plus new shotcrete.





6.
Rotunda brickwork:
shotcrete strengthening was concealed in the corner piers behind the plaster and brick.

8.
Looking up into void between existing brick wall and the lath-and-plaster finish in the Reference Room where shotcrete was located.

'In plane' shear tests at multiple locations on all the walls were carried out first. This non-destructive test measures the force required for a jack inserted vertically between two bricks to just start to move the brick.

The results were then correlated to an ultimate shear strength for the wall. The average value obtained was 145psi (well above the code allowable value of 10psi), which had provided sufficient strength for the building to withstand the earthquakes it had lived through to date.

An iterative analysis of the lateral system under the site-specific ground motions was carried out. In the first phase the entire existing structure was modelled, together with proposed new walls, showing which parts of the brickwork would be overstressed in the earthquake. The model was then run through a second phase where the stiffness of the cracked elements was omitted; stress levels were again checked and the forces were stable. A final phase considered the maximum credible condition where all brickwork had cracked and only concrete walls were carrying the loads. The building design was based on an envelope of all the load conditions to represent the range of actual building performance.

7.
Shotcreting the Reference Room.

Installing the shotcrete strengthening

Not only was the exterior brick of the Library to be preserved, but the major interior public spaces, including the 300ft x 60ft x 65ft tall Reference Room, had plaster wall finishes aligned with historically significant plaster ceilings above, which could not be realigned to accommodate new shotcrete on the brick. When the survey started, the plaster walls were found to have been built up to 2ft inboard of the exterior brick, enough for access ladders up into the ceiling. This gave the space needed to add reinforced shotcrete backing to the brick, reinstall plaster finishes, and maintain the original lines.

The Rotunda

One particularly challenging area was the Rotunda, an octagonal brick structure which lets natural light into the Central Reading Room from above. The interior has very attractive decorative brick, coloured plaster, and tiles on the walls - all to be preserved - but neither the structural nor mechanical systems supporting the space were up to the required standards.

The Rotunda was supported by eight non-ductile concrete piers linked by a beam above and sitting on a ring beam below. Analysis showed the concrete reinforcing as inadequate for the seismic forces that could be generated by the heavy tile roof and brickwork above. Since there was a strong desire to minimise impacts both the inside and outside the structure, several alternatives for structural strengthening were studied. One was to remove the exterior brick and apply new steel, but the outside appearance would have been altered.

Another scheme was to cut away vertical strips of the least decorative plaster inside, apply shotcrete, and replaster to match the original. This was feasible, but decorative brickwork would have to be removed at the base connection for dowel installation - a potentially visible repair. After consultation with the contractor, a procedure was developed that gave repair work indistinguishable from the existing.

The Rotunda's existing ductwork system gave only heating and ventilation, which needed to be upgraded to full air-conditioning. The ducts were buried in the wall, taking air from the fan room above and voiding it through low-level ornamental grilles in the tilework. Calculations showed that bigger ducts were needed, but there was no way to modify the existing, so Arups proposed to modify the round grilles. To achieve proper air distribution, the outlet velocity was increased by partially blanking off the opening. Straightening vanes were placed behind the ornamental opening and the feed to the outlet designed to reduce air flow turbulence. Black paint concealed the new devices and there is now controlled air distribution, making a more comfortable space.

The Reading Room ceiling

The Northridge earthquake in January 1995 was an unanticipated test of Arups' design. The contractor was ahead of schedule and had just completed shotcreting when the region experienced horizontal accelerations of 0.27g and vertical accelerations of 0.15g. Damage to non-structural elements was the major problem after the earthquake and plans had already been made to brace terracotta partitions which cracked. However the damage to the historic plaster ceiling in the Reference Room was a major challenge.

It consists of a pattern of cast plaster units reinforced with jute over the c300ft x 60ft area. The centre is a dome, flanked by flat panels at high level sloping down to a perimeter cornice over the remainder. During the earthquake a dome cornice piece had fallen and cracks between elements were visible. The plaster panels are attached from 1in black iron channels fixed to the back by wads of jute and plaster. These channels are suspended by wires or jute encased in plaster from a grid of small black iron channels, in turn wired back to the roof trusses. Inspection showed that the earthquake had broken some of the plaster attachments, causing panels to move and break loose.

Earthquake repairs to State-owned buildings are funded by the Federal Emergency Management Agency (FEMA) and since final completion of the Library was not possible without access to the Reference Room, approval for funding had to be gained as quickly as possible to minimise delays. With the help of Arup R&D in London and local specialists, various options were studied, but no direct precedent for the problem was found in a seismic zone. Since there is no lath in the plaster, any invisible fix would depend on the bonding of backing and hangers to the back of the existing plaster. A suspended net, bolting hangers through the panels, epoxy backing, and replication were all considered.

A.
Ceiling damage from the Northridge earthquake

Since the room will be occupied by many students 16 hours a day, safety was of paramount importance; however the preservation community felt that replication was not an option. Load tests for new backing systems and hanger capacity were carried out, and it was concluded that a hybrid solution would provide maximum confidence for life safety in future earthquakes and be acceptable to the historians.

This incorporated preservation of all the special decorative hand-painted panels which would have a glass-fibre backing applied to hold the pieces together in case of cracking. The rest of the ceiling would be replicated in the present-day equivalent, glassfibre reinforced gypsum plaster, cast in moulds from the original pieces and hand-painted to match. This would be attached to a new metal stud backing by cast-in fixings with the retained elements held in place by edge clips.

B.
Plaster-coated hangers suspend ceiling panel from black iron frame and steel structure above.

UCLA obtained approvals for this approach and the work is now complete. The quality of craftsmanship has been truly impressive, showing how modern materials can match historical standards, and improve the historical community's confidence in what can be achieved. When the Reference Room finally opens it will be a spectacular space once more.

Library environment

Besides seismic strengthening and HVAC upgrade, UCLA wanted this project to include provision for numerous computer workstations in open areas of study rooms, reading rooms, and classrooms in the Library. Reprogramming the Library space gave more individual and group study areas. A lot of book stack area disappeared, the IT revolution having allowed much Library material to be stored electronically. New electrical systems were needed within the constraints of the historic fabric to power this expanded computer use.

All the primary electrical distribution systems were replaced. Distribution of power and telephone/data outlets for the open areas was studied carefully. Raised floors were inappropriate, and distribution from the ceiling down was ugly, so poke-through floor units were selected for most areas. These required coring holes in the existing slab at predetermined locations and at a density limited by the National Electrical Code. The unit is installed flush with the floor, but protrudes into the ceiling void below and so can be used wherever an accessible ceiling occurs below.

Where decorative plaster ceilings below prevented use of poke-throughs, we located a series of 1in x 1.5in channels, cut into the top of the existing slab from the wall to an outlet in the space. This was a costly and limited solution, but allowed electrical power to be supplied to every space in the Library.

In the main Reading Room, computer usage and current expectations of environmental comfort required a cooling system as well as improved heating. The 65ft height meant that supply from the ceiling was not practical and no space was available in the walls. Arups devised a perimeter heating solution of radiators encased in the historic bookcases, with specially designed insulated metal enclosures and metal scoops to limit temperature effects on the wood. Cooling for the heat generated by computer use is provided by eight free-standing air-conditioning units also built into the cabinetwork. These are next to the readers, so close collaboration with the manufacturer and Arup Acoustics was needed to design a unit to meet the size and noise requirements. The tempered fresh air is also distributed through ductwork and grilles in the existing bookcases. By thus concealing the systems, there is no evidence of the technology hidden within.

Construction

The contract was let as a lump sum low bid, as required by UCLA. For a renovation, this means the documentation needs to be as comprehensive as possible to minimise the potential for changes due to unforeseen site conditions. Extensive verification of the existing MEP systems was therefore carried out, and surveys of critical but inaccessible structural areas. Having very detailed structural sections was invaluable in the field when the inevitable occurred. Arups had an engineer on site two days a week - not common USA practice - to help solve site problems.

Construction started in 1993 and as a result of close teamwork between the owner, contractor and design team, the project was completed on budget and ahead of schedule - a great achievement for such a complex project.

The main portion of the Library was completed in 1995, and the ceiling renovation finished for re-opening in summer 1996. Through sensitive design, the building has retained its historical character, but offers facilities that will take the Library into the next century.

Credits:

Client:
University of California, Los Angeles
Architect:
Moore Ruble Yudell
Consulting engineer:
Ove Arup & Partners California
Donna Clandening, Nancy Hamilton, Morgan Lam, Gary Lau, King Le Chang, Catherine Wells, Atila Zekioglu (structural)
Alan Locke, Jacob Tsimanis (mechanical)
Dan Ursea (plumbing)
Rahim Peyvan, Gregory Morrison (electrical)
Contractor:
Morley Construction
Illustrations:
1: Ove Arup & Partners
3, A: Moore Ruble Yudell
4: Trevor Slydel
5: Seymour Liao
6, 2, 8, B: Tom Sadowski
7: Morley Construction

City of Hope: Steel moment connection development

King-Le Chang
Hossein Mozaffarian
Atila Zekioglu

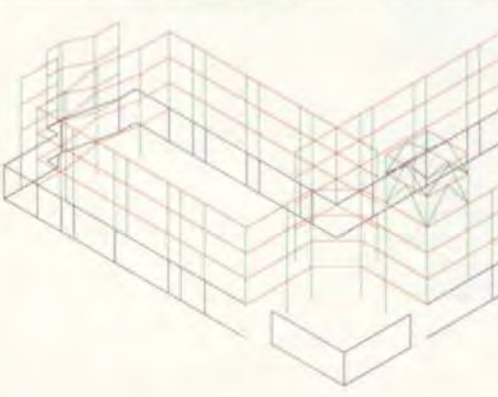
1. Model of City of Hope project.

Introduction

The City of Hope National Medical Center in Duarte, California, founded in 1926 through donations from individuals and organisations, is world-renowned for advancing the detection, prevention, and treatment of cancer and other serious diseases. Sited 25km east of Los Angeles, its existing premises range from single-storey wood-framed buildings for patients to three-storey 'non-ductile' concrete ancillary support buildings. The magnitude 5.9 Whittier earthquake in 1987 generally caused minor damage. Bobrow/Thomas & Associates, the project architects, completed a facility master plan in 1992 with new buildings for patients, research, medical offices, outpatient clinic, and central plant (Fig.1).
Arups' LA office was commissioned as structural, mechanical, electrical, plumbing and acoustics engineers.

2. Potential seismic sources in site region.

3. 3D view of steel moment frame system - Patient Pavilion.



Site seismicity and seismic codes

The potential seismic forces for this site are defined by the region's moderate and major faults (Fig.2): the Sierra Madre and San Andreas faults are 3km and 50km distant respectively. The proximity of the former, with a postulated magnitude 7.5 event, indicates that the site will be subjected to higher ground accelerations associated with 'near fault' phenomena.

Present US building codes require a conventional structure to be designed for a prescribed seismic force which is a small fraction of what the building would actually experience during a major event, on the premise that the structure has adequate inelastic ductility to deform without fracture and to dissipate seismic energy safely.
It is assumed that the system will be ductile and well-behaved and that the actual seismic energy input will not exceed the post-yield energy absorption capabilities of the plastic hinges. This approach fails to incorporate actual site characteristics, and has no basis to capture the effects of system redundancy, member/connection ductility, or deformation limits for 'acceptable performance'.

One of the most difficult questions to answer after an earthquake is whether a given structure has performed well or not. It would be easier to answer if the structure was initially designed to meet a certain performance objective, but this requires the establishment of quantifiable acceptability criteria. Currently, efforts are under way to develop performance-based seismic codes to address the current code deficiencies.

Initial approach

Discussions about site seismicity and the seismic code shortcomings with the client and project architects during October 1993 (prior to the January 1994 Northridge earthquake), led to the performance objective being set for the essential buildings, such as the patient pavilion, as follows:

'To provide a structural system capable of resisting the expected ground motion, 10% probability of being exceeded in 100 years, without critical damage to the structural system. Critical damage is, for this purpose, defined as that which could render the building unusable by the local or state agencies.'

Initially, the steel moment frame system was reviewed, which identified the overall system stability (Fig.3) and beam-to-column connection behaviour as the critical components in satisfying the performance objective.

The acceptability criteria were then set as follows:

'• The moment frame system will be proportioned to have sufficient strength and deformation capability to resist the design basis ground motion. This shall be demonstrated by non-linear superstructure analyses.

• The beam-to-column connections will be detailed on the basis of non-linear superstructure analysis results, plastic hinge rotations, and the 1988 enhanced moment connection test results.'

Three pairs of time history acceleration records, filtered and scaled to match the upper bound response spectrum, were prepared by the geotechnical consultant. The original selection process considered site similarities and focused on longer duration events . The non-linear analyses indicated a maximum plastic rotation of 1.6% radians.

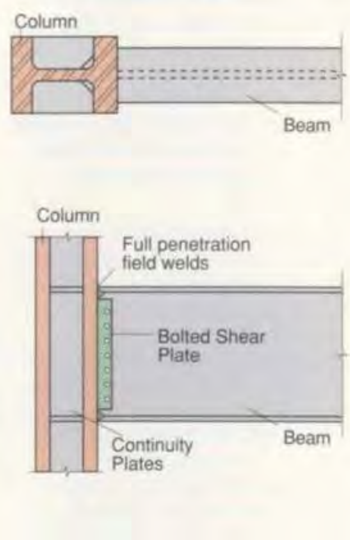
This maximum rotation served as the serviceability check for the connection development process.

Northridge earthquake

Steel moment frame systems, offering plan flexibility and construction time/cost advantages, have been widely used in California. Before Northridge earthquake (magnitude 6.6), they were regarded as one of the best seismic systems with sufficient ductility to resist major ground motion.

The limited published data on the pre-Northridge beam-to-column connection (Fig.4) tests, which indicated problems with its reliability, were mainly unnoticed and/or disregarded as being rare cases not expected to be duplicated in real life.

4.
Plan and elevation of 'Pre-Northridge' moment connection detail



Northridge earthquake - a relatively brief ground motion with moderate to major intensity - was the first real test in the LA area for steel moment frames, whose beam-to-column connections sustained failures ranging from cracks at beam flanges and/or welds to fractures through column flange and web (Fig.5). The failure rate was relatively high: over 200 steel moment frame buildings were damaged, out of an estimated 500-700 subjected to moderate ground motion.

In the first six to nine months following the earthquake, efforts were made to revive the prescriptive 'Pre-Northridge connection', with a combination of improved welding techniques and tougher weld wires. This proved unsuccessful and the code authorities prohibited its use. The Federal Emergency Management Agency (FEMA) then funded a four-year research program to reduce the earthquake hazards of steel moment frame structures.

The challenge

By October 1994 it was clear both that there were no quick fixes and that current projects with steel moment frames could not be put on hold until after the FEMA program. Two choices remained: to redesign the projects using alternate lateral force-resisting systems, or develop an acceptable moment connection. Arups decided to pursue the latter, more challenging, option.

Connection development methodology

On a project-specific development structural engineers, unlike researchers, do not have the luxury of learning from initial physical testing of concepts followed by modifications and laboratory retesting. In simple terms, 'learning from failures' to obtain a better product is not an option - no client wants to know or even hear about it. The learning process had to be based on studies of unsuccessful testing by others and damaged connections from Northridge.

5.
'Pre-Northridge' moment connection: Fracture through column flange.

The following key steps were identified:

- Study and learn from others' failures.
- Develop a concept for new connection.
- Perform non-linear analyses to compare variations of the concept.
- Select the new connection to be tested.
- Fabricate test specimens under simulated field conditions.
- Perform physical testing of multiple specimens.
- Correlate analytical and experimental results.
- Develop simple procedures to allow efficient design of new connection.

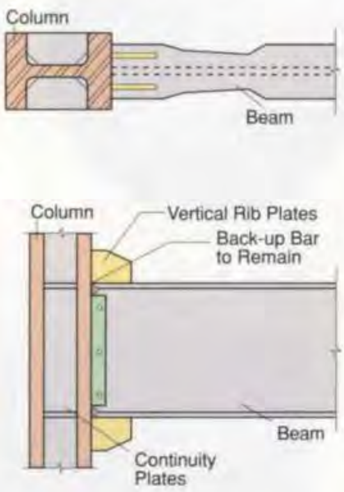
The first two tasks led to the following definition of basic connection features (Fig.6):

(1) Reduced beam flange area to predetermine a plasticity zone away from the field welds at the column face. Because the design of most moment frames is based on stiffness rather than strength, this would not require a heavier beam section. The beam flanges can be shaped either by flame-cutting or using drilled holes to achieve this area reduction.

(2) Welded vertical steel plates at beam-to-column interface to reduce through-thickness stresses at column flange.

(3) Welded shear plate at beam web, as opposed to a standard bolted connection, to transfer the bending and shear stresses in the beam web directly to the column. The welded shear plate also effectively mitigates secondary stresses should bolt slippage occur.

6.
Plan and section of new moment connection detail.



Finite element analyses

Non-linear finite element analysis (FEA) using the ANSYS program was used to investigate the proposed connection performance. Since material properties could not be estimated accurately, the FEA work was based on 'average beam yield stress + one standard deviation'. Various configurations were analysed to evaluate and proportion different features, as well as verify the simplified connection design procedure (Fig.7). Figs.7a and 7b illustrate models with constant and varying diameter drilled holes in beam flanges; Fig.7c shows taper-cut beam flanges. Flanges are reduced according to the beam plastic moment gradient to ensure yielding in this 'structural fuse' region.

These results indicated that the taper-cut flange scheme achieved the target plastic beam rotation of 3.5% radians - an artificial capacity required by the state agency - at much lower strain levels. Plastic straining in the reduced beam flange region was uniform, as opposed to strain concentration regions occurring at the beam flanges with drilled holes. The taper-cut flange scheme clearly performed better than the drilled-hole schemes; further, the behaviour of the model at a rotation of 1.5 to 2% radians was without apparent local buckling. Also, the beam-column interface stress levels were about 65% of the beam yield stress. The taper-cut flange scheme was thus selected for full-scale connection testing.

7 **(a) (b) (c).**
Non-linear finite element analysis.

(a) 'constant size drilled holes'

(c) 'shaved' flange

(b) 'varying size drilled holes'

Specimen fabrication and testing

Three beam-to-column flange specimens were detailed for fabrication, together with welding requirements to assure simulation of actual construction procedures. Beam sizes tested were rolled wide-flange US shapes, W36x150, W33x152 and W27x178 based on ASTM A572 Grade 50 specifications. The column sizes were W14x455 and W14x426. These heavy sections were used for three reasons:

- to force all the inelastic behaviour into the beam and allow for full strain-hardening
- to subject the column flange to the maximum possible stresses
- to consider the increased size of milling defects in larger rolled steel shapes.

The beam was laterally braced at the top and bottom flanges, at the end of the taper-cut region furthest from the column, to simulate the actual 'as-built condition-to-be'. This minimises lateral torsional buckling of the beam and allows it to develop its maximum flexural strength, thereby subjecting the column and the field welds to the highest possible stresses.

The test loading (Fig.8) was applied at the end of the beam in a slow cyclic fashion, and stopped at each peak displacement to observe the yielding and local behaviour (Fig.9).

The specimen responded excellently, achieving large beam plastic rotations beyond the 3.5% radian target.

Three different beam-to-column flange connection specimens tested during October and November 1995 exhibited similar results.

8.
Specimen in test frame.

9 **(a) (b) (c)** Below:
Connection specimen appearance at a rotation of 1.5 to 4.5% radians

(a) 1.5%

(b) 3%

(c) 4.5%

Correlation

The ANSYS FEA model was translated into a DYNA3D model to perform cyclic analysis runs matching the displacement history used in testing. The actual material properties, obtained from tension coupon (steel sample) tests, were used in this analysis.

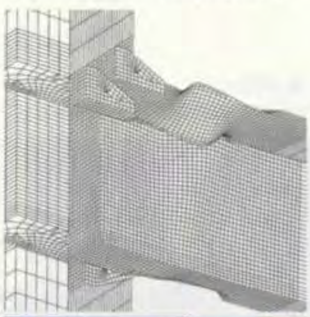
The close match between the computer simulation (Fig.10a) and the specimen after physical testing (Fig.10b) is noteworthy. More importantly the close match of force vs displacement results from the FEA and the test further verifies that the new connection behaviour is predictable by computer modelling.

Beam-to-column web connection

So far, the development and testing of the beam-to-column flange connection has been discussed. It is important to note that in most moment frame structures, due to plan configuration and/or to obtain a desired behaviour, it is typically necessary to have beam-to-column web moment connections. Before Northridge, very limited web connection testing was performed in the US. Since that event, Arups has been the only firm to develop and test this configuration. The concepts utilised in this development (Fig.11) were similar to the flange connection; the test results have been excellent, and similar to the column flange connections.

10 **(a) (b)** Below:

(a) Connection computer model appearance at 6in displacement.



(b) Specimen appearance after test.

Conclusions and future

Arups' new steel moment connection satisfies and surpasses the capacity requirements set by local and state building agencies. It is currently being incorporated in the new outpatient clinic at City of Hope (Figs 12 & 13), and will be used for the upcoming patient pavilion and diagnostic and treatment centre. The premium costs for the new connection over the 'Pre-Northridge' connection are reasonable, though it is problematic to consider this as a premium since the previous connection did not perform.

The beam and column sizes tested so far have been based on fairly large and heavy sections typical to moment frame construction in seismic zones. This connection could be used in new constructions with smaller or slightly larger sizes without any further physical testing, since no apparent 'scale effect' issues exist. It is intended to modify and further develop this new connection for application to skewed connections and upgrading of connections in existing buildings.

Arups' LA office gratefully acknowledges the co-operation of City of Hope in this development process and for agreeing to make this information public. The test results are being published through the American Institute of Steel Construction.

Reference

(1) POPOV, E, *and* TSAI, KC. Steel beam-column joints in seismic moment-resisting frames. Report No. UCB/EERC - 88/19, November 1988, University of California, Berkeley, California.

11.
Beam-to-column web connection specimen.

Credits

Client:
City of Hope National Medical Center

Architect:
Bobrow/Thomas & Associates

Consulting engineers:
Ove Arup & Partners California
King-Le Chang, Atila Zekioglu, Hossein Mozaffarian, Limin Jin (structural)

Other consultants:
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Law/Crandall Inc., Marshall Lew (Geotechnical)

Testing:
Professor Chia-Ming Uang, University of California, San Diego

Specimen fabrication:
The Herrick Corporation

W&W Steel Co.

Eagle Iron Construction Inc

Welding inspections:
Smith - Emery Company

Illustrations:
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3, Keith Chung
4, 6: Sean McDermott
5: Melani Smith
9b, 9c: UCSD
2, 10b: Atila Zekioglu
7, 12, 13: Paul Clifton
8, 11: David Hewitt
9a: Hossein Mozaffarian
10a: Alex Sturt

13.
Outpatient clinic:
moment connection after welding.

12.
Outpatient clinic under construction at City of Hope, utilising the new connection.

The Øresund Link

Jørgen Nissen

The Danish straits

The Danish straits are of special importance because they provide the only natural connection between the Baltic and the open seas. Until the Kieler Canal was completed (just in time for World War I) across the neck of Schleswig-Holstein some 70km south of the German/Danish border, most of the nine nations surrounding the Baltic could only gain access to the oceans through the straits.

The straits also function as hydraulic links. The Baltic is a brackish body of water, with two main layers: the upper less saline and more oxygenated than the lower. From time to time there are influxes from the open sea through the straits which bring saline, oxygenated water to the bottom layer. The straits are profoundly important for the maintenance of water quality and survival of marine life within the Baltic, and any scheme for crossing them must obstruct the water flow as little as possible.

There is heavy traffic across those straits which lie wholly within Denmark's boundaries; ferry services were established early and are among the world's busiest. The Little Belt, between the mainland peninsula of Jutland and the island of Fünen, was bridged in 1935 and again in 1970, and construction of a tunnel and bridge link across the Great Belt from Fünen to the largest island, Zealand, started in 1987 for completion in 1998. Attention is now focused on links across the Danish borders. The first contracts for the Øresund Link between Denmark and Sweden were let during 1995 and completion is scheduled for the year 2000. 1995 also saw the start of preparatory feasibility studies for a fixed crossing of the Fehmarn Belt between Denmark and Germany. All these crossings are for both rail and road traffic.

Each of the links in and around the waters of Denmark is of a similar size, and together they form a major investment programme which will change the transport patterns of the whole region. The Øresund and Fehmarn projects are, like Eurotunnel between Britain and France, part of a group of major trans-European links now being planned and built.

Crossing the Øresund

To build a fixed link across the Øresund is not a new idea. As early as 1888 an 'underwater bridge' was proposed, in which a railway was to run through a large pipe resting on the sea bed across Øresund between Elsinore and Helsingborg, 50km north of the final route. More realistic schemes were prepared in the 1930s, when a Danish-Swedish contractors' consortium presented a proposal for a road and rail bridge from Copenhagen to Malmö via the island of Saltholm. World War II interrupted all planning for the Link, but in the 1960s attempts were again made to advance the idea. A number of proposals were put forward but all were subsequently rejected. They included combinations of road and/or rail links between Elsinore and Helsingborg and/or Copenhagen and Malmö in tunnel and/or on bridge, and some included plans for moving Copenhagen Airport to Saltholm.

With the Great Belt Link under way, the Danish and Swedish governments finally entered into a binding agreement to establish a fixed Øresund Link in a Treaty dated 23 March 1991. The reasons for now backing the proposal were that the Link would not only create safe and effective traffic connections between Scandinavia and the continent of Europe - Denmark had joined the European Union in 1973 and Sweden was to join in 1995 - but also to lay the foundation for the Øresund region to become an attractive growth area fully capable of competing at an international level.

The Link's form was specified in some detail in the Treaty. It would carry a dual two-lane motorway and a twin-track high speed railway between Copenhagen and Malmö, but on a line south of Saltholm, which had been declared a nature reserve. There would be an immersed tunnel under the Drogden channel adjacent to Copenhagen Airport, an artificial transition island south of Saltholm, and a high-level bridge with spans over the Flinttrännan and Trindelrännan navigation channels, both in Swedish waters. The Link would be financed through loans guaranteed by the two states, and the construction and operating costs would be recovered through tolls. In autumn 1992 the two governments formed Øresundskonsortiet, a company owned jointly and equally by them, to be responsible for the financing, design, construction, and subsequent operation of the Link.

1. The Danish straits.

The competition

Øresundskonsortiet invited engineers and architects to take part in a competition at the end of 1992. The ASO Group was formed by Arups with SETEC of France, Gimsing & Madsen and ISC of Denmark, and Tyréns of Sweden, and was selected for the competition in February 1993 together with five other international groups.

The competition was in two parts: a bid for project management services and a design competition. The brief for the latter defined a form of the crossing similar to that set out in the Treaty, but it also invited proposals based on a more open brief to give scope for improving the 'reference project', as it was called.

The closing date for entries was in April 1993.

The competition was to be judged by three independent panels - aesthetic, environmental, and technical - which indicated that the client would attach equal importance to these factors. Under the Treaty the Link had to be designed and constructed with due consideration of 'what is ecologically motivated, technically possible and financially reasonable to prevent any detrimental effects on the environment'. Environmental issues thus took priority, and the major concern was the marine environment in the Baltic. With the need for water flow to be obstructed as little as possible, the limit to blocking by the Link's elements was subsequently set by the environmental authorities in both countries at 0.5%.

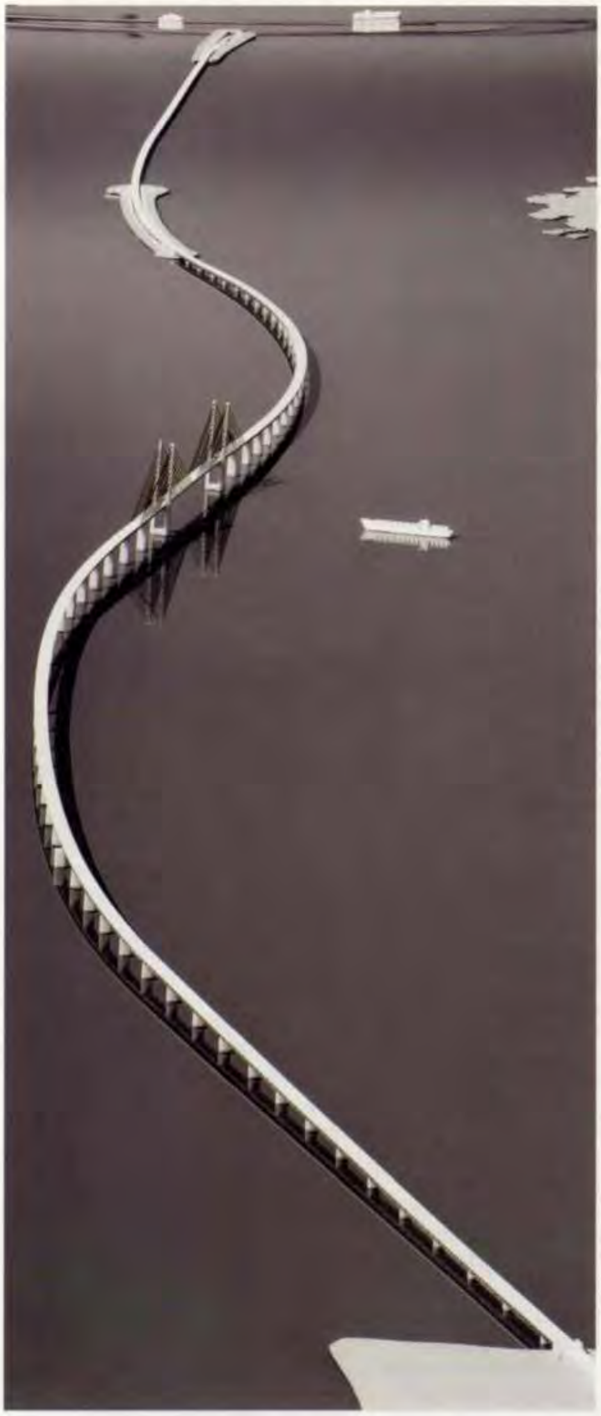
Competition scheme

The design presented special problems. There were to be three very different elements: a tunnel under the sea, an artificial island where the motorway and railway surface, and a long bridge with at least one large navigation span. This would be very large, standing in a seascape without natural forms for it to be set against. The landscape on either side is gentle and friendly, with small and rolling hills and curved coastlines where the land meets the sea. The Link will often be seen at a distance: from the shores, the sea and the air, whilst those on the Link itself will mostly see it at speed. The construction strategy was to have detailed design and construction carried out by different contractors on the various parts of the Link, breaking the continuity of the design process.

The reference project had only partially addressed these special issues, and the ASO team decided that the design could be improved in several ways to achieve a functional, economic, and elegant solution that would minimise the crossing's environmental impact and enhance the experience of using it. The strategy was to create a simple, rational, straightforward form; to express function without unnecessary details; and so to produce a strong and robust unity capable of being divided into smaller parts which could be detailed by different contractors and still be a harmonious whole.

2. The 'underwater bridge' proposal from 1888.

3.
Model of the competition design, showing the S-curve, and at the top of the picture, the two islands joined by the low roadbridge.



Key features

• *An alignment curved in plan*

The 1991 Treaty specified an alignment which was simply a straight line from the artificial island south of Saltholm to the landfall in Sweden. ASO proposed an S-curve alignment for the bridge to give users of the Link continuously changing views of the sea, the islands, the coastlines, and the bridge itself, and also to allow a nearly perpendicular crossing of the Flintrännan navigation channel.

• *Two smaller artificial islands, and a railway tunnel and low road bridge between them*

The Treaty specified a single 4km long artificial island in the waters south of Saltholm. ASO changed this to two smaller separate islands: the one to the east taking the railway from the main bridge to a tunnel while the road was kept above sea level until it reached the western island, where it joined the railway in tunnel. Although they were located in shallow water in the shadow of Saltholm, the two smaller islands reduced the blocking effect significantly below that of the larger single island. The proposal also made a total separation of road and railway traffic possible across the islands.

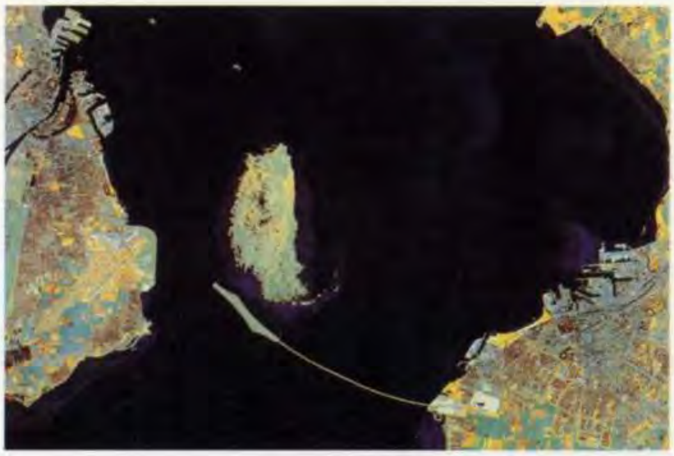
• *A two-level bridge structure between the eastern island and Sweden, with one generous navigation span across Flintrännan*

Separating road and rail traffic over the whole Link was a unifying theme in the ASO proposal. The Treaty specified bridge structures with the road and railway side by side, but in ASO's two-level proposal the motorway is above and the railway below. With this arrangement the most economical structural solution is to use steel trusses with diagonals connecting the upper and lower decks.

These trusses are uniform throughout the bridge, but modified at the cable-stayed main spans so that every other diagonal has the same direction as the cables. The 20m bay length of the truss is constant along the bridge and imposes a modular discipline on all the spans. The deep girders lead naturally to longer spans, which have environmental as well as visual advantages.

ASO proposed a single navigation span of 490m over Flintrännan instead of the 330m and 290m spans over Flintrännan and Trindelrännan specified in the Treaty. A truss sufficiently deep to accommodate the railway is naturally stiff enough to act as a deck for a cable-stayed span considerably longer than that required by the brief, so the opportunity was taken to provide one at Flintrännan which generously exceeded the minimum shipping requirements, thus avoiding the need for the other over Trindelrännan.

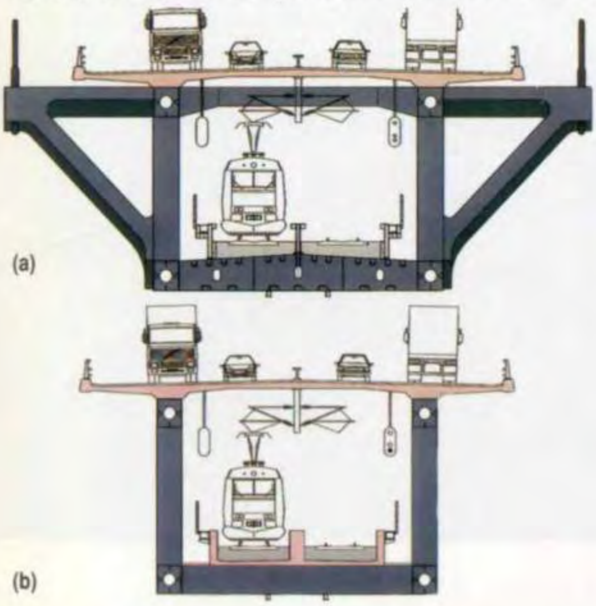
The inherent stiffness of the truss deck was also a factor in choosing a harp configuration for the cables. The live load moments in a slender deck are sensitive to the vertical stiffness of the cable system, which strongly suggests a fan arrangement. This does not apply to the truss deck. Its repeating geometry has also a natural affinity with the harp, which can be emphasised by adjusting the angles of the diagonals to match those of the cables. This resulted in simple details where the force from the cable anchorages is delivered to the bottom chord through sloping brackets. (Arups previously used the concept of aligning truss diagonal and cable angles in 1988 in a competition entry to replace the Williamsburg two-level suspension bridge across the East River in New York. In the event, the existing bridge was repaired.)

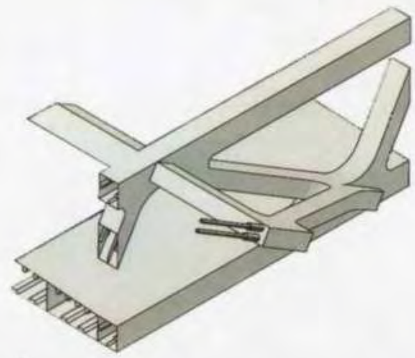


4.
Satellite image of Øresund, showing Copenhagen (left), Saltholm (middle), and Malmö (right).
The final route and island are graphically superimposed. During design development the final alignment was changed to a shallow C-curve, and the two islands combined.

5.
Drivers can experience the principal feature of the crossing, the main bridge, from anywhere on the motorway. The proposed lighting scheme includes low lighting at parapet level and special illumination lights for the main towers.

6 (a) & (b) Below:
The 30.5m wide cross-section for the main bridge **(a)** is similar to that for the approaches **(b)**.
The approach girders are steel lattice girders 10.2m deep with concrete decks for the road and the railway.
The lower concrete deck is shaped to contain derailed trains. The 20m bay module is constant throughout.





8.
The stay cables are anchored at brackets that transfer cable forces to the girder structure. The cable planes are vertical and placed at a distance from the carriageway edge, making maintenance easier and reducing risk of accidental loads on the cables.

The harp system has a visual formality, particularly apparent when cable planes are vertical, and the towers were designed to express this. The effect is further enhanced because each cable plane is supported by independent towers unconnected above deck level. This requires the cable plane to coincide with the centroidal axis of the tower, which is 2.3m from its face at roadway level. The cables are thus at a safe distance from the roadway and protected from accidental damage.

Using only one cable-stayed bridge also avoided the unfortunate visual effect of two similar but differently-sized bridges near to each other. The Link would have only one visual peak, the importance of which would be emphasised by the longer span and higher towers. The two-level structure also has operational advantages as it keeps the high-speed trains away from the road users and allows both modes of transport the flexibility of cross-overs, so that sections of track or carriageway can be by-passed when necessary. Both car and rail travellers also have a better view of the Øresund.

7.
The main bridge is a harp cable-stayed bridge with two side-spans. The diagonals are modified in the main bridge to suit the layout of the stay cables.

The pylons are vertical, with individual towers not connected at the top.

The centres of gravity of the towers are on a vertical line, so the inner faces will slope slightly outwards to counteract the sense of an overall inward slope

9. Graphic superimposition showing the completed link, post-2000, from the Danish coast .

The consistent form of the simple and strong horizontal truss girders is the principal feature that gives the design a sense of unity. The overall effect is a clear statement of structural purpose: strong horizontal girders supported on concrete piers at the approach sections and, at the main span, the cables which continue the line of the truss diagonals to the 200m high masts of the two supporting towers.

From competition to tender

The result of the competition was announced in July 1993 when two design proposals were chosen to be developed further. They included two bridge alternatives: ASO's two-level design with the railway below the motorway, and the ØLC Group's proposal for a single-level bridge with the railway between the two road carriageways.

Both competition designs were developed in parallel beyond what had been possible in the short competition period. Technical and environmental studies, and consultations with the relevant authorities in both Denmark and Sweden, were also undertaken and the designs modified as required. In the summer of 1994 it was decided that both bridge alternatives should be taken forward to tender on a common alignment and with common designs for the tunnel and the artificial island.

The two-level bridge proved a robust design and fundamental changes to the concept were not found necessary. However, the idea of a railway tunnel and a low road bridge running between two small islands did not survive the further studies - mainly on economic and environmental grounds - and the S-curve was altered to a C-curve when changes were made to the alignment at Copenhagen Airport in order to simplify construction through the Airport.

Although the changes were 8km from the western end of the bridge, they altered the alignment direction at the artificial island; this, combined with bird sanctuary restrictions south of Saltholm, made the S-curve unfeasible.

Most legislation, rules and regulations, codes of practice, etc, differ between Denmark and Sweden, and much work was done to develop special documentation for the project. As a basis for the design the Eurocode system was adopted.

However, not all the relevant Eurocodes are complete and many exist only in draft form, so to adapt them to this project a set of project application documents was prepared giving amendments to individual Eurocodes. Partial safety factors and load combination factors were calibrated, and accidental load cases identified and defined to satisfy the operational risk acceptance criteria developed for the completed Link.

Many items in the design basis document are based on a rational assessment of risks. The acceptable risk and safety levels were taken to be comparable to similar traffic installations in Denmark and Sweden. Specific risks of fatalities and total disruption were assessed and compared with similar for road and rail transportation on land, and risk acceptance criteria were defined accordingly. Risk investigations were carried out to define the frequencies of various critical hazards. For the bridge these are ship collision with the bridge piers, ship collision with the bridge girder, aircraft collision, fire on the road, and fire on the railway.

Reasonable safety levels on the motorway and the railway are primarily achieved by designing structures and installations to minimise the frequency and consequences of accidents. There are facilities for rescue, fire fighting, clearance, and quick reopening of the bridge for normal or alternative traffic, and it has been designed to accommodate a traffic management system with traffic detectors, variable traffic signs, traffic lights, automatic weather stations, etc.

This system is connected to the overall Øresund Link traffic management system which will be the SCADA-type (Supervision, Control And Data Acquisition). All information is collected in a Link control

Time schedule:

Detailed planning and design
Dec 1995 - Feb 1997

Establishment of work areas
Dec 1995 - Jan 1997

Caisson and pier elements
Jul 1996 - May 1999

Deck elements
Sep 1996 - Jun 1999

Marine works
Nov 1996 - Sep 1999

Eastern approach bridge
Feb 1996 - Jul 1999

Western approach bridge
Jul 1997 - Jan 2000

Cable-stayed bridge
Feb 1997 - Jun 1999

Completion
Apr 2000

Some statistics:

Total length of Øresund Link:
15840m

Length of artificial peninsula:
430m

Length of tunnel:
3510m

Length of artificial island:
4055m

Western approach bridge:
3014m (four 120m spans and 18 spans of 140m)

Main bridge:
1092m (490m navigation span plus 160m and 141m side-spans on both sides)

Eastern approach bridge:
3739m (three 120m spans and 24 spans of 140m)

Total length of bridge:
7845m

Main quantities for the bridge:

Earthworks:
300 000 m^3

Structural concrete:
250 000 m^3

Reinforcement steel:
40 000 tonnes

Cable-stay steel:
3 000 tonnes

Structural steel:
90 000 tonnes

Contract sums:

Tunnel contract:
3766M DKK (at February 1995)

Dredging and reclamation contract:
1388M DKK (at February 1995)

Bridge contract:
5350M DKK (at June 1995)

centre from where it will be possible to take immediate action when traffic flow irregularities occur. The railway traffic will be managed by the Danish and Swedish railway authorities according to agreed requirements.

Tender

The client decided to let the works in design-and-construct contracts. These have some obvious advantages, but clients have little direct influence on designs as long as the functional requirements formulated in tender documents are met and the design conforms with the design basis.

In this case, extensive consultations were held with authorities in the two countries on matters like aesthetics, environment, road and railway operation, navigation, safety, etc, and risk analyses were carried out to identify major risks and measures to alleviate them. The designs prepared by the consultants were markedly influenced by these activities and included many features over which the client wished to maintain control. Some of these could be expressed as functional requirements but some important design features could not. The tender documents therefore included not only the usual design and construction requirements but also Definition Drawings which described the design features, geometry, and materials that should be retained in the contractor's design. The tender documents also included a set of illustrative design drawings as an example of a comprehensive design which fulfilled the client's requirements.

The two bridge proposals were issued for international tender in November 1994. Each was in two packages, one for the approach spans and the other for the cable-stayed elements comprising the centre span and its flanking side spans. By tendering the bridge in two parts the client could combine the two minimum bids for each proposal, which could be expected to lead to a lower total price than one bid for the whole project. The bids were returned in June 1995. The seven bids for the cable-stayed spans showed a price variation of 42% from the lowest to the highest, whilst the seven for the approach spans had an even broader spread of 56%. The lowest bids for both parts were both for the two-level concept.

After lengthy evaluation of all the bids, a single contract for the whole length of the two-level bridge was signed with Sundlink Contractors in November 1995. The tender designs conform very closely with the illustrative design.

Constructing for the Millennium

A fundamental principle adopted in developing the conceptual design was that it should allow for economical construction, resulting in the minimum possible adverse environmental impact.

The objectives the ASO team set itself were to demonstrate that the design was practical, that it could be built within the implementation programme set by the client, and that it would allow competitive tendering for the proposed design-and-construct contracts. This was achieved by incorporating scope for the following:

- factory condition prefabrication of large sections of each element of the Link
- large-scale erection operations for which tenderers would be able to utilise existing plant.
- repetition of detail design and construction details.

The selected contractor has to a high degree based his tender on these assumptions and will prefabricate concrete caissons and columns on land in Malmö. Complete bridge spans including the concrete decks will also be prefabricated on land, the approach spans at Cadiz in Spain and the cable-stayed spans at Karlskrona in Sweden. The main in situ parts will be the towers to be cast in climbing formwork.

The construction programme has to take into consideration environmental issues and must not interfere with shipping traffic in the Øresund. The migration of herring through it sets limitations on the programme, and the dredging and reclamation works in the area have to be scheduled precisely. The shipping traffic requirement is very restrictive; in particular, the requirement that work in the existing Flintrännan navigation channel cannot commence before completion of the realigned navigation channel has a considerable influence on the programming for the whole Link.

Throughout the construction period the ASO Group will monitor and audit the contractor's design and construction activities.

When it is completed in the year 2000, the bridge will be the longest railway bridge in Europe, and the 490m span the world's longest cable-stayed span to carry both road and railway traffic.

Credits

Client:
Øresundskonsortiet

Members of the ASO Group:
United Kingdom: Ove Arup & Partners

France: SETEC Travaux Publics et Industriels

Denmark: Gimsing & Madsen A/S; ISC Consulting Engineers A/S

(Tyréns Företagsgrupp AB, Sweden, were members of the Group until December 1993, when they withdrew as it was then clear that ASO would not be involved further in the tunnel and artificial island.)

Architect to the Group:
Georg Rotne

Sundlink contractors:
Skanska, Sweden
Hochtief, Germany
Højgaard & Schultz, Denmark
Monberg & Thorsen, Denmark

Designers to the contractor:
COWIconsult, Denmark
VBB Viak, Sweden

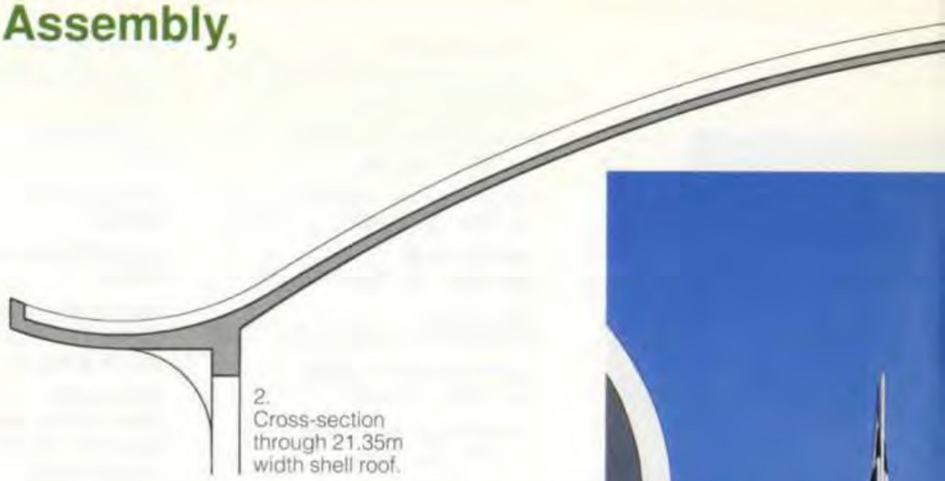
Arup team:
Andrew Allsop, Gert Andresen, Chris Barber, Allan Delves, Klaus Falbe-Hansen, Don Fraser, Clive Gaitt, Graham Gedge, Ajoy Ghose, Fraser Gillespie, Mark Howell, Chris Humpheson, Naeem Hussain, Peter Knight, Margaret Law, Phil Lee, Ai-Ling Lim, Angus Low, Chris Manning, Alain Marcetteau, Charles Milloy, Strachan Mitchell, Paul Morrison, Jørgen Nissen, Douglas Parkes, Line Petersen, Jeremy Shapley, David Snowball, Lene Strand, Ian Wilson

Illustrations:
1, 2: Peter Speleers
3, 5-8: ASO Group
4, 9: Øresundskonsortiet

New Chamber, National Assembly, Gaborone, Botswana

Simon Nevill

1. Banknote illustrating original National Assembly.



2. Cross-section through 21.35m width shell roof.

3. Original building on the left and the new chamber on the right, all fronted by the extended colonnade.

Introduction

In 1965 Ove Arup & Partners were appointed by the Government of the British Bechuanaland Protectorate (now Botswana) to design and supervise the construction of a Parliament Chamber and supporting offices, for completion in time for Botswana's independence on 30 September 1966. The design work was undertaken by the Bulawayo office but with assistance from the London office on the main chamber design[1].

Nearly 30 years later, in 1993, Arup Botswana were approached by the Department of Architecture and Building Services to design a new and larger chamber for the expanded parliament. This allows for appropriate representation of a population that has grown from 500 000 to 1.4M in the intervening period. The project commenced on site in January 1994 and was essentially completed in November 1995.

Design proposals

The National Assembly is located at the centre of the Government Enclave in Gaborone, Botswana's capital, and is overlooked by a statue of Sir Seretse Khama, the first President of Botswana. The original development consisted of the main chamber with its shell roof, and a single-storey complex of offices and support facilities. In the intervening years a chamber for the House of Chiefs and a two-storey office building have been added, and a shell roof colonnade links the whole complex together. Immediately to the north of the chamber was an area of open ground used as an unsurfaced informal car park.

The design proposal for the new chamber was to create a much expanded but similar shaped building to the original chamber on the site of the car park, with the colonnade extended to tie the whole National Assembly together. A formal and more secure car park would be constructed around the new chamber. It was clear that the central feature of the new chamber was to be the shell roof, twice the width of the original.

The original chamber

This double-volume space has a gallery overlooking the chamber and two levels of meeting rooms, press rooms, and support facilities at the east end. The building is covered by a shell structure, 24.4m long with a width of 10.7m and a rise of 2.1m. The shell thickness varied from 150mm at the edge beam to 89mm at the crown, and was stiffened by three beams across the chamber with a top profile identical to that of the roof. The building was constructed by a Rhodesian company for the sum of £111 000.

The new chamber

The architectural solution was to create a scaled-up version of the original, increasing the seating capacity from approximately 40 to 76 seats for Members of Parliament. As with the original, the main chamber was conceived as a double-volume space with galleries. In the initial scheme there was a full basement car park, but this was later deleted. A basement plantroom was included but this was located immediately outside the west gable wall of the main chamber.

The final solution for the chamber roof was a shell structure 30.5m long with a width of 21.35m and rise of 3.86m. For these parameters the shell thickness varied from 140mm at the edge beam to 100mm at the crown. At the sides, the roof was extended a further 2.3m beyond the column line, the whole shell being supported on edge beams 300mm by 600mm deep on 300mm square columns at 3.05m centres. The shell was stiffened by three transverse beams spanning across the building.

When the original roof was designed in 1965 the use of shell structures was very much more common than in 1993. In 1965 a number of technical design papers were available, giving both empirical and analytical solutions, with the design information presented in the form of graphs and tables. Computer solutions were not really available. In 1993 the initial scheme designs were based on the empirical solutions. However, the final detailed design was based on a GSA grillage analysis.

The colonnade

The 18 original shells along the front of the National Assembly have been joined by an additional nine shells of exactly the same geometry to link in the new chamber. The shells are 75mm thick, 5.5m long, width 3.05m, and have a rise of 700mm; they are supported on a series of 300mm by 150mm columns. A design check was undertaken to ensure that the original details were still valid in terms of the current standards, the area of most concern being the slender support columns. They were vindicated, though during construction it was decided to clad them in granite and make them square.

Integrated into the colonnade was a 20m high clock and bell tower and a barrel-vaulted entrance portico, the latter being a tied concrete half-cylinder 16.6m long, with a width of 6.1m and a radius of 2.9m. The concrete barrel is 75mm thick and the ties are 25mm diameter stainless steel. Suspended from the crown of the barrel is a half-ton bronze and stainless steel globe with Botswana highlighted.

4. New building under construction.

5. Interior of new chamber.

Ground conditions and foundations

In Gaborone the general soil conditions are fill, transported soils, ferricrete, residual soils, and granite bedrock. The transported soils are interesting in that they have a very open grain structure, which gives high strength when the material is dry. However, when the soil becomes wet and load is applied there can be a sudden and significant collapse settlement as the grain structure breaks down. The foundations for the building were therefore taken down through the potentially collapsing soils and onto the soft granite rock with an allowable bearing pressure of 1000 kN/m^2.

Construction

Building the shell roof was the most interesting and demanding element of the contract. The contractor intended originally to construct it in three sections using a mobile scaffold; however, the design concept is that the shell spans along its length and this method would have compromised the structural integrity, so it was necessary for the contractor to prop fully the whole roof. Curing the concrete was a key concern, as it would have a very thin cross-section and be cast potentially at very high temperatures - the summer maximum in Gaborone can be over 40°C. Fortunately, it was cast in the middle of winter when the temperature was much more reasonable. The contractor was not using a top shutter and thus used a very low slump concrete mix design. Generally this worked well but in one area it led to extensive honeycombing of the concrete which had to be cut out and recast, requiring significant re-propping of the roof.

Conclusion

At the beginning of the construction period several significant changes were made to the project to enhance the quality of the internal finishes. The overall effect is to create an open and warm environment by the use of extensive wood panelling, bronze fittings and marble. The building was handed over to the National Assembly in time for the opening of parliament in November 1995 at a final cost of Pula 12.5M (£3M).

The opportunity to work on this project with the historical connections back to the original scheme was rewarding, and being able to discuss and review details with a member of the team that undertook the original design was a great benefit. 30 years on, the original Arup drawings and calculations, as well as some of the project correspondence, made interesting reading.

Reference

(1) WALKER, E. Botswana. *The Arup Journal*, *2*(3), pp2-5, 1967.

Credits

Client:
Botswana National Assembly Department of Architecture and Building Services

Architect:
Mosienyane & Partners

Civil and structural engineers:
Arup Botswana Dan Adorisio, Noah Banda, John Blanchard, Olise Mhone, Ian Miller, Simon Nevill (structural)

Doug Walton, Keith Harwood (civil)

Mechanical engineers:
Lasco Engineering

Electrical engineers:
Stewart Scott International

Main contractor:
Green Industrial Enterprises Corporation

Illustrations:
2: Dave Bryant
3-5: Illustrative Options

Johannesburg Athletics Stadium

James Burland Alan Jones Rob Lamb

1. Visually and practically, the stadium locks into the heart of Johannesburg.

Introduction

In mid-1993 Johannesburg City Council invited bids for the design and construction of a world-class athletics and football stadium close to the existing Ellis Park rugby stadium and indoor arena. It was conceived as part of the City's bid to host the 2004 Olympic Games - a domestic contest between Johannesburg, Durban, and Cape Town which the latter won in January 1994[1]. The seating capacity required was 40 000, with the facility to upgrade to 80 000. A team was formed with Arup Associates, in association with local architects RFB, as principal designer, and Ove Arup Incorporated's Johannesburg office as civil and structural engineers.

Stadia are always in danger of becoming formula buildings, imported and nationally anonymous, which would have been particularly inappropriate for Johannesburg now. The design team took great care to ensure that the stadium could be built with local materials and resources; also, its shape was driven both by the needs of international athletics and by the desire for a sensitive solution within the landscape setting of East Johannesburg: crucial for the successful regeneration of the Ellis Park precinct.

The client favoured Arups' design because it resolved the planning issues, but the price submitted by the contractor-led team was unacceptable. After negotiation, Arups' professional team under the leadership of project managers Pro-Crit was appointed by the Council, the design being cut back considerably to meet an extremely tight budget. The seating capacity was reduced to 38 000, and the upgrade to 55 000.

The site

Other urban design consultants had prepared sketches and concepts for the new Ellis Park precinct, linking the athletics and rugby stadia and creating a remarkable setting for international events. The site slopes eastward towards the airport, with the Central Business District skyline as its western backdrop, and the seating bowl is landscaped to extend a planned green corridor from the edge of the city. The landmark potential for the stadium structure and geometry, the powerful lighting required for television, and the combining of parks and open belts in the valley between the flanking Witwatersrand ridges to the north and the south, all challenged the architects to produce a strong new international image for Johannesburg.

All the following were important:

- the impact of the stadium on the inner city area
- the effects of scale and massing, both the long-distance view and from the rear
- lighting halation
- safe operation in the tight site boundaries, even with 55 000 spectators.

The ideal geometry - circular perimeter and oval arena - was exploited to create varied scale in the bowl itself, from 9m to 36m high, surmounted by a tracery of masts and cables supporting a 'paper-thin' roof. The masts are a repeating motif varying from 30m-50m high around the bowl, signalling entrances and orientating approaching spectators.

The brief

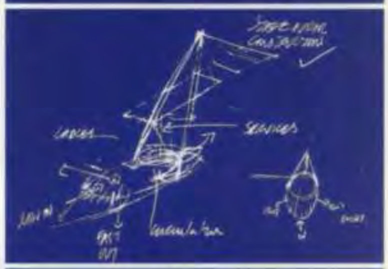
The stadium's main use is for international athletics; other types of events include soccer, concerts, and spectaculars. The detailed brief evolved during evaluation following the initial competition, and in response to the budget subsequently set. Vertical circulation, excavation, and the size of the roof (especially its cantilever) were all rationalised to achieve significant cost savings without losing architectural expression, massing, and planning concepts.

International athletics has security and organisational characteristics that must be enabled by shell and core in planning the stadium; the demands of accreditation necessitate separate areas for athletes, media, VIPs, officials, and spectators. The competition sketches took these factors into account whilst demonstrating how the stadium could cater for much smaller regional events, concerts, spectaculars, carnivals, and product launches.

All of them need:

- access for heavy vehicles into the arena
- egress for thousands of spectators from the arena floor
- large areas for stage requirements
- the ability to convert swiftly from sports to concert use and back again.

2



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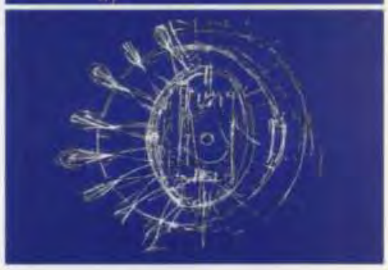
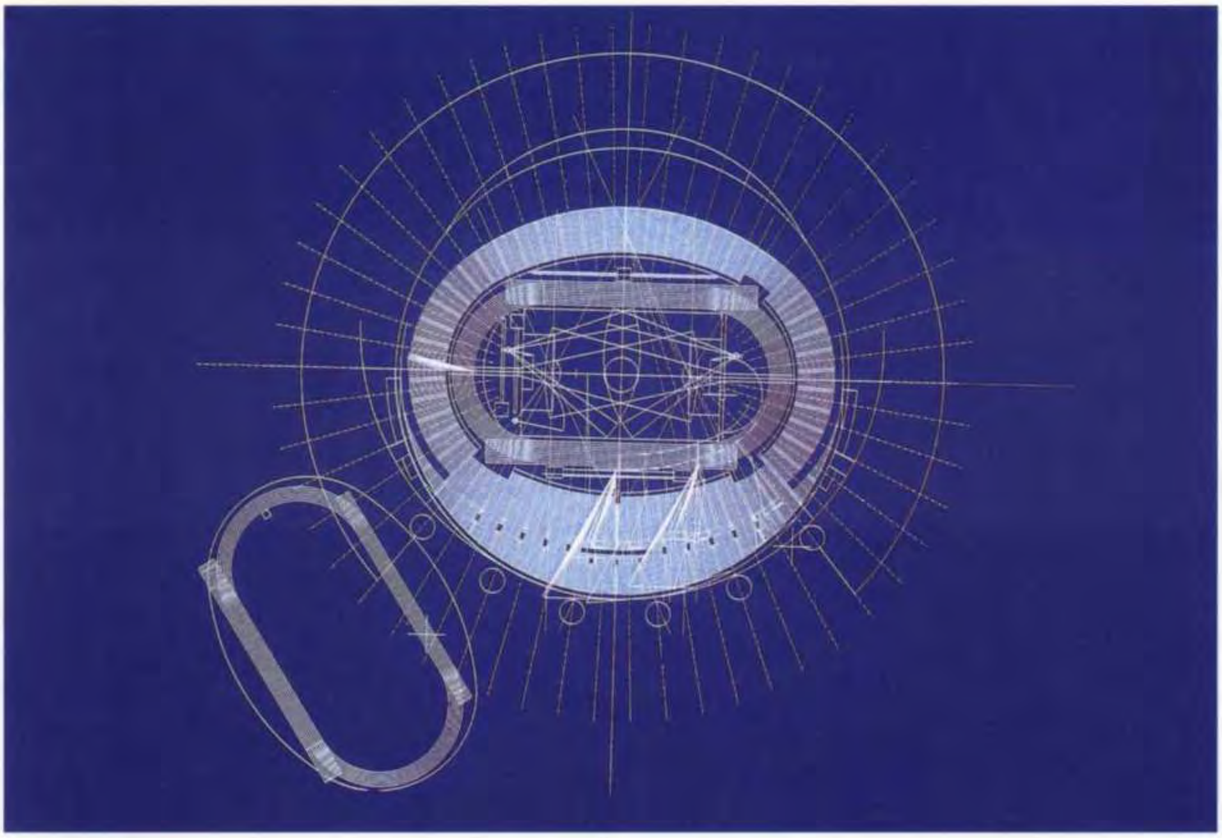


4

CONCEPT DEVELOPMENT

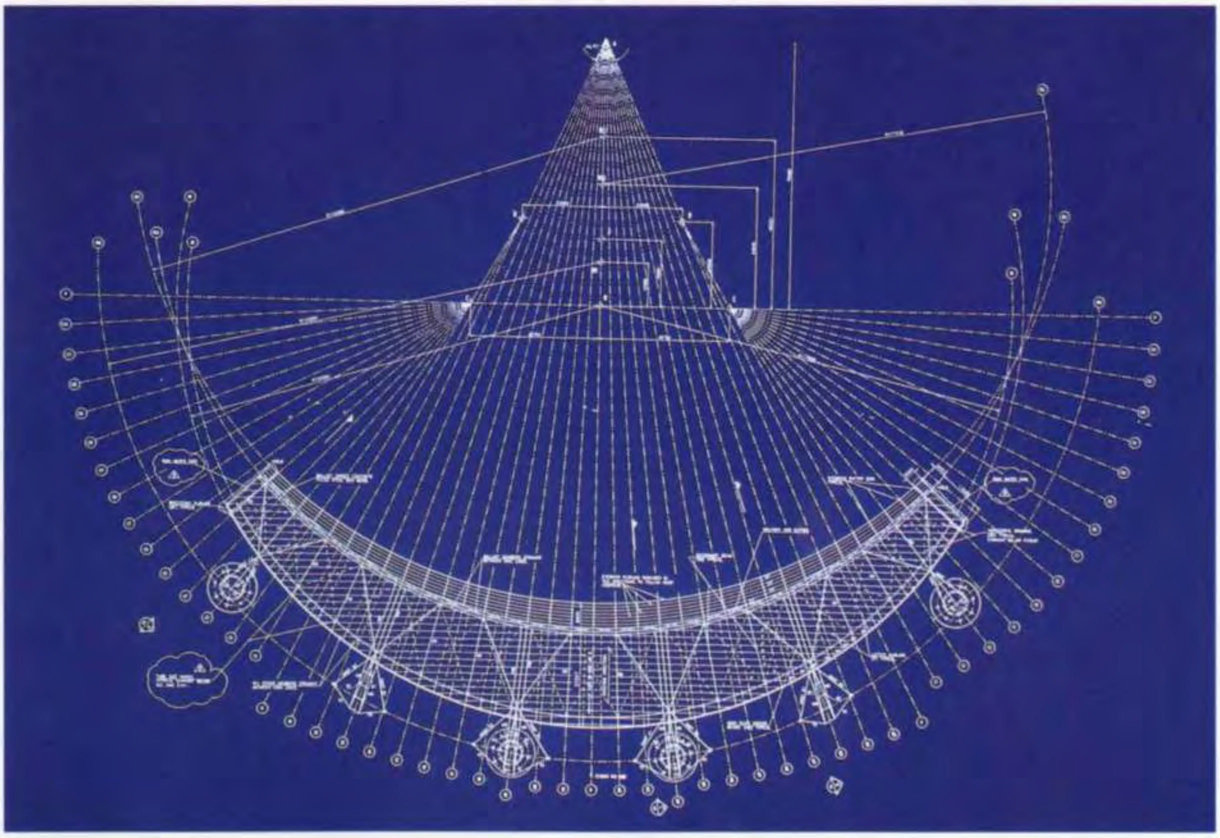
2-4.
Johannesburg grew from the wealth generated by the mining of ridges that run east-west and structure the city. The stadium's gantry-like masts, combined with the continuous ridge - environmentally favourable in design for athletics - are a deliberate visual correspondence with this aspect of its history.

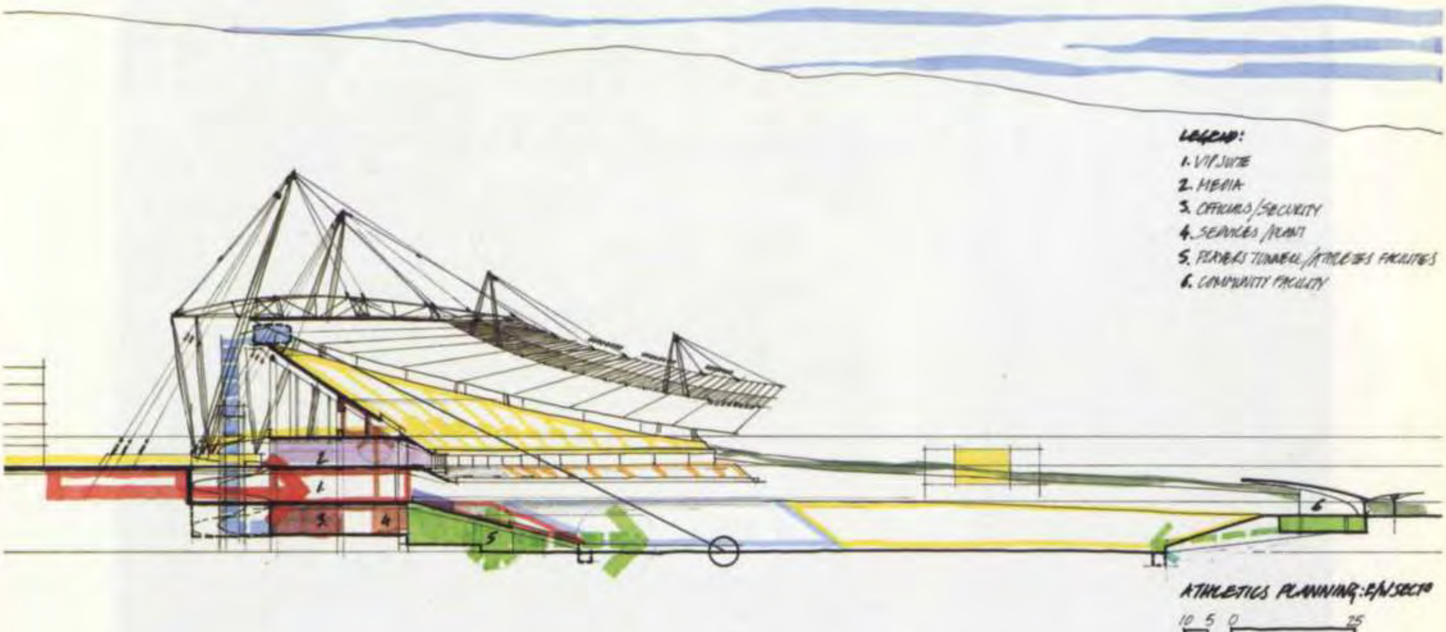
Good sightlines, generous concourses, and clear ramped circulation all enhanced the orientation, comfort, security, and safety of the spectators. Engineering for crowd safety was particularly pertinent on this restricted urban site, as well as in the overall city context. Media requirements are highly important, especially in internationals where as many as 180 countries may be represented. Arups designed for expansion and contraction of space according to different types of events, the proximity of the media centre in the stadium, and provision of dedicated cabling routes.



5.
Stadium plan, with the practice track at the bottom left.

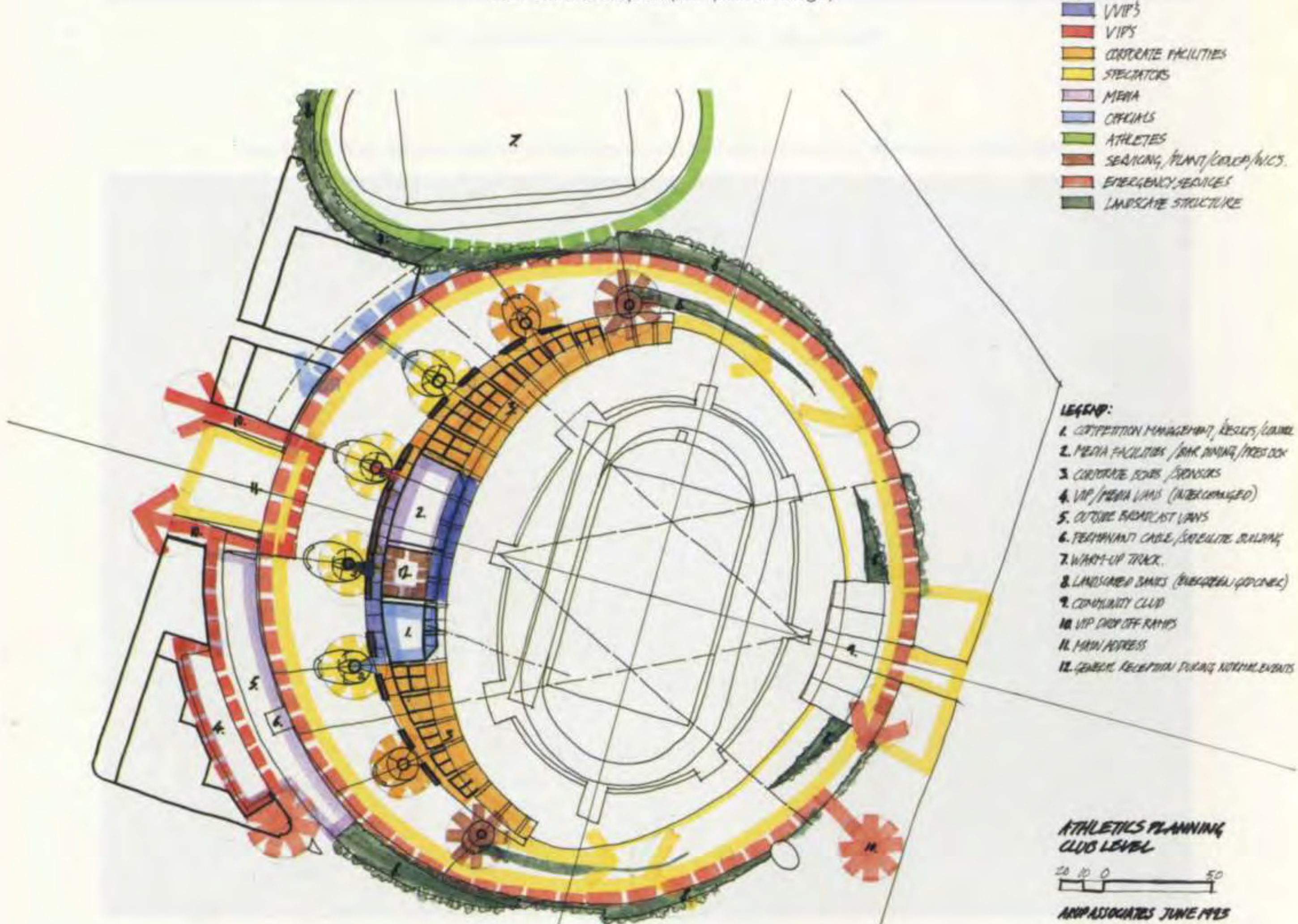
6.
The elliptical plan of the mast-and-cable roof curves vertically to create a shallow oyster shell form.





7.
East-west section (competition stage).

8.
Main concourse plan (competition stage).



The design

Site planning strategy

The stadium location allows essential ancillary areas to be placed around its perimeter, each with a separate entrance and adaptable to the specific needs generated by the size and type of event. Particular features of the planning are:

- a warm-up field near the start line, with ramp access so that athletes do not have to negotiate steps
- VIP parking behind the best seats
- parking for outside broadcast trucks close to stadium and media facilities
- perimeter spectator concourse for safe circulation
- perimeter emergency vehicle route for crowd control and security.

For small events these areas can be given over to other uses, but without them the security, safety and programme requirements for world-class events would be compromised. The service areas are concentrated on the west side, which allows for the future expansion to 55 000 capacity without radical re-organisation.

The seating plan

Though athletics can create onerous sightlines and spectator desire patterns, it can be exploited to generate a very attractive and intimate seating bowl. At Johannesburg this incorporates:

- a circular external perimeter within the maximum viewing distance to the centre of the field
- an oval arena for excellent all-round sight lines
- a 38 000-seat bowl with emphasis on providing seats along the finish straight, the most popular zone for athletics
- overlapping tiers of seating to minimise the overall perimeter
- seating as close to the track as security will allow
- the maximum safe angle of seating rake (34°) for the upper bowl.

Landscape and orientation

The stadium is oriented 15° east of north. This, combined with its slope to the south-east and the lowering of the field below natural ground level, is enough to gain grade circulation into the mid-level of the seating bowl on the west. The fact that the bowl forms a crater in the landscape, rising to a crest on the west, places many seats on the ground, which economises on superstructure and cladding. The main stand being in the west also protects the majority of spectators from the afternoon sun.

Section

This has the following features:

- four main levels (services, main concourse, club level, and upper concourse)
- roof cantilevering over 43 seating rows and shaped to give all seats an unobstructed view of the arena and the scoreboard
- ramp circulation integrated with the A-frame masts supporting the roof
- overlapping seating tiers to bring the perimeter in as close as possible. This economises on the total area and enhances the arena's intimacy.
- security moat between the arena and seating, giving discreet access for athletes to field and relay events and accommodating the services cabling to the in-field events timing and measuring equipment
- a leading edge to the roof to soften the sharp shadow cast across the field, so that TV cameras can cope better with harsh, contrasting sunlight conditions.

'Club' concourse

The club level is an independently serviced concourse with access to approximately 1500 seats. This gives a flexible arrangement for corporate boxes, competition management, the press, club/dining, and media bar.

Crowd safety and access

Recent severe problems in some European football grounds have prompted careful reassessment of the design of large-scale facilities.

The main considerations incorporated at Johannesburg are:

- good seats with clear sightlines and adequate space for every spectator
- elimination of lengthy unprotected staircases
- use of shallow gradient ramps for vertical circulation, which can also accommodate vehicles
- wide and clear upper level concourses
- visible entrances
- clear routes from the stadium perimeter to seats
- generous external concourse encircling the entire stadium
- emergency vehicle access permanently clear of obstructions around the external concourse.

The stadium is designed to clear in eight minutes. General spectator seats are in the upper bowl on the west side and the lower bowl to the north, south and east. Concessions and amenities are widespread, and tucked under the seating rake or into earth embankments. The upper tiers are designed with vertical aisles and vomitories with no lateral gangways to minimise crowd disruption because of frequent spectator movement related to timing and location of track and field events. The shallow ramps to the upper bowl allow spectators to circulate at high level, bypassing the VIP and other accredited entrances on the west side.

The plan/section design has clear horizontal division with vertical distribution ramps regularly spaced around the perimeter. This helps to fulfil the safety requirements, and also allows for a flexible segregation of media, VIPs, officials, and competitors for athletics and other activities.

Lighting for image and atmosphere

The approach and siting of the stadium, combined with its sweeping roof line and visible seating bowl, all build up a dramatic image against the dark ridge to the north of the site.

The lighting strategy provides 1500lux for high quality television, and reinforces the roofline. The lighting masts to the east lean forward over the track and field and punctuate the silhouette.

Athletes

Marshalling the athletes and participants for pre- and post-event co-ordination becomes more efficient and secure if it is compactly arranged between the warm-up field and the start line - an important factor when bidding for some of the world's most prestigious events.

The key provisions for the athletes are:

- generous WC amenities by the pre-event holding areas
- mixed media interview facilities on the finish line with an attractive backdrop across the track and field
- medical facilities at track level close to the finish and the emergency exit route
- dope testing facilities, secure from external interference, near the finish line area
- discreet moat access to events away from the start line
- provision of seats for athletes in the main seating bowl
- excellent access for disabled athletes (especially relevant for the Para Olympics).

Media

The intensity of media activity varies widely depending on the type of event, so the stadium structure is designed with core facilities that can expand as required, space being re-allocated for major international events. Independent media circulation and proximity to an outside broadcast area are provided. OB vans can link up with a cabling building from where cables run through an underground service tunnel to the media areas and the moat. A large area, normally for general circulation, behind the wedge of seats at the finish line can be taken over for host broadcasting activities. The press box at club level, with a view over the arena, has a bar, press rooms, interview rooms, a briefing area and dining-room. This is the nucleus of the media zone, a crucible for the generation and circulation of information - both formal and informal.

9.
Johannesburg's Central Business District forms the western backdrop to the stadium.

10. Site progress, October 1995.

Officials

These broadly divide into those handling competition management, and technical personnel. The plan/section concept allows for close and secure contact between officials during events - essential during large events both for the complex organisation of track and field programmes and speedy release of results.

The photo finish, competition control and results rooms are all at Club level, looking directly across the finish line. Officials are normally volunteers, and long hours are involved, so good rest facilities with a view of the track and field are important.

VIPs

The VIP function rooms, at the upper service level under the lower seating bowl, are provided with secure access either from the service passage or from the VIP entrance at the main concourse level. A terrace let into the lower seating bowl provides excellent viewing and entrance to the VIP seating area. The VIP section has dedicated vertical circulation to the other levels of the west stand and to the field for prize-giving and presentations. The number of seats can be expanded outwards from the core as required.

Disabled spectators

The use of ramps and the arrangement of the main concourse at the top of the lower bowl allows good generalised and non-segregated viewing space and amenities for the disabled.

Engineering design

Creating the bowl

Some 200 000m^3 of material, an andesite with relatively high moisture content, had to be excavated to form the bowl; this had compaction problems, overcome by impact rolling. The material was stockpiled during this contract for subsequent backfilling to structures.

The site's restricted geometry made it difficult to accommodate the stadium and warm-up track within the boundaries, so much adjustment was needed to achieve a reasonable balance between cut and fill, and to fit the stadium within the perimeter constraints without creating unacceptable embankments round the edge.

During these refinements it was decided to carry part of the warm-up track on structure over the external concourse.

The water table - 6-10m below natural ground - varied from 5m above to 5m below field level. Ten 25m deep wells were sunk to dewater the west grandstand during construction. Now, a permanent grid of subsoil drains carry all groundwater to the moat, which also acts as the main stormwater drain for the lower bowl and the field, discharging by gravity to the stormwater reticulation to the east.

Should total blockage of the piped systems occur, the marathon tunnel would form an overflow to prevent flooding of the lowest seating levels.

Track and field

For an athletics stadium to be graded 'International' or 'National', the track must have a synthetic surface and a layout complying with International Amateur Athletic Federation (IAAF) requirements. For the Johannesburg Stadium, Athletics South Africa (ASA) and the client also imposed additional requirements.

As at all major sporting facilities, practice and warm-up activities had to be catered for, and it proved possible to lay another full-sized track near the pre-event area of the main stadium. This has been designed to allow two lanes to be added to both the track and straight should the need arise, and can also used by the local community, schools, and clubs.

The total provision for international track and field events is as shown below in Table 1.

The main stadium is orientated 15° off north-south - near ideal for such a facility. All events except the high jump have run-ups in alternative directions, to avoid problems from wind and setting sun. Site constraints

Table 1:	Main stadium	Warm-up
Track	9 lanes	6 lanes
Straight	10 lanes	8 lanes
High jump	two	one
Triple/long jump	four	one
Pole vault	four	one
Javelin	two	one
Discus/hammer	two	one
Shot	two	one
Steeplechase	one	-
Soccer field	105m x 68m	105m x 68m

dictated that the warm-up track runs east-west: this was not ideal, but acceptable to ASA and the client as it is unlikely to be used for International or National events.

The ground under the tracks was of poor quality, and particular attention had to be given to the design of the track layers, to meet the stringent tolerance requirements. The final solution included ground stabilisation, gravel sub-base and base layers, and an asphaltic base layer under the synthetic surfacing. A full polyurethane and rubber granule in situ installation, together with EPDM UV-stabilised granule topping, was used for the 8300m^2 main stadium track surfacing. A similar combination of prefabricated and in situ material was used for the 4440m^2 of warm-up track surface.

Surface water runs off into a polymer concrete slotted drain within the track oval. This has frequent connections into an underground pipe collector system, which also takes the discharge from a comprehensive sub-surface drainage network. As a result, drainage time for both field and track after a storm has been shortened.

Concrete structure

Due to the slope on the site and the excavation for the bowl, the west grandstand is a semi-basement, giving three levels of accommodation below original ground level and up to four levels above. It is supported on piled foundations. Temporary anchored pile retaining walls (with gunite arches) up to 11m high were built to allow rapid excavation of the semi-basements; in the finished building the piles span between floor plates which carry the lateral forces back to permanent shear walls.

The suspended structure is generally 280mm thick flat slabs spanning 7m-8.5m, supported on 600mm diameter columns with mushroom heads, and partially post-tensioned in the radial direction only. For architectural reasons, raking beams to support the seating were provided on every second grid, the precast seat elements spanning 12-14m. The heaviest of them - prestressed to reduce depth and mass - weigh 8 tons; in all, 11.4km of seating elements were cast on site in three separate casting beds. An in situ concrete walkway (ridgeway) tops the upper seating bowl, giving an additional circulation route with spectacular views of the stadium.

Four cast in situ spiral ramps, cantilevering from eight circular columns on the inside radius, provide pedestrian and light vehicle access to all levels of the west stand. The northern ramp, most of its bulk concealed underground, allows athletes to descend 17m from the warm-up track to the sprint start. Most of the seating for the lower bowl comprises a stepped surface bed cast in situ on the earth embankment.

Various structures for toilet and concession buildings have been buried in the earth embankments to avoid clashing with the sweeping lines of the seating bowl. This concept was extended when an expensive 9m high earth embankment giving access to the upper public concourse was replaced by a two-storey office building providing 4000m^2 of lettable space to increase the project's viability. The 7.5m wide x 4.5m high marathon tunnel allowing athletics and heavy vehicles through the east embankment was cast in situ, and supports 4m of fill.

One feature that had a significant bearing on construction was the 2.5m wide x 3.5m deep moat around the track. This involved a 4.5m deep excavation at the toe of the 23° earth embankment, which in turn gave the contractor notable temporary works problems, particularly where the excavation was below the water table.

The roof

The 200m long roof over the west grandstand is the most dramatic element of the stadium, with its thin roof plate hovering over the seating bowl and suspended by a delicate tracery of cables and masts. The pretensioned cable net option proved too expensive under local conditions; instead, the plate hangs from tension cables from the tops of six A-frame masts, leaning forward over the roof to improve the efficiency of the front ties. To further reduce costs, the original cantilever of 45m in the centre was reduced to 36m, tapering to 18m at each end.

The crescent shape in plan derives from the 'circle around an oval' geometry of the stadium. On elevation, the roof plate is a circular curve chosen so that the roof hugs the ridgeway around the top of the upper seating bowl.

The roof plate itself is made of lattice purlins spanning to plate girder rafters, all contained in an 1100mm depth, the rafters supported by struts at the ridgeway end and by the front tension ties. A 900mm x 900mm steel box girder, spanning between the front tie connection points and ballasted with in situ concrete after erection, resists wind uplift. The roof plate is braced in-plane with tubular cross-bracing, set slightly below the plate to avoid clashing with the lattice purlins, and is clad with a narrow flute steel sheeting laid parallel to the line of symmetry

The roof slopes 1.5° to the rear for drainage, whence a box gutter carries all stormwater to the ends for discharge into large conical receivers.

A feature of the roof is the lighter leading edge or 'eyebrow', projecting 8m beyond the ballast girder. This consists of 100 mm diameter tubular purlins spanning between more closely spaced cantilever rafters, and is clad with 50% translucent polycarbonate sheeting to provide a transition for TV cameras from the deep shadow of the roof to the harsh sunlight. The stadium floodlights and loudspeakers are mounted under the eyebrow sheeting, and are accessed from a walkway that also serves as a gutter attached at the soffit level of the ballast girder.

At each A-frame mast, three front ties support a pair of rafters on the centreline and one rafter on each side at the third point of the masts. The tie-back system was taken down to terminate at the feet of the A-frame. This avoids the need for large tension anchor blocks, as the vertical component of the forces in the masts and the ties largely cancel each other. The A-frame is stabilised by raking struts between the ends of the extended roof rafters and the ends of the concrete raking beams of the upper bowl. The thrusts from the raking struts are carried down to ground level via ties and shear walls in the concrete structure, and the horizontal components are equilibrated with the forces at the foot of the A-frames via buried concrete tie elements. As a result, the net forces on the A-frame foundations are small, so that each leg can be supported on a single 610mm diameter driven pile.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11 - 14.
Two elements of the A-frame roof supports with their 'organic models'.


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15, 16.
Bracing element and A-frame, as built.

17.
Completed interior of the stadium, showing wide pre- and post-event vomitories and (Left to right, mid-way up the lower bowl) the three dedicated terrace areas for media, VIPs, and officials.

The A-frames scale down in size towards the ends in sympathy with the height and width of the roof, those at the centre straddling the spiral ramps whilst the small outer ones are supported on top of the end spiral ramps. The A-frame legs taper over the outer thirds of their length. They are susceptible to vortex excitation at low wind speeds but the friction in the spherical bearings at the foot of each provides sufficient damping to prevent oscillations reaching appreciable magnitude.

The tension ties are all Macalloy 460 architectural range bars with the associated fork ends and turnbuckles. The largest tie required three 85mm diameter bars. Careful attention to the main A-frame and tie connections resulted in elegant but buildable details.

Wind tunnel tests by the CSIR in Pretoria determined roof wind loads. One surprise was that the proposed 'Melbourne' slot on the leading edge of the eyebrow did not have the expected effects, possibly due to a similarly-sized slot further back where the walkway provides access to the floodlights. Based on these tests, the Melbourne slot was omitted. The roof was analysed as a 3D space frame on GSA, with the jacking loads modelled by thermal loads.

18.
Night-time illumination enhances the drama of the structure.

Roof erection

This was the contractor's responsibility, but the procedure suggested by Arups in the tender documentation was eventually adopted due to reservations about the contractor's proposal.

The A-frame masts, including raking struts and all rear ties, were preassembled on the ground behind the main stand.

The assembled masts were lifted into position by a 225 ton track-mounted crane and supported by completing the connection of the raking struts to the raking concrete beams of the bowl structure. The largest A-frame was 52m high by 20m wide at the base, with 1.4m diameter tubes, 16mm thick, and weighed nearly 80 tonnes.

The roof rafters were erected on their permanent supports at the rear, and on temporary props near the ballast girder at the front; the temporary props were supported on the bowl's raking concrete beams. The horizontal cross-bracing between rafters, the curved lattice purlins and the ballast girder completed the erection of the roof plate.

Pouring the concrete ballast into the steel box girder was done in two stages, due to limitations on the capacity of the temporary props and the supporting raking beams. After the first stage, all loads in the props were removed by jacking the front ties. The temporary props were removed after first stage jacking and the second stages of both ballast and jacking were carried out on the unpropped roof. The jacking detail ensured that this was safe. After second stage jacking was complete, all front tie forces were within 5% of the values predicted by analysis.

19.
Linford Christie wins the 100m from Frankie Fredericks on the opening night, 23 September 1995.

Conclusion

Work began in March 1994. Construction took 16 months and was completed within the extremely tight budget: a reflection of local commitment and expertise, though it was a race to finish in time for the opening function on 23 September 1995, at which Linford Christie ran the 100m in 9.97 seconds and Frankie Fredericks the 200m in 19.93 seconds.

The stadium is significant for the people of Johannesburg and South and Southern Africa. It combines state-of-the-art international stadium planning principles and design with local relevance.

Its shape is driven not only by the needs of international athletics but also by the desire for a sensitive solution within the landscape setting of east Johannesburg. It is a crucial element in the successful regeneration of the Ellis Park Precinct.

Reference

(1) BOSTOCK, M. *et al.* Cape Town's Olympic Games bid. *The Arup Journal, 29*(3), pp.10-14, 1994.

Credits

Client:
Johannesburg City Council

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RFB Consulting Architects

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Ove Arup & Partners Andrew Allsop, Paul Cross, John Hirst, Chris Wise

Project managers:
Procrit

Mechanical & electrical engineers:
CA du Toit & Partners

Quantity surveyors:
Roos & Roos Incorporated

Main contractor:
Goldstein Building Transvaal

Roof contractor:
Genrec/Dorbyl Consortium

Bulk earthworks contractor:
Stocks Roads

Piling and lateral support contractor:
Frankipile

Track construction:
Lonerock/Fintrex

Field construction:
Protech Projects

Illustrations:
1-8, 12, 13, 16, 17: James Burland
9, 18: Leon Krige
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