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Ove Arup Partnership
13 Fitzroy Street
London W1P 6BQ

Tel: +44 (0) 171 636 1531
Fax: +44 (0) 171 580 3924

Editor:
David J. Brown
Art Editor:
Desmond Wyeth FCSD
Deputy Editor:
Beth Morgan
Editorial:
Tel: +44 (0) 171 465 3828
Fax: +44 (0) 171 465 3675

Front cover:
The Wandoo 'B' offshore oil installation, North West Shelf, Australia
(Photo:Stevenson Kinder and Scott)
Back cover:
Interior of Radsporthalle, Berlin (Photo: Gerhard Zwickert)

Radsporthalle, Berlin **3**
Mike Banfi
David Deighton
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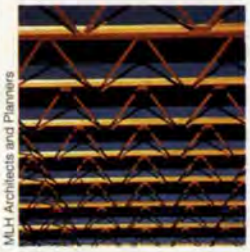
Raj Patel
Alan Tweedie
Mohsen Zikri

J Willebrand

Originally conceived in connection with Berlin's bid to host the 2000 Olympics, the new Rad- und Schwimmsporthalle in what was East Berlin is a symbol of the reunited Germany. Arups carried out the structural, geotechnical, mechanical, electrical, and acoustic design, as well as undertaking further aspects like communications, controls, commissioning, water treatment, specialist lighting and, for the services, the quantity surveying, cost control, and construction management. This article describes the 115m clear-span velodrome, the first part of the complex to be completed.

The Bellville Multipurpose Stadium, Cape Town **11**
Andrew Hakin
Neil MacLeod
Ugo Rivera



MLH Architects and Planners

Also linked to an Olympic bid - this time Cape Town's for 2004 - the Bellville indoor sports and entertainment facility was designed and completed within six months for the July 1997 World Junior Track Cycle Championships. Arup (Pty) Ltd designed the 120m span arch structure enclosing an existing open air cycle track, as well the acoustic panel treatment to the roof.

East Midlands Airport Passenger Terminal **14**
Gary Marshall
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Graham Bolton

Peter Mackinven

As one of the fastest-growing airports in the world, East Midlands needed expanded facilities to cater for the anticipated increase in passenger traffic. Ove Arup & Partners was appointed for the full engineering design and procurement of Phase 1, the major constituents of which are a new Departures Building and extension to the Arrivals Building. Arups' commission included designing the new baggage handling system - 32 check-in desks with all the associated conveying equipment and screening processes.

Wandoo concrete gravity substructure **18**
Gordon Jackson
Robert Care

Stevenson Kinder and Scott

The Wandoo 'B' offshore oil installation on Australia's North West Shelf is the first concrete gravity substructure in Australian waters. The project was also the first in the Australian oil and gas business to proceed on an alliancing basis. Ove Arup & Partners, designer of the CGS and casting basin, was a member of the Wandoo Alliance. This article describes the design, construction and installation of the CGS, and shows how construction cost was reduced through making consideration of buildability integral to the design.

The Nigg dry dock upgrade **26**
Alan MacLeay
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DGS Films Ltd

This refurbishment of a facility in north-east Scotland for building off-shore oil rigs drew upon expertise from many parts of Arups: for the structural design of the quay wall, the dock slopes, and other minor elements; the design and specification of the dock gate bearings and seals; draughting and reinforced concrete detailing; computer modelling of earthworks; advice on treating contaminated soils; design of rock armouring and fendering specification; advice on concrete mix design, concrete repairs and steel pile corrosion; finite element modelling of the quay wall; and abseil inspection of the dock gate.

Smart technology in construction: Dream or reality? **32**
Tony Sheehan

Tony Sheehan

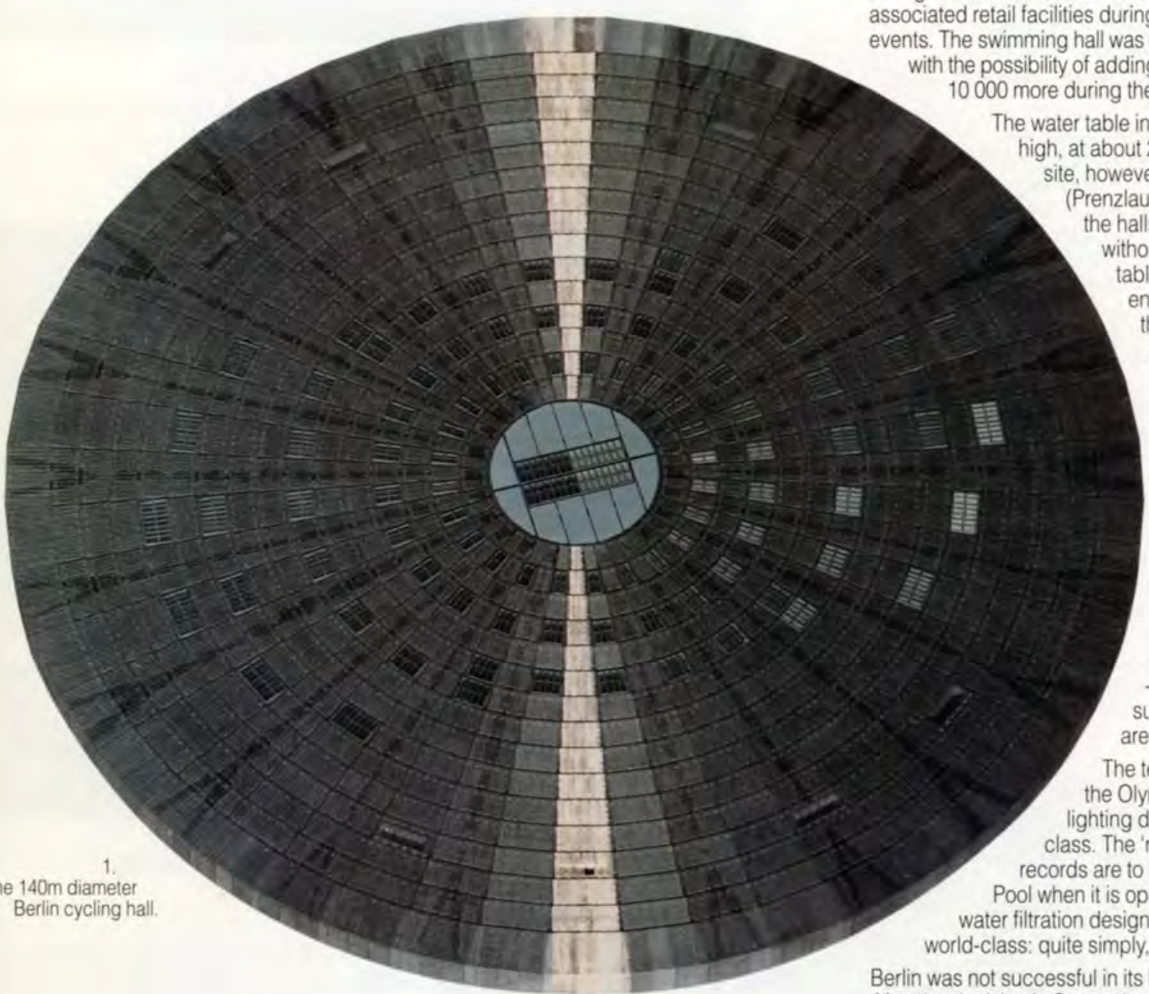
This article looks at developments, in various contexts, of 'smart' technology. These include the sensing of changes in structural performance; the modifying by actuators of structural behaviour; the combination of both sensors and actuators in active control systems; and the development of ' smart' materials which can sense and respond to external change. All these are assessed against the background of continued requirements for safety, durability, low cost, low maintenance, and predictable properties.

Radsporthalle, Berlin

Mike Banfi David Deighton Paul Nuttall
Raj Patel Alan Tweedie Mohsen Zikri

Introduction

In November 1989, West and East Germany were no more, following the symbolic destruction of the Berlin Wall. Now, bounded by the old Lenin Allée and Ho Chi Minh City Straße in what was East Berlin, stands the new Rad- und Schwimmsports- halle, a monument to the city's determination to show the external world its new face. After 1989, things moved quickly. In 1990, it was announced that Berlin could once again be the capital of a reunited Germany. What better way to confirm this to the world but to be its focus in the new Millennium and host the 2000 Olympics? In summer 1992, international competitions were announced for the design of the new facilities needed for such a prestigious event. Primarily, the competitions were for Olympic facilities, but the sites were chosen so that the new buildings could play key roles in the urban regeneration of areas of Eastern Berlin.



1.
The 140m diameter Berlin cycling hall.

The architect Dominique Perrault asked Ove Arup & Partners to assist him in his competition entry for the new Olympic cycling and swimming facilities, on a 500m x 500m site then occupied by an old Communist meeting hall and a cold store for an adjacent abattoir. In October 1992, the team was notified that it had been declared the winner, with a daring approach to the site that added significantly to its urban quality.

In November 1992, contracts were signed, with Arup GmbH being responsible for the structural, geotechnical, mechanical, electrical, and acoustic design, as well as all the interdependent disciplines such as communications, controls, commissioning, water treatment, specialist lighting and, as far as the services were concerned, quantity surveying, cost control, and construction management.

The initial programme was tight. The decision about the host city for the 2000 Games was to be made in September 1993, with the Olympic Committee visiting Berlin in June 1993. It was imperative that several large value construction contracts be let and that work had started on site to demonstrate Berlin's commitment.

The race had begun.

The concept

To avoid the urban problems associated with inserting two Olympic-scale sports facilities into an environment dominated by housing, Dominique Perrault chose to sink the stadia below ground. Long-span roofs would cover them and have their upper surface at ground level. The rest of the site, at ground level, would be turned into a Normandy apple orchard and the two roofs would be covered with a shimmering stainless steel chain mail surface to represent two lakes within a park.

Simple. But the scale of the project was huge: the brief for the cycling hall was for 5800 fixed seats, and flexible enough to accommodate 10 000 - 12 000 people and associated retail facilities during non-cycling indoor events. The swimming hall was to have 4000 fixed seats, with the possibility of adding temporary seating for 10 000 more during the Olympics.

The water table in Berlin is generally very high, at about 2.5m below ground. This site, however, was on a small hill (Prenzlauer Berg), which permitted the halls to be sunk below ground without encountering the water table - though not deep enough to accommodate the four storeys necessary. Ground level has, therefore, been built up by approximately 5m over the complete area of the site to give the effect Perrault required.

The servicing requirements for the underground halls were both exceedingly complex and onerous. Berlin has dramatic climate conditions, with long -15°C winters and +35°C summers; spring and autumn are just rites of passage.

The television requirements for the Olympics meant that the lighting design had to be world class. The 'requirement' that world records are to be broken in the Olympic Pool when it is open meant that the pool water filtration design had to be better than world-class: quite simply, the best.

Berlin was not successful in its bid for the 2000 Olympics. After the decision in September 1993 for the Games to go to Sydney, the project was re-evaluated and the budget reduced to DM550M.

This was done by eliminating some surface area of the complex primarily associated with the pools, but the technical design remained untouched; it was too late by then to change fundamentally the cycling arena. The retaining walls and excavation were progressing on site, the detailed design of the roofs was complete and the contract let.

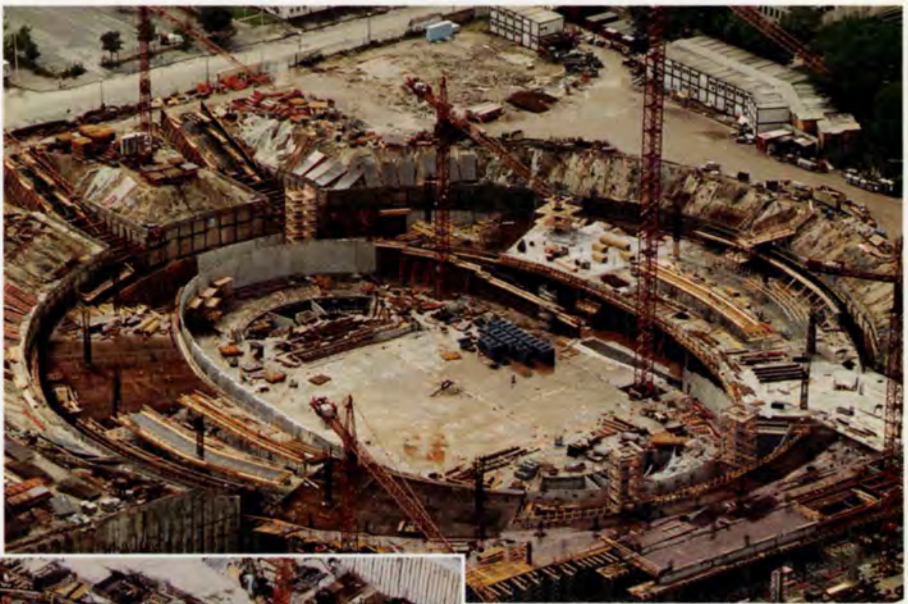
2.
The site in December 1993:
Berlin winter temperatures can sink to -15°C.

3.
Secant pile walling (shown here) supporting the railway,
and the contiguous piled walls to the entrance ramps, are left exposed in the finished building.

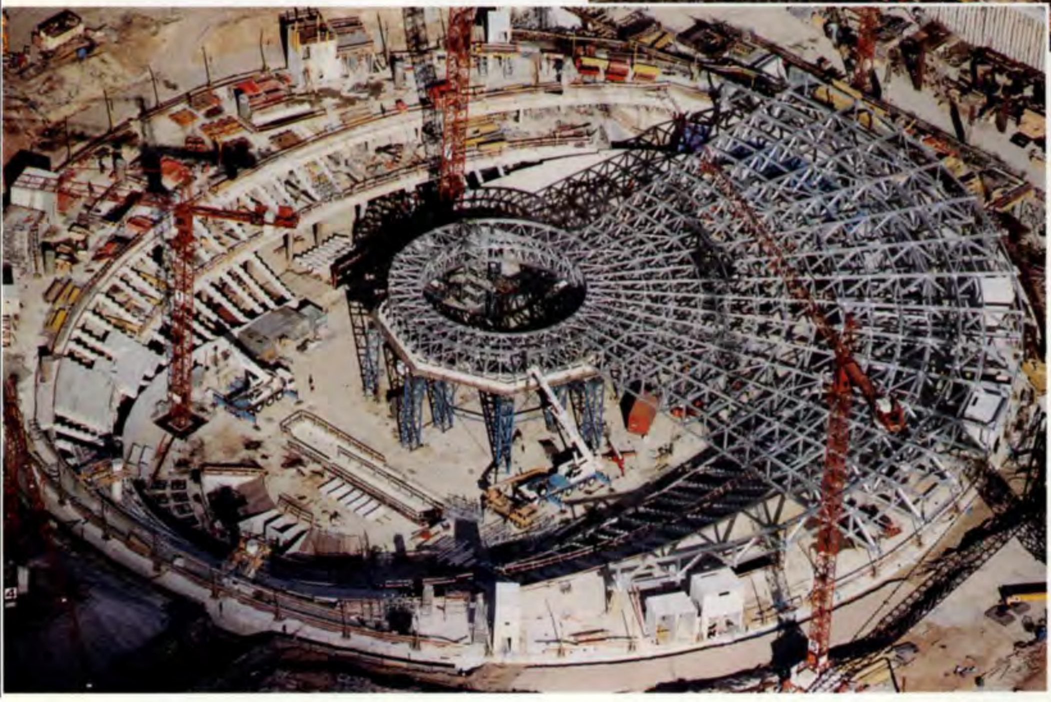
Excavation and substructure

The site geology consists of some 4m of fill on top of 15m of boulder clay containing sand lenses and perched water. Below this lie the typical Berlin sand layers. The general depth of excavation on site was 13m from existing ground level, with some deeper basement areas requiring a further 2m of dig. The lowest level therefore lies some 20m below final ground level, so substantial earth-retaining structures would be required.

The most efficient design would combine temporary works requirements with the needs of the final structure, and it was decided to use a combination of anchored contiguous and secant pile walls. Along the line of the adjacent railway a secant pile wall was adopted to limit movements. In the remaining areas outside the circular arena, contiguous pile walls were used. The 450m long secant pile wall consists of 900mm diameter piles and has one row of permanent anchors. The contiguous piled walls are also of 900mm diameter piles typically at 1.9m-2.25m centres with four layers of permanent anchors. A 200mm thick shotcrete panel then spans between the piles. Permanent monobar anchors (Gewi) with working loads up to 550kN were used. Each wall was designed to cope with the failure of a single anchor, with earth pressures redistributed to neighbouring anchors via arching or a capping beam if the top anchor should fail. 5% of the anchors are monitored by load cells to check the anchor force during excavation, the effect of the 5m build-up of the garden, and their long-term performance.



4 above:
Before roof construction, June 1995.



5.
Clockwise construction of the radial trusses approximately one quarter complete, October 1995.

Suitable excavated material was stored on site for later use in building up the garden level, but lack of space limited this. Material removed from site was transported to fill sites (an ex-opencast lignite mine) via the adjacent rail link - which helped relieve congestion on the already busy Berlin road network. During the project 1077 trains, each with 23 wagons full of excavated material, left the site - a total length of 300km of train. 330 000m^3 of material was excavated at the Radsporthalle alone.

The main structure of the Radsporthalle is a reinforced concrete braced frame with those parts of the structure below the water table or in contact with the earth designed as watertight concrete. There is no external waterproof layer. In the circular area under the steel roof, a different approach to the temporary works and building stability was taken, as it was decided that the circular shape of the building could be used to advantage. In this area a conventional retaining wall was used, propped by the floor slabs. The circular floor plates work in diaphragm action as compression rings which resist the earth pressures.

6.
The site in its urban context.

This had the advantage that large stability walls at right angles to the perimeter walls could be avoided in the main circulation and occupied areas, contributing significantly to the quality of the public spaces. Floors were designed as flat slabs with a theoretical structural grid of 7.2m x 7.2m, but the effect of the circular shape and the asymmetric ellipse for the track meant that this simple grid was never reproduced in practice.

The seating in the Radsporthalle consists of a series of precast L-shaped beams spanning circumferentially onto precast toothed beams more or less at right angles to the track. These toothed beams in turn span between the top of the track wall and the upper floor slabs. Precast elements were used, as the seating area is visible and a better finish quality could be guaranteed.

Moreover each unit is repeated several times, with a consequent significant saving on construction time gained from offsite production.

Roof design

The shape of the roof was dictated by the architecture of the whole scheme, and the main task was to find an efficient structure for the 140m diameter flat disk that would seem to hover above the cycling track. The top surface of the structure was fixed by its relationship to the surrounding ground, so there was a large incentive to minimise its depth to avoid an increase in excavation. At the early stages of design the depth had been 5m: 3m had been considered, but this was structurally extremely inefficient and did not provide the depth needed for roof plant and plantrooms. The final depth was approximately 4.1m overall, giving a span/depth ratio of about 28:1.

Various numbers of columns between 6 and 32 were investigated before 16 was decided upon, which gave reasonable economy in the roof and could be accommodated architecturally in the spectator areas below. The various options of framing the roof in terms of number of radial trusses and the type of circumferential structure were studied at the same time.

The final structure consists of 48 radial trusses, connected together via a 14.4m radius ring truss at the centre of the roof. At 57.6m radius 57.6m the trusses are supported either directly by one of the 16 concrete columns or indirectly via a circumferential truss. The tie downs which restrain each truss are located at a radius of 65.16m, beyond which there is a relatively lightweight canopy structure. Secondary beams in both the top and bottom surfaces are spaced at typically 3.6m centres and span between the radial trusses.

The inner ring is in fact two rings, positioned at radii of 10.8m and 14.4m. These are connected together by horizontal bracing in the top and bottom planes as well as by radial members and diagonals. The inner ring also has vertical bracing which occurs typically only in the top third of the truss to permit access routes. Inside there is a central rooflight with a movement joint between it and the inner ring to prevent it forming a link across the eye of the roof and attracting forces.

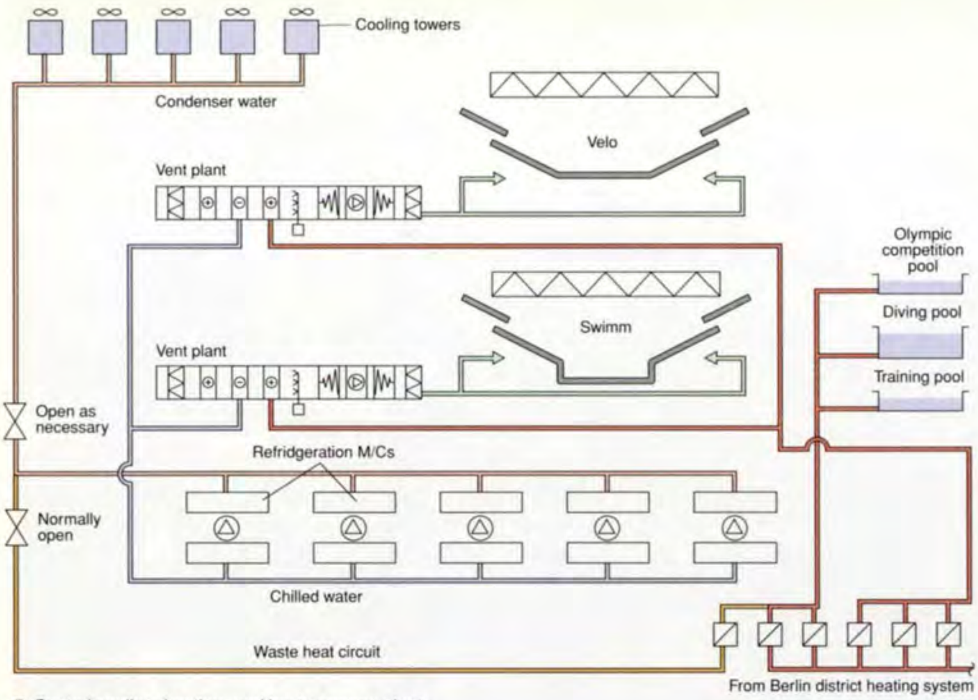
Between the roof and concrete columns, bearings allow relative horizontal movement between the steel roof and the concrete substructure. 12 of the bearings allow movement in both directions and four are restrained in a horizontal direction to provide a means of stability to the roof.

The majority of the truss members are HD sections rolled by Arbed. Together with suitable Arbed HE sections these offered a wide range of section weights with the same internal dimension between flanges. By turning the chord members on their sides the web members and chord members could be easily connected by welding and the need for stiffeners was also reduced. The size of the truss members varies from HD400 x 287kg/m for the chords at the column line to HD260 x 54kg/m for web members near the centre. The secondary beams are HE sections either 240mm, 280mm, or 300mm deep.

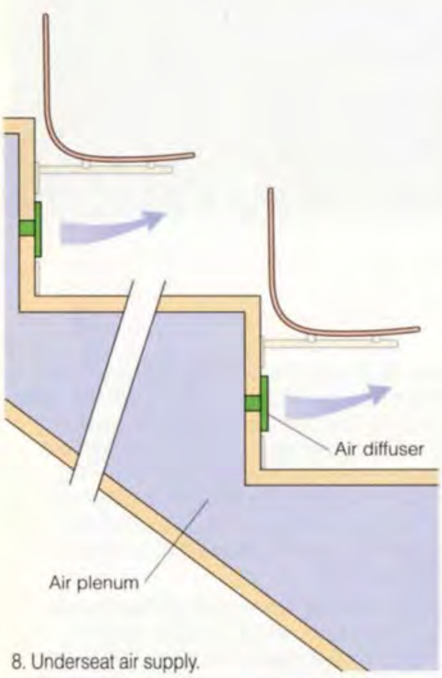
In addition to loads from cladding, maintenance, snow, and wind, the structure has to support the construction of plantrooms in the 14.4m wide annulus of roof inside the columns. The four main plantrooms enclose 10 tonne air handling units. There are also smaller plantrooms in the canopy supporting fans. Hanging equipment for events was allowed for by incorporating 50 separate 1 tonne loads in the design.

The roof was erected in winter 1995-96 during persistent temperatures of -14°C. Erection followed the principles described in the tender documents and the structure was precambered by 240mm to compensate for dead load deflection and provide a level roof in the final condition. A temporary support was erected in the centre of the Radsporthalle and the inner ring was erected on top of it.

The roof was built in a clockwise direction, firstly by erecting the circumferential ring truss between the columns, followed by the radial trusses and secondary beams. After erecting all the steelwork, the tie downs were stressed and the central temporary support lowered using 32 hydraulic flatjacks. A check on the roof level showed it to be within 9mm of the theoretical level once depropped.



7. Central cooling, heating, and heat recovery plant.



8. Underseat air supply.

9. Seating units showing air diffuser holes.

Plant strategy

The overall strategy is based on centralising heating and cooling plant, but decentralising air systems to meet the individual needs of the spaces served.

Air distribution system

The strategy aimed to achieve three key objectives; to minimise energy use, to achieve good comfort conditions, and to fit with the architectural concept of the roof. An important roof feature is daylight penetration and this led to limiting the amount of plant and ductwork therein.

The traditional method of overhead air supply, often used in similarly large spaces, was considered but rejected due to its high energy use, unpredictable comfort conditions, inflexibility, and the need for large plant and ductwork. Instead, a low-level air supply system was conceived to convey the air gently beneath the seats in order to create a local microclimate in the occupied zone, rather than a 'blanket' pool of air to treat the whole space. Computational Fluid Dynamics (CFD) was used at the conceptual stage to confirm the viability of this proposal and to analyse comfort criteria. This was performed for different occupancy levels, against a complex matrix involving climatic variations, changes in internal loads, and different occupancy patterns, arising from multi-use of the space.

The design was rationalised so that two systems only could handle the wide spectrum of uses. These are:

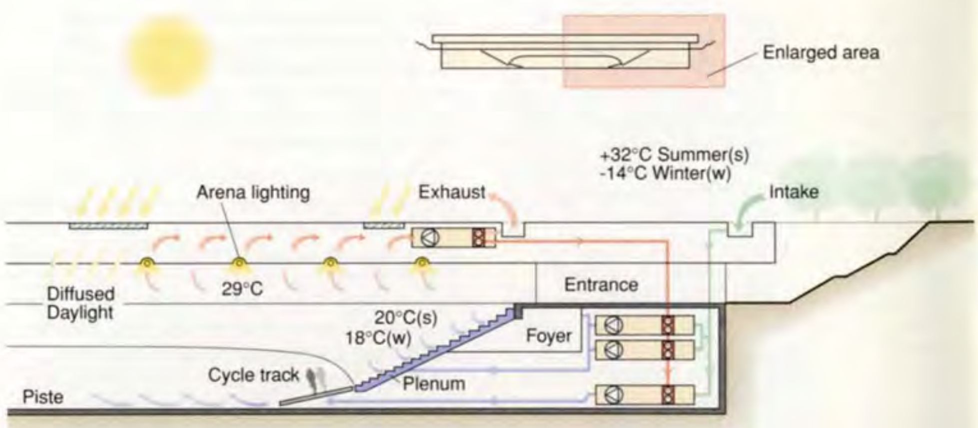
- *Seating/foyer air supply - two units provide 56 000m³/hr each, and two units 66 000m³/hr each.*

Low-velocity air is supplied beneath the tiered seating areas to create a comfortable microclimate. Extract is at high level and is assisted by natural buoyancy. Frequent and lengthy intervals involve the use of the adjacent foyer area. To deal with this economically, air serving the seating area is temporarily transferred to the foyer.

- *Piste air supply - four units capable of 34 000m³/hr each.*

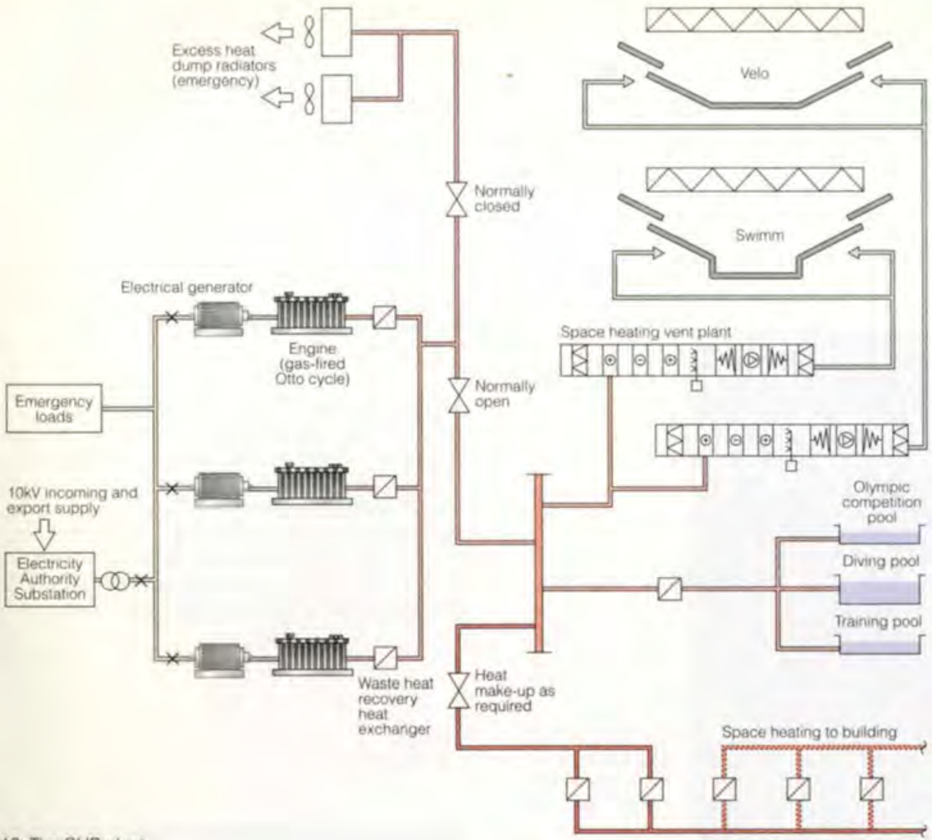
Air is supplied all round the piste area at low level using a dedicated low velocity system and is extracted in the same manner as described above. The main ventilation plant is at the lowest level and linked to outside via dedicated fresh air shafts. Supply air to tiered seating is via plenums connected to individual outlets beneath the seats. Smoke extract from the main space uses the air intake shafts in reverse to save space and costs.

- *Main extract - via eight roof units capable of 45 000m³/hr each.*



10. Overall ventilation strategy.

11. CFD plot of summertime temperatures.



12. The CHP plant.

Heating

The prime source is 1800kW available from the combined heat & power plant (CHP), whilst the base source is 13 200kW available from Berlin's district heating company (BEWAG) via five plate heat exchangers.

Cooling

The total cooling capacity is 3600kW provided by five centrifugal refrigeration machines capable of delivering two different capacities, in order to match the cooling load profile. Three machines provide 900kW each and two machines deliver 450kW each.

All machines are capable of heat recovery.

Heat rejection plant

The refrigeration plant is served by five close-circuit cooling towers with total heat rejection of 5000kW. Towers are suitable for heat recovery.

CHP

The heating and electrical power demand of the Schwimmhalle, as well as the emergency power requirements of both the Radsporthalle and the Schwimmhalle, were used as a basis for selecting the CHP plant. It consists of three electrical generators driven by spark-ignition gas-fuelled engines (Otto cycle engines). The generators are capable of generating 310kW of electricity each and the total heat recovered from the engines is 1800kw approximately. Although the full heating capacity from three engines was allowed to be exploited, in the case of the generators the team were only permitted to allow for two in their emergency power calculations. However, under non-emergency conditions electricity is available from all three generators.

The CHP plant is designed to run continuously and its heat and electricity output will be fully utilised to heat the pool water and serve the pool ventilation fans. Efficient operation of the CHP plant is assured, given that the Schwimmhalle requires a fairly constant level of heating of fresh water makeup all year round to maintain a normal pool temperature of 28°C. Fans run continuously to control humidity and temperature levels.

There are no written regulations covering this type of installation and each design has to be discussed with the certifying authorities. The Otto cycle engines cannot load up as quickly as diesel engines and accept only 30% of their full load capacity after 15 seconds. To meet the DIN requirements for places of public assembly, two engines always have to be run during occupation, regardless of load demand, so that under emergency conditions power is immediately available.

The final major consideration was how 'black starting' - ie no electricity available at all - was controlled. Given the stage load acceptance of the generators, it was necessary to prioritise the loads into 90kW blocks and switch them on line as generator capacity became available. This control was totally under the central load management system of the CHP plant.

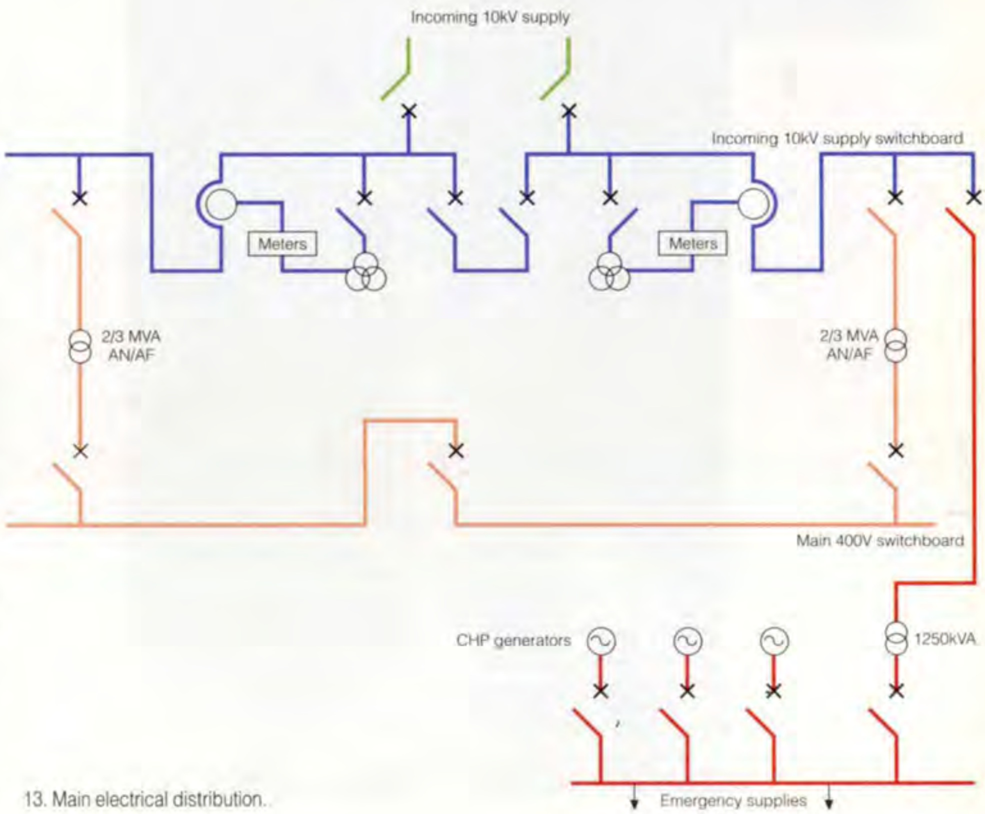
Energy-saving measures

- The CHP plant deals with base heating load and offers on-site electricity generation.
- Fresh air rates and hence total ventilation loads are reduced whenever extreme external conditions occur.
- Refrigeration machines are coupled to three plate heat exchangers to recover waste heat. This amounts to 1300kW and is used for pre-heating the pool water. This system is also used in reverse to protect the cooling tower circuit during extreme cold weather.
- Cooling towers are used in mid season/mild winters to generate 'free' chilled water at 6°C via plate heat exchanger.
- A BMS with fully addressable DDC outstations monitors and controls energy use.

Electrical systems

The electrical systems were divided into two: strong and weak current. Strong current includes normal power distribution, lighting, emergency lighting, and lighting control. Weak current covers the remainder, a very broad set of uses including fire alarms, intruder detection and CCTV monitoring, clocks, voice alarm, sound reinforcement, telephones, data, the ticket systems, the disabled toilet alarms, the timing and results systems, and multi-access TV and display boards.

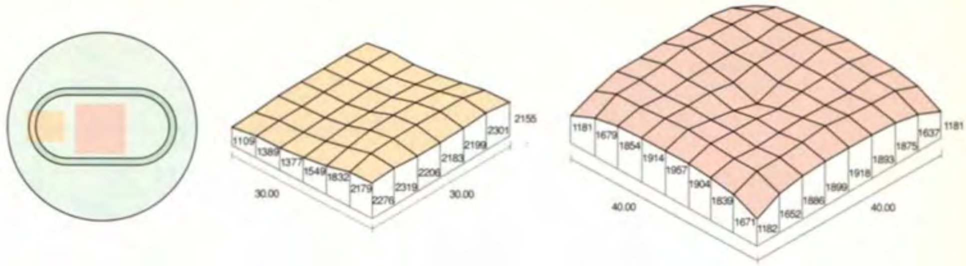
Total site electrical demand for the Radsporthalle and Swimming complex is 6.5MVA, of which 6MVA is supplied from the electricity company via two 10kV incomers and 0.5MVA is supplied from the on-site CHP system; the latter also acts as the emergency generation required for the life safety system. The electrical distribution is TNC-S, with the combined neutral installed to the local distribution board.



13. Main electrical distribution.

Lighting

The arena lighting in the Radsporthalle is achieved by using two different types of floodlights arranged concentrically, in keeping with the roof structure. All floodlights are fixed to the side of maintenance walkways in the roof. The architectural restraints encountered (ie limited location for the positioning of the floodlights, and the fact that no floodlights can be angled) made the design of the asymmetric reflector very specific.

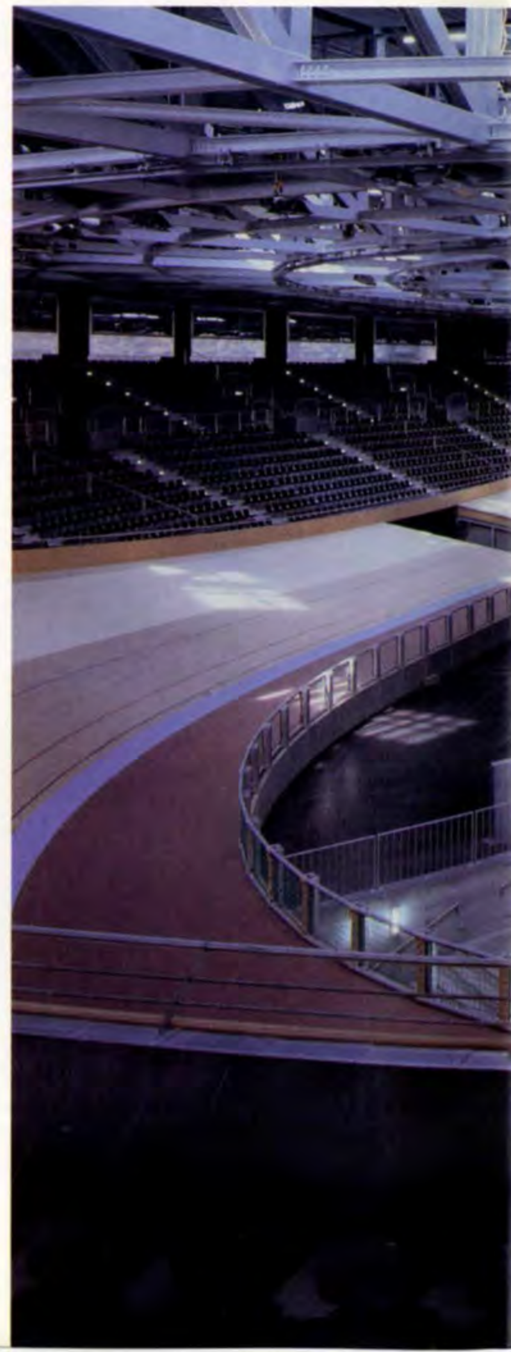


14 above:
Horizontal illuminance levels.

Floodlights, each fitted with two 400W lamps, are used to provide the general illuminance for the non-televised events. Supplementary lighting is provided by 2kW floodlights to achieve a higher level of vertical illuminance for the televised events. The design illuminance for the television broadcasting was about 1000 lux vertical. These design figures were below the standard requirements for the high definition television (HDTV) broadcast,

15.
Stainless steel chain mail covers the walls and roof.

17 below:
The completed interior.



16.
Column/roof connection detail.

but advice at the time was that this illuminance requirement would be reduced as new technology for HDTV developed. All lamps are metal halide types with colour temperature of 4000K and colour rendering index (Ra>65) to conform to the CIE standard for television broadcasting.

A computerised scene-setting lighting control system is provided to allow different lighting scenes to be created and selected according to functions in this multi-use arena. Each lamp is controlled by an addressable contactor to provide complete flexibility for different settings to be programmed in, prior to the staging of any events.

To avoid lengthy black-outs and minimise disruptions to major events following a power interruption, 50% of the arena lighting is on maintained supply back-up by CHP generators and instant restrike facilities are incorporated in the rest of the lighting installation.

Fire detection

The fire detection system was a basic addressable system with mainly manual break glass alarms.

This appears surprising considering the number of people that are potentially located underground; however, the strategy is that the building is manned continuously, and should an alarm be initiated the fire brigade anticipate being on site in less than five minutes. The use of automatic alarms was not thought appropriate, as it could result in frequent false alarms.

Drainage

Being an underground facility, the drainage aspects of the design were critical. The depressed location of the Radsporthalle entrance meant that the surface water system had to be designed for 1 in 50 year storm with a peak intensity of 170mm/hr.

The design criteria was 300 l/s/ha and a restriction of 80 l/s/ha was imposed on the local discharge. Because of this, rainwater storage was required prior to discharge into the sewer.This was provided via a 0.5km long 1.2m diameter retention pipe, which provides a storage capacity of approximately 600m^3. The roof drainage system is of the syphonic type. All sanitation water is collected in a series of below ground dry well chambers and pumped to the external system.

Water systems

The sports complex features central and local as well as point-of-use hot water systems.

The central system has a capacity of 21 000 litres per hour and serves the Radsporthalle and Schwimmhalle to cover normal events.

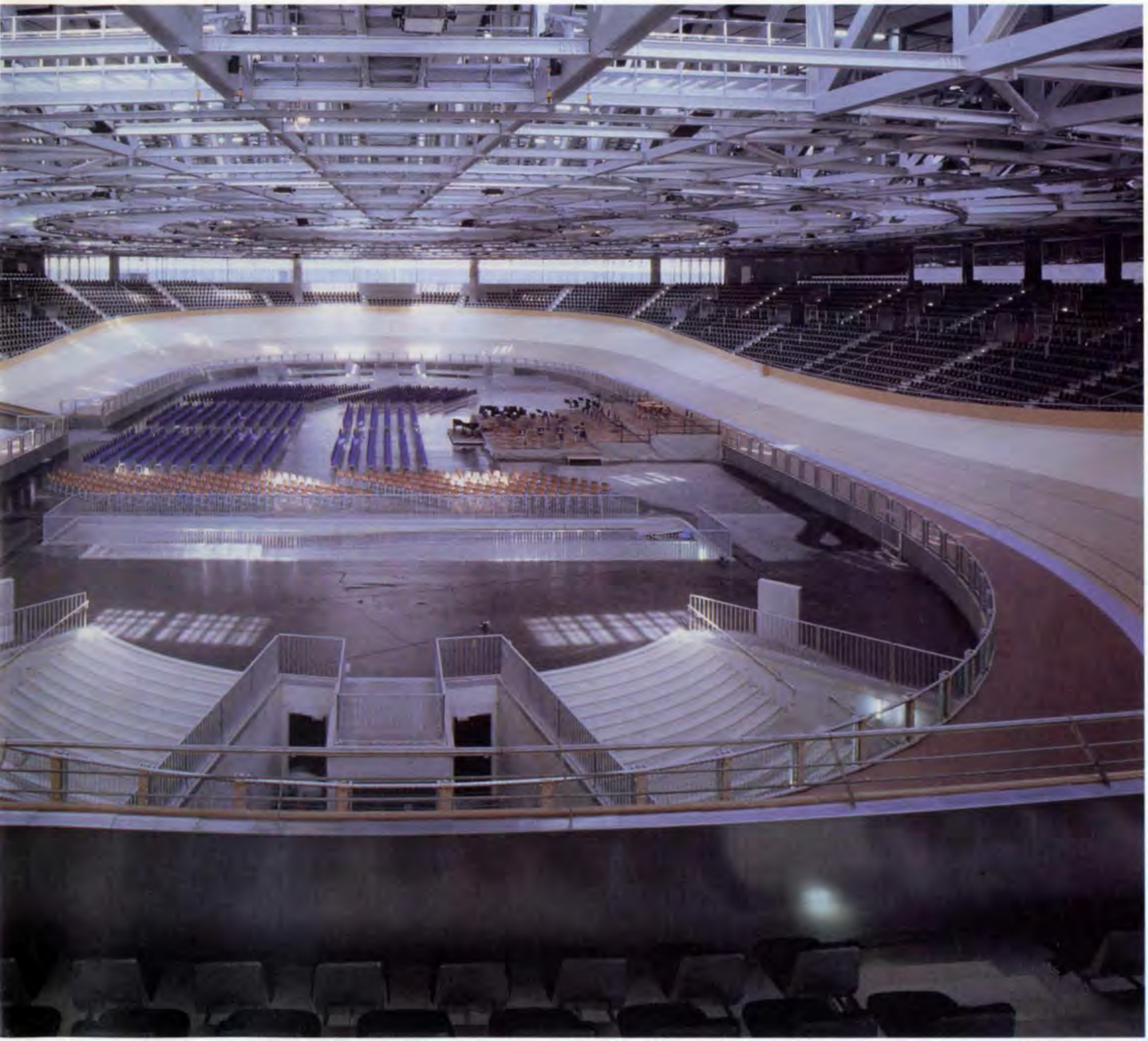
For special events a local system provides a capacity of 4000 litres per hour. When events are not staged the in-house areas are served by point-of-use systems to save energy.The cold water installation serves directly all domestic appliances and fire systems. The installation is zoned to allow intermittent use of the complex.

Acoustics

Three key design issues were addressed to achieve the required acoustic performance:

- ***Room acoustic response***
  The reverberation time of the arena bowl was analysed to ensure an appropriate room response, whilst the geometry and acoustic performance of the arena bowl finishes were studied to prevent late echoes and reflections. The building-wide acoustic response was analysed to meet the voice alarm system design standards.
- ***Sound insulation***
  The performance of the building envelope was analysed to ensure that noise break-out from the building does not result in any significant change in the environmental noise at the nearest residences.
- ***Electro-acoustics***
  In the case of the voice alarm systems, the correct room acoustic response coupled with the type, location, orientation, and performance characteristics of the individual loudspeaker types was essential.

In general, the materials used within the arena bowl environment were acoustically reflective. Achieving acoustic absorption is often limited to choosing an absorptive seat and through design of the roof absorption, and the Radsporthalle is no different in this respect.



In addition, the roof is also the main sound radiating surface of the building, and hence the main influence on the noise radiated externally.

The roof design consists acoustically of two elements:

- ***The external roof deck:*** Through optimised mass this provides a level of sound insulation and some low frequency absorption through panel resonance.
- ***A liner tray system:*** This consists of 65% open wire mesh panels suspended 1.5m below the ceiling, with 100mm of acoustic insulation located in the tray. The combined system provides a high degree of mid and high frequency absorption whilst the hanging system optimises the low frequency performance.

The result of using this system is a mid-frequency reverberation time of 2.1 seconds flat across the frequency spectrum, avoiding the common low frequency 'boom' associated with arena spaces.

The large flat vertical surfaces within the bowl had to be minimised to avoid the unwanted reflections which, due to the curved geometry, can be focused and thus result in distinct echoes. Where possible, panels were made sound absorptive. At the upper levels of the seating bowl, where the envelope is glazed, the panels were angled such that reflections were directed onto the absorptive roof panels.

The voice alarm system design criteria ensured a speech transmission index (STI) of 0.45 in all areas of the building. This not only required the introduction of acoustic absorption within the different building spaces but also attention had to be paid to the design of the loudspeakers. In spaces where it was not functional or cost-effective to introduce absorption, for example in non-performance and non-public circulation areas, directional column loudspeakers were installed to ensure that the design targets could be met.

The performance sound system in the arena bowl consists of a fully distributed, high power, full frequency range system capable of the highest quality reproduction. Three different types of loudspeaker are used in the bowl, positioned in a concentric ring arrangement covering the event floor, lower bowl seating, and upper bowl seating. The system is controlled by a central computer; this allows the user to select the type of event which is to occur and the system signal processing is automatically set up to allow for that event configuration. This facility is particularly useful in the concert modes, where a side stage or end stage arrangement is used. In this situation, specific time delay relative to the stage location is required in order that the listener places the source of the sound on the stage and not the nearest loudspeaker. This function allows the system to be used in conjunction with touring sound systems (as used by rock / pop bands) to improve the sound coverage, particularly at the upper levels of the bowl.

Organisation

The architect was based in Paris up to the end of scheme design. All technical design work was handled from London during this phase, with a small client liaison team based in Arups' Berlin office. At the end of scheme design the architectural focus moved to Berlin and the Arup Berlin team was increased to handle client and architect co-ordination, the main Arup production team still being based in London. In summer 1994 an Arup project office was set up on site with IT links to the London office to facilitate the transfer of drawings and other information. A team is still on site dealing with the construction of the Schwimmhalle. In all, over 340 Arup staff contributed to the whole project, spending more than 32 000 man-days on design, co-ordination, and management. Space does not permit all to be credited below, but they have not been forgotten.

Conclusion

The Radsporthalle officially opened on 23 January 1997 by hosting the 86th Six-Day race. This is not only a sporting event but an occasion for a party, with plenty of beer and music. The fact that local hero Olaf Ludwig led and eventually won the first event in the new Radsporthalle on this historical cycling site added to the general air of excitement around the new venue and the huge success of the event. The arena has since accommodated both rock and classical music events and at the time of writing is hosting an indoor windsurfing event. In September 1997 the Radsporthalle roof was awarded the ECCS 1997 Steel Award for Germany.

All this is, of course, only half the story. The structure of the Schwimmhalle complex is now finished on site and services installation well under way. The combined complex will be more than simply the sum of the two halves.

Watch this space!

Credits

Client:
OSB Sportstättenbauten GmbH, Berlin

Architect:
Dominique Perrault, Paris
APP, Berlin

Engineers:
Ove Arup & Partners/Arup GmbH
Paul Nuttall (project manager)
Adam Chodorowski, Helen Dauris (geotechnics)
Carl Bauer, Jenny Burridge, Dieter Feurich, Ron Jacobs, Keith Jones, Duncan MacIntyre, Malcolm Turpin, Faith Wainwright (substructure)
Mike Banfi, Matt King (roof steelwork)
Trevor Baker, Jean-Paul Velon, Ray Young (CAD)
Nigel Dore, John Fisher, Barrie Gould, Bill Horn (detailing)
Kate Benton, Chris Clifford, Jim Shaw, Alan Tweedie, Laurence Vye (structural, Berlin)
David Deighton (services co-ordination)
Matthew David, Jim Deegan, Ken Ma, Richard Nowicki, Ann O'Sullivan, Mohsen Zikri (mechanical)
David George, Lidia Johnson (public health)
Florence Lam, Alexandra Wolf (electrical)
Nicos Peonides, David Pritchard (controls)
Tom Fernando, Graham Naylor-Smith (communications)
Steve Jolly, Clodagh Ryan, David Wall (services, Berlin)
Richard Cowell, Malcolm Wright (acoustics)
Raj Patel, Neill Woodger (electro-acoustics)
Fida Berhanu, Jens Meirich, Michaela Praetsch, Falko Reinhold, Kurt Schimanski, Michael Schmidt (site management)

Steel roof contractor:
Krupp Stahlbau Berlin GmbH

Concrete contractor:
Wolff & Müller RKK GmbH & Co. KG, Berlin

Retaining walls contractor:
Aufschläger

Spoil removal subcontractor:
Schauffele

Mechanical services subcontractor:
Stangl

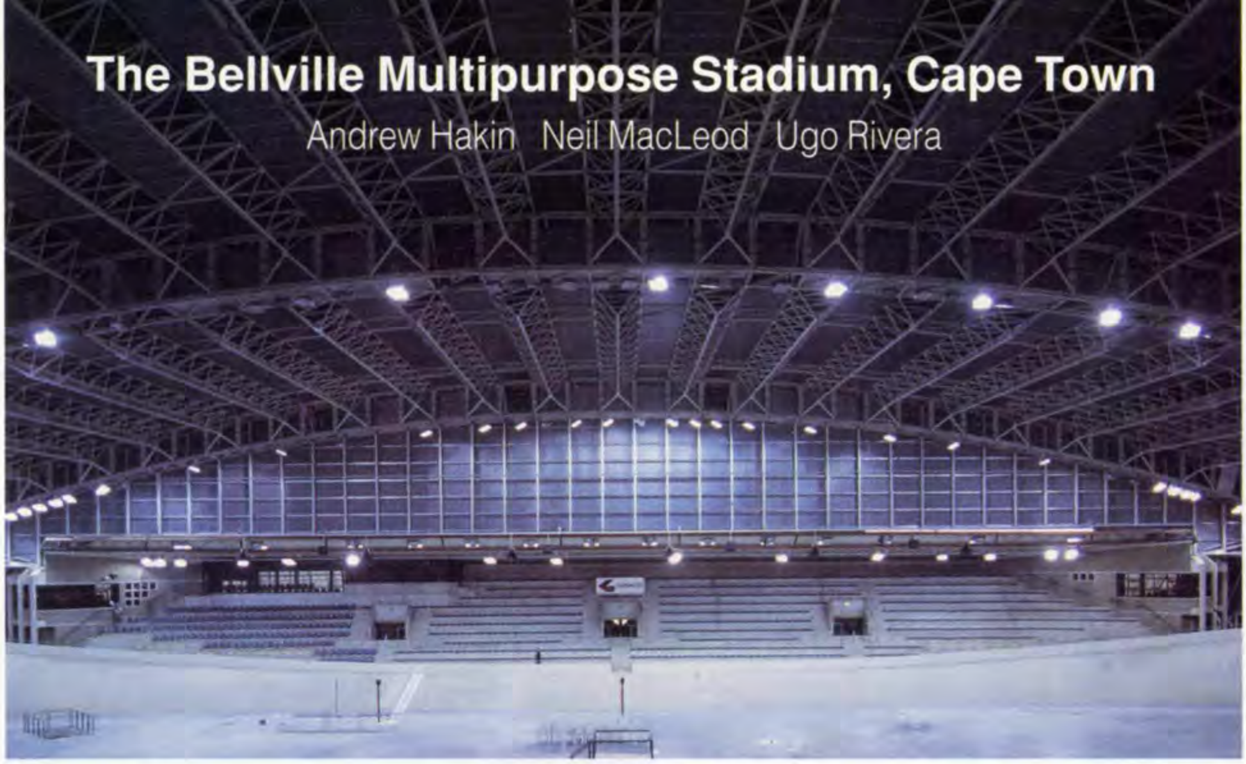
Electrical services subcontractors:
Siemens
Altmann & Böhning

Illustrations:
1, 4-6, 18: R Grahn/OSB
2: Faith Wainwright
3, 17: Gerhard Zwickert
7, 8, 10, 12-14: Denis Kirtley/Martin Hall
9, 11: Ove Arup & Partners
15, 16: Ulrich Schwarz

18 below:
When complete, the whole Rad- und Schwimmsportshalle complex should complement rather than dominate its urban context. The swimming hall, still under construction in the foreground, will feature in a future *Arup Journal.*

The Bellville Multipurpose Stadium, Cape Town

Andrew Hakin Neil MacLeod Ugo Rivera



1. The completed interior, July 1997.

Introduction

The Bellville Multipurpose Stadium is an indoor sports and entertainment facility enclosing an existing 250m cycle track and providing additional spectator seating. The initial priority was to meet the requirements of the July 1997 World Junior Track Cycle Championships, but the facility was ultimately designed to cater for Olympic track cycling competition (if Cape Town's bid to host the 2004 Olympics was successful) and a yet-to-be-defined brief as a multipurpose indoor hall for sporting and entertainment functions, exhibitions, conferences, etc.

The two significant factors which guided the design of the structure were the speed with which it had to be designed and erected to be ready for the international cycling event, and the stringent acoustic requirements for the building envelope. The velodrome is in a valley surrounded by residential areas and commercial hotels, and correct acoustic treatment of the building envelope had to be designed into the project to allow its future use as a pop concert venue. Tina Turner became the design standard for the team!

Background

In the early 1990s the then Bellville Municipality (located some 20km east of the centre of Cape Town) built a sports centre consisting of an athletics track back-to-back with a cycle track. The dividing pavilion provided spectator seating for each of these and other facilities such as club rooms, changing rooms, toilets, administrative offices and a small gymnasium. The 250m cycle track was accepted by the UCI, the world cycling governing body, and for some time it was the only accredited track in Africa. It was thus the obvious venue for cycling in Cape Town's 2004 Olympic Bid, although it was acknowledged then that for the city's climate it would require a roof and that ultimately a timber track would be desirable.

In 1993/94, when Arups was deeply involved in Cape Town's Olympic Bid[1], the firm was approached by an architect who had plans for a large roofed stadium in a different part of the city. Arups diverted him into exploring the possibility of roofing the Bellville Velodrome, which they then pursued together. The result of these endeavours, as well as the desire of the City of Tygerberg (which the Bellville Municipality had now become) to create an indoor facility at the venue, was that a proposal call was prepared by Tygerberg to find a private sector financier and operator for the complex, who would also develop some 4.2ha of commercial land.

Various submissions were received but ultimately no deal could be struck with the leading contenders and the process was unsuccessfully repeated. At this stage Arups was part of the team that had the favoured solution, but with whom a commercial agreement could not be concluded.

The resultant time loss in this abortive process left the City with little alternative if they were to host the World Junior Cycling Championships, scheduled to commence on 29 July 1997. Arups, together with other members of the favoured team and others, were called to a meeting on 21 December 1996 and asked whether they considered it possible to complete the project in time. The short answer was that if the proposal call design and the team were kept substantially unchanged it would be possible. This included negotiating the contract with the contractor.

The city councillors met the next day and on 23 December the team was summoned to a meeting and advised that they were appointed. This was a brave and extraordinary decision by a public sector client.

The penalty for failure would have been a severe blow to Cape Town's Olympic aspirations, with the Championships occurring just before the September decision by the International Olympic Committee. In addition the Bid Company's R5M contribution towards the cost of the venue would be forfeited. The challenge now facing the team was to complete the facilities - for 2500 spectators, some 30 teams, media, and officials - in just over six months by 6 July 1997, ready for the Championships to begin three weeks later.

Design

The structural solutions chosen for the various elements of the new stadium enclosure were all informed by the need for swift erection procedures and simple construction methods, whilst maintaining elegant, economic, and efficient overall structural systems. As Arups was also responsible for engineering the panelised acoustic cladding treatment, an integral design was developed which greatly speeded the erection and closing of the roof.

The building can be split into component parts for simplicity of description: the foundations; the brickwork façades; the reinforced concrete framed façades; the reinforced concrete buttresses to the arches; the steelwork roof structure; the closure to the existing stadium; and the acoustic panel installation.

Foundations

Two significantly different foundation conditions occur at the site: weathered Malmesbury rock in the western and central areas, and deep fill in the east. The buttresses restraining the main roof arches receive significant horizontal and vertical forces and are founded on large diameter oscillator piles, some of which are raked to resist the horizontal loads. The perimeter wall foundation solution varied with the site conditions, pad footings being used where possible on the rock, and small diameter driven piles in the fill areas to the east.

To reduce costs and maintain trade consistency, retaining walls were constructed in reinforced cavity brickwork.

2. The original Bellville sports facility in 1992.

3. Erection of the first arch, April 1997.

4 Above: Roof cladding in progress, May 1997.

5. Interior, May 1997.

Façades

Part of the overall structural proposal was to separate the façade structure from the roof support structure to ensure that the fast pace necessary for roof erection was not compromised by the slower activity of constructing the brickwork façade. A system of piers and precast lintel horizontal support beams was developed for the self-supporting brick façade walls, using the staggered plan configuration of the façade and the buttresses for out-of-plane stability. Regular returns in the brickwork limited the need for movement joints, further saving time and construction costs. The wall, which in places is up to 9m high, was also designed to incorporate an acoustic inner liner as part of the future construction phase and also offer support to the fibreglass gutter system at the roof eaves.

The highest part of the façade of the building is to the north, where the central area is up to 16m above ground level. A reinforced concrete 'colonnade' of columns and beams support this wall. The columns were cast in situ to their full height with special brackets designed to be cast in to receive precast beams erected later between them - a solution which the contractor was able to implement swiftly and cleanly.

The top 4m-6m of the north façade is supported by a light steelwork frame stabilised in the plane of the wall by cross-bracing. The steel columns support the end rafters spanning onto the wall from the northern-most arch, and the façade is sheeted above the level of the first colonnade beam, an arrangement which enables the structure to accommodate the temperature movements of the long-span steelwork roof structure. Generally, this façade was designed as an inexpensive temporary structure to allow a change to a final solution in the next building phase, as this would in future become the principal entrance to the stadium.

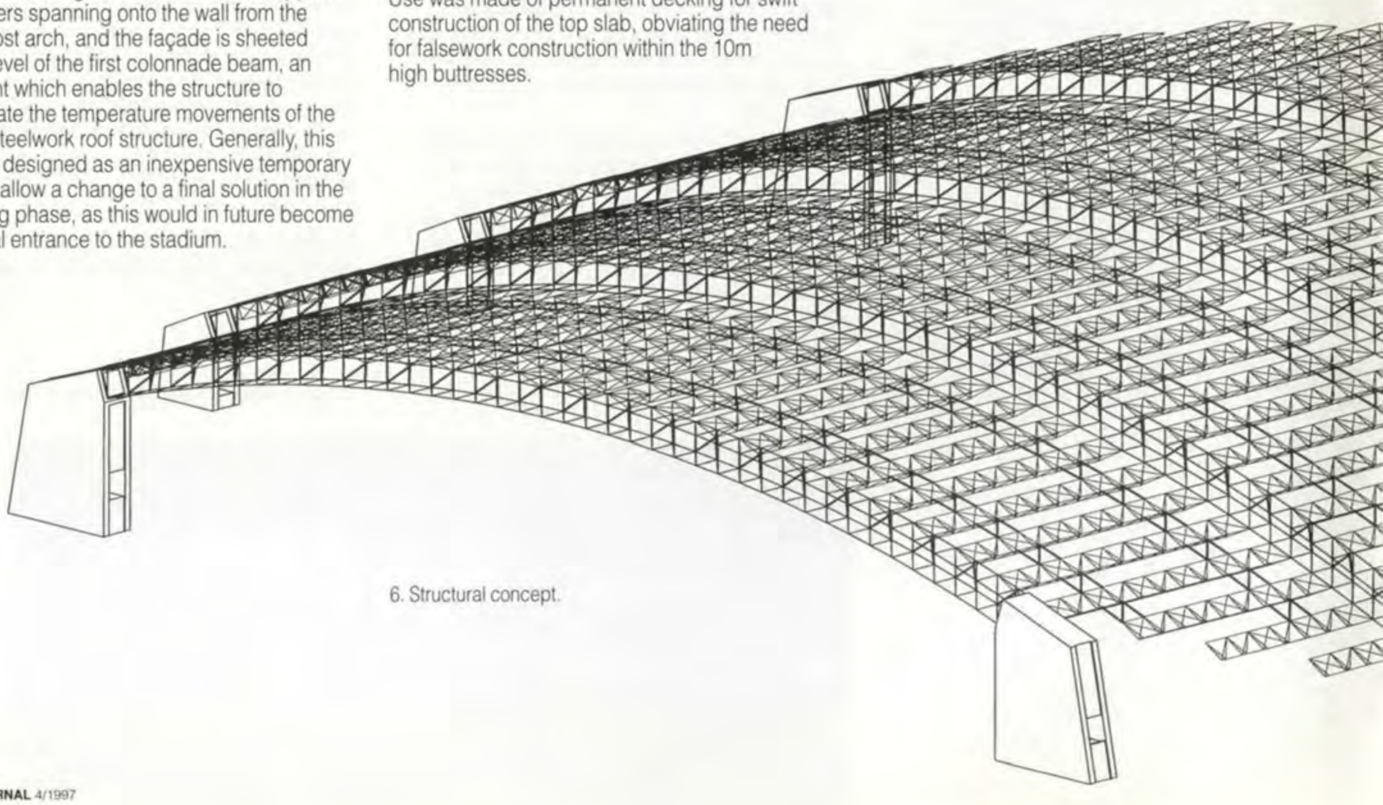
Reinforced concrete buttresses

The size and shape of the reinforced concrete buttresses were developed to contain the air-conditioning plant, and thus consist of box structures with main load-bearing side walls 550mm thick. Cross-walls link the side walls and enable the buttresses to do their job in giving the roof lateral stability in both directions. The shape of the buttresses was further constrained by the close proximity of the site boundary, particularly to the east, and the need to maintain access routes around the building.

To reduce costs and speed up construction there are only two buttress configurations, with four repetitions of each making up the eight buttresses. Use was made of permanent decking for swift construction of the top slab, obviating the need for falsework construction within the 10m high buttresses.

The roof

The roof structure consists of four main support arches with a system of closely spaced triangulated trusses between them. The centre two arches span approximately 120m with a rise of 13m, whilst the northernmost and southernmost both extend 85m with a rise of 7m. The roof steps at each arch position by an amount equal to the arch depth (2.25m) enabling the roof to follow the general rise in slope of the ground level to the north, as well as to allow additional space for the projected future spectator grandstand. The overall effect is of a series of four barrel vaults.



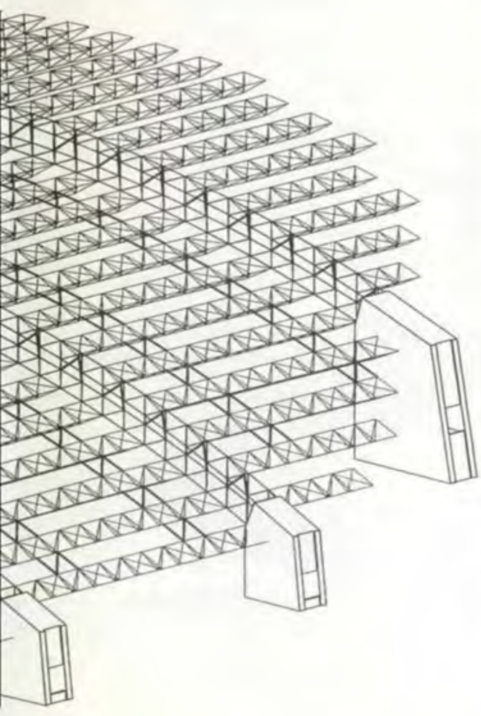
6. Structural concept.

7. View of completed exterior, July 1997.

Each arch consists of standard section, hot-rolled, grade 300W steelwork in a framed box girder format with vertical I-sections linking top and bottom chords. The horizontal lattice between bottom chords and between top chords is trussed with light I-section bracing. The verticality of the members linking the top and bottom chords enabled the transfer of consistent dimensions between steps at arches and a high degree of repetition was achieved in the trusses linking the arches. The vertical members are positioned at 2.25m spacings as chords of the main arch radius of 150m.

Throughout the design, consistency and simplicity in setting out the roof was a high priority; its success would depend on the ease and speed with which both fabrication and erection could take place. The triangulated trusses between arches are made up of angle section twin top chords and single bottom chords; the configuration in section is a 2.25m top chord to top chord spacing with a 1.125m truss depth. The bottom chord angle was thus used on its back to make unnecessary the use of cleats at the lattice-to-bottom-chord connections. Simple repetitive connection details were developed for all other connection points.

With these secondary trusses spaced at 4.5m, their top chords provided support lines at 2.25m centres across the surface of the vaults, making a tertiary layer of purlins unnecessary. This led to time savings in erection through fewer large 'pieces' of structure having to be lifted.



For lateral stability the roof relies on the trusses and arches forming a large vierendeel structure on plan with an overall depth of 90m. Calculations showed the arrangement to be suitably stiff under wind loads and stable against arch buckling effects.

Acoustic panel installation

The strict regularity of the grid not only produced time and cost savings due to the repetition in the steelwork, but also facilitated a prefabricated, industrialised approach to designing and installing the acoustic cladding system.

For the new stadium to be truly multi-purpose - able to host concerts as well as sporting events - a high degree of acoustic protection was required. The building fabric had to insulate the stadium operations from the surrounding residential areas as well as afford a reasonable sound quality for those within the building. The resultant acoustic requirement was a build-up of material to be incorporated within the fabric of the roof which increased its normal self-weight by 150%. The approved acoustic sandwich, weighing 60kg/m^2, was engineered into a unitised framing system, partially pre-installed onto the triangulated trusses and erected with them. The intermediate panels between trusses were designed as drop-ins as part of the secondary fix, in situ on the roof. The pre-installed panels provided access bridges for safe and smoothly-handled fixing.

Two additional acoustic layers were then applied to the roof surface before the roof sheet installation. The profiled metal roof sheet was rolled in single lines of up to 130m length directly from the on-site mill up onto the roof surface. The sheet is the clip-on type, with brackets connected through the acoustic sandwich for a full fix onto the steel structure below. The clip system enables the thermal movement of the roof sheet to be accommodated without distress to fixings.

Closure to existing stadium

The structural closure between the southernmost arch and the existing stadium is a simple system of vertical posts with top hat section girts spanning horizontally to carry acoustic panels and side cladding. The posts join to a transfer beam at the level of the existing roof structure and brackets from the beam connect to the existing trusses. The brackets are designed to accommodate differential vertical movement between the new and existing roof structures whilst transferring lateral forces from wind on the new vertical face to the existing roof below. The existing roof structure was strengthened to accommodate the additional loading.

The structural steel for the new roof construction weighed in at an economic 32kg/m^2 with standard plated sections used throughout and no tubular sections. Excellent economy of structure was therefore achieved by vigorous analysis, judicious choice of standard (and thereby inexpensive) steel sections, and careful consideration of erection procedures during the design period. The roof steelwork was fabricated in nine weeks and the whole roof of 400 tonnes erected in eight weeks.

Conclusion

The special achievement of this building was its realisation from commencement of design to construction in six months - a remarkable combined effort from a committed design and construction team. The project also represented a good example of inter-office co-operation between Arups in Cape Town and Johannesburg to ensure that the relevant expertise was applied and the deadlines met.

The R45M (£5.5M) project was successfully handed over on the due date and has proved a major marketing asset for the bid company. The World Junior Track Cycling Championships were very successfully held from 29 July to 3 August 1997 and the building was praised highly by the visiting teams and officials.

Earlier in July, Cape Town was host to the Fencing World Championships, also held in a new venue built on a similar timescale and for which Arups was the structural engineer. These two events did much to show Cape Town's ability to host major sporting competitions successfully and made good impressions on those whose votes counted in the September election in Lausanne. Even so, sadly Cape Town did not win the 2004 Olympics, but as a spin-off from the bid, the city now has a world-class championship cycling venue.

Reference

(1) BOSTOCK, M *et al.* Cape Town's Olympic Games bid. *The Arup Journal, 29*(3), pp10-14, 3/1994.

Credits

Client:
City of Tygerberg

Project managers:
Lurie Yates Partnership:
Proman Management Services (Pty) Ltd

Architects:
MLH Architects and Planners

Quantity surveyors:
Senekal Allen & Partners
Letchmiah Daya Varachhia

Structural engineer:
Arup (Pty) Ltd Andrew Hakin, Dennis Jacoby, Gregory Leon, Neil MacLeod, Ugo Rivera, Radivoy Sendic, Franz Simak, John Smith, Pieter Swart

Mechanical engineer:
Watson Edwards
R & B Controls

Electrical engineers:
Gibb Africa
Johardien & Associates

Civil engineers:
V3 Consulting Engineers

Acoustic consultants:
Jongens Keet Associates

Main contractor:
Grinaker Building Cape

Illustrations:
1, 2, 3, 7: Ugo Rivera
4, 5: The architects
6: Andrew Hakin

1.

East Midlands Airport

Gary Marshall
John Read

Introduction

East Midlands Airport (EMA) lies close to Britain's M1 motorway within the triangle defined by Derby, Nottingham, and Leicester, and serves these three cities and the surrounding region. The runway was laid down in World War 2 and the commercial local airport opened in 1965. The facilities grew steadily over some 30 years while in joint ownership of the three local authorities; more recently, significant expansion has happened as the regional importance of air travel and particularly freight traffic has increased, coinciding in 1995 with privatisation. East Midlands is the third largest airport in the UK for pure freight and one of the fastest-growing in the world.

In September 1994 Arups were invited to assist the Airport planning team to prepare a feasibility study and cost plan on the basis of existing outline proposals for future development of the passenger terminal. In 1994 the Airport handled 1.62M passengers, and it has an annual growth rate of 10%. The aim was to expand the facilities to cope with the immediate demand and allow for up to 3M passengers pa by 2001. The scheme also provides for further expansion to cater for the 4.5M passengers anticipated by 2005. The main requirement identified at feasibility stage was to relocate check-in desks and baggage handling facilities in a substantial new building linked to the existing terminal. The successful conclusion of this initial study, prepared in three short weeks with the help of Arup Associates, was a commission for the full design and procurement of the phase 1 works, described in this article.

Phased redevelopment

The existing terminal contained small-scale check-in and departure lounge facilities, including a basic baggage handling system. The feasibility study identified a programme and cost plan for the phased redevelopment required to achieve the increased throughput while maintaining full operation at all times.

Phase 1 included the new Departures Building containing all check-in and baggage handling, a new arrivals extension to the east of the terminal with an additional baggage reclaim carousel, and major infrastructure changes. The new hall provides 32 check-in desks and incorporates full hold baggage screening in an automatic handling system. Infrastructure changes included a new boilerhouse, gas supply to the site, upgrading of power supplies, a new road system, main car park layout, and landscaping. All main water, fire main and most drainage services were also diverted or upgraded.

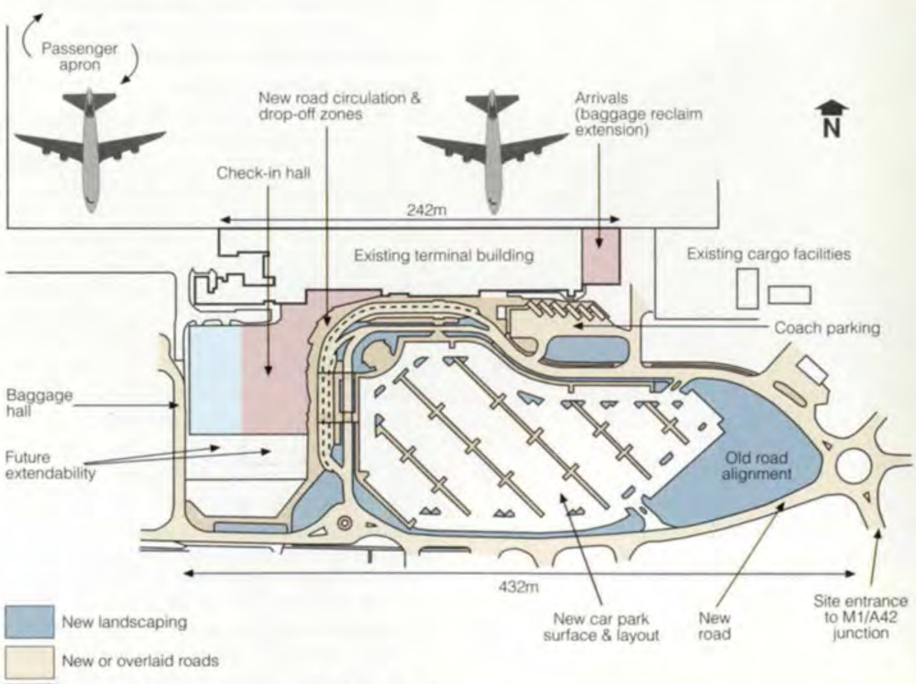
The design had to be tested against proposed longer-term expansion plans to ensure that it would be readily adaptable to future needs. In particular there are proposals for extension of the apron west of the new baggage hall and provision for extendability of the new Departures Building at the southern end.

Phase 1a followed on from the Phase 1 works and involved linking the new construction at Arrivals with the Departures Building by overcladding the existing façade and repaving the frontage to the existing terminal. Phase 2 involved an interior refit of the existing departure lounges, public areas and catering, and did not involve Arups. The commencement of Phase 2 followed completion of phase 1 as the departure lounges were expanded, making use of the space freed by relocation of check-in and baggage handling.

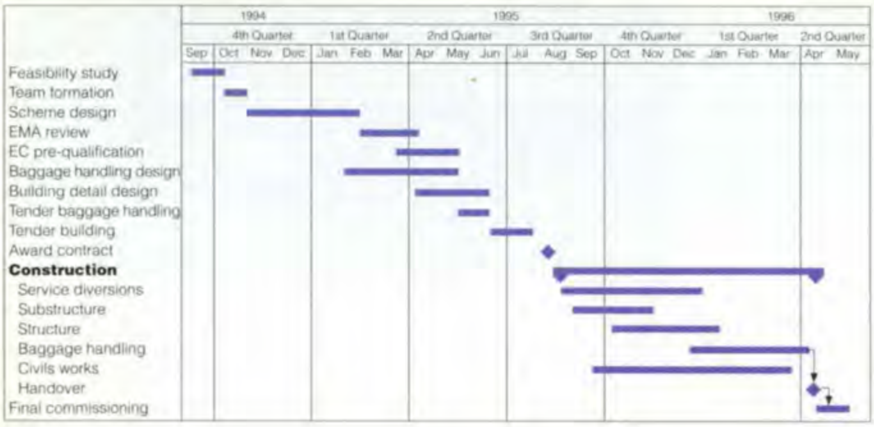
Layout planning

EMA proposed positioning the Departures Building southwest of the existing terminal. Development to the north or west would have severely limited future development of aircraft stands or reduced existing apron space, whilst development to the east is limited by the existing and future cargo handling facilities. The requirement to develop to the south had one severe disadvantage, however, in that it interrupted access to both the plant areas and servicing zones; several options for transferring these facilities to maintain landside access were considered. The final solution was to rationalise the service zone and incorporate a loading dock north of the Departures Building with distribution of services south into the new building and east into the existing. This required careful consideration of security and control issues, as servicing vehicles need to pass through the secure airside zone.

The airside baggage hall size is extremely dependent on the layout of the baggage handling system - described in detail on page 16 - whilst the



2. Site layout plan.



3. Project timescale.

landside area is defined by queueing and circulation space. To operate in its present form - prior to apron extensions - baggage cart access must be to the north to reduce the length of the external air side boundary and travel distances to existing aircraft stands. The building arrangement allows for future direct access doors to be introduced to the west, and provides the height and suspended loading capability necessary for auto-sort machines to be introduced in the future.

The airside security boundary runs north to south through the Departures Building, allowing for future apron space to the west and passenger circulation to the east. The boundary also provides fire compartmentation and is penetrated only by two main baggage conveyors, and at the out-of-gauge area. This also serves as the staff channel, so only one security area in the hall has to be manned.

A high level of integration of architectural, building engineering, and baggage handling constraints was required to define the building footprint successfully.

Management

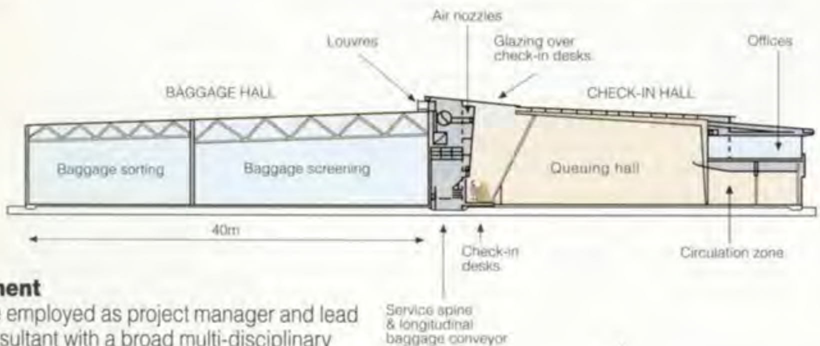
Arups were employed as project manager and lead design consultant with a broad multi-disciplinary role and a brief to promote team working and integration. The brief included full cost control and monitoring. A subconsultant architect - the Nottingham-based firm Crampin and Pring - was appointed, following interview by Arups and the EMA client team. The architect was required to provide a strong design and planning lead to ensure the integrity of the building design whilst working as part of a fully integrated group.

All design team members were allowed direct access to their counterparts in EMA's organisation to develop particular requirements. Regular design team meetings reviewed the developing scheme to ensure full input from all disciplines and achievement of functional requirements. Cost plans were updated regularly to give EMA the opportunity to assess value for money and appreciate the allocation of cost as the design developed. This applied particularly to the baggage handling contracts, where EMA were closely involved in contractor selection and directly responsible for the choice of screening equipment.

Although the Airport is privately owned (by National Express) it falls under the European Community Utilities Directive. This required the works to be advertised in the *Official Journal of the European Community* (OJEC), and formal procedures to be adopted for procurement. The restricted procedure was employed with questionnaires being prepared so that contractors could be ranked and shortlisted. Over 60 enquiries were processed for the building contract and shortlisted contractors were interviewed jointly by Arups and EMA. AMEC Building was the successful tenderer for all Phase 1 building and civil engineering works, with Logan Fenamec securing the baggage handling works. Contract values were £7.4M and £1.3M respectively.

The project's timescale was constrained by a programme for introducing hold baggage screening agreed between EMA and the Department of Transport (DTp) and by the practical restrictions imposed by the seasonal demands of passenger usage. All Phase 1 facilities had to be in use by Easter 1996. Scheme design was completed in February 1995 but not finally agreed by EMA until April. Detailed design and construction was completed in a one-year period (Fig 3 above).

4 below: Cross-section of Departure Building.



To achieve this timescale it was necessary both to pre-order some elements before tender and to incorporate contractor design for selected specialist packages. The adopted form of contract was ACA2 (Association of Consulting Architects) which gives clear definition of contractor-designed elements and the timescale for design development. To ensure co-ordination, weekly design workshops were held and AMEC engaged Faulks Perry as their architect to co-ordinate design by subcontractors. Full involvement of the design team ensured successful solutions were achieved. The project duration was successfully reduced, although to achieve this the design team input during construction was significant.

Full-time site supervision, with a lead electrical engineer on site throughout to reflect the high services content, allowed control of contractual interfaces and a rapid response to design queries. The interface between baggage handling and building contracts was managed smoothly despite inevitably conflicting requirements, although site management of the extensive infrastructure works involved significant liaison and co-ordination difficulties. Maintaining the Airport in full operation throughout the works called for close co-ordination, achieved by formal site meetings and regular client liaison meetings. The last weeks of the project involved significant overtime working on site to achieve the scheduled completion, culminating in an all-night vigil and final morning preparations to greet the first passengers.

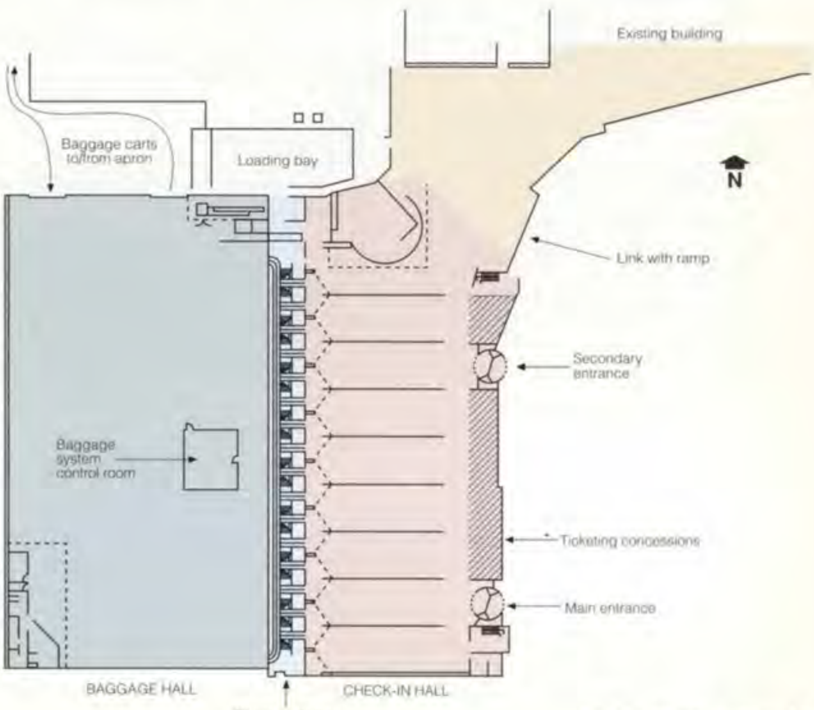
Design overview

A decision was taken to express separately the architecture of the two parts of the Departures Building and allow them to reflect their differing functions externally as well as internally.

The baggage hall roof consists of simple lattice truss main spans with cold formed tubular bracing members and column section chords. Internal columns are positioned in the sterilised areas formed within the reclaim carousels. Externally this area is clad in profiled powder-coated steel to express its functional nature.

The check-in hall addresses the brief to provide an exciting yet calm atmosphere internally and to maintain a friendly personal scale. The architectural approach was to bring the first floor offices to the frontage so that passengers enter a single-storey space under a curved ceiling which then expands upwards into the airy check-in hall. The clean, light quality of the interior is enhanced by the wide band of roof glazing, fabric ceilings, and lightweight ▶

5. Plan showing the two main parts of the Departure Building: The baggage hall and check-in hall.



Baggage handling systems

Graham Bolton

Operational requirements

The baggage handling system is designed to accommodate a peak throughput of 1450 departing passengers per hour, based on flow forecasts prepared by the Airport for the predicted future growth. An average of one bag per passenger is assumed, taking account of passenger surveys and the planned mix of scheduled and charter flights.

The system layout was developed on the basis that the two baggage handling agents already based at the Airport would transfer their operations to the facility, with potential to accommodate an additional agent in the future. Both British Midland and Servisair, the two initial agents, handle both scheduled and charter flights. On day one it was defined that eight of the 32 check-in desks would be allocated on a long-term basis to British Midland, primarily for scheduled flights, with the remainder of the desks allocated as required to match demand.

System overview

The departures baggage handling system comprises 32 check-in desks, feeding bags on a conveyor system through two security screening lines to reclaim carousels for manual sorting of baggage and loading into trailers for onward transport to aircraft. From a controls and operation perspective the system is split into north and south sub-systems which can operate independently. There is provision to inject transfer baggage into the system upstream of each security screening line. A separate line with an independent X-ray screening machine for out-of-gauge baggage (eg large, fragile, or awkward items) is provided at the north end of the facility.The system is planned to facilitate a number of alternative modes of operation, which can be summarised as follows:

Symmetric operation: operating the complete system, with 16 check-in desks feeding each of the screening lines and reclaim units. This is expected to be the normal mode of operation.

Asymmetric operation: allowing an unbalanced split of check-in desks between the two halves of the baggage system. This arrangement provides the airport with flexibility when supporting two or more handling agents.

Reduced operation: allowing operation of the system with a reduced number of check-in desks, a single screening line or reclaim carousel. This arrangement ensures continuous operation of the baggage handling function in the event of a sub-system or component failure, but may also be used as an energy-saving measure in off-peak periods.

The north and south sub-systems are controlled by independent programmable logic controller (PLC) systems, located in separate main control panels. A single SCADA (supervisory control and data acquisition) computer is used to provide a management information and control system (MICS). A separate, hardwired control panel allows operation of the system in a fall back mode.

Security screening

After the terrorist bomb attack on Pan Am flight 103 over Lockerbie in December 1988, the international air travel industry was forced to respond with improved security procedures. Whilst sophisticated screening techniques for passengers and cabin baggage were already well established, less rigorous procedures existed for hold baggage. Within the UK, the DTp mandated that 100% screening of hold baggage be introduced for all departing international flights, requiring major new systems to be introduced in all UK international airports. Screening systems rapidly establish that hold baggage does not contain explosive substances. Once this is established the bag is considered 'cleared'.

A: Baggage hall.

The screening process involves several levels, combining state-of-the-art 'smart' X-ray technology with more traditional manual screening and search techniques. Bags that cannot be 'cleared' at one stage are directed to the next for more detailed analysis. The process comprises:

Level 1: automatic examination of the bag using 'smart' X-ray technology, which will automatically clear 70-80% of bags

Level 2: operator examination of an X-ray image, which will clear most of the remaining bags

Level 3: more detailed screening (of about 1% of total throughput), using a combination of more sensitive scanning equipment and air sniffers

Level 4: reuniting the bag with the passenger, to allow a manual search. This occurs in a very small number of cases (typically <0.1%)

In late 1994, implementation of hold baggage screening (HBS) in UK airports was in its early stages. Whilst the basic screening technology was established, an appropriate method of integrating the equipment in an overall system had to be developed. BAA, after a series of trials, developed a working system for Glasgow Airport in summer 1994 which formed a starting point for planning other systems. Ongoing development of the screening machines was also being undertaken in the USA, allowing more space and cost-effective solutions to be considered, subject to proving the technology.

Based on a review of the Glasgow installation, and Arups' experiences on the Heathrow Transfer Baggage System, several possible concepts were developed for screening at EMA, taking account of anticipated improvements in technology. Following a review with BAA and EMA security staff, two concepts were selected for detailed development and evaluation, using computer simulation (see below), before selection of the final configuration. This employed the latest 'Matrix' system architecture from the US system supplier Vivid. This allows operators in a single control room to view images from multiple X-ray machines, giving improvements in system throughput whilst minimising staff numbers.

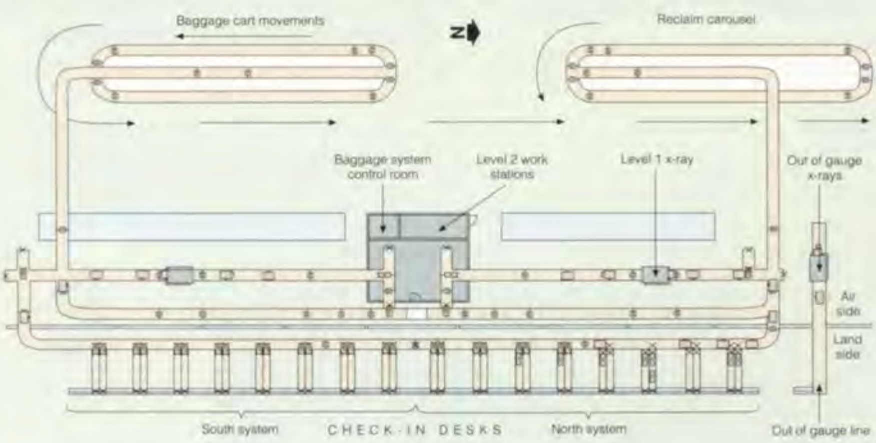
Thanks to the efforts of the baggage system contractor and screening equipment supplier, EMA is believed to be the first UK airport successfully to implement this technology.

System simulation

Effective performance of the system in a range of operating conditions is critical to the EMA terminal's smooth running. To evaluate the system performance early in the design process, the Arup team developed a computer simulation, using the AutoMod package. This commercial package - widely used for modelling materials handling and manufacturing systems - allows the creation of dynamic 3D models to visualise and analyse system behaviour. In this application, the model was used to assess system behaviour under normal operation for various input conditions, and under potential failure scenarios. Being able to compare different configurations of the security screening system was particularly beneficial, considering the impact of queue lengths and the number of operators on system throughput. The team was also able to test the system's robustness to changes in key assumptions, such as screening pass rates and decision times.

Conclusion

The baggage handling system was handed over for operation in April 1996, matching the completion of the main building works. For the Arup automated systems team it was a landmark project, both in coming to terms with the introduction of Hold Baggage Screening, and in the use of computer simulation to prove the design.



B: Plan of baggage handling system.

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canopies above the check-in desks. The desks are backed by a sloping screen which separates the check-in hall from the conveyor route at low level and service route above. The screen is constructed as separate timber panels with wide joints infilled with perforated metal, allowing a glimpse into the service space beyond. The structural form of tapered steel rafters supported on inclined tubular props springing from precast piers subdivides the 32 desks into groups of four, spatially defusing the point of check-in and providing a relative intimacy to the transaction. The simple openness of the space and direction of structure eases the check-in process and aims to avoid confusion. Integrating the new works into the existing required partial demolition through two bays of the original 'clasp system' structure, trimming the existing to open out the very closely-spaced existing columns. The link forms a gently sloping ramp which starts the transition from open hall to retail areas and catering zones on the way to passport control and security. Integrating building services was a key issue and it was aided by creating the service spine behind the check-in desks. Air-handling units are mounted on raised platforms north and south within the baggage hall and feed ducts at high level which throw air across the full width of the check-in hall via high level nozzles in the screen wall.

6.
Check-in hall showing the division of check-in desks by pre-cast concrete piers.

Warm return air is either recirculated or dumped into the baggage hall in cold weather as an energy-saving feature. The step in roof line immediately above the service spine provides louvre areas for air inlets and discharge. These approaches stemmed in part from the height restriction for development caused by the adjacent control tower and result in the building having very clean lines externally.

Key design issues

The multi-purpose service spine was central to the effective use of the building but posed some problems of detail design in accommodating all requirements, as was the case with the desk access. Access to the back of the check-in desks is via a mezzanine walkway in the spine with steps down to each pair of desks. The steps, however, are designed to lift up giving clear maintenance access to the main conveyor which runs in a trough at low level.

It had been hoped to route diverted services and new links to the offices through a service tunnel under the link ramp, but costs and security concerns necessitated a high level service bridge, the integration of which into the fabric ceiling created difficulties due to the large number of service and structural/architectural interfaces.

The structure is expressed in the check-in hall using several exposed elements. Precast concrete piers, constructed in a white aggregate with a combination of smooth and bush-hammered finishes, divide the check-in desks. The shape of the piers is formed by sectioning intersecting arcs, giving a slender tapering form to a substantial structural member. The tolerances involved in constructing and erecting the piers was a significant concern, addressed successfully through design and erection with no problems encountered on site. The raking tubular props supporting the rafters have cast steel tapered ends and forked pinned connections with stainless steel cover plates. Tolerances were again a concern and the roof geometry required the angles of the forked ends to differ by a few degrees at the ends of each prop. A full method statement for erecting the piers and check-in hall steelwork was provided in support of the CDM Regulations. The procurement time for both the castings and the precast piers had significant programme implications and both packages of work were ordered, and their construction begun, in advance of the main contract. The rafters and their supporting columns involved significant fabrication to form cut and rewelded tapering members, and again to meet the programme, advanced ordering of the main sections was arranged.

Road circulation was a major concern, both in the final built context and during phased construction so that access was always maintained. A six-phase programme was needed for roads and car parks whilst the layout was completely reconstructed. Traffic studies and design options had to address numerous conflicting scenarios including controlled usage during security alerts, and the scheme adopted uses an unusual anti-clockwise circulation route.

The new offices and retail outlets created in the check-in hall required heating and cooling, which was piped to fan coil units from the existing chilled water system and the upgraded heating system. To reduce fuel storage area, fuel costs, maintenance tasks, and to reduce emissions, the existing coal-fired boilers were replaced by gas-fired boilers. This also meant that two chimney stacks obscuring the control tower's view could be removed. The switch from old to new was accomplished as part of the works and a progressive changeover was effected for the new and existing circuits. Similar upgrading work for electrical power included providing an 880kVa standby generator for the whole terminal site. This was linked to a new package substation constructed in the north end of the baggage hall.

Artificial lighting in the check-in hall is by 1500W uplighters concealed above the check-in desk fabric canopy and the lip of the office area ceiling. The diffuse lighting created with the fabric ceiling is very effective.

Postscript

The project was marked with both sadness and joy, the former stemming from the sudden and unexpected death of Patrick Sides, EMA's Managing Director, during the scheme's design development. Joy arrived when the project received the distinction of 'Medium Sized Project of the Year' in last year's Quality in Construction Awards. We hope that Patrick would have been proud that his vision has been followed through to such a satisfactory conclusion.

Credits

Client:
East Midlands Airport

Structural, civil, mechanical, electrical, fire, acoustic, and baggage handling engineers:
Ove Arup & Partners
Paul Geeson, Gary Marshall (project management)
Mark Davidson, Duncan Overy, Roger Pickwick, John Read, Julian Thew, Vinh Tran, Chris Webb, Paula Wilson (structural), Peter Court, Andy Davies, Terry Dix, Stuart Hood, Jennifer Innes, Glen Irwin, Martin Lesh, Diane Saddleir (mechanical)
Simon Averill, Ken Sharp, David Stanley, Jared Waugh (electrical)
Dempsey, Geoff Griffiths, Jane Heslington Neil Jeffries, Will Sims, Peter Terry (civil engineering)
Bob Kelly (resident engineer)
Graham Bolton, Adrian de Vooght (baggage handling)
Adrian Collings, Robin Lee, Jonathan Roberts (geotechnics)
Mike Armstrong, Chris Jones (quantity surveying)
Andrew Gardiner, Jackie Perryman (fire engineering)
Raj Patel (acoustics)

Subconsultant architect :
Crampin & Pring

Main contractor:
AMEC Building (now AMEC Construction)

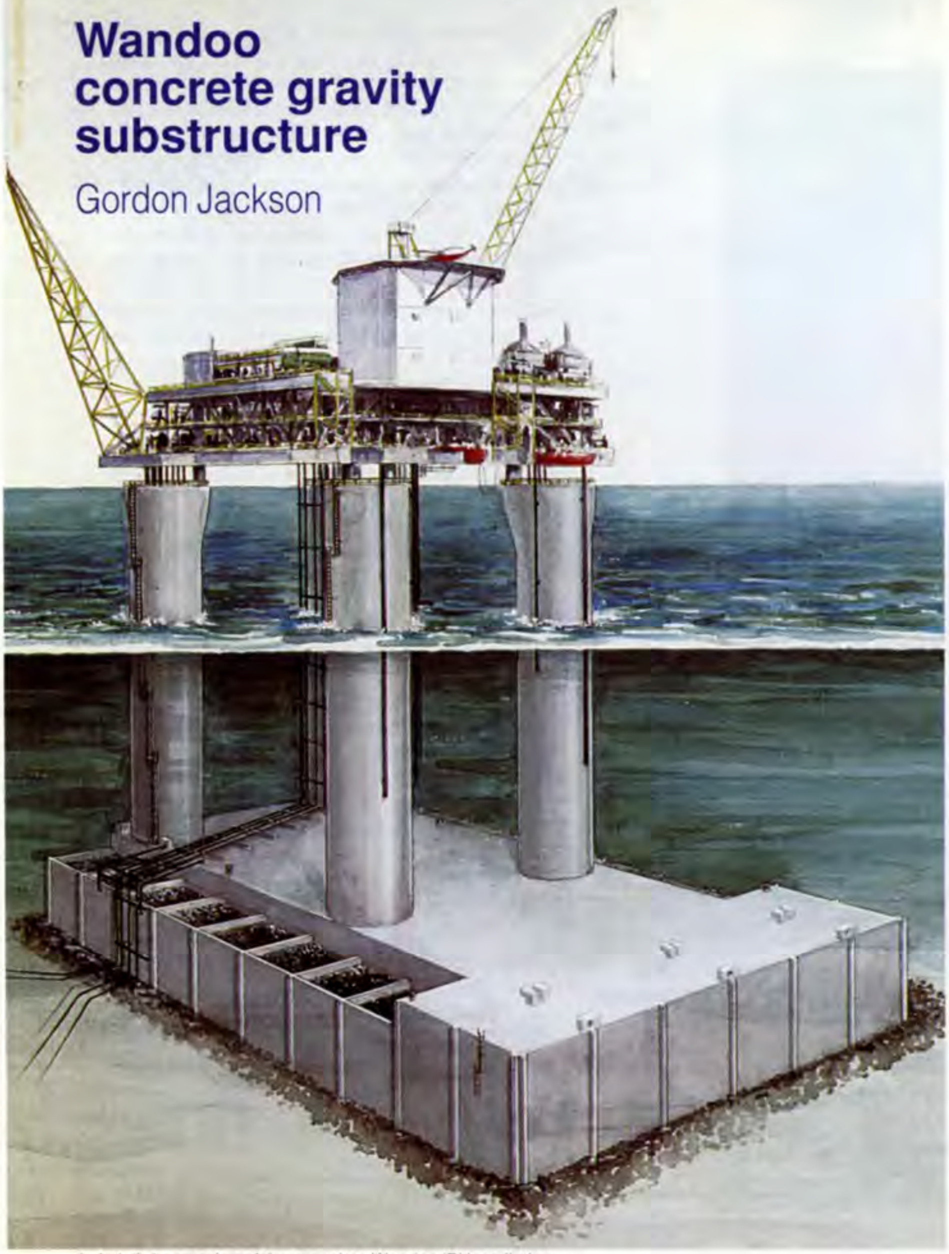
Steelwork fabrication:
Robinson Construction

Illustrations:
1, A, 6, 7: Peter Mackinven
2 - 5, B: Emine Tolga

7. General view and approaches to terminal building.

Wandoo concrete gravity substructure

Gordon Jackson



1. Artist's impression of the complete Wandoo 'B' installation.

The Wandoo field

In 1991 Ampolex Ltd discovered the Wandoo oilfield in the Carnarvon Basin, some 75km north-west of Dampier on Western Australia's North-West Shelf. A marginally economic field to develop, its potential yield was estimated at 75M barrels over 20 years; the oil is heavy (19.5 API) but wax-free and therefore relatively easy to store and transport by pipeline. The reservoir lies in a narrow pay-zone of uncemented sands. It was considered difficult to develop, so Ampolex decided on a two-phase approach.

The first phase - Wandoo 'A' - started as an extended production test and operated for four years. Wandoo 'A' comprised a braced monopod steel jacket, production facilities mounted on a jack-up, and pipeline offloading via a CALM (catenary anchor leg mooring) buoy to a tanker. The second phase was the full field development described below.

Wandoo full field development

Before establishing the Wandoo Alliance in September 1994, Ampolex evaluated a range of development concepts for the Wandoo B platform, including steel jackets plus lifted topsides with oil export either by tanker or pipeline; FPSOs (floating production, storage and offtake); and concrete platforms. Their preference was for a converted crane barge carrying the topsides equipment (McDermott's DB100), supported on a concrete gravity substructure (CGS) with integral oil storage. Following the Alliance's formation in September 1994, the technical feasibility of using the DB100 was examined, a target price was established, the Alliance contractual basis was developed and other concepts were examined to be sure nothing had been missed.

By November 1994, the Alliance had narrowed the options down to a CGS with 300 000 barrels storage and barge-installed integrated deck, and a steel tanker FPSO unit. The Alliance prepared cost estimates in December 1994. The CGS was shown to have a marginally lower capital cost than the FPSO, as well as being cheaper to operate. Consideration of schedule risk was, however, the principal factor that led to concrete being selected; the Alliance was able to commit with greater certainty to a two-year schedule to first oil with a CGS than an FPSO. Other factors were the greater Australian content and the fixed platform's capacity to support future development. Ampolex accepted the Alliance's price and detailed design commenced in January 1995 with a total project budget including the drilling (not part of the Alliance scope of work) of AUD$480M.

Wandoo facilities

Wandoo 'B' comprises a CGS in 54m of water, a two-level integrated deck (Hideck) of 9400t operating weight providing processing facilities for 120 000 barrels per day throughput and 40 000 barrels per day oil production, and three flexible flowlines and dual export lines to a CALM buoy at which the produced oil is loaded onto tanker.

Table 1. The Wandoo 'B' CGS: key facts

Concrete (50MPa cylinder strength)	28 000m³
Reinforcement (460MPa)	8200t
Prestressing	550t
Mechanical outfitting	780t
Iron ore ballast (dry weight)	39 000t

The concrete was a blend of 35% cement and 65% blast furnace slag for low heat gain with local sand and granite aggregate. The reinforcement was specially made with a 460MPa yield strength rather than the standard Australian 400MPa. The cost penalty for adopting higher strength reinforcement was more than recouped by savings in quantity and steel fixing costs.

The CGS has a rectangular base caisson 114m x 69m in plan and 17m high, containing oil storage in two compartments. A buffer tank houses an additional stage of oily water separation in the ballast water handling. A closed water displacement oil storage system was adopted with topsides provision for full ballast water clean-up. A closed system, where there is no direct communication between the storage compartments and the sea is considered the best solution in terms of minimising the risk of pollution. Four 11m diameter shafts rise to a total height of 69m, offset towards one end of the base caisson to permit jack-up drilling through one of them. Two others house oil storage pipework and diesel respectively; the fourth shaft contains no facilities and is simply filled with seawater.

Arups' role

Arups' role in the Alliance was to design the CGS and the casting basin in which it was constructed, and to support the Alliance in areas where Arup skills were relevant, such as installation engineering of the CGS and to a lesser extent the deck. The firm also participated fully as an Alliance participant at Board level and within the Alliance Project Management Team. The Alliance contract creates the possibility of developing a strong synergy among all participants through all stages of the project, and it should be understood that the results outlined in this description of the CGS are due to the efforts of the Alliance as a whole, not merely those of individual firms.

Conceptual design development

A CGS design should be cost-effective, able to accommodate changes, and simple to construct and install, so in the early stages of concept development the designer must understand how it will be installed and built so as to satisfy these criteria. The design evolves rapidly as schemes are conceived, tested, and refined, and the designer's experience in structural engineering, naval architecture, and geotechnics in particular is essential to the creative process towards the preferred concept. This design process is one of the most highly interactive of any type of project. The conceptual engineering must also embrace a thorough understanding of project economics; knowledge of the likely construction productivities that will be achieved for the elements being considered. Through the Alliance contract mechanism, the designer, constructor, and installer of the CGS worked together as equals from the outset.

Within a month of the decision to pursue the 300 000 barrel CGS and Hideck option, the concept was confirmed, and throughout detailed design remained unchanged. At a detailed level, however, the constructor was able to initiate improvements to suit the construction method as well as absorb alterations to improve the topsides and pipeline construction and installation methods. It was equally important that the scheme be flexible enough to incorporate innovative, cost-saving ideas. As these ideas emerged they were assessed against the project economics and incorporated if the net present value was improved.

Text continues on page 21 ▶

The Wandoo Alliance

Robert Care

What is an Alliance?

An Alliance is a particular method of bringing companies together in a working partnership to produce results to the mutual benefit of all participants. Alliance participants focus on a common goal or outcome for the project. This is achieved through a contractual framework whereby each party's reward or profit is at risk, based on the performance of the whole project and not just their portion of it; individual organisations are only rewarded for achieving the project's goals - not their part of it. To negotiate this form of contract, a relationship must be developed between client and contractors based on trust, dedication, common goals, and understanding of each other's individual expectations and values.

The Alliance contract establishes the framework for sharing the financial rewards among the Alliance participants when improvement on the target price is achieved. Equally, the risk of cost overruns is shared among the participants. This provides a very focused commercial incentive for the team to identify solutions rather than each organisation dwelling on whose fault any problem may be.

To achieve success with alliancing, it is crucial that not only are the corporate objectives of the parties understood and aligned, but there is ongoing education and alignment of the management, professionals, and individuals involved. This works towards developing a culture of collaboration, mutual respect, integrity, innovation, and no blame: all concerned focus on results, willing to challenge conventional paradigms. This culture should ideally be created separate from the culture of the parent organisations or participants, even though the cultures may be similar.

To be successful, the Alliance should be a separate entity with its own identity, and most important be supported and championed by the client as well as all participant organisations. It should be considered as a 'virtual corporation'. Above all else, alliancing offers the opportunity to use 'breakthrough' principles - to challenge and change past ways of doing business of all the parties in the Alliance. The approach allows corporate and individual 'baggage' to be shed, and new opportunities and possibilities generated to achieve extraordinary results - well beyond what could be predicted given past performance and the notion of 'business as usual'.

Alliance principles

The Alliance operated in a framework of six principles to create a receptive atmosphere for good ideas, to effect breakthroughs, and to guide team members towards acting in the best interests of the Alliance:

- *to work together in a spirit of openness and co-operation*
- *to use innovative methods to bring the Wandoo field on stream at the lowest possible cost whilst meeting the design basis, operating standards, and schedule*
- *to disclose to each other cost and technical information - an open book approach*
- *to bring full commitment to effective interfacing between the parties*
- *to strive for continuous improvement*
- *to integrate staff from one party into another where it best suited the project needs - on a best person for the 'job' basis.*

Achievements

Besides constructing the first CGS for Australian waters, the Wandoo Alliance delivered other 'firsts' for the industry:

- *first casting basin built in Western Australia for the offshore oil industry*
- *first offshore installation of an integrated deck carried at high level on a transportation barge with a CGS installed on the seabed*
- *first application of 'smooth bore' flexible pipelines for oil offloading*
- *first oil within 26 months of project sanction compared to an industry norm of 36 months*
- *maximum daily oil production - 20% above design oil production capacity (47 000 bopd vs 40 000 bopd) within four months of first oil.*

The alliance culture successfully broke down the usual barriers between designer, constructor, and installer. For example, design team members worked in the construction teams at both Bunbury (CGS construction) and Singapore (topsides construction) and were members of the installation team. Supervision of one party by another or man marking was avoided, as were multiple project management layers. Before starting on site, the constructors worked with the designers to ensure the design was tailored to the respective construction methods at the two fabrication sites. Above all, those who worked in the Alliance found it a rewarding experience in terms of fulfilling their own potential contribution to a highly successful project.

The Wandoo Alliance participants

The Wandoo Alliance comprised five companies which collectively had not previously worked together, although each had some previous working relationships with one or more of the others:

Ampolex: *Operator*

Ove Arup & Partners:
CGS and casting basin design

Leighton Contractors:
CGS and casting basin construction

Brown & Root:
Topsides and pipeline design, all installation activities, procurement, and project services

Keppel Corporation:
Topsides construction and living quarters design and construction.

Ampolex commenced the project as operator and 100% owner of the Wandoo permit area. In 1995, 40% of its interest in the Wandoo permit was sold to a consortium of mainly Japanese companies, the largest being Mitsui. Late in 1996, Ampolex was bought by Mobil.

Schedule

Wandoo was completed faster than any other dry-built CGS: from the start of concept design to sailaway took only 22 months, due considerably to the alliance working method. Although this is believed to be the fastest schedule yet achieved, the project demonstrated that opportunities clearly exist to reduce the program by a further three months for a similar platform.

Project timetable

September 1994:
Alliance created.

November 1994:
300 000 barrel CGS with barge-installed deck concept selected, along with a steel FPSO.

December 1994:
300 000 barrel CGS concept sanctioned.

January 1995:
Detailed design commenced.

February 1995:
Casting basin construction commenced.

July 1995:
CGS construction commenced.

August 1995:
First base slab poured.

October 1995:
Wall slipforming commenced.

December 1995:
Decision to increase storage to 400 000 barrel.

March 1996:
Outfitting works commenced.

July 1996:
Concrete works completed.

September 1996:
CGS sailaway.

October 1996:
CGS installation.

January 1997:
Topsides placement.

March 1997:
First oil.

The contribution of John Kava, Project Integration Manager, to this section is gratefully acknowledged.

Casting basin design and construction

Site selection

A new casting basin was required to construct the Wandoo CGS since none existed in Western Australia that could accommodate a structure of this size. Several ports have relatively deep harbours and approach channels: Albany, Bunbury, Fremantle, and Port Hedland all have at least 12m dredged depth at lowest astronomical tide. Bunbury was nominated by the Wandoo Alliance when it was formed in September 1994, based on previous studies carried out by the Alliance participants.

In addition the Western Australian government had been promoting a 'Concrete Offshore Structures Industry' (COSI) in conjunction with the Australian Cement & Concrete Association: a COSI basin site study undertaken by local consultants and sponsored by Ampolex and others confirmed Bunbury as COSI's preferred site in November 1994. The Wandoo Alliance determined from the outset that the casting basin should be a 'hole in the ground' rather than an elaborate construction containing elements of permanence beyond the immediate means of the first project.

Advantages of Bunbury

The factors considered most important in the selection of Bunbury were:

- highly sympathetic port authority
- sufficient land for development
- location in an area of future port expansion, thereby adding a tangible asset
- favourable characteristics for basin development indicated from recent harbour extension
- ample land available for spoil disposal
- outline environmental approvals already existing
- exit channel from the inner harbour wide enough not to need dredging
- temperate climate
- good local industrial infrastructure
- the proximity of Perth and its industrial infrastructure.

Site location

This was influenced by competing considerations. Basalt dipping to the east made the extreme east of the inner harbour most favourable if excavation in rock was to be avoided, but an existing road and railway limited the extent of the basin eastwards. The CGS was to be floated out and turned through 90° in the inner harbour - and there was more sea room the further west the basin was located. The Alliance concluded that float-out was feasible from the eastern limit of the inner harbour and the location was finalised.

Basin size and facilities

The basin was to provide minimum facilities for constructing the CGS. Investment in more fixed facilities such as permanent gates could not be justified on economic grounds for a one-off project and also could reduce the flexibility of the site for future users. The size of the basin is primarily governed by the size of the CGS and construction considerations. The surrounding floor had to allow large crawler cranes to service the centre of the CGS. 15-20m is normally sufficient and 20m was selected here. To allow float-out, a finished floor level of -12.0m AHD was selected. (Australian Height Datum is 0.64m above chart datum which normally equates to lowest astronomical tide.)

The landward basin batters were 1:2 with 2m wide benches at intervals to permit installation of dewatering facilities; the sea bund batter was 1:2 with a 15m wide bench at -7m AHD. The existing batter of the sea-bund into the inner harbour was less steep at 1:3. On the western side a 1:10 sloping access ramp 10m wide was provided.

Geotechnical design

The basin's detailed design ran concurrently with the CGS conceptual design, to ensure the construction schedule could be met. A detailed site investigation in November 1994 revealed highly variable geotechnical conditions. The upper strata were alluvial deposits of loose clayey silty sand with pockets of soft to firm sandy silt and clay. The fill overlying these deposits was recent - loose silty sand. Most of the lower strata were interbedded medium dense to dense clayey sands and soft to hard, high plasticity clays. At the western end, older very dense sand/gravel size clasts of slightly weathered basalt and quartz in a very stiff to hard claybound matrix overlaid the basalt bedrock. The latter was a strong fine-grained dark grey to blue grey rock with local vesicles and columnar joining, not encountered above basin floor level within the curtilage of the basin, and not at all at the eastern end. Here, soils at basin floor level comprised very dense silt and sand.

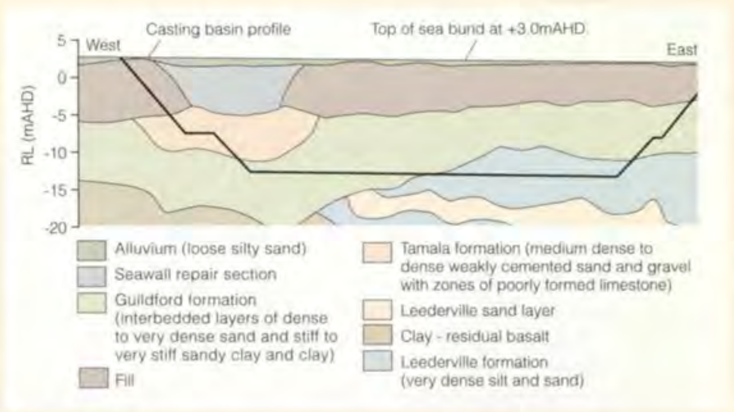
The geotechnical design confirmed the landward batters as stable with appropriate dewatering. The sea-bund was designed with a bentonite slurry cut-off wall to -7.0m AHD - just below the relatively permeable alluvial fill material, and extended each side of the basin some 100m. Total predicted water ingress to the basin was 70 litres/sec assuming fairly conservative permeabilities for the materials found.

Basin construction

Basin construction began in late February 1995. The intended construction schedule was five months with the total volume to be excavated around 440 000m^3.

Excavation of the upper part was largely by scrapers, selected for their speed to the spoil grounds and thus low cycle times. In due course, however, ground conditions became too wet for their efficient operation and excavation was completed by large excavators and articulated trucks.

Groundwater was initially controlled by dewatering spears jetted into the basin perimeter which depressed the water table by around 3m, enough to permit construction to around -7m AHD.



A. Geotechnical section.

Small areas of limestone, already found in the geotechnical investigation, were encountered between -6m and -7m. At -7m AHD, a stream appeared in one end of the sea-bund. Initially controlled with erosion protection material, it remained stable for around 36 hours, but then flows suddenly increased, rapidly eroding the bund and breaching it on the morning of 29 April 1995. No one was hurt, largely due to site management planning and the foresight of the general foreman.

This was a major setback and delays upwards of three months were considered likely. The major problem was finding and mobilising appropriate equipment - and the Alliance principle of 'No Blame' was put to the test for the first time. In the event the strength of the Alliance approach was fully demonstrated. Rectification work started the same day as the breach occurred and was carried out so fast that concrete construction began only four weeks later than scheduled. Locally sourced sand was end-tipped into the breach and vibro-compacted. A new bentonite cut-off was commenced and taken to -17m AHD or rock head to ensure that all sand lenses were intercepted. The basin was drained and excavation recommenced on 29 May 1996.

A review of the breach suggested that locally heavy seepage occurred either through, or beneath, a thin limestone layer in beds of clean sand infilling an old river channel; both limestone and channel were identified during the initial site investigation. Permeability tests in the sands at that time were used as the basis for basin design and did not predict local flows large enough to cause failure. It was concluded that some pre-existing water path or internal collapse caused the relatively sudden breach.

After the remedial work, excavation from -7m AHD to formation level of -12m AHD proceeded as originally planned. This required careful management of groundwater levels. The soils under the basin were shown to contain confined aquifers and the water pressure had to be brought under control to prevent the basin floor boiling as overburden pressure was removed. The western half was underlain by basalt, sufficiently weathered and jointed to act as a saline aquifer; the eastern half was underlain by two confined aquifers in the Leederville and Yarragadee formations, the first partly saline and the second fresh water. Bore pumps installed around the basin perimeter to draw down the water levels in the aquifers during construction permitted excavation to proceed to formation level and perimeter drains to be installed. Passive wells around the basin perimeter discharged from the aquifers into the perimeter drain, and two sets of pumps discharged the water via a settling pond into the inner harbour. A series of bore pumps incorporating well screens was installed in the sea-bund. The fresh water and seawater pumps had to be segregated as iron precipitation in the freshwater occurred if flows were mixed and the pumps and well screens rapidly became clogged.

Once basin excavation was complete, tests confirmed predictions from analytical models and permitted the dewatering pumps to be decommissioned, since adequate control of the aquifer head was being achieved through the passive relief wells at the toe of the batters.

Construction was completed in September 1995, having overlapped with CGS construction by two months. Although the latter start was delayed by the breach, the overall delay was minimised by having a phased handover of the site.

CGS construction

To ensure the continued stability of the sea-bund and the safety of construction personnel, daily monitoring of water ingress, pump operation, and piezometric levels took place. A contingency plan was drawn up to safeguard personnel safety in the basin. This comprised staged withdrawal of the workforce should pump breakdowns not be rectified within a specified period. This management strategy proved very effective and in the event no stoppages were required during CGS construction. The bund and groundwater regime behaved as predicted. Overall flows into the basin were less than the design values: during the summer 52 litres/sec entered the basin; flows increased as the winter rains arrived but remained below the design value of 70 litres/sec.

B. Casting basin construction, April 1995.

2.
Caisson slipforming in progress, December 1995.

3.
Progressive decking-out and concreting roof slab, with first shaft half-completed, March 1996.

4.
Shaft slip-forming, April 1996.

Text continued from page 18.

Design justification

Once a concept is finalised, one of the main tasks is proving its adequacy and preparing justification calculations. Equally important, though, is that designer and constructor continue to work on each detail to ensure the CGS is simple to construct. Arup staff from the Australian, Malaysian, Nigerian and British offices worked on the design justification in the Alliance project office in Perth and Arups' London office before the focus shifted to Bunbury, Western Australia, when construction commenced. Detailed design and verification of the concrete to ensure all structural elements were satisfactory involved careful consideration of a myriad of load cases using advanced analysis and verification tools. These tasks accounted for less than half the total design effort, however. Steelwork had to be detailed both inside and outside the concrete structure. Ballasting systems to install the CGS and then act as oil storage systems had to be laid out inside. Electrical systems were incorporated to both control the temporary installation phases as well as monitor and control the oil in store during the life of the field. The estimated weight and the centre of gravity position were continuously tracked to ensure the maximum float-out draft was never exceeded and that the structure would remain stable in all conditions when it was floating. Installation sequences were gradually refined and finalised in response to the weight and centre-of-gravity changes as design and construction proceeded.

Structural form

The simple structural form of straight walls and flat slabs is the main reason why CGS concepts like Wandoo are straightforward to build. Equally important is to minimise the number of different section sizes and shapes, since repetitive geometry makes formwork more efficient. The inherent robustness and flexibility of the Wandoo 'B' CGS stem from structural arrangements and details refined over several years.

Whilst its outer appearance is similar to Arups' earlier Ravenspurn North design[1], significant changes were made to the internal layout and operational systems to simplify the design. As a result Wandoo is substantially cheaper in real terms than its predecessor the Ravenspurn platform built nine years earlier.

Element sizes

The hydrostatic loads experienced during installation govern the thicknesses of slabs and walls. An 11.25m square grid was selected, based on the thicknesses needed to carry these loads. Other considerations were:

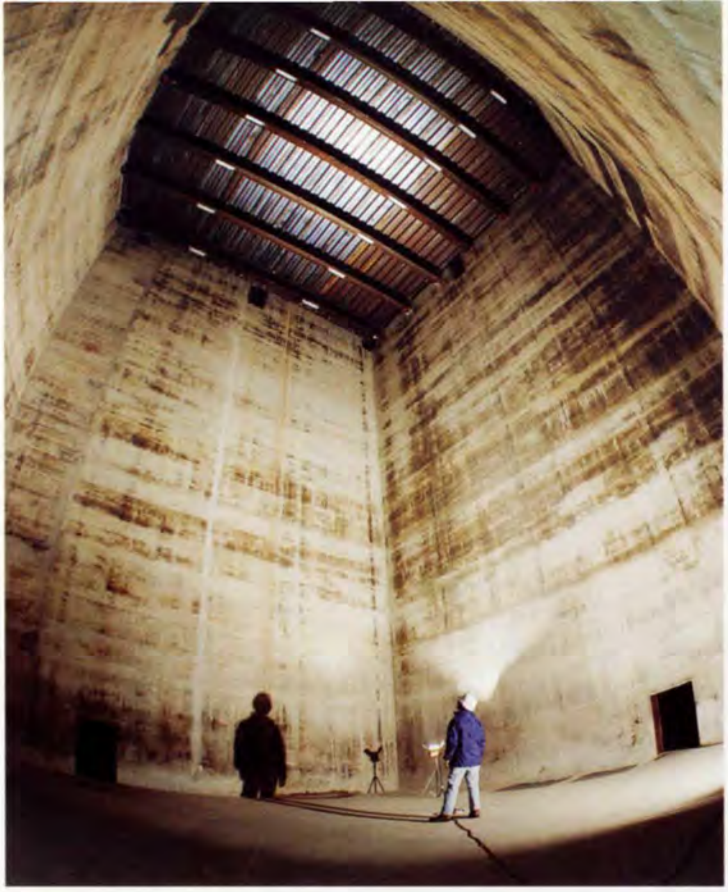
- the capacity of the shafts to withstand the imposed loads with a diameter that fitted into the square grid
- the use of cast-in pipework in all slabs
- the number and spacing of conductors
- the size of barge needed to carry the integrated deck at high level and fit between pairs of shafts.

Prestressing

The horizontal prestressing in the caisson comprises straight tendons in the walls and roof. The prestressing anchorages were located in external buttresses at wall intersections to minimise congestion in this area. Vertical prestressing was provided to the shafts, anchored in the base slab and cap slab on top of each shaft. Access to the base slab anchorages was through specially constructed access tunnels under the CGS, an arrangement preferable to anchoring the bottom of the tendons inside the shaft.

Reinforcement detailing

All reinforcing drawings and bending schedules were prepared by the designer to suit the agreed construction method. This process is essential for efficient construction of CGSs.



5.
Interior inspection of closed cell during hydrotesting, July 1996.

Compartmentation

Full advantage was taken of the inclined installation technique pioneered on Ravenspurn. Two 16-cell groups formed large central compartments, with open-topped cells and trim tanks around them to protect against impact damage during transportation. The few compartments reduced pipework reticulation and numbers of valves, only seven being needed for the CGS installation.

Cast-in pipework

Pipework for the oil storage and installation systems was entirely cast into the slabs, which eliminated follow-on work, potential for pipe damage, and the need for puddle flanges in water-tight walls: these need great care and attention if they are not to leak. However, to obtain these benefits requires a high degree of co-ordination between concreting and outfitting works early in the project, and may result in pipework procurement coming onto the critical path.

7.
GRE pipework for oil storage system inside substructure.

6.
The installation in July 1996: eight open cells in foreground, and closed cells behind.

8.
Exterior pipework on shafts.

Innovation

To maintain their competitive edge, CGS designs must continually develop through innovative ideas. To introduce innovation successfully on a fast-track project is a tremendous challenge: developing and testing novel ideas needs time to ensure that all their aspects have been examined. The Alliance culture helped this process, through its recognition that cost savings could be realised through greater efforts in the design office, and this environment allowed the Wandoo 'B' CGS design to incorporate successfully several innovative ideas.

Oil storage system

This was based on the principle of minimising equipment in the utility shaft, and the chosen design is perhaps the simplest oil storage system ever developed, evident by the low operational maintenance required - occasional inspection of just four valves. This compares to substantial plantrooms down shafts on earlier CGSs.

Glass-reinforced epoxy (GRE) pipework was used throughout the oil storage system on both the ballast water and oil sides - probably its most extensive use on a CGS with integral oil storage. GRE ensures the durability of the storage system pipework through the life of the field, whilst its lightness, and ease of handling and jointing make it straightforward to use alongside concreting trades.

All instruments are arranged to be fully accessible and replaceable through the life of the field. Pipework is cast into the base caisson walls and extends the full height of the shafts to act as conduits for the instruments - believed to be the first such configuration of instrumentation on a CGS.

Foundation

The foundation design allowed the CGS to be built directly on the casting basin floor, allowing an early start on base slab and wall construction. Foundation arrangements depend on soil conditions, those at Wandoo comprising calcareous sand over calcarenite. Sliding is the governing mode of failure for CGSs on such soils and skirts are often provided to ensure sliding takes place in sufficiently strong soil. On Wandoo it was possible to eliminate them, apart from a perimeter skirt 300mm deep to limit scouring before rock dumping could take place. A roughened base slab profile, plus a novel series of strip drains to minimise pore pressure generation in the calcareous sand, gave the necessary frictional resistance.

Permanent roof falsework

A system of steel beams and metal decking to support the weight of concrete before it gained strength was adopted. Steel saddles set on the top of the wall into which steel beams could be simply dropped were fabricated, thus allowing immediate access for setting the metal decking.

This allowed roof slab steel fixing to commence almost immediately after stripping the wall forms compared to the usual 1-2 weeks.

Modular outfitting

Details minimising work in spaces where cranage and access are restricted greatly benefit construction schedule and productivity. A modular steel tower was developed for the utility shaft to support all the oil storage pipework, routing for controls and instrumentation, and ladder access to the shaft base. The sections were fabricated offsite and extensively outfitted at low level before being lifted in for completion. Being self-supporting for vertical loading, the tower required few connections to the shaft walls and virtually no temporary scaffolding - another construction interface best minimised - to carry the transportation and installation forces it experienced.

Design flexibility

The Wandoo concept proved robust.

Accommodation of design development and agreed construction details changed concrete quantities, weight distribution, and vertical centre of gravity, but throughout this process the float-out draft and installation sequence remained virtually unaltered.

With construction already under way the design was asked to be even more flexible. The client responded to an Alliance suggestion that 33% more oil storage was possible - up to a total of 400 000 barrels - with minimal disruption to the completed works. The Alliance provided this by converting trim tanks and open cells into storage compartments at the southern end of the structure where construction had just started. A week after the decision, new reinforcing drawings and bending schedules for the base slab were ready. Construction proceeded with only minor disruption and no impact on the target completion date.

Construction

Overall method

This was conventional, with the caisson walls slipformed in eight sections and the shafts slipformed in pairs. The base slab was also in eight sections, and the roof slab in six sections.

Concreting

The largest concrete slab pour, 1600m^3, was completed within a day, whilst another was achieved on the hottest day of the year, in shade temperatures peaking at 42°C. Special provisions were needed for concreting in such hot weather; the necessary elements in the mix design and the construction control exercised during concreting resulted in good thermal characteristics in the concrete, even in section thicknesses up to 3.5m. The disadvantage of providing concrete suitable for summer working is rather slow setting time. A change to the setting accelerator during shaft slipforming pushed rates up to 250mm/hour, and adopting a similar mix for the later wall slips gave more than 100mm/hour.

Reinforcement

Reinforcement fixing rates were good throughout, consistently bettering original estimates. In the slab work, rates below 10 man hours/tonne were achieved - around 50% better than on past projects. All this was helped by the low average reinforcement density of 290kg/m^3. Conventional patterns could be employed for slab and wall construction with normal shear links, whilst T-headed reinforcement was used in the thicker shaft areas that support the tubular steel deck connection stab-ins, after concluding it would be cost-effective because of good fixing productivity.

Outfitting

Another key element of efficient CGS construction is management and control of the construction and outfitting scaffolding. Wandoo 'B' needed only minimal quantities; access provision was integral in the slipforming process so scaffolding is not required. The modular tower sections in the utility shaft had full access provision included, thus avoiding internal scaffolding to the shafts. External outfitting works used a combination of fabricated access towers and mast climbers.

Transportation and installation

The CGS was deballasted in the previously flooded casting basin on 11 September 1996. It had a draught at float-out of 10.74m and the final weight was within 1% of the estimate - a testament both to the quality of the dimensional survey and the method of establishing concrete densities through correlation of cylinder testing to selected element coring.

Sailaway commenced at first light on 12 September 1996. The CGS was manoeuvred out of the basin using one main tow tug for forward propulsion and three harbour tugs for sway and yaw control. After eight hours it was clear of the Bunbury channel, the second tow tug of 146t bollard pull was attached, and the tow to field commenced.

It was decided that an escort tug was not necessary despite the length of tow and the proximity of the route to the coast. On leaving Bunbury, the weather immediately worsened and exceptional storms were encountered, with significant wave heights up to 5m and 50 knot winds. One of the sections of the main tow lines, where synthetic material had been used, parted and had to be re-connected before the tow could re-commence. Despite the weather, however, the tow to site was completed in 22 days, only six more than anticipated.

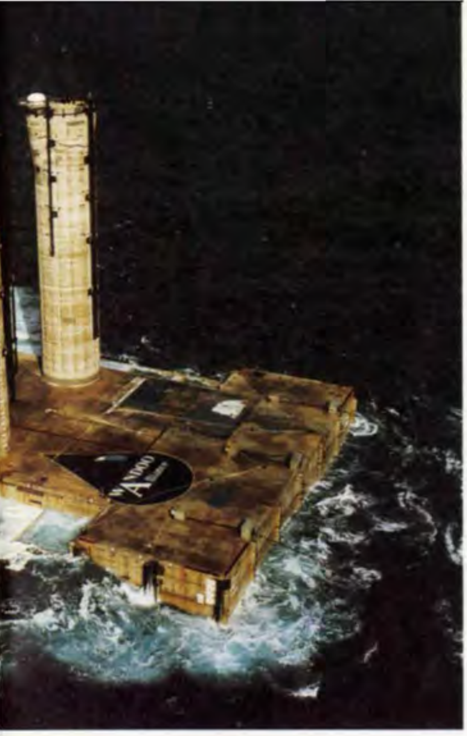
Conditions at site for installation were good, with waves under 1.5m and wind speeds less than 20 knots throughout. The inclined installation sequence began at 10:00 on 5 October 1996 and the CGS's leading edge touched down at 17:30.

The CGS was rotated to the horizontal and fully touched down at 14:00 on 6 October 1996. A crushed rock scour protection blanket was placed around the structure followed by iron ore ballast into the open cells. This work, in conjunction with pipeline stabilisation work, was completed on 24 December 1996.

9.
The CGS route from Bunbury to the Wandoo field.

10 & 11.
Leaving the casting basin.

12.
The float-out from Bunbury drew widespread local attention.



Completion

The topsides was installed on 11 January 1997 after a two-week delay due to cyclones in the Wandoo area, and first oil was achieved on 10 March 1997, some three weeks ahead of the agreed schedule. The project duration of 26 months from the start of detailed design to first oil is exceptional for a project of this nature and shows once again the achievements possible with alliancing. It compares to an average of 36 months established when bench-marking Wandoo 'B' against other comparable projects world-wide. Also, the project was completed for less than the target price and the participants have benefited from their share of the savings achieved.

Future projects

The Wandoo CGS successfully demonstrated that such projects can be delivered on time and budget in Australia. A significant part of this success can be attributed to the Alliance way of working.

The Australian oil industry is now aware that a CGS designed and constructed in Australia can compete successfully against an imported FPSO, and other concrete platforms are likely to be selected for the North West Shelf. The CGS + barge-installed deck concept is finding favour throughout SE Asia, since it avoids reliance on heavy lift vessels. Wandoo has shown that the cost of a casting basin does not make the overall economics of such a field development unattractive. This is expected to be a key consideration throughout SE Asia since the region has no suitable casting basin facilities at present.

In the North Sea, the success of the Wandoo project is expected to increase oil companies' interest in concrete as a viable development alternative. Recent experience supports this view with the rate of dry-built CGS construction worldwide increasing steadily:

Ravenspurn North	1989
F3-FB	1992
Harding	1995
Halfweg Q1	1995
Wandoo 'B'	1996
Bream B (completed at a quayside)	1996
West Tuna (completed at a quayside)	1996
South Arne	1998

The Wandoo 'B' CGS demonstrates the flexibility of concrete both to offer different solutions and accommodate changing thinking. It is sure to stimulate the trend towards more CGSs being adopted for field developments around the world.

Reference

(1) ROBERTS, John. Ravenspurn North concrete gravity substructure. *The Arup Journal, 24*(3), pp2-11, Autumn 1989.

Credits

Client:
Ampolex Ltd

Other Alliance Partners:
Brown & Root
Keppel Corporation
Leighton Contractors
Ove Arup & Partners
Robert Addlesee, Ade Adekunle, Steve Armstrong, Raphael Arndt, Andrew Bannink, Terry Bell, Phil Bramhall, Ian Brooks, Vivienne Brophy-Smith, Jane Bushaway, Des Butler, Henk Buys, Robert Care (Arup Australia project director and Alliance board member), Simon Cardwell, Peter Chamley, Shen Chui, David Clare, John Clinton, David Collier, Mark Collier, Mike Cook, Eddy Dunne, Fred English, Ian Feltham, Mike Francescon, Karen Gates, Tony Gourlay, Andrew Grigsby, John Hamilton, Daire Hearne, Stephen Hendry, Trevor Hodgson, Charlie Houston, Chris Humpheson, John Innes, Ashraf Issak, Gordon Jackson (engineering manager), Mike Jacobsen, Tyran Jones, Chris Judd, Bernard Kelleher, Peter Lees, Mike Locock, Angus Low, Ken McDonald, Alan MacLeay, Andrew McNulty, Chris Martin, Charles Milloy, David Moorehead, Hugh Muirhead, Christopher Owen, Heleni Pantelidou, Oscar Paredes, Darius Pavri, Bernie Pemberton, Tony Phillips, John Redding, John Roberts (Ove Arup Partnership project technical director and alternate Alliance board member), Dan Ryan, Tony Ryan, Martyn Scott, Paul Scott, Paul Sexton, Tony Sheehan, Rob Simpson, Rob Smith, Don Sylwestrzak, Allan Teh, Nick Thompson, Judy Tomelty, Noel Tomnay, Kim Vivian, Rob Wallis, Niall Watson, Ian Webb, Kaye Welsh, Stanley Yuen, Jacqui Zugg

Illustrations:
1. Fred English
2-15, B: Stevenson Kinder and Scott
9, A: Martin Hall

13 above:
In rough seas off Perth.

14.
Touch-down on 5 October 1996.

15. Positioning the topsides, 11 January 1997.

The Nigg dry dock upgrade

Alan MacLeay John Mott

Introduction

In January 1996, Morrison Construction Ltd approached Ove Arup & Partners to join them in a design-and-construct bid for upgrading Nigg Dry Dock on the north shore of the Cromarty Firth on the north-east coast of Scotland. This was no ordinary contract of its type as the client, BARMAC, stipulated that they wished to form an alliance agreement with the winning contractor to share responsibilities jointly for the project's technical and economic viability. 'Risk-and-reward' contracts between client and contractor are now commonplace in the oil and gas sector but not in civil engineering, and it is thought that this was one of the first in the UK. Tenders were submitted in April 1996 and, following minor revisions to the functional specification, a revised tender was lodged in June. Morrison Construction was awarded the contract in July 1996.

The design programme started immediately and various specialist groups throughout Arups had to be involved, due to the diverse nature of the project. It was managed from the Aberdeen office supported by:

- Arup Geotechnics in Edinburgh and London for design of the quay wall and dock slopes
- Civil Engineering in Edinburgh for ground modelling of earthworks using MOSS software
- the Environmental Group in Edinburgh for advice on treatment of contaminated soils
- the London Maritime Group for design of rock armouring and fendering specification
- Arup Research &Development for advice on concrete mix design, concrete repairs and steel pile corrosion
- the Abseiling Group in Cardiff for roped access inspection of the dock gate
- finite element modelling of the quay wall by Arup Energy in London
- engineering support staff from the Coventry office.

The Aberdeen office carried out:

- structural design of the quay wall and other minor elements throughout the dock
- design and specification of the dock gate bearings and seals
- draughting and reinforced concrete detailing.

Site staff were drawn from the Glasgow office and Arup Energy in London.

Investment in the redevelopment totalled £7.5M, some £3M of this coming from the European Regional Development Fund and Ross & Cromarty Enterprise, with the remainder shared between Brown & Root and McDermott, the parent companies of BARMAC.

2. Nigg dry dock looking north before the start of the refurbishment works.

1. Location plan.

Background[1]

The graving dock at Nigg was developed in the early 1970s for constructing offshore platforms - initially steel jackets fabricated in the dock and floated to the field, and latterly self-floating structures. However, the advent of skidding jackets onto transportation barges made the dock's days of activity numbered, and the last job departed in 1987. The dock lay idle until the upgrading works started in 1996.

In 1971 Brown & Root and George Wimpey Co Ltd had joined forces to build and operate a construction facility for producing major offshore structures. Inside 20 months the dock and yard had been designed, built, and equipped. It measured 305m x 176m in plan, with 12m of water depth at Mean Sea Level. The side slopes at 1:1.5 were protected on the top surface with an asphalt layer, whilst a dewatering wellpoint system was installed within the slopes to aid stability during the dry dock state and de-watering of the dock. The dock gate - 125m long x 15m wide and 15m high - was of a caisson type, 52 cells formed from reinforced concrete. The gate acted as a gravity structure sitting on nine rows of bearings along the sill and bearing against a dock-side nib. Lip and caisson seals, along the base of the gate, ran vertically up the junction with the roundheads and were the first line of defence against water ingress. The roundheads either side of the gate were in mass concrete, incorporating 1.2m square flooding culverts controlled by electrically-operated penstocks, and 3m x 2.5m balancing culverts with simple inclined flap gates to ensure minimal differential in head between sea and dock sides.

The dock was (and still is) the largest in the world, with a similar statistic for the caisson dock gate.

The dock de-watering / gate move / float-out structure / gate back / flooding cycle, undertaken over the years to 1987, took toll of the slopes and the dock gate bearings. The slopes partially collapsed in places, requiring repair and stabilisation with gabion baskets. The sill bearings suffered shear failure and had to be replaced by divers during the dock de-watering / flooding cycle. The dock nib bearings totally failed and were latterly replaced with timber-bearing plates. The dock cycle was taking up to 10 days to achieve.

The essential upgrade

In 1991 Brown & Root's joint venture partner Wimpey pulled out of the Nigg operation, while soon after, 20km (as the crow flies) to the south, J Ray McDermott, the operators of the Ardersier fabrication yard, was experiencing their worst down-turn, forced to survive on a care-and-maintenance basis. Market demand was shifting away from the yards' traditional base of fixed platforms to floating and subsea solutions, whilst at the same time over-capacity in the UK's fabrication industry was being made worse by increased European competition. This signalled the need for rationalisation and in February 1995 Brown & Root and McDermott entered into a joint venture, BARMAC, to run the two yards' complementary facilities and skills.

The new market opportunities identified by BARMAC included the construction and outfitting of floating production storage and offloading vessels (FPSOs), floating storage units (FSUs), production jack-ups, tension leg platforms (TLPs), semi-submersible platforms, and concrete gravity structures such as the Wandoo installation in Australia. To compete in these markets the decision was made to upgrade the dry dock facilities at Nigg. There were three major upgrade requirements:

1. construction of a new quay wall against which the floating production units could be moored. This wall was to be capable of supporting lifts of up to 1000 tonnes by the yard's Lampson *Transilift* crawler crane.

2. re-shaping the dock plan area to accommodate the largest-known floating production unit. This meant lengthening the dock to accommodate 350m long vessels.

3. Dry dock flooded following completion of the new quay wall (left in picture) and slope works.

3. reducing the dock cycling time and associated operating costs. The target was to reduce the former from 10 to three days and to make a similar reduction in the operating costs. Several steps were to be taken to enable this to happen:

- improving the dock gate bearings and seals
- refurbishing the structure and mechanical outfitting of the dock entrance works
- re-grading and protecting the dock side slopes to maintain stability during flooded, dewatering, and dry dock conditions without a wellpoint system and with accelerated dewatering
- installation of permanent high capacity dewatering pumps to increase the rate of dock dewatering.

The quay wall

The new quay wall on the west side of the dock is 240m long, with a retained height of 15m and a design life of 20 years. The design selected for development was a reinforced concrete counterfort wall with counterforts at 5m centres and a base width of 16m. The basic statistics are 14 000m^3 of grade 40 concrete and 1650 tonnes of high yield reinforcement steel (40mm maximum diameter). It is designed to support the Lampson crane with a hook load of 1000 tonne at a maximum of 25m in front of the wall, producing track pressures of 920kN/m^2 immediately behind the wall. Additionally the design had to cater for rapid dock dewatering with the introduction of drainage layers within the backfill and weep holes through the wall.

The wall is founded on rock strata (weathered sandstone) for half of its length. The rock level drops to the south and the overlying sand, which was inadequate to support the loads, was replaced with a grade 25 mass concrete infilling.

Seven load cases were considered for the geotechnical design. The worst for sliding and overturning were those which considered the cranage sited at the virtual back of the wall, and the bearing pressure at the wall/formation interface was found to be at its most when the cranage was sited directly behind the front wall. The geotechnical design required the stiffness of the wall structure to spread the crane load over a greater length of wall than that mobilised by distribution of load within the soil. A length of at least 60m was required by the overall stability design.

4. NASTRAN FE analysis of the quay wall.

The structural design had to accommodate this geotechnical requirement by making the concrete elements torsionally adequate to transmit the loads over the required 60m. To assist in this the wall was built as continuous without any movements joints throughout its 240m length. A finite element analysis was performed using NASTRAN to assess the global effects and give forces within the front wall and base, and hand calculations supported by GSA grillage analyses and spreadsheets were then employed to optimise element sizing. The sizing was compared with Morrison's construction rates to identify the optimum ratio of concrete volume versus reinforcement density on the basis of buildability and costs.

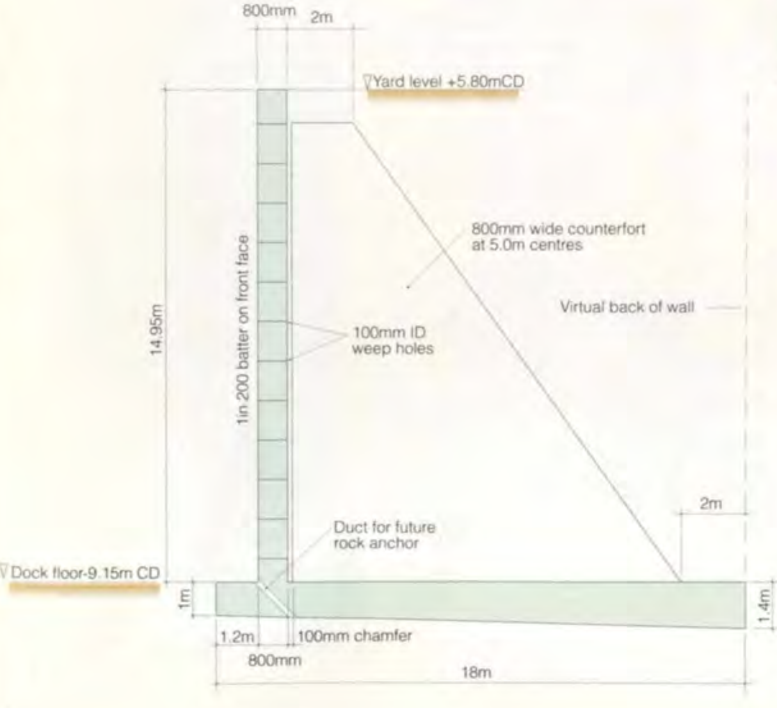
Morrison made an early decision to go for 'T-shaped' full height pours inclusive of 5m width of front wall plus the full counterfort. This required purpose-designed steel shutters to accommodate the 178m^3 pours over a minimum of nine hours. A production line approach with two full shutters was evolved to achieve the construction of the wall over a period of 20 weeks - an impressive achievement.

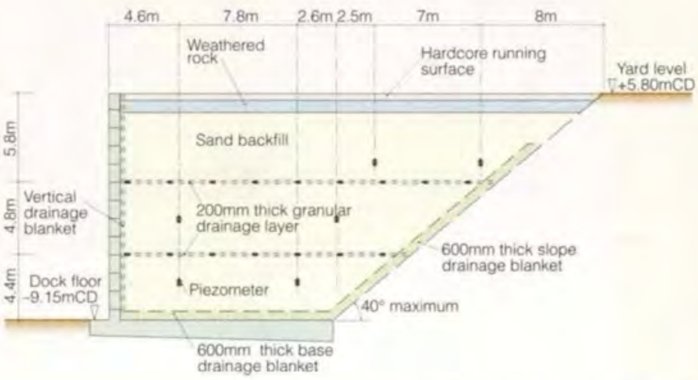
The 14.8m single vertical lift is believed to be the largest attempted ever by Arups; the wall is one of the highest retaining walls of its type ever built, and its size and load-carrying capacity are believed to make it the worlds strongest.

6. Counterfort reinforcement.

7. Back filling of the wall.

5. Typical section through quay wall.





8. Section through quay wall infilling and drainage makeup.

Survey and assessment of the dock gate

Bob Cather Graham Gedge

Continued satisfactory performance of the gate structure was vital to the dock's long-term future, so a thorough survey of its condition was required to assess the need for immediate repairs and future maintenance requirements. These needs arose from the harsh, marine atmosphere to which the gate had been continually exposed - for most of its 20-year life it had one surface almost entirely submerged in the sea. The landward face had only been fully immersed during the dock's intermittent flooding. The ballast cells would also have remained flooded throughout this time, so had effectively been permanently immersed below the level of ballasting. The landward side, the top of the gate, and the seaward side above high tide level had been exposed to airborne seawater spray and wave overtopping.

For overall programming reasons, survey and repair of as much of the gate as early as possible were vital, the areas accessible for survey being essentially those not submerged whilst the gate was in place:

- above the mean high water level on the seaward face
- top surfaces
- within the ballast cells above water level
- the entire surface of the landward face.

Helpfully, many of these areas are also those most prone to chloride-induced reinforcement corrosion; areas permanently immersed in seawater are not generally considered to be at risk.

Although chlorides will be present in submerged concrete, the diffusion rate of oxygen through saturated concrete is so slow that corrosion rates are negligibly small. In non-permanently immersed conditions, considerably higher chloride concentrations can build up and the drying periods allow oxygen access. An inspection procedure was developed to gain as much relevant information as possible in the short time available - some four days of site work. The size of the gate and time constraints dictated concentration on those areas where access was difficult and the risk of chloride-induced corrosion considered greatest. To optimise data collection it was decided to undertake the survey using rope access, or abseiling techniques (Fig D). CAN(UK) undertook this, supervised by an access engineer from Arups' Cardiff office, and assisted by the presence of a materials engineer from Arup R&D. The ability to put together such a team at short notice allowed a flexible and directed approach to data collection that could be readily modified in light of the survey findings as it progressed.

They concentrated their efforts on the landward face; a limited amount of inspection was also undertaken in the ballast cells and on the seaward face.

The data collection included:

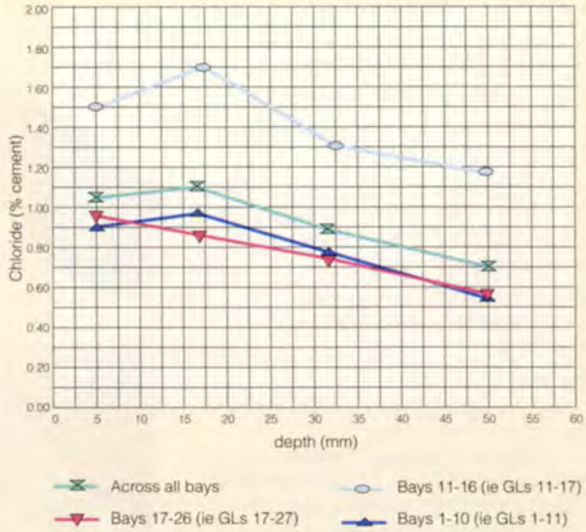
- a full visual survey
- a covermeter survey of the landward face
- drill samples at various depths on the landward face for subsequent chloride analysis
- cores taken above and below the water line for subsequent petrographic analysis.

The landward face was divided into 26 bays and chloride samples taken in the upper, central and lower parts of each bay. For the cover survey the bays were further divided into 1m squares which were scanned and the lowest cover in each square recorded.

The results of the visual survey (Fig A) were surprising, given the exposure condition and age of the structure. There was very little sign of cracking and spalling of the concrete cover attributable to corrosion. There were small areas where bars with low cover were exposed.

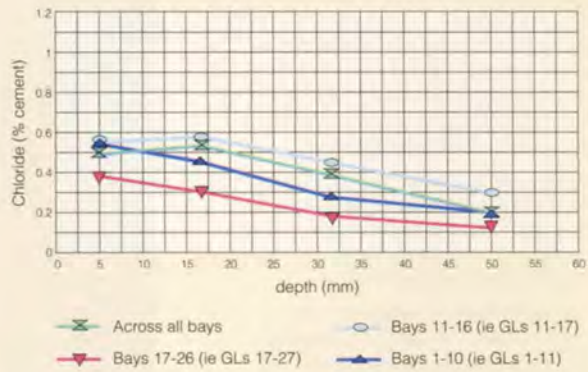
Towards the base of the wall a more pronounced and repeating pattern of spalled concrete and corroded bars was noted, their pattern, location and appearance indicating construction defects in the original slipforming rather than degradation with time.

The observations of generally limited corrosion and spalling were made more confusing as other survey results became available, particularly the chloride and cover surveys.



B. Chloride profile with depth - lower zone.

C. Chloride profile with depth - upper zone.



The initial conclusions from these were:

- that the cover was variable but not surprisingly so, with an average of 30-35mm
- that the chloride ion concentration at the level of the bars generally exceeded 0.2% by weight of cement at the reinforcement level
- that chloride levels increased from the upper to lower parts of the gate, in some places to more than 2% by weight of cement.

The source of the chlorides was considered to be the general marine environment, augmented by wave action washing over the gates during storms, rather than from original mix constituents.

For parts of the gate beneath the ballast cells' overflow pipes, enhanced levels of chloride were found (Figs B & C).

From other research and experience, these findings suggested that corrosion of the bars should have been occurring and causing distress to the structure. Certainly, had this been a highway structure on a UK motorway, cracking and spalling would be expected at these levels of cover and chloride. There was no immediate explanation for the apparently contradictory evidence from the visual survey and other tests. Petrographic analysis had shown nothing particularly special about the concrete - typical of the early 1970s and now regarded as rather too porous and susceptible to the adverse effects of chloride ingress in a marine environment.

In chloride-induced corrosion of concrete reinforcement several key stages affect the overall time to deterioration. The first relates to ingress of chloride ions and when it exceeds a critical threshold concentration at the steel surface for corrosion initiation. The second phase relates to rate of steel corrosion and generation of sufficient bursting forces from the expanded corrosion product to crack the concrete.

From the analysis results large parts of the gate surface should have passed the initial stages and it was thus necessary to investigate those factors affecting the rate of propagation of corrosion.

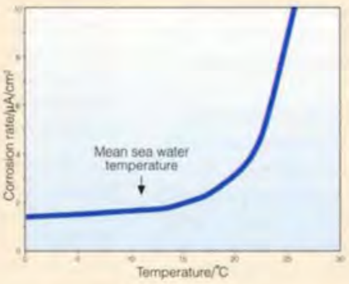
The key to any potential corrosion problem is to understand the service environment and how this might affect possible corrosion mechanisms.

A. Survey results.

D. Abseiling survey of the dock gate.

Small changes in the micro environments close to a metal surface often cause corrosion. However, the general, or macro environment, can also have a significant effect and this was so with the dock gate.

A study of the dock's geography revealed that the gate itself had its longitudinal centreline on an east-west gridline. This resulted in the landward face being due north and below the surrounding ground level. This orientation was discussed with the lighting group in Arup R&D, who concluded that the landward face would receive virtually no incident sunlight at any time of the year, and thus experience no temperature rises from solar gain. It is thus reasonable to assume that the reinforcement temperature would never exceed the local ambient temperature. As the reverse face of the gate is permanently in contact with the sea, this water temperature would strongly influence the temperature of the reinforcement. Data from the Maritime Group in London showed the mean summer water temperature in the Cromarty Firth to be +10°C. This is highly significant. Corrosion reaction rates, in common with all chemical reactions, are strongly influenced by temperature. The temperature: rate relationship is not linear, but tends to follow an exponential curve.



E. Corrosion rate of reinforcement -v- seawater temperature.

Published corrosion rate data for reinforcement in concrete show that rates below 15°C are reasonably constant with temperature and something like five times less than those at 20°C. It can be proposed that the critical temperature at which rates start to accelerate is around 15°C (Fig E). This data offered an explanation for the apparent contradiction between the visual survey, the cover / chloride analysis, and experience on other structures in more temperate conditions. The reinforcement was corroding, but, due to temperature control of the corrosion reactions, the rate was insufficient to have initiated cracking or spalling.

With corrosion rate data available and making assumptions about the mechanical properties of concrete, it is also possible to use mathematical models to predict time to cracking and spalling, though they should be used with caution, being relatively unproven on actual structures. Models can only predict the time to cracking once the critical chloride level has been exceeded (ie corrosion has been initiated).

Knowledge of when initiation would have occurred was not available for the gate so it was necessary to assume that this level had been exceeded for five years. Despite the limitations, the models are useful in providing a 'coarse' estimate of time to cracking and as such are useful in estimating maintenance needs over more than five years.

Using typical corrosion rate data (at 10°C) from the literature, it was possible to show that:

- Major cracking or spalling was unlikely at covers in excess of 30mm within five years.
- The risk of cracking increased at times in excess of 10 years.
- At covers of 15mm or less, cracking was likely within the next five years.

This assessment of time to cracking enabled a priority list for repair and maintenance of the gate to be prepared. This included areas of low cover (15mm or less) and damage to be repaired immediately, and also areas (in terms of cover) at risk in the medium and long term that would need to be dealt with as part of the gate maintenance programme.

A further programme of work on the currently submerged zones is proposed for when the gate is next floated out.

As for cost, this assessment represented a major saving for the client: a new gate was estimated to cost £4.5M, compared with £2.5M for alternative remedial measures based on cathodic protection.

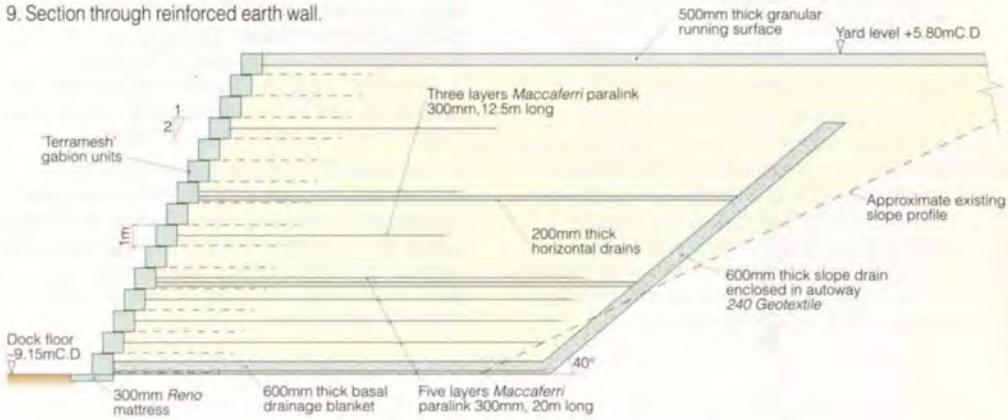
The remedial works in the present contract cost around £100 000, with additional comparable sums allocated at five-yearly intervals for the next 20 years.

▼ *Text continued from page 27*

Earth-reinforced walls

Between the south end of the new quay wall and the existing west roundhead, a 15m high reinforced-earth wall was designed by Maccaferri, checked and adopted by Arups. It is believed to be a first in the UK in terms of its height and ability to resist the dock dewatering cycle. It comprises a front face, at a slope of 2:1, of *Terramesh* rock-filled baskets and *Paralink* grid reinforcement strips located within the backfill. The latter is generally locally-won sand with rock drainage layers enclosed in geotextile to aid the flow of water out of the 'structure' during dewatering.

9. Section through reinforced earth wall.



10. Completed *Terramesh* and counterfort retaining walls.

Side slopes

The original 1:1.5 side slopes were regraded to 1:3, lined with geotextile and topped with varying grades of rock sizes and thickness. The slopes were designed to be stable during the dewatering cycle and also capable of resisting wave attack with the dock gate open. Below the wave attack zone, geotechnical considerations of slope stability were paramount, giving 500mm of small size rock protection, ie weight to hold the slope in place. Within the wave attack zone, a computational wave modelling study was carried out by HR Wallingford to give wave conditions within the dock. This data determined the size and thickness of rock armour to be adopted around the dock slopes to adequately resist forces from a 1 in 1 year return period storm.

The design of the new dock slopes was undertaken using the OASYS SLOPE program with the worst case being the dewatering condition. This was approximated by assuming a groundwater table parallel to the slope surface at the junction between the slope soil and the overlying rockfill ballast.

11. Typical section through dock side slope.

Wave conditions within the dock were computed from the following procedures:

- Wave conditions in the Moray Firth (from the north-west side of which the Cromarty Firth forms an inlet) were derived from the UK Meteorological Office (UKMO) European wave model.

  In addition, waves generated locally in the Moray Firth were computed using wind data recorded at RAF Kinloss. From these two sets of data, the wave energy reaching the mouth of the dock from the Moray Firth via the Cromarty Firth was computed. OUTRAY, a wave refraction model was used for the computation.
- Waves at the mouth of the dock generated locally in the Cromarty Firth were computed by the JONSEY wind / wave prediction model using wind data recorded at RAF Kinloss.
- Wave conditions at different locations within the dock were computed using the PORTRAY wave disturbance model to assess diffraction and reflection effects of the waves entering the dock.

Rock sizes and thickness of armour layers to cater for the waves were calculated using Van der Meer's equation.

Rock for the slopes was initially won from the Nigg Quarry some 500m from the site. However, availability of the required rock sizes and quality forced the need to import from another source 30km away.

The dock gate

The main items of work on the dock gate encompassed:

- refurbishment of all appurtenances, eg winches, fairleads, bollards, etc.
- upgrading as necessary all electrical control and instrumentation for the gate operation
- replacement of ladders, stairways and rubbing strips
- provision of new bearings and replacement seals
- inspection and repair of concrete on the gate.

The latter two items of work required the most input from an engineering point of view. Historically, there had been many failures of the existing sill bearing fixings due to a combination of excessive deformation of the bearing and corrosion. In addition, the existing north nib bearing pads had failed by crushing. However, although the bearings had by and large failed, the caisson and lip seals appeared to be functioning correctly.

13 Left.
Completed refurbishment works with the Lampson crane sitting on top of the new wall.

14 Right.
Modified entrance ramp with rock armoured side slopes. The Lampson can be seen lifting a redundant 800 tonne crane base from the dock in background.

The decision was taken to completely redesign the sill and north nib bearings, and to replace on a like-for-like basis the caisson and lip seals.

The main requirement for all the work on the sill was to minimise the labour needed to replace the elastomeric components, as it all had to be completed under water. This included devising new sill bearings that would not shear off during opening and closing of the gate yet would still have a minimum of fixings. The existing bearings had in total over 3000 fixings.

A composite bearing was selected, comprising a base layer of neoprene and a top layer of ultra-high molecular weight polyethylene (UHMW-PE). The theory was that the UHMW-PE layer interfacing with the underside of the dock gate would have a lower frictional resistance than the lower rubber layer to sill concrete interface, ie the gate would tend to slide on the top of the new bearing before inducing shear deformation through the bearing to the fixing.

By virtue of the composite nature of the bearing, almost all the vertical compression occurs within the lower rubber layer and the thickness of this layer was selected to give the same compressed thickness of bearing pad as the original pads. This allowed the same design of caisson and lip seals to be used. Portmere Rubber, the suppliers of the new bearings, made extensive tests of the design at Southampton University to confirm the theories.The sill bearings were each designed for a working load of 2900kN and an ultimate load of 7250kN.

The number of sill bearings was reduced from the original nine continuous strips to five rows of stiffer discrete pads measuring 1.4m x 300mm, and the number of fixings was reduced to 270. Structural analyses of the gate were completed to be sure that it would function satisfactorily with a greatly reduced bearing area.

The new north nib bearings were of similar composite design to the sill bearings and were attached to hinged steel frames, which allowed these bearings to be fixed in the dry and rotated into place by the divers after flooding. Although these bearings were discrete lengths, they formed a virtually continuous bearing along the length of the north nib. These bearings were each designed to a working load of 1660kN and an ultimate load of 4980kN each. The lip and caisson seals have the same geometry as the original seals. However, an alternative material was specified to allow resealing within 24 hours of load removal.

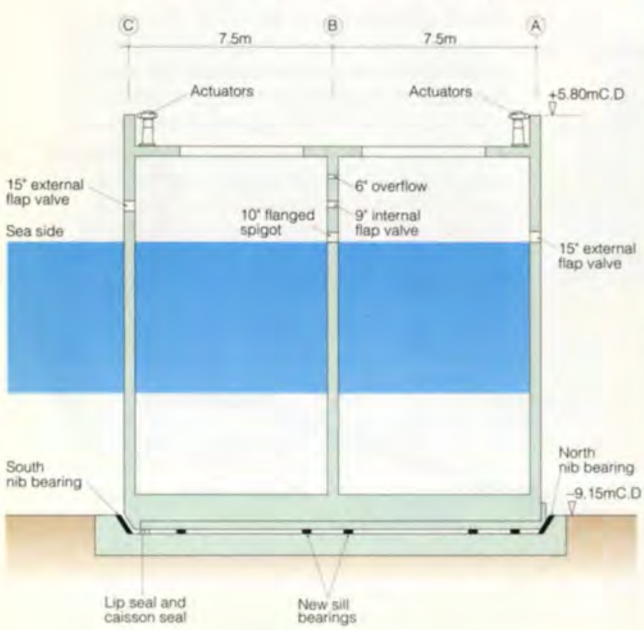
The repairs to the concrete were carried out by Morrison themselves, with the north face and top surface being treated whilst the gate was in operation. The south face above low water level, the ends, and the internal cell walls were repaired when the gate was afloat and moored against the completed quay wall.

Environmental issues

A past diesel spillage in the yard to the north of the dock was known to have polluted the subsoil material and water table; diesel seepage through the north slope of the dock was caught by petrol interceptors. The dock upgrading required regrading of this north slope as well as excavating a pocket to accommodate the largest floating production units.

It was anticipated that contaminated material would need to be separated out and removed to a treatment area. A diesel remediation area was constructed within the yard, comprising a lined pit and gravel drainage base layer upon which the contaminated material was spread. Remediation was achieved by leaving the material exposed to the atmosphere with wetting and rotivation employed to leach out the diesel, the leachate

12. Section through concrete caisson dock gate.



15.
Diving operations on the sill at the east roundhead following flooding of the dock and removal of the dock gate.

being then passed through the drainage layers into a full retention interceptor. A maximum of 7500m^3 of contaminated material is being treated by this method and is noted to be achieving the desired effects with some of the upper layers now clear of diesel. At the time of writing the remediation process was to continue for a further 12 months.

It was anticipated that the diesel 'pocket' to the north of the dock would be a continuing problem, with contaminated groundwater flow into the dock during dry dock conditions. Three by-pass type interceptors were installed on the drainage line along the toe of the dock slopes, so that clean water passes finally into the existing sumps at either side of the dock gate where 'jockey' pumps continually cut in to remove it into the Cromarty Firth. In addition, oil booms and skimmers will be used to catch any seepage within the dock whenever it is flooded.

Dolphins live in the immediate sea areas, and concern that some could get trapped in the dock at the start of the dewatering cycle prompted BARMAC to initiate a study by the Zoology Department of Aberdeen University to develop a procedure to protect them and ensure as far as practical that they are out of the dock before dewatering. This procedure will be adopted as part of the Dock Operations.

Mechanical and electrical services

Significant refurbishment and renewal of mechanical and electrical services were required around the dock to meet the demands of the new facility, not the least of which was the provision of two submersible pumps of 1.1m^3/sec capacity, capable of de-watering the dock in 64 hours.

These pumps, however, only provide half of the pumping capacity required to meet the required dewatering time of 32 hours, the remaining pumping capacity being provided by hired pumps. These utilise siphonic action to improve their efficiency, and require 1MVA of power supply.

New electrical sub-stations were provided on both the east and west sides of the dock. In addition electrical services were supplied to six outlets along the back of the new quay wall. Mechanical services, ie compressed air, firewater, potable water, oxygen, and propane gas, were provided to the quay wall outlets from existing services within the yard. All were designed to accommodate ground movements associated with the trafficking of the Lampson crane.

Dock/floodgate dewatering

On completion of the quay wall in early June 1997, the dock was flooded to permit removal of the dock gate. The gate was de-watered, floated out from the position between the roundheads, and moved by tug and winch to a berth alongside the new quay wall. At this location, concrete repairs to the internal cell walls of the gate were easily undertaken. Meanwhile divers removed the old bearings and seals, replacing them with completely new-designed bearings and replacement seals. These activities took six weeks and by mid-July the gate was moved back into the dock entrance and re-flooded to rest on the new bearings.

16.
Existing lip and caisson seals on the quoin of the west roundhead.

17.
Replacement north nib bearings on their support frames.

Dewatering the dock was hampered by technical problems with the electrical supply and the pumps. However the maximum dewatering rate was achieved for the latter half of the exercise, which was the most critical event for the design stability of the dock slopes and the quay wall. The rate of water drainage from the quay wall structure was monitored via ceramic piezometers buried at various levels within the backfill, which registered that the groundwater level immediately behind the quay wall followed the dock water level virtually exactly, ie no lag between water levels, proving the efficiency of the quay wall drainage layers. The slopes, apart from minor movement in localised areas, remained stable. The reinforced earth walls also functioned as designed and settled on average by 150mm.

The dock gate caisson and lip seals functioned well with only very minor leaks on both the vertical edges against the roundhead quoins. These leaks gradually closed up. The north nib bearings can be observed and they appear to be working as designed, satisfactorily taking up the slight variation in gap dimension between the gate and the nib.

Conclusions

The upgrading works were successfully completed in July 1997 and the dock handed back to BARMAC at a prestigious opening ceremony on 1 August in the presence of 220 guests.

This was a challenging project both from the design and the construction viewpoints, in particular the quay wall which was complex on both fronts but turned out to be straightforward. However the other aspects of the project were equally challenging.

Financially, Arups entered into a 'gainshare / painshare' risk-and-reward agreement with Morrison Construction, based on the final economic viability of the project. The 'Seamless Arups' philosophy was successfully demonstrated and the firm's considerable resources in the UK were utilised to the full. The alliance worked well and cost-effective solutions were brought forward through the encouragement of openness and teamwork in a number of aspects and although Arups opted not to become a full Alliance member, the firm embraced the working methodology successfully.

Reference

(1) WORTH, REJ. and TRENTER, NA. Graving Dock at Nigg Bay for Offshore Structures. Paper 7815, *Proceedings of the Institution of Civil Engineers,* Part 1, pp361-376, August 1975 .

Credits

Client:
Brown & Root McDermott Fabricators Ltd. (BARMAC)

Engineer:
Ove Arup & Partners
John Hamilton, John Mott (project management)
Robert Addlesee, Graeme Anderson, Tom Barr, David Colley, David Collier, Louise Dowling, Andy Duncan, Greg Haigh, Sai Lung Ho, Andy Horton, Alan MacLeay, Peter Meenan, Gary Milne, John Pyper, Bob Souter, Clon Ulrick (civil / structural)
Chris Martin, Rob Smith (FE analysis)
Hillary Cameron, David Clare, Allan Garrard, Heleni Pantlidou, Alan Richmond (geotechnics)
Justin Abbot (environmental)
David Cross, Andrew Grigsby, Chris Owen (mechanical / electrical)
Bob Cather, Graham Gedge (materials)
Steve Hold (abseiling)
John Mott (planning supervisor)
Craig McBain, Gordon McIntyre, Lynne Ritchie (drafting)
Steve Armstrong, Neil Gatenby (site team)
Linda Tweedie (project secretary)

Main contractor:
Morrison Construction Group plc

Principal sub-contractors:
Albion Drilling
Atlas Diving & Marine Services
G & A Barnie Electrical Contractor
Caithness Scaffolding Services
Cromarty Firth Diving Services Ltd
Glenfield & Kennedy Ltd.,
ESL (Inverness) Ltd
George Morrison Building Contractor
Maccaferri
Macdonald Scaffolding Services
Portmere Rubber Ltd
Rocklift Ltd
Seaboard Marine Services
Weir Pumps Ltd
Wimtec Environmental Ltd
Woodside Engineering.

Arup sub-consultants:
HR Wallingford Ltd
CAN (UK) Ltd

Illustrations:
1, 5, 8, 9, 11, 12, A-C, E: Emine Tolga/Peter Speleers
2, 3: DGS Films Ltd
4: Ove Arup & Partners
6, 13, 16: Alan MacLeay
7, 10, 15: Steve Armstrong
14: Allan Garrard
17: John Mott
D: Steve Hold

Smart technology in construction: Dream or reality?

Tony Sheehan

Introduction

In construction, materials and structures must generally provide:

- safety (during construction, in use, and during demolition)
- durability (typically lifetimes of 60-120 years are required)
- low cost (generally initial rather than 'life cycle' costs)
- low maintenance
- predictable properties.

The usual response to this challenge is to use tried and tested materials that are well documented in national and international codes of practice. Compliance with such codes often gives some comfort to the designer, in both acceptability of the design method and longevity of the final structure. Such structures are dependable, yet provide few ways (if any) to, for example, either detect loads applied to them or compensate for these loads.

'Smart' technology can, in theory at least, offer a whole range of new opportunities and approaches to construction. 'Smart behaviour' essentially describes the ability of a material or system to sense a change in its local environment, eg in temperature or light conditions, and respond to that change automatically, as by opening a window or changing from clear to opaque (Fig 1).

The challenge for smart technology in civil engineering and general construction is to provide sufficient 'added value' to justify adopting a smart approach. It is essential that the demands for durable, predictable behaviour of materials and structures are not compromised by this approach; rather that significant advantages can be identified, to justify what is almost always an increased initial cost.

The key applications for smart technology in construction can be summarised as follows:

- ***Sensors:*** *to increase a building element's 'intelligence' by providing real time feedback on performance (eg for bridge monitoring)*
- ***Actuators:*** *to provide a means of modifying structural behaviour (eg for vibration isolation)*
- ***Smart materials:*** *to create new materials capable of sensing changes in the localised environment and responding to them (eg intumescent paints for fire protection)*
- ***Smart systems:*** *to combine sensors and actuators through an active control system in order to both assess the action required and act accordingly (eg active noise suppression).*

These concepts can and have been applied to a range of complex civil engineering problems in many different ways. Examples include:

- ***Structural monitoring:*** *increasingly intelligent materials providing real time feedback on performance*
- ***Structural vibration control:*** *passive actuators and smart systems used for earthquake control*
- ***Acoustics:*** *the use of smart systems, for example involving active noise control*
- ***Smart environmental control:*** *through active and passive façades*
- ***New design approaches:*** *through use of smart materials and revised design philosophies.*

To best summarise the potential in these fields, it is worth considering each in turn.

Structural monitoring

The costs and consequences of periodic inspections to structures have stimulated the production of systems for remote monitoring. This is, in a sense, the lowest level of 'intelligent structure'; sensors or systems detect changes in key properties, but when and how to act remain firmly in the hands of the engineer.

In the UK, a *corrosion monitoring system* based on linear polarisation resistance and half cell potential has been used for remote monitoring of concrete repairs to Wolvercote Viaduct on the A34 Oxford ring road. Information from site is fed directly back to the monitoring engineer's office, where the effectiveness of the repairs can be assessed.

Optical fibre sensors have been the subject of much research in recent years, and now monitor factors as diverse as quality of construction, strain, corrosion, temperature, and so on. A concept called 'optical domain time reflectometry' has been applied, in which the time delay between sending out a light pulse and receiving a reflection can successfully predict where along a fibre any change (eg cracking) occurs.

Major developments in optical fibre communications technology allow vast quantities of data to be sent across extended distances with negligible signal loss. Optical fibres can thus offer the potential for measurement along extended lengths (through use of optical fibre sensors) and remote monitoring (through use of optical fibres as communication devices). Optical fibre sensors monitor bridge and other civils structures worldwide for:

- *quality of construction, eg assessment of grouting of prestressing ducts in concrete construction*[1]
- *corrosion monitoring*[2]
- *structural monitoring of structures*[3,4].

The ability to access such information remotely has been effective on the Winooski One hydroelectric dam in the USA, where a network of optical fibre sensors were cast into the structure[5]. This can be monitored directly from office computers or indeed on the Internet, either method making considerable cost savings in inspection. By combining such communications technology with, for example, a network of embedded sensors to track the rate of carbonation or chloride ion ingress into a concrete structure, an increasingly 'intelligent' structure can be obtained.

TRIP steels also offer much potential in providing structural performance data. As these materials are strained, they undergo a progressive, irreversible phase change from a non-magnetic, face-centred cubic to a ferromagnetic body-centred cubic phase. (*TRIP steels* are so called from this Transformation-Induced Plasticity.) Direct measurement of the state of magnetism of *TRIP steels* allow peak strains experienced by the material (and any structure it is attached to) to be assessed over time or following an extreme load condition like high wind or earthquake. This technology can be applied to peak strain measurement of large structures such as bridges (as on several in Georgia, USA[6]), and has also been exploited for structures subject to earthquakes. Localised measurement of key elements (eg peak strain sensing of rock bolts in tunnels) has also been considered.

Much has been written on how sensor materials like optical fibres and *TRIP steels* can monitor existing structures. Often, however, clients are reluctant to incorporate such systems either on new structures (since they should be designed and built properly in the first place) or on existing structures unless clear signs of degradation are present. The use of new structural materials like polymer composites, however, offers considerably more potential. Much of the use of materials in construction depends on long-established rules and codes of practice that are only readily available for traditional construction materials like steel, concrete, brick, etc. For materials which do not have codes of practice, eg polymer composites, combining them with sensors to assess their condition allows them to be used in construction with increased confidence[7]. In this way, barriers to innovation in their use may be reduced.

This has been done on several prestressed pedestrian bridges in Germany, notably the Notsch Bridge, Schiessbergstrasse Bridge, and Marienfelde Bridge (see Fig 2 above right), where real time feedback on the performance of non-metallic prestressing has been made possible by using sensors.

1. Smart glazing: clear to opaque.

2.
Use of optical fibres to monitor strain in new materials.

Structural vibration control

For vibration control of civils structures, it is clear that considerable levels of load and frequency control are required. The initial design approach to such problems will almost always be to build more robust structures, capable of resisting nearly all conceivable load cases. In practice, however, the creation of improved design processes, the need for more cost-effective structures, and the demand for 'architecturally acceptable' slender elements have led to the use of less material and the creation of more flexible structures. Reductions of the potential impact of severe wind, earthquake or (in the case of offshore structures) wave loads may be essential for continued structural integrity in such structures.

There is a clear need to establish either improvements to existing design methods or new methods for coping with structural vibrations. This being so, there is considerable interest in the concept of a 'smart structure' that can adjust its stiffness to minimise the effects of extreme conditions (Fig 3).

For all tall buildings, there is a need to reduce the level of *wind-excited vibration* which could result in discomfort of occupants, disturbance to equipment or, at worst, structural damage and even collapse. The external shape and size of a building are the key factors determining performance in wind; dynamic properties are usually considered unimportant (with the exception of very slender structures).

Devices able to limit the effects of wind-induced structural vibrations include tuned mass dampers and tuned liquid dampers. Both are simple, passive devices which modify structural behaviour to single frequency cyclical loading either through modification of building mass or through reliance on liquid motion in a container to absorb and dissipate vibration energy.

In *earthquakes*, however, a building's dynamic properties (fundamental period, damping and mass) are vital, and any methods which can influence such properties will be of considerable interest in earthquake-affected regions. The challenge of efficient earthquake design is that buildings are almost always one-off structures with different properties in each case. To further complicate the situation, there is often considerable uncertainty in predicting the amplitude, direction, and frequency of earthquake motion at a particular site and in predicting structural response to these.

Generally, structures are designed to prevent collapse during the worst credible event and to limit structural damage in other cases. A range of standard seismic design solutions are available worldwide to cope with such situations. Tuned mass dampers are limited in use to a narrow range of frequencies. Base isolation is also used; here the building is essentially constructed on bearings of low lateral stiffness which combine with some form of energy dissipation in an attempt to reduce the deflections between building and substructure to manageable levels. The Osaka International Conference Centre will utilise special passive hysteretic dampers (unbonded braces) to convert vibrational energy to other forms, (see Fig 4: on the next page) and a range of passive dampers - viscous, viscoelastic, friction and hysteretic - with similar objectives are widely available.

Additionally, a considerable research effort has addressed the issue of 'smart' or 'active' earthquake-resistant buildings, with particular interest in the US and Japan. Essentially, an active control system involves monitoring ground motions and building response and then by computer control altering the building's stiffness to reduce the earthquake-induced motions. The system's effectiveness depends critically on its ability to both measure and compensate for the experienced movement (allowing for any time delay that may be present). Clearly, if an active system were to become 'out of phase' with a real earthquake, the very real fear is that it may make the situation worse, not better.

Nonetheless, four systems of active building control are available:

- ***Active Tendons,*** which alter the tension of cross-bracing between floor levels to resist any imposed acceleration or deformation
- ***Active Mass Dampers,*** *which involve movement of a heavy mass (generally at roof level), again controlled by actuators linked via computer to a network of sensors on the structure*
- ***Active Variable Stiffness,*** *where a series of extra stiffening elements (braces) may be either linked into or remain free from the main structural frame, varying stiffness accordingly*
- ***Active Airjet Control,*** *where a compressed air source can create jets of air in an attempt to counteract earthquake motions.*

Buildings actually constructed using these techniques are currently few. An active mass driver system was installed by Kajima at the top of the Kyobashi Seiwa building, an 11-storey structural steel office tower in Tokyo. The building is extremely slender, and justified using active control to reduce vibrations and give occupant comfort during wind and relatively minor earthquakes. It experienced two major earthquakes in its early life, performed well, and illustrated good correlation between predicted and actual performance[8].

Kajima were also involved in constructing a three-storey building incorporating active variable damping (KaTRI no.21) in 1990. The system links sensors detecting seismic ground motions to a controller which can then adjust the stiffness of the structure to minimise vibration response. The variable stiffness machinery requires only 20W of electricity per device; it can thus work successfully with a small stand-by generator in the event of power failure, a considerable advantage over high powered systems.

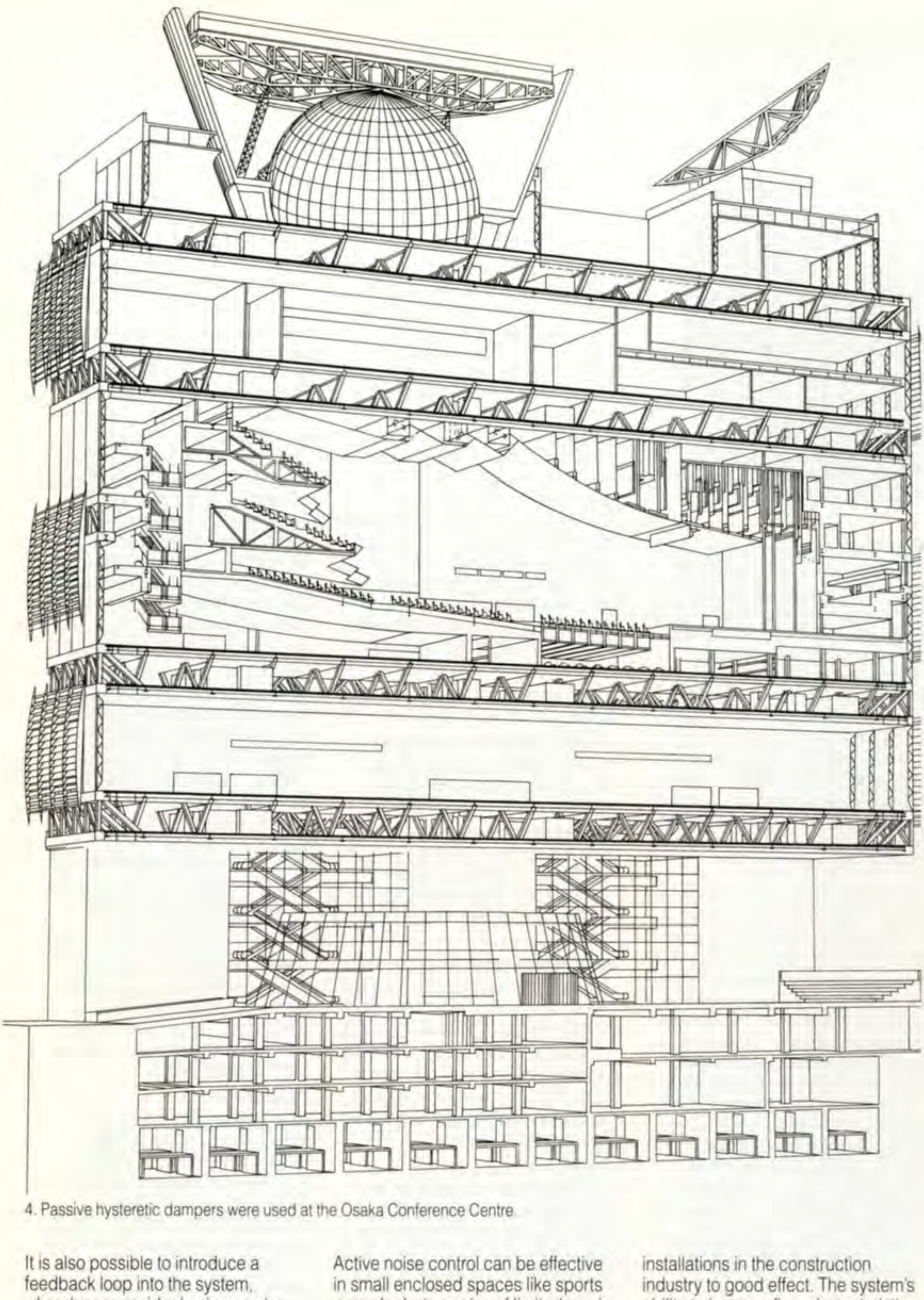
There is a great need to upgrade earthquake resistance of existing structures in earthquake regions. Retrofitting creates even more of a challenge as existing structures have not been designed for active control. Work by S J Shelley *et al*[9] considered the application of an active mass damper to a steel truss highway bridge in Columbus Ohio . Clearly older structures will exhibit time-dependent characteristics, and degradation of their components must be both predicted and accommodated in any active control system. Nevertheless, this work illustrated reasonably consistent performance and reliability of a retrofit system in a rapidly varying environment; its potential could be extended in the future.

Acoustics

Traditional sound insulation is achieved through use of heavy constructions that have sufficient mass to control building vibrations, and hence sound transmission, to prescribed levels. Doubling the mass of a wall increases sound insulation by only 5dB so a requirement for high sound insulation can mean very heavy constructions. An electronic method for increasing sound insulation or providing noise control is thus highly desirable.

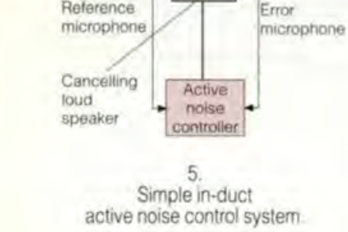
A patent for reducing noise through the creation of equal and opposite 'antinoise' was issued as far back as 1936 to P Leug in the US. The application of the concept was, however, restricted by the limited response time of electronic circuitry. This situation had improved dramatically by the early 1980s. The concept is simple enough; the wave-form of noise in an enclosed space is monitored by a controller which generates an equal and opposite signal which is fed through a loudspeaker to create destructive interference with the original signal.

3.
Tacoma Narrows Bridge - The need for smart structures?



4. Passive hysteretic dampers were used at the Osaka Conference Centre.

It is also possible to introduce a feedback loop into the system, whereby any residual noise can be identified, the loudspeaker signal modified, and the effectiveness of the system improved (Fig 5).



5. Simple in-duct active noise control system.

Active noise control can be effective in small enclosed spaces like sports car cabs but remains of limited use in construction. If the environment containing the noise varies, the anti-noise signal may not remain the equal and opposite of the original signal and amplification of the noise can result. For effective cancellation, the loudspeaker has to be close to the original source of noise. The effective range of frequencies is also limited, and the durability of loudspeakers in what may be quite aggressive construction environments must be carefully assessed.

Nevertheless, active noise suppression is extremely effective at low frequencies (eg transformers and diesel generators) whose sound is characterised by long, well-defined wavelengths which are easier to cancel. The concept has been applied to mechanical ventilation installations in the construction industry to good effect. The system's ability to improve flow characteristics and reduce energy losses may, indeed, create a net energy saving, providing a unique advantage which may justify use.

In the future, there is potential for providing active noise control in walls, aiming to reduce transmission of sound waves by vibrating the wall with an appropriate anti-vibration signal. Concepts like smart panels and indeed smart wallpaper (incorporating an array of piezoelectric actuators) have been developed and tried out in a small way with some success. The challenge of matching such a system to the range of noises (eg road traffic), angles of incidence of noise to a wall surface, and indeed the complexity of control signal prediction, is considerable, but the concept is gradually becoming a realistic prospect.

Smart environmental control

In an environmental control context, an 'intelligent' building is one designed to operate in an energy-efficient manner by balancing external environmental influences with the internal environmental requirements. In many cases, this may result as much from a 'smart' design approach as from any materials used.

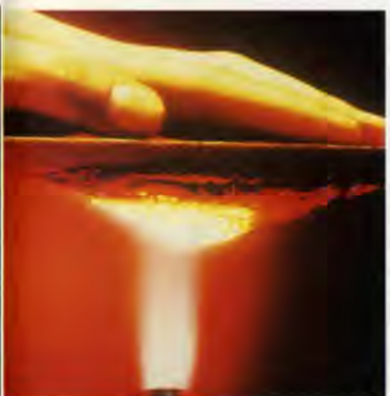
Active façades can often be created to control the movement of air, radiation and humidity from reasonably simple materials which act as a simple 'buffer' between internal and external environments. More recently, interest has also moved towards more adaptive façades made up of materials with more specific molecular behaviour.

Options offering considerable potential for more adaptive façades include:

- ***Angular glazing,*** *which can be created to allow light to pass through in only certain carefully selected directions. Glare control can be carried out fairly simply by lamination of solar control films.*
- ***'Super-windows'*** *comprising multiple layers of glass and/or plastic which may be film-coated or filled with low conductivity gases like argon. Such systems can be such good insulators that they can gather more energy than they lose over a 24-hour period in winter. Ventilated double skins comprise two sheets of glass separated enough to allow the air between to rise up and (potentially) allow some control over solar gain.*
- ***'Air curtains'*** *as at RMC's headquarters at Egham, where one is used to separate a humid swimming pool from a reception area. The effect is created by air travelling from a supply grill in the ceiling to an extract in the floor.*
- ***Photochromic glass*** *which becomes increasingly opaque when exposed to strong sunlight. The size of photochromic panes has been somewhat limited, and there is also concern about the glass responding to incident light rather than atmospheric conditions, which may not be the building user's true 'need'.*
- ***Automated blinds:*** *photovoltaic cells (to simply soak up the sun's energy for use in the building) have been around for some time. Recently, they have been used to good effect by Arup Façade Engineering at the University of Northumberland. The simple use of photovoltaic cells to generate a more active façade has been successfully applied at Kaisertechnik's Business Development Centre in Duisburg, where cells have been combined with a system of blinds such that excessive sunlight can be used to power blinds to reduce solar gain.*

- ***Electrochromic glass*** *consists of electrochromic layers laminated with float glass that can be turned from clear to opaque by an applied electric field at the flick of a switch. Once the change has taken place, the potential does not have to be maintained, and both size of pane and external durability of these materials are finally reaching realistic levels. This approach offers perhaps most potential since most office workers prefer to control their own environment rather than have it fully automated.*

The high initial cost of some of these can inhibit their use, but they may give real benefits, not only in terms of health, comfort and resultant productivity, but more specifically when considering operational costs. The potential savings in air-conditioning costs for active glazing, for example, have been estimated as up to \$10/ft^2 of glazing[10]. Advertising has also been suggested as a way to justify increased costs; several London stations now use simple display screens for advertising, although this falls some way short of a truly smart façade. Banks of LCD screens or projected images may show more potential. Such financial incentives may help justify increased use of these systems in the future.



6. Intumescent paint; smart response to fire.

New design approaches

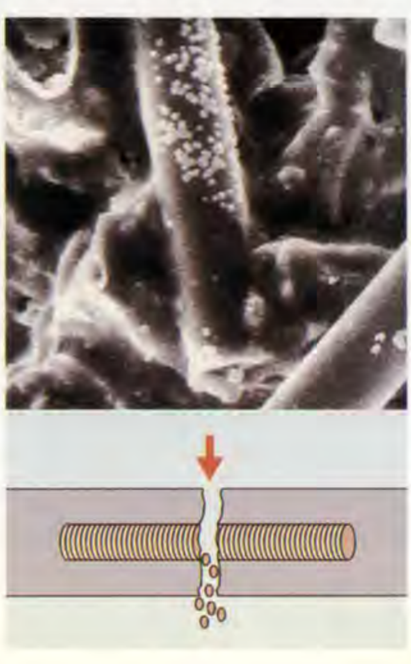
Smart design inevitably involves a multidisciplinary approach, and appropriate experts in the individual areas described here are available throughout Arups worldwide. In almost all cases, the smartest design approach will involve a simple rather than a complex solution in the first instance.

The truly smart materials of the future will be able both to sense changes in their environment and respond to those changes without need for further input. Some ideas with most potential are far more straightforward than many of the concepts already described. Intumescent materials provide an example of a reasonably smart material already widely used in construction. Here, decorative paint coatings to structural steelwork are able in the event of a fire to both sense the fire and, at an appropriate temperature, foam up to form an insulative char to separate the structural element from the heat (Fig 6).

7.
Hydrogels: smart response to water.

Products of similar potential include *hydrogels*, (Fig 7) materials that swell up when activated by either moisture or some other species. These are now available commercially as water-stops, where they can be used in applications like swimming pools as a 'smart seal' between concrete elements, preventing water seepage through gaps formed during construction. Further applications will undoubtedly follow. One currently in development at the University of Strathclyde[1] incorporates hydrogel in sensors to detect adequate filling of grouted post-tensioning ducts in concrete structures.

Looking further to the future, materials which *self-heal* in damaged structures could see increased application. These would be able to sense damage and effect self-repair automatically - a 'biological' behaviour, parallel to skin healing after a cut or graze. The wider field of 'biomimetics' will help us to learn many engineering lessons from biological behaviour in the near future. In the short term, however, self-healing is far nearer than one may think; Arup R&D has strong links with researchers at the University of Illinois who have developed a range of hollow fibres which can be filled with resin or even corrosion inhibitors, then cast within other matrices (eg of concrete or polymer). Significant cracking of the matrix leads to cracking of the fibre, release of the resin, and some form of repair carried out (Fig 8). Such concepts are at an early stage, but are attracting research funding from the National Science Foundation in the USA to assess their real potential.



8:
Self-healing materials:
hollow fibres filled
with active chemicals.

Conclusions

Overall, smart materials and systems will only find widespread application in future construction markets if they:

- are sufficiently durable (civils structures often require 60-100 year life with little or no maintenance)
- provide real technical advantages over tried and tested alternatives (perhaps solving problems that are not currently solvable)
- are cost-effective.

The apparent goal of the totally automated façade or civils structure may, in reality, have some drawbacks. For many years, photochromic glazing (where glass darkens automatically in response to incident radiation) was thought to be the ideal for controlling light/heat ingress in office buildings. In fact, recent research has concentrated more on electrochromic glazing (where the glass is controlled by an electrical switch) as office workers prefer to control their own environment. Similarly, fear of 'making the situation worse' leads to passive rather than active earthquake protection solutions being adopted. Future 'smarter' solutions will only find real use in civils applications if they respond to real needs rather than technology for technology's sake.

Nevertheless, a range of complex demonstrator projects and real life projects have shown considerable potential for smart civil engineering structures. For newer materials, the use of advanced monitoring techniques offers an excellent opportunity to take advantage of their unique properties without being inhibited by codes of practice; real time feedback on performance can permit safe use. It is clear that the need for safe, durable structures will remain essential; the expansion of knowledge on smart solutions is likely to help us to achieve such objectives in the future.

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Arup contacts

Smart materials/Systems/General:
Tony Sheehan

Smart acoustics:
Raf Orlowski

Smart earthquake systems:
Ted Piepenbrock

Smart façades:
Andrew Hall

Intelligent buildings:
Bill Southwood, Martyn Harrold

Illustrations:
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