Wentworth: The new tennis pavilion

Arup Associates Group 2

Following their successful submission in an invited competition involving six international architectural practices, in March 1989 Arup Associates were asked to develop design proposals for the new clubhouse and sporting facilities at Wentworth Golf Club in Surrey. The proposals focussed around a master plan for the site which established locations for a new 7400m^2 clubhouse, screened parking and improved facilities for the tennis club within the framework of an overall landscape strategy. The clubhouse was designed on three levels to provide restaurant facilities, a health and fitness spa, together with meeting rooms, club offices, lockers and a shop. The restaurant and lounges were planned to overlook the existing green and also to relate to the fine woodlands on the site, whilst the overall form of the building was kept low in order to respect the established natural setting.

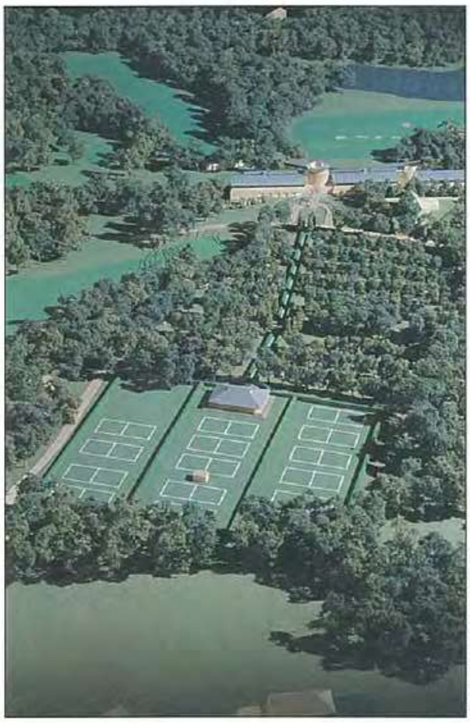


Photo: Arup Associates

The new pavilion, which was completed for the 1990 season, is the focus of the tennis club at Wentworth. Sited alongside a group of new and refurbished tennis courts, it provides a range of facilities including a lounge and bar with locker rooms, tennis professionals' office and a small shop for sporting equipment. The single-storey building has been planned so as to overlook the five new courts and the nine others which already existed. An outdoor terrace, partly sheltered by the overhanging roof of the pavilion, provides both players and spectators with good views of the playing areas. Framed in Douglas fir with double beams of laminated redwood supporting a pitched roof of slate, the scheme attempts to develop and extend the tradition of timber pavilions housing recreational and sporting facilities in park-like settings. Arup Associates were also responsible for the interior design and the scheme creates a series of spaces which are finished in white and timber. The steel structure is expressed against a ceiling of Douglas fir, whilst the floor is finished in Beech.

Wentworth Tennis Pavilion recently won the New Built Award in the Runnymede Annual Design Awards.

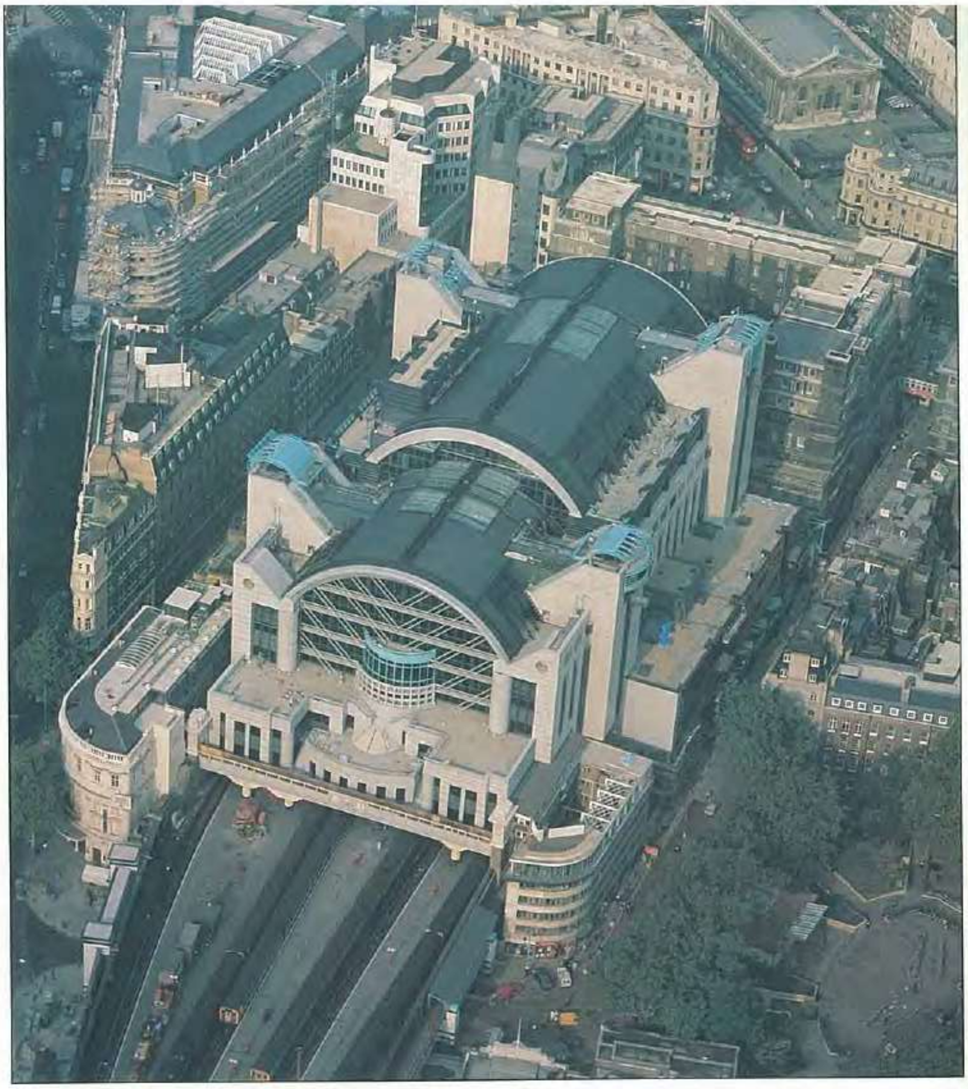
Photo: Crispin Boyle

Photo: Jonathan Moore

EMBANKMENT PLACE

Architect: Terry Farrell & Co. Ltd.

Malcolm Barrie
John Crack
Mark Facer
Graham Phillips



Introduction

In the autumn of 1985 we were asked by the developer Greycoat to help Terry Farrell formulate a proposal for building an office block in the air space over the platforms of Charing Cross railway station in central London.

1. Site location plan. 2. Aerial view of the finished building.

The British Rail Property Board had given Greycoat one year to produce a feasible scheme and conclude a development agreement with them. Our initial brief was simply to provide the necessary engineering back-up to achieve this. A year seemed a generous period of time at first. However, not only did we have to conceive a technical solution, but also to convince the various departments at British Rail that they could accommodate the proposals. These included the BR engineers responsible for the existing structure, the station management who looked after the smooth running of the station, and the traffic planning staff responsible for the timetables. In parallel, the architect had to gain planning permission for a major office development (where none had previously existed), on a site of major prominence on the Thames skyline abutted on three sides by a conservation area. The challenge had been set.

The real constraints began to emerge. There were simple items, such as the fact that Charing Cross handles 120 000 passengers a day through its six platforms — nearly three times the passenger density of any other London main line station. This explained the initial reluctance of the station management to allow any intrusion, columns included, through the existing platform environment.

Where could we site the lift shafts to service 40 000m^2 of office space on a 100m × 50m floor plate? Ideally they needed to be central and thus had to pass through the centre of the platforms. History came to our rescue. Examination of old records showed a ramped cab road which rose from the old stables in the brick vaults below to serve the passengers at platform level. It had long since been decked over to provide space at platform level for ancillary buildings to house station staff. Here was an opportunity, a place where a hole could be carved out of the existing brick vaults without upsetting the integrity of the remaining structure, an existing obstruction at platform level which we might remove and replace with one of our own lift shafts.

The idea for the lift shafts through the cab road solved the problem in the office above but posed a new design challenge. They were 70m from Villiers Street and the front door. How could the architect design an entrance hall 10m wide and 70m long with the row of columns necessary to support the superstructure down the middle?

We discovered that parts of the original station structure had been founded on timber piles which had rotted causing considerable differential settlement. This had been stabilized with raking minipiles which protruded underneath our site area, further reducing our scope for foundation solutions beneath the site.

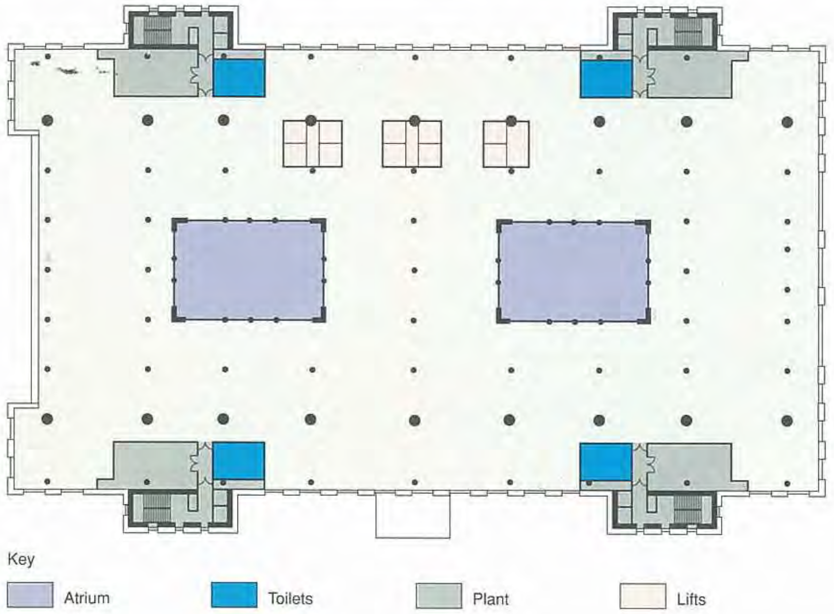
Did the drawings of a major Underground siding for the Northern Line under the station show just a scheme or had it been built? There were certainly abandoned tunnels beneath our site but how extensive were they? Where could we put the foundations and columns?

The architect delved more deeply into the history of the site. Whilst the Players' Theatre had operated under the arches only since 1946, it could trace its pedigree back to 1858 when Gatti's Music Hall started in the old Hungerford Market which preceded the railway station on the site. Clearly the theatre had to be retained. How could we get an auditorium to work in a 10m wide brick vault if we put a row of structural columns down the middle, as commonsense dictated we should?

Columns — the questions always seemed to come back to the columns.

Thus the structural design of the building began to emerge, guided by the inherent constraints of the site. The building was to be supported on as small a number of columns as possible, placed to minimize disruption to the station. The ideal solution from a planning point of view was for only two rows of columns 36m apart, their location being determined by the site constraints. This overcame the difficulties in the vaults by providing a huge area free of vertical obstructions and therefore suitable space for the theatre and entrance hall. The preferred scheme for the superstructure was a series of huge steel tied arches built at roof level with the office building suspended below. Plantrooms were located within the vaults wherever possible and four service risers like giant buttresses were positioned outside the building, bypassing the station. The scheme, which combined both civil and building engineering concepts, fitted in comfortably and lent distinctive form to the architecture of the building. The object of the exercise was achieved and the various agreements fell into place.

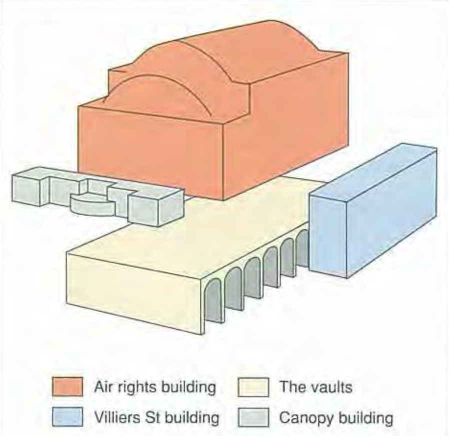
Now we had actually to design it in detail so that someone could build it. Not only should it be possible to build, but in an overheated construction market it was necessary to make the concept easily understandable so as not to frighten off subcontractors or attract price premiums. It was also necessary to build at a speed comparable to the other large developments under construction in London at that time which were potential competition for tenants. The stage was set for the next challenge — the detail design.



3. Typical floor plan.

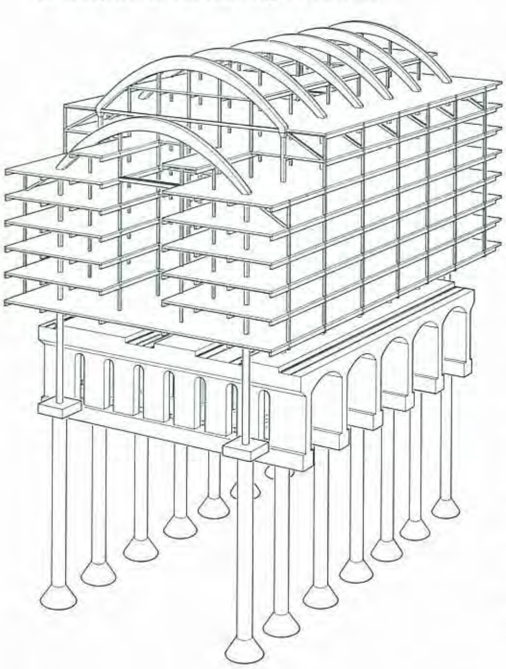
Detail design

The highly visible building in the air rights over the railway platforms houses nearly 40 000m^2 of office space. There is also nearly 20 000m^2 of additional accommodation located within the old brick arches which houses plant space, the theatre, an impressive entrance hall, car parking, service facilities and a number of retail units.



4. Exploded schematic of the four distinct parts of the building.

5. Diagram of the building's structure.



The engineering for this hidden part of the development, whilst less spectacular than the office building above the platforms, was just as complex in its requirements for custom-crafted details to solve the myriad of engineering problems which are a part of construction in, under, and around old buildings.

The development of the design led to the building being considered in four distinct parts:

The air rights building: the main bulk of office space located above the platforms suspended from the tied arch at roof level.

The canopy building: so called because it started off its design development as an acoustic canopy on the front elevation and grew into a two-storey extension to the main office building.

The Villiers Street building: a low-rise, conventional building constructed on the space between the station and Villiers Street and conceived architecturally to appear as three low-rise buildings sympathetic to the Villiers Street-scape and also to mask the enormous bulk of the air rights building behind; internally it is part of the main office space.

Lastly, the vaults, which encompass all the development below the tracks.

Air rights building

This forms the major area of office accommodation and is the structure most obviously influenced by the site constraints. A variety of different structural solutions were investigated and costed, varying from traditional four- and six-column schemes to large-span two-column schemes. As already noted, the ideal was for only two rows of columns per vault to pass through the platforms on either side of the station, so it was decided to pursue this option, with column-spacings of 36m and 10.5m or 12m respectively across the station and along its length. Such an arrangement required a large transfer structure which, under normal circumstances, would be positioned at the lowest level above the station and the building constructed on top. However, this would have required a storey-height truss, creating a floor unsuitable for high specification office letting. Since there was a height limit on the building, this would have amounted to a lost floor, so other methods of bridging the space were investigated.

The possibilities examined and costed included full frame vierendeels, portal frames, A-frames, and mast and guy solutions. The favoured solution of hanging the building beneath a series of steel tied arches provided an efficient structure and maximized the usable office space, whilst maintaining the rights of light to adjacent properties, and creating the distinctive barrel-vaulted appearance. Whilst simple in concept, the scheme was inevitably complex in detail, both in design and construction terms.

The arches spring from the eighth floor, some 40m above street level, and span 36m with a rise of 10m. They consist of steel box sections 2m deep by 1m wide at the crown, and 1m square at the springing points. The flanges of the boxes are 55mm thick, whilst the webs are 30mm. The springing points are joined together with a post-tensioned concrete tie beam. Between the springers and the associated columns are large bridge bearings, allowing sliding and rotation at one end, and rotation only at the other. Intermediate columns hang from the arches at 6m centres.

The arches

6. A full-scale test was carried out on two of the arches prior to delivery to site.

7. Detail at top of the main column, showing the arch springer, the ends of the tie beam prestressing bars, and the diagonal bars.

8. One of the arches being erected.

9. Placing of the arch 'key-stone'.

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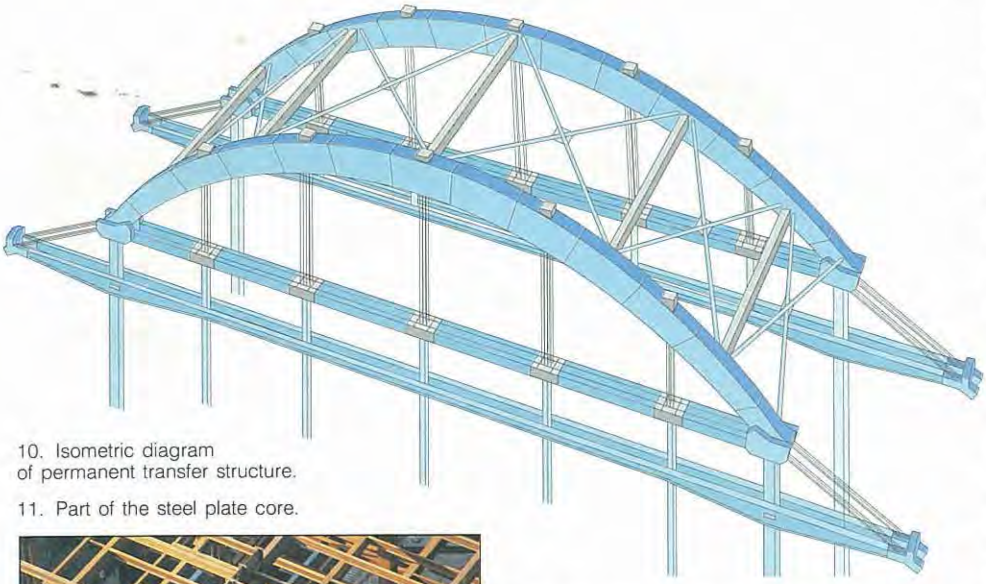


10. Isometric diagram of permanent transfer structure.

11. Part of the steel plate core.

To maximize use of the site footprint, the building cantilevers 8.5m to each side of the main columns. It was decided to support this area using a triangulated system of struts and ties at high level. Unbalanced loads between one side of the building and the other are counterbalanced by means of 13.5m long beams that cantilever beyond the main columns in towards the centre of the building, and react against the dead load of the floors supported by the arches.

The transfer structure produced very high column loads, well beyond the capacity of British rolled sections. The works contractor was given a choice of using a welded box section or rolled sections produced by Arbed of Luxembourg. He chose the latter. These have 125mm thick flanges and 80mm webs and have compact dimensions. Below the first level above the station, the column sections become 1m diameter reinforced concrete members possessing sufficient stiffness to extend unrestrained 18m down to foundation level.

Lateral restraint to the building is provided by two steel braced cores and a large steel plate core which houses the main lifts. Alternative proposals for a steel braced core were costed but the steel plate option proved the most economic for providing the required stiffness.

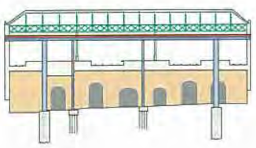
Several options for floor beams were investigated. Tapered steel plate girders were chosen for the primary beams in place of normal rolled sections. Savings in weight more than made up for increased fabrication costs and they were adopted for use in the building. For a 12m span, they are typically 750mm deep at mid-span with a 7mm web and 15mm flanges.

Noise and vibration from trains and platforms beneath the building was considerable and special isolation measures were necessary. The front elevation was double-glazed using up to 19mm thick glass separated by a 12mm air gap; the first slab over the tracks was concrete plus insulation, with a further concrete slab on top. Despite being separated from the station structure, the main columns were subject to some vibration transmitted via the sub-soil. Analysis of the arch proved it to be an efficient filter to the forcing frequencies for all areas suspended from it which was a large part of the floor area. The vibration problem was restricted to areas attached to the main columns. Although the problem was considered to be borderline, anti-vibration bearings were introduced where practical to minimize the risk of unacceptable vibrations.

In order to build the structure in a conventional sequence it was necessary to have additional temporary columns through the station. These were of a very limited capacity. The bare steel frame was erected, together with its metal decking, to roof level and then the arch transfer structure was built. Once the arch was in place flatjacks at the top of the temporary columns were released and the arches passively picked up the load of the superstructure. The deflected shape of the building was controlled by jacking points at the top of the hangers and stressing of the post-tensioned arch tie beam. The concrete floors were then cast in a sequence that was coordinated with additional stages of stressing of the arch tie beam (Figs. 12-17).

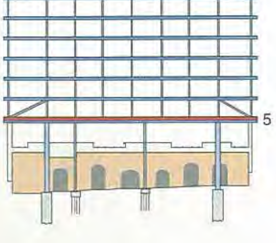
The large column loads (up to 2700 tonnes), necessitated a novel approach to the foundation design. The ground conditions, typical of a large part of central London, consist of fill, flood plain gravel, and London Clay followed by the Woolwich and Reading Beds. Large diameter

ERECTION SEQUENCE

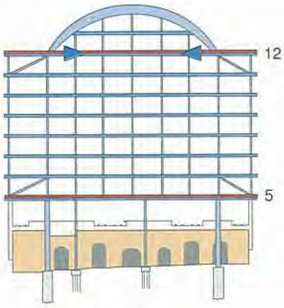


12. First level (5) above station is erected beneath existing station roof during weekend track possessions. After concreting, no further BR restrictions apply.

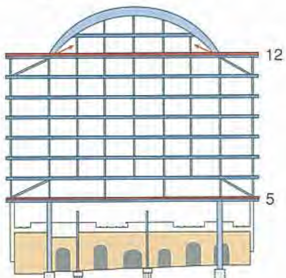
13. The station roof is removed and the temporary transfer structure is erected.



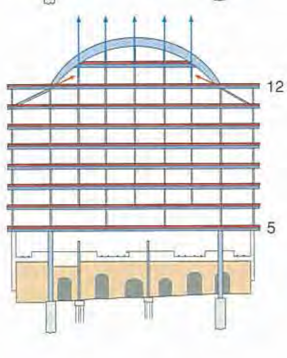
14. The frame is erected and decked up to level 12. Level 12 only is concreted to form an erection platform for the arches.



15. The arches are erected and the prestressed concrete ties to the top transfer structures (arch and strut/tie system) are stressed.



16. Flat jacks at the top of the temporary columns below level 5 are lowered to transfer load from the bottom to the top transfer structure. Temporary columns between level 5 and 6 are removed as the load within them goes from compression to tension by simply tapping out bolts at the end connections. The temporary diagonals are similarly removed by stressing the top permanent diagonals.



17. The frame sag is removed by stressing the top verticals and diagonals. The loads within the prestressed concrete ties are topped-up prior to concreting the remaining floors. The sag resulting from concreting is removed as before and then the prestressed concrete tie load is topped up for a final time prior to cladding, fit-out and live load being added.

18. View of the brick vaults beneath the station during construction.

19. One of the building's permanent columns where it passes through the vault roof.

20. The Player's Theatre nearing completion.

bored piles would have been the normal solution but the height restrictions within the existing station vaults precluded the use of a boring rig. It was decided to use hand-dug piles extending up to 35m below ground level. The piles were up to 2.5m diameter with an under-ream of up to 6.6m diameter. The capacity afforded by the under-ream enabled the shaft diameter to be kept to a minimum, and enabled the depth of excavation to be kept well above the water-bearing Woolwich and Reading Beds, thus reducing the risk of the under-ream 'blowing' during excavation as a result of sub-artesian pressures. These piles were described in the Summer 1988 edition of *The Arup Journal*.[1]

The vaults areas

The 125-year-old brick vaults upon which Charing Cross railway station is built hide extensive and complicated structural works. Two storeys of reinforced concrete flat slab and column construction were shoe-horned into each vault to provide extensive plantroom areas, the theatre, a wine bar, car parking, retail units and the main entrance to the building. These structures are totally separate from the existing brick vaults, both to fulfil legal requirements that the station may be demolished at any time, and also to provide vibration isolation for the new structures.

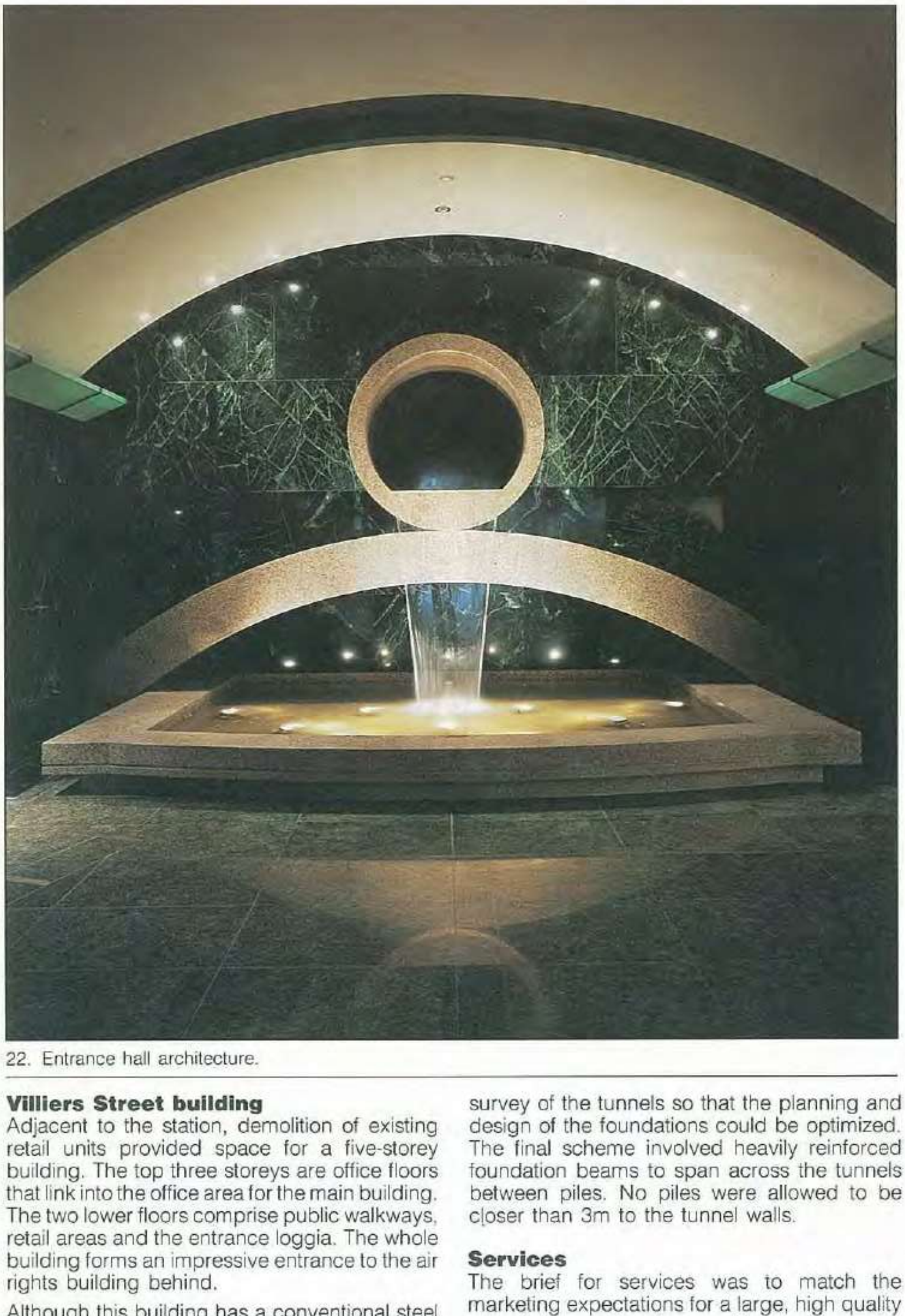
The replacement of the cab road with the main lift/stability core for the Air Rights Building was a major undertaking in its own right. It was designed and detailed to maintain stability of the existing station structure both during construction and when complete.

Within the vault areas, excavation of considerable depths of fill material was required to enable construction to proceed. Very little factual information on the nature or depth of the foundations for the existing station vaults was available. It had not been possible, due to the timing of the project, to carry out trial pit inspections in the time between vacating of the vaults by tenants and start on site. Based on estimates of the loads on the foundations and old drawings, assumptions regarding their depth and the nature of the ground were made. As excavation proceeded, the existing foundations began to settle excessively. The reason for this was a very high existing bearing pressure (by present-day standards) coupled with unexpectedly weak ground, which resulted in an unusually high reliance on the surrounding overburden pressure for the foundations' bearing capacity and performance. Further consideration showed that the problem only existed temporarily. In the final condition adequate stability of the existing footings would be preserved. Nevertheless, temporary settlement control was effected using a system of inclined mini-piles through each affected foundation.

The canopy

The front of the building adjacent to the River Thames has a stepped appearance and this is formed by a separate three-storey canopy building. Braced frames or shear walls could not be used to provide lateral stability, and initial sway frame designs proved too flexible. Adequate stiffness was achieved using a steel and reinforced concrete composite portal frame within the vault level and columns cantilevering up to the first level above the tracks. The columns are broken just below this level with an array of rubber pads that provide vibration isolation for the office areas above.

21. Top office level showing vaulted ceiling.



22. Entrance hall architecture.

Villiers Street building

Adjacent to the station, demolition of existing retail units provided space for a five-storey building. The top three storeys are office floors that link into the office area for the main building. The two lower floors comprise public walkways, retail areas and the entrance loggia. The whole building forms an impressive entrance to the air rights building behind.

Although this building has a conventional steel frame structure the positioning of foundations for the Villiers Street building required careful planning because the running tunnels for the Northern Line pass diagonally across the building footprint. London Regional Transport were commissioned to carry out an accurate survey of the tunnels so that the planning and design of the foundations could be optimized. The final scheme involved heavily reinforced foundation beams to span across the tunnels between piles. No piles were allowed to be closer than 3m to the tunnel walls.

Services

The brief for services was to match the marketing expectations for a large, high quality speculative shell and core office and a smaller area of retail accommodation.

The offices are air-conditioned and plant is sized to cope with the heat generated by large dealer spaces, as well as normal office equipment. Air-conditioning allows the facades to be sealed against noise, particularly from the adjacent railway. The retail areas are provided with service connections for future tenants.

All the floor area of the air rights building was potentially valuable lettable space. It was therefore decided to locate as much plant as possible in the less commercially attractive vault areas. Riser positions are limited because the office areas are separated from potential plant space in the vaults by the station platforms. Air-handling units are therefore positioned at each level, adjacent to the office areas they serve. They are connected to central plant in the vaults by service cores which are positioned outside the plan of the station platforms, at the four corners of the building.

The office air-conditioning system is a conventional, ceiling-mounted, variable air volume system with perimeter heating by either radiant panels, built into the facades, or reheaters in the areas where this was not practical. Typically each floor has four separate air-handling pods ranging in size from 16m^3/sec on the larger floors to 5m^3/sec on the smaller floors. These allow each floor space to be easily divided into four tenants' areas.

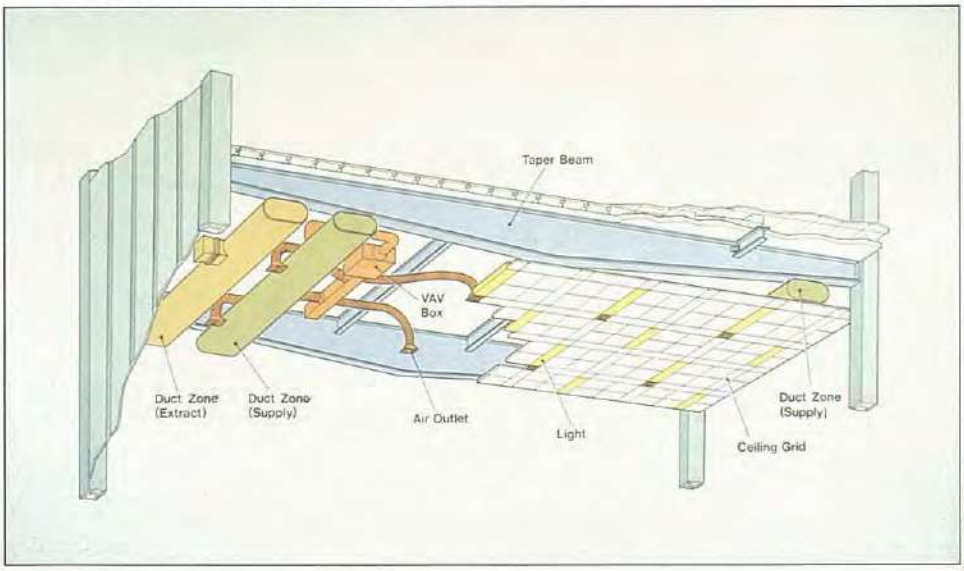
Central plant, such as boilers, chillers, storage tanks, substations and standby generators, are positioned in the upper and lower level vaults under the station. From these areas services are routed horizontally at high and low level in service corridors to the vertical risers positioned outside the station boundaries on the edge of the building. Each of these service cores incorporates toilets, service risers, a fireman's lift, an escape staircase and an air-handling pod. The position of the air-handling pods adjacent to the outside perimeter wall made it relatively easy to provide full fresh air capacity. Cooling towers and spaces for future tenants' plant are positioned at the top of the cores. Incorporated into the facade, on each side of the service core buttress, are the intake and discharge louvres which are neatly disguised by architectural metalwork.

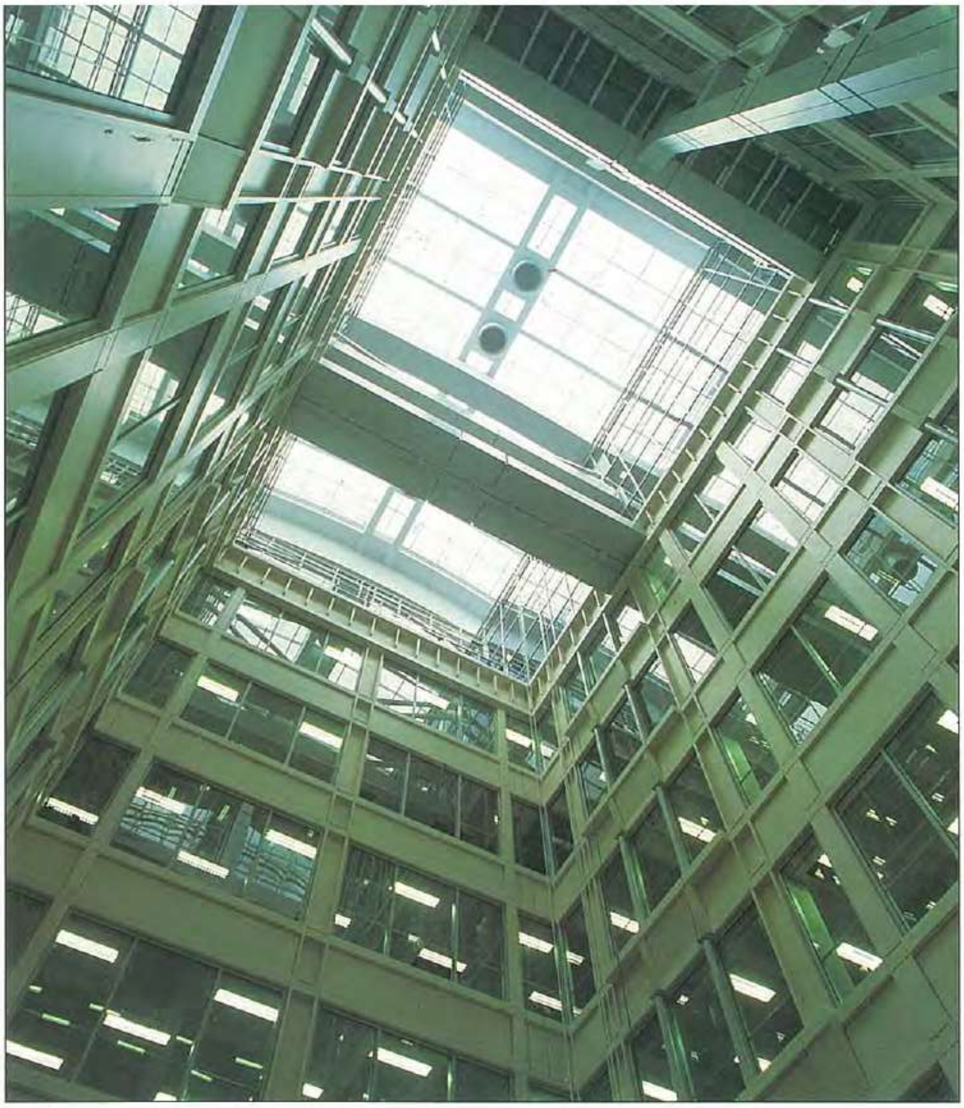
The services design incorporates numerous features to provide future flexibility. A raised floor provides a floor void of 150mm clear on every floor except the two lowest where 300mm clearance is provided for potential dealer use. The heating and chilled water circuits incorporate multiple pump control and two port control valves to provide cost-effective capacity control. Each air-handling unit can handle 100% fresh air for energy-saving purposes and each can be separately controlled to suit tenants' requirements. The building is fitted with a building management control system.

Electrical power for the development is provided via a 22/6.6kV primary substation established by London Electricity. From here cables are routed to the high voltage switchroom and

23. Tapered beams at the edge of the atrium.

24. How the structure and services are co-ordinated within the floors.





25. Inside a completed atrium.

then through the vault areas to six substations, two supplying landlord's equipment and four for the office floor tenants, each located near the base of a service core. Electrical distribution for the offices is via rising bus-bars, located in the service cores, with tap off facilities directly feeding a sub-switchboard at each level.

A landlord's standby generator of 750kVA capacity is positioned in the vaults where there is also space for future tenants' generators. The generator engine is cooled through a heat exchanger linked to the roof-mounted cooling towers. As far as noise was concerned, attenuation was provided on the generator room air inlets and outlets and on the exhaust, to comply with the specified limits of 65 dBA at 1m.

The exhaust flue rises through the building to discharge at roof level.

On the security aspect, a central console is located in the building management suite, which incorporates the main fire alarm panel, fire fighting telecommunications, lift intercommunication, CCTV monitors, intruder detection indication and smoke extract fan override switches. The CCTV cameras are located around the ground level perimeter together with intruder detection facilities on all accessible external doors.

Designated routes, for communications and information services, are included throughout the vault areas and service cores.

Fire safety

Because the building is large and incorporates two central atria, it requires special fire safety features. It is sprinklered throughout with extra cover around the perimeter and atria to avoid fire spread. Escapes are designed to be capable of evacuating all the occupants in 2½ minutes and a comprehensive addressable fire alarm system is installed. The atria are fitted with smoke extract fans at roof level to avoid the possibility of fire spreading via an atrium. Because they are enclosed, make-up air is supplied mechanically into their base in the event of fire. This is provided through a bypass duct connected to the normal air-handling units.

On most floors the normal air-handling units also limit the build-up of heat and smoke by operating in the event of fire in lieu of opening windows as required conventionally.

Conclusion

Despite the complexity of Embankment Place, the project has been brought to a successful conclusion. Practical completion for the shell and core works was achieved in October 1990.

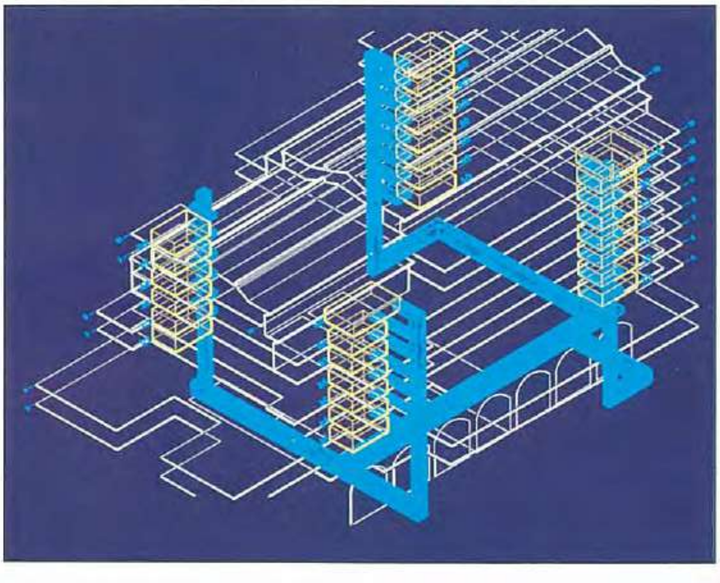
Credits

Client:
Greycoat Group plc
Architect:
Terry Farrell & Co. Ltd.
Structural & services engineers:
Ove Arup & Partners
Acoustic consultant:
Arup Acoustics
Quantity surveyor:
Gardiner & Theobald
Services quantity surveyor:
Mott Green & Wall
Construction consultant:
Schal International Ltd.
Management contractor:
Laing Management Contracting Ltd.
Theatre consultant:
Sandy Brown Associates
Project:
Final cost: £80m approximately
Start on site: July 1987
Practical completion: October 1990
Gross area: 60 000m²
Photos:
Project logo, 2, 21, 22, 25: Courtesy Greycoat
6: Northern Photographic Services
7, 8, 9, 23: Ove Arup & Partners
11, 19, 27: Peter Mackinven
18: John Mitchell
20: Gerry Loader

Reference

(1) GROSE, B. Hand-dug piles in London. *The Arup Journal, 23* (2), pp.9-10, Summer 1988.

26. How services are routed from vaults to floor-by-floor pods.



27. Charing Cross station during rush hour.

John Lewis, Kingston

Architect: Ahrends Burton & Koralek

Peter Evans

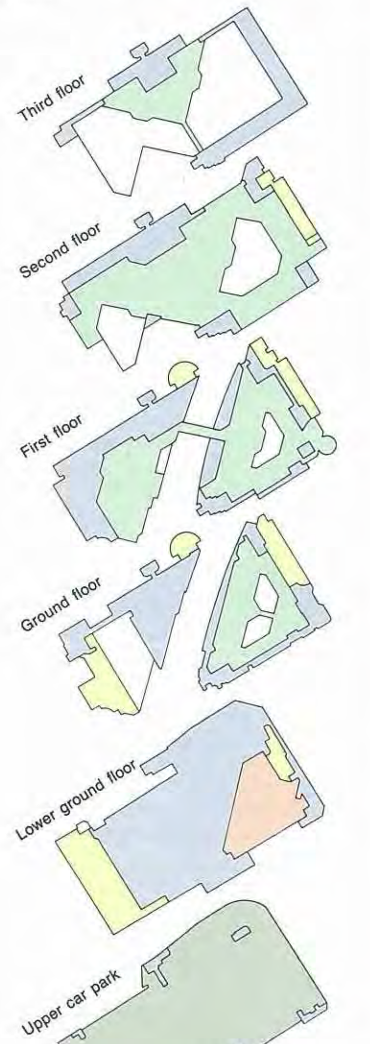
Introduction

Over the past 10 to 20 years there has been a considerable amount of retail development, mainly in the form of shopping malls, refurbishments, or out-of-town developments. There have, however, been relatively few purpose-designed department stores, such as this new John Lewis, which opened recently, after a 25-year search for a suitable site to the south of London.

The John Lewis Partnership were particularly concerned about good roads and especially customer access. An early possibility was Kingston upon Thames, but because of its traffic problems other sites were examined.

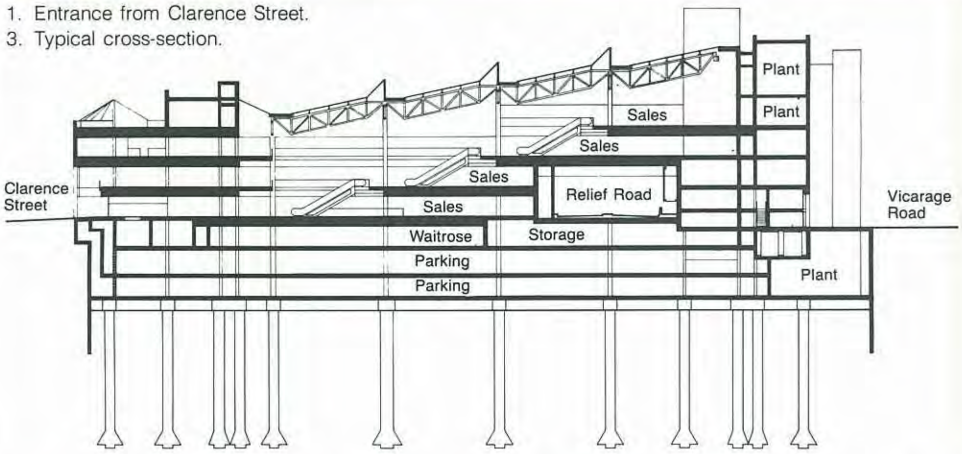
1. Entrance from Clarence Street.

2. Levels.



John Lewis selling
John Lewis non-selling
Lettable units
Car parking
Waitrose selling

3. Typical cross-section.



However, in 1979 they returned to Kingston, which had been developing plans for road improvements and pedestrian schemes to restore its reputation and position as a premier shopping centre for the south and west of London.

Lengthy public enquiries then followed because of the sensitivity of the site, adjacent to the River Thames by Kingston Bridge and with the Kingston Town Centre Inner Relief Road planned to pass through the development. There were also complications over the funding for the road and it was not until April 1986, when the Royal Borough of Kingston took over the GLC's highway responsibilities, that a Building Agreement was signed and planning consent obtained. The Building Agreement contained various terms and conditions that had implications for the engineering design, particularly with regard to the Relief Road and the archaeological remains on the site.

ABK started work on the project more than 10 years ago, when Arups were given a multidisciplinary commission. However, as the John Lewis Partnership built up their own in-house services design team, this commission changed to the provision of civil, structural and public health engineering services.

The development

In addition to being a John Lewis department store, the building also contains a Waitrose supermarket and the area by the riverside has been set aside for public amenities, a river walkway, restaurants and a public house.

The site slopes, but where the ground is highest, the building has three levels of substructure 13m deep, the lowest two as parking for 700 cars, and the topmost designated as lower ground floor for storage, a loading bay, the Waitrose, and riverside amenity areas. Above the basements are four sales floors which have large openings, tiered back and linked by escalators.

A special feature of this store is the open glazed roof that provides daylight to the sales areas. There are a further two perimeter levels of plant space above them.

The site investigation showed a general succession of about 1.5m to 2.5m of fill, overlying 3m to 5m of gravel above the London Clay. The groundwater is about 2m below ground level, the level of the Thames at the riverside.

The foundations are provided by about 300 large diameter bored piles of diameter 1.2m and 1.5m with underreams. Some less heavily loaded columns are founded on smaller piles, 600mm and 900mm straight shafted. Pile depths range from 20m to 30m below ground level in the London Clay.

The basement retaining walls are 800mm thick reinforced concrete formed by the diaphragm wall method under bentonite. The toe is embedded 6m into the London Clay below the undercroft that provides a heave zone under the lower car park. The basement structure is reinforced concrete with the top surface of the car park slabs laid to falls with a brushed finish.

The Relief Road passes diagonally across the site at ground level, falling from Kingston Bridge to Wood Street, forming, in effect, a suspended tunnel through the building. The Building Agreement required that the road be designed as though it were any regular highway bridge, following the usual Department of Transport procedures and standards. A dispensation was however received to use the Ove Arup & Partners specifications for those structural elements that also comprised the road and its supporting structure, rather than have to use DTp specifications which would have complicated the work. Nevertheless these elements had to be checked against the usual codes of practice for building regulations and the bridge codes, which led to a number of conflicts concerning requirements for fire resistance of a basement structure and long-term requirements for durability, particularly regarding concrete grades and cover to reinforcement. This followed from the Building Agreement condition that the road could remain independently in the (unlikely) event that the surrounding store be demolished. While it would be possible to add bracing and other such support, the basic design and materials were more fundamental and had to be appropriate for this eventuality.

The Building Agreement also stipulated that the road tunnel be lit, using two independent power sources, and that the levels of light be controlled by a sensor, dependent on the brightness outside.

The superstructure is also reinforced concrete, well suited to follow the complex geometry and detailed profiles required for the brickwork cladding. The floors are generally coffered slabs supported on concrete columns. The stair, lift and service cores do not have stiff concrete shear walls and the stability of the building is therefore achieved by frame action. The absence of shear walls not only has a construction benefit, but the complications that would have followed from having stiff elements in a building with no expansion joints were also avoided. The building is about 155m × 85m but the provision of an expansion joint (or joints) was effectively precluded by the roof being two adjacent squares, set over the road which then divides the store across opposite corners. The structure was checked to confirm that it would accommodate thermal effects.

With the loading bay at lower ground floor level and the road passing across the site at ground floor level, large span transfer structures were needed at first and second floor levels. In the loading bay, the headroom requirements permitted storey-height beams to span the 24m involved, but the Building Agreement applied across the road, where headroom restrictions were more limiting of beam depths, some 2.5m for spans of between 22m and 25m.

4. Sales floor.

Noise and vibration

Tony Stevens

An important part of the design brief was to evaluate the significance of the transmission of traffic noise and vibration into the building. It is protected against airborne noise generated by traffic on the Relief Road by the appropriate design, materials and mass of its fabric.

But the level of vibration that might be detected inside is also important. Criteria at Kingston were determined by reference to measurements taken at other John Lewis stores from which acceptable conditions were established. While the levels of vibration perceptible to people are well-known, it was necessary to determine how sensitive display stands might be affected, for example glassware or mirror-lined boxes on glass shelves. Information about traffic vibration was obtained from published research papers and by taking readings on similar bridge structures.

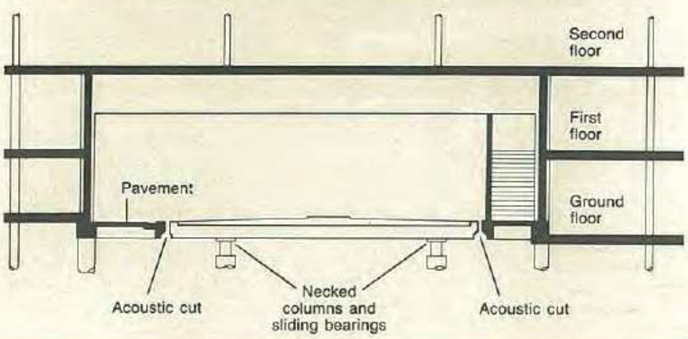
A computer model was devised to simulate vibration behaviour in the structural framework of the road and the floors. The computer results showed that the excitations from the passage of a range of vehicle types on the model structure would give vibrations in the floors, particularly on the sales floors at road level, that would not meet acceptable standards.

The possibility of mounting the road slab on rubber isolating bearings was considered but, apart from the structural complexities and the amount of warehouse headroom below that would be lost, bearings would not have dealt with the full range of frequencies involved. But the computer simulation run with the road slab itself separated from adjacent sales floors at ground floor level only, showed that standards of vibration would be met, albeit with a narrow margin. The attenuation is achieved by the increase in length of the path for the vibration through the structure that a separation joint either side of the road affords.

Other measures taken to lessen the effects of traffic were to make the road slab and supporting columns as massive as possible. In order to allow for lateral temperature movements, the tops of the supporting columns in the middle section of the road are 'necked' to allow for an element of bending. At the ends of the road, steel sliding bearings allow for greater movement. This movement has had to be accommodated in the design of the materials across the separation joint, which differs from a conventional bridge type expansion joint by virtue of its shearing action. Therefore to make the building weather-tight the joint is built up using a number of materials to give a barrier with back-up systems, while at the same time preserving the acoustic cut.

Both user perception and vibration measurement confirm that predictions of the computer simulation were valid. Neither staff nor customers are conscious of the passage of road vehicles even on sales floors adjacent to, and at the same level as, the Relief Road.

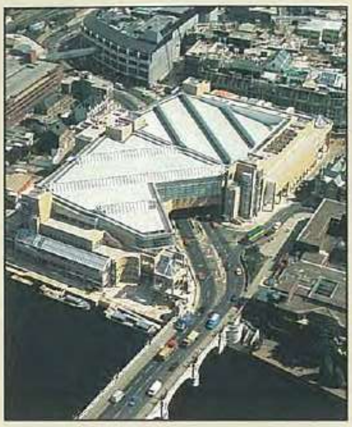
A. Relief road and below (B) a typical cross-section.



Roof drainage system

Don Barron

The design development of the rainwater collection and removal system at main roof level established that the glazed roof structure should be free from any outlets and pipework unless these were located in suitable positions around the perimeter of the building.



C. Aerial view of the roof.

An aspect of the glazed roof design was a series of access ways, also acting as gutters to intercept rainwater, located at regular intervals diagonally across the roof, laid flat, and sized to have sufficient spare capacity to accommodate and provide retention for a 1 in 150 years storm with intensities between 220mm/hour for two minutes and 100mm/hour for 15 minutes durations. At each end of a gutter, rainwater outlets are located in sumps to assist their hydraulic performance, with pipework dropping to high level before interconnecting with other outlets and then passing down the building.

Essential safety features incorporated include standing overflows adjacent to each outlet, overflow weirs through parapet walls at each end of the gutters which discharge into drained perimeter channels, and also large diameter common trunk overflow pipes from each low point on the roof. These trunk overflows run independently down the building and discharge at ground floor level. Moisture detectors, each located in the standing overflow pipes, also provide an alarm as part of the building management system in the event of a flood or surcharge condition.

At the north end of the road the beams are diagonal to it, involving a span of 33m with an allowable depth of 3.5m. The transfer beams are supported by concrete walls that run parallel with the road.

The perimeter of the glazed roof is bounded by a reinforced concrete box section smoke duct, which also runs down the centre of the building, dividing the roof into the two squares. Each half of the roof is set at 45° to the main building grid, sloping with the direction of the tiered floors.

The roof is made up from tubular steel sections with a system of primary trusses spanning 11.3m diagonally onto circular hollow section columns which vary in height from 8m to 12.5m. Secondary trusses, which span 17m onto the primary trusses, are at 5.6m centres and in turn support the purlins which carry the roof glazing. Incorporated into the design of the roof was an arrangement of channels, gutters and overflows to cope with extreme storm conditions.

Although there is some metal cladding, most elevations are in intricately detailed brickwork.

5. Interior.

6. View across River Thames.

Contract arrangements

With planning consent granted in April 1986, the Building Agreement required the Relief Road to be open for traffic in October 1988, when the rest of the Kingston Inner Relief Road was due for completion, and the store to be open for 1990 Christmas trading. Programme studies therefore indicated that an early start on site should be made in September or October 1986.

Since there was insufficient time to progress the design from its preliminary stages to full tender documentation, an initial foundations works contract, Contract 1, was organized, comprising site preparation, bored piling, diaphragm walling and river wall sheet piling. This was awarded to John Mowlem & Co. in September 1986.

While this was in progress, the documentation for the main building work, Contract 2, was prepared and tendered in February 1987. Because Contract 1 was proceeding well, with good prospects for an early completion, and because Contract 2 was tendered slightly later than planned, to avoid the site being idle, even for a short period, Contract 1 was extended to include the initial basement bulk excavation. While this work was in progress Contract 2 was awarded, also to Mowlems, who were able to continue on site with a flying start for the main building works.

Basement construction

The construction of a large basement in a town centre is a major operation and the design must address not only the support of the perimeter walls but also ground movements. At the design stage it was necessary to consider both the permanent and the temporary conditions during excavation and construction. There are now accepted standards by which to assess the effects of ground movements on buildings. Studies showed that with care in construction and with props provided at the levels of the floor slabs, the ground movements would be acceptable.

With an initial foundation contract the design of the piles and perimeter walls was committed early, placing certain constraints upon following operations. It was envisaged that the centre portion of the basement would be constructed in an open excavation, leaving perimeter embankments. An overall top-down method of basement construction was not obviously appropriate, although such possibilities were recognized, particularly for the perimeter works. However, the need to expedite the Contract 1 site start meant that no specific provisions were made for top-down construction, which could expend time and money with no guarantee of benefit in the following contract. The later extension of Contract 1 to include the bulk excavation, leaving the perimeter embankment and access ramps, further constrained the construction method.

The specification for Contract 2 set out performance requirements for the basement construction in terms of propping levels and loads and also limitations on ground movements. Inclinometer tubes had been cast into the diaphragm wall during Contract 1 to enable wall movements to be monitored in conjunction with a surface traverse survey. As a means of describing these constraints a characteristic method statement was prepared, using traditional raking props and waling beams at two levels (upper car park and lower ground floor). Such an exercise was also necessary to confirm basic feasibility and for cost plan information. Tenderers were invited to adopt this scheme or develop their own alternative schemes to comply with the specified performance criteria — in terms of supporting the wall and limiting ground movements.

The method proposed by Mowlem was a semi-top down system, in which construction in the centre of the site was raised with the embankment at lower and upper car parks and lower ground floor, which was then built outwards to the diaphragm wall cast on the berm and on staging supported on the embankment. When sections of the slab had been completed across the site to achieve a load balance, construction proceeded beneath by mining away the embankment. The upper car park was similarly cast on the berm and on staging, and then construction proceeded to the undercroft beneath the lower car park, to complete first the pile caps and then the lower car park slab. The construction of the permanent columns in the perimeter then followed, the slabs having been temporarily supported on an arrangement of structural steelwork brackets and beams carried by steel columns, founded on sacrificial piles, between the permanent piles. Such a system requires a design not only for the permanent condition, but also for the intermediate support system. These requirements resulted in particularly complex arrangements in smoke vent and stair, lift core and car park ramp areas. For the latter cases it was expedient to leave large openings in the top-down phase and fill these in bottom-up.

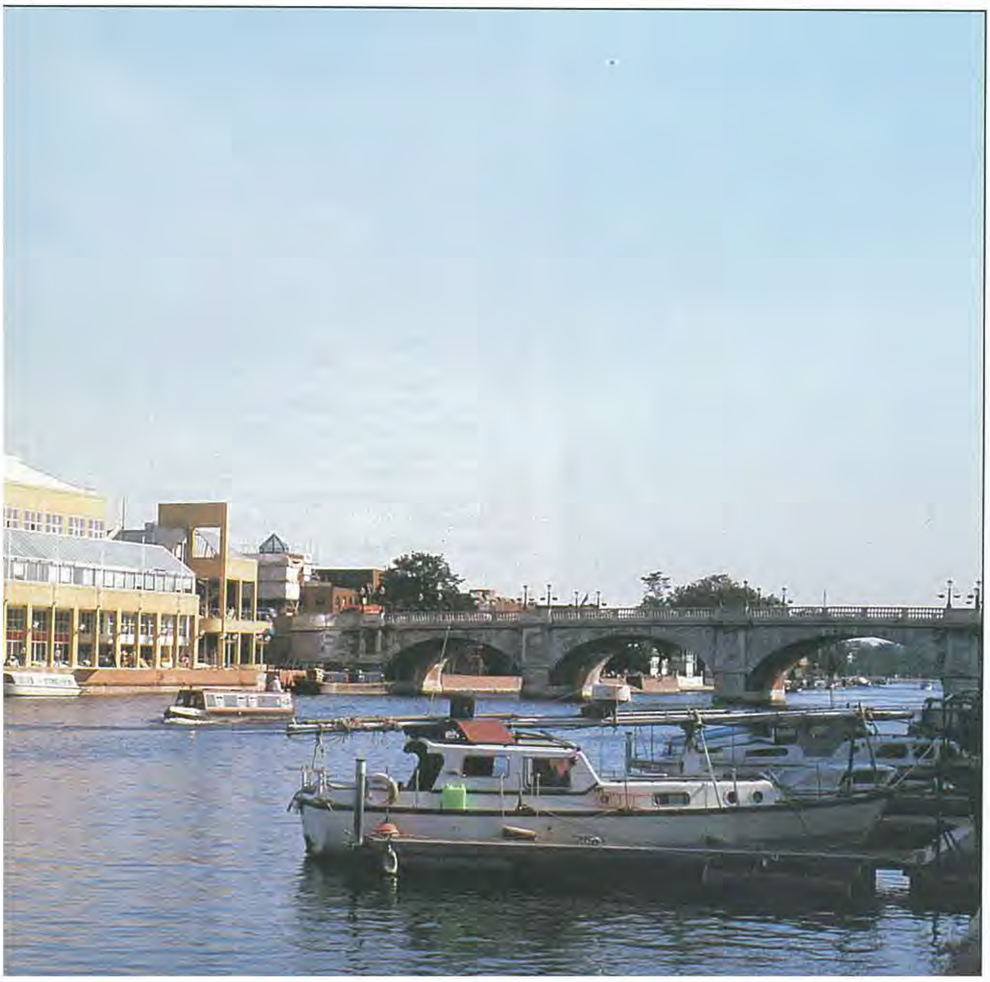
The development of the temporary works design within the framework of the established constraints, and the design of the permanent works, were very much related, requiring close collaboration between Mowlems' temporary works engineers and the structural design team.

The monitoring of the ground movements showed the movements to be comfortably within the predictions.

Preservation of archaeological remains

The known existence on the site of mediaeval remains of some rarity was regarded as highly important. Their safe removal, storage and eventual relocation within the new building was one of the conditions of the planning consent, with the Royal Borough of Kingston contributing to the costs.

The remains were an undercroft, or vaulted cellar, and the causeway walls, piers and abutments from the original Kingston Bridge. In ancient times Kingston was a major route to London because, in the 12th century and for many centuries after, the bridge was the only permanent crossing upstream of London Bridge. The removal of these remains was carried out by Pynford (South) Ltd. during the early part of Contract 1, an operation that was the subject of an earlier *Journal* article[1]. These remains were brought back to the site in September 1988 into the completed loading bay area by Pynford, who then jacked and slid them on grease skates into their final position in the riverside amenity area.



Conclusion

Traditionally, John Lewis have a low-key style and when they open a new branch it is without ceremony — and Kingston was no exception. The doors were simply opened on Tuesday 11 September 1990 and a patient queue was admitted at 9.30 am sharp. However the opening had been marked the previous day by walk-around press conferences and an evening reception for the Mayoress.

Credits

Client:
John Lewis Partnership
Royal Borough of Kingston upon Thames

Architects:
Ahrends Burton & Koralek

Structural engineer:
Ove Arup & Partners

Mechanical & electrical engineers:
John Lewis Partnership

Quantity surveyor:
Davis Langdon & Everest

Client's representative:
Clarson Goff Associates

Main contractor (Contracts 1 & 2):
John Mowlem & Co. plc

Bored piling and diaphragm walling (Sub-contract):
Stent Foundations Ltd. and Stent/Soletanche

Structural steelwork (Sub-contract):
Tubeworkers Ltd.

Illustrations:
1. Photo: Martin Charles. 2. Courtesy John Lewis Partnership. 3., B. Courtesy the architect.
4, 5., A. Photos: Peter Mackinven. C. Photo: Barry Bulley. 6. Photo: Chris Gascoigne/Arcaid.

Reference

(1) McVITTY, I. *and* EVANS, P. Preservation of medieval ruins. *The Arup Journal*, *22*(2), pp.12-14, summer 1987.

A △

B ▷

C ◁

D ◁

E ▷

F ▽

G △

H ▷

I ▷

1990 Awards and Commendations

Institution of Structural Engineers Special Award
A. *San Nicola Stadium, Bari, Italy*
Architect: Renzo Piano, Building Workshop
Structural and lighting engineers:
Ove Arup & Partners

Sunday Times/Royal Fine Art Commission Building of the Year
B. *Imperial War Museum Redevelopment*
Arup Associates Architects + Engineers + Quantity Surveyors

RIBA Regional Awards (Winners)
Eastern: *Rank Xerox Research Building, Welwyn Garden City*[1]
Architect: Nicholas Grimshaw & Partners
Structural, civil and services engineers:
Ove Arup & Partners

London: *Whiteleys, Bayswater*[2]
Architect: Building Design Partnership
Structural and services engineers:
Ove Arup & Partners

C. Wales: *Parc Tawe Hothouse, Swansea*
Architect: Percy Thomas Partnership
Services engineer: Ove Arup & Partners

British Construction Industry Awards Highly Commended (Civil Engineering Category)
D. *Ravenspurn Concrete Gravity Substructure*
Design, construction management and site supervision: Ove Arup & Partners

Highly Commended (Building Category)
St. Enoch Centre, Glasgow[3]
Architect: Reiach and Hall/GMW Partnership
Structural engineer: Ove Arup & Partners

Ravenspurn also won an ICE Northern Counties Merit Award and a Concrete Society Civil Engineering Award, and the Imperial War Museum won an Eternit International Prize for Architecture (Renovation Category) and the Museum of the Year Award. Arup Associates won the first Art at Work Award.

Brick Development Association Structural Brickwork Awards Commendation
Guesten Hall (Avoncroft Museum)[4]
Architect: Associated Architects
Structural engineer: Ove Arup & Partners

Civic Trust Awards (Non-Metropolitan areas) Awards
E. *Oakhill House, Hildenborough, Kent*
Architect: Aukett Ltd.
Structural engineer: Ove Arup & Partners

F. *Builth Wells High School, Powys, Wales*
Architect: Powys County Council
Structural and services engineers:
Ove Arup & Partners

Commendations
G. *St. Michael Financial Services Ltd., Kings Meadow, Chester*
Architect: Aukett Ltd.
Structural engineer: Ove Arup & Partners

H. *1000 Parkway, Solent Business Park, Fareham, Hants*
Architect: Aukett Ltd
Structural engineer: Ove Arup & Partners

I. *Hay Craft Centre, Hay-on-Wye, Powys, Wales*
Architect: Burgess & Partners
Structural engineer: Ove Arup & Partners

Rank Xerox, Welwyn Garden City, was also Commended.

City Heritage Award
Lutyens House, City of London[5]
Architects: William Nimmo & Partners, Peter Inskip and Peter Jenkins
Structural and services engineers:
Ove Arup & Partners

Photos:
A: Jack Zunz. B: Arup Associates.
C: The Architects. D: Niki Photography.
E: Architects. F: Bryan Williams.
G, H, I: Architects.

Illustrated in The Arup Journal:
A. Autumn 1990; B. Winter 1988/89;
D. (1) Autumn 1989; (2) Winter 1989/90;
(3) Spring 1989; (4) Forthcoming issue;
(5) Summer 1990.

Modelling wind flow over a factory with CFD

Ian Ong

Computational Fluid Dynamics (CFD) is rapidly gaining acceptance as a design tool in the engineering community. It is becoming a cost-effective method for modelling many types of engineering problems involving fluid flow, although its use has not yet become routine.

A recent example of its application has been the study of wind flow over a factory roof for Shimizu UK Ltd., developers of a new manufacturing facility for Nippondenso at Telford, Shropshire. Various volatile organic and non-organic compounds, by-products of manufacturing processes, would be discharged via roof-mounted stacks. Wrekin District Council expressed concern that introduction of a parapet as an architectural feature on two edges of the building would interfere enough with the wind flow regime over the roof to produce unacceptable concentrations of pollutants in the environment. An additional possibility was the re-entrainment of exhaust gases into the roof-mounted air-handling units.

CFD involves solution of the numerical forms of the equations which describe fluid flow. Many thousands of such equations have to be solved and, hence, powerful computers are needed. The objective is to predict fluid flow characteristics such as velocity, temperature, static pressure, and species concentration. After reaching a converged solution, these variables are presented as vectors or contours on a number of user-selected cutting places in the computational mesh. The technique allows the engineer to use his engineering (*not* computing) skills to identify particular flow phenomena of interest, such as stagnation zones, separation points, or areas of recirculation. It allows him to optimize his proposed designs *prior* to the construction of any prototype or mock-up.

Following hand calculations to assess possible problem areas and define the worst-case boundary conditions, a CFD model of the roof, comprising a mesh of cells in which the calculations are performed, was generated to address the problems. A southerly prevailing wind was considered, with the velocity profile based on UK wind data corrected for the local topography. In this external flow problem, the building is represented as an obstruction around which the mesh is formed. Calculations are actually performed in cells around and far out in all directions from the building, but these have been omitted for clarity (Fig. 1).

The mesh was refined in the critical areas where the exhaust stacks are located, in order to track the paths of the plumes in detail. In areas where only a general description of the flow field is required, a coarse mesh is employed for computational economy.

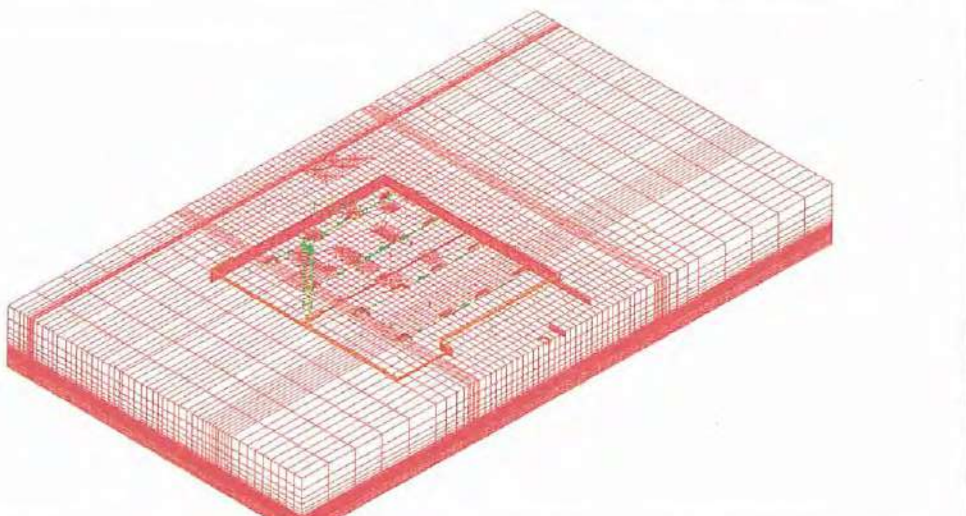
The computational mesh consisted of some 130 000 elements, approximately twice the size of Arups' next largest CFD simulation. To cope with this, the analysis was performed on our Convex C210 mini super-computer, where meaningful results were achieved after running the problem over a weekend. The analysis in total required 30 hours of Convex Central Processing Unit (CPU) time. Some of the resulting plots can be seen in Figs. 2-4.

Conclusion

CFD, supplemented with empirically-based calculations, has demonstrated that the influence of the roof parapet on wind flow patterns gives no cause for concern, and that dilution levels are sufficiently high to ensure that concentrations of stack emissions at air-handling unit intakes are within acceptable limits. The results were used by Arups and Shimizu UK to reassure the local Environmental Health Officer and Nippondenso that pollution levels would be within acceptable limits.

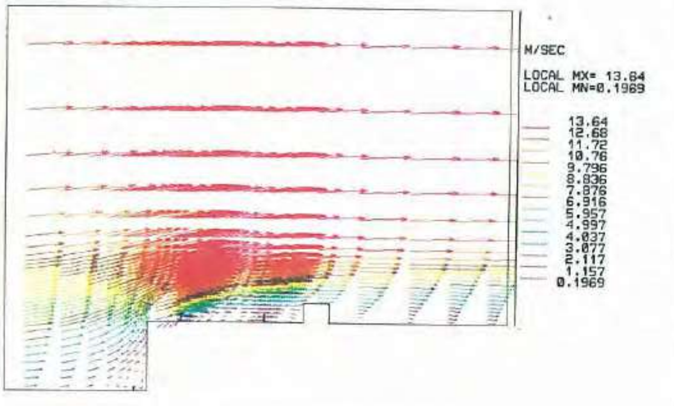
Credits

Client:
Shimizu (UK) Ltd. (for Nippondenso)
CFD consultant:
Ove Arup & Partners



1

Prostar 2.0 Velocity magnitude



2

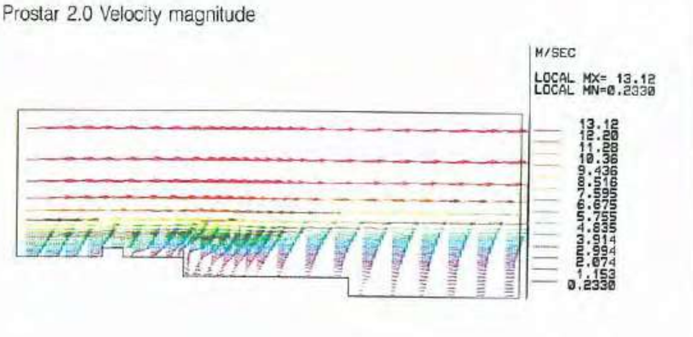
1. Computational model, showing mesh projected onto the Nippondenso building, Telford.

2. Velocity vector plot of the flow field at the leading edge of the building; the two exhaust stacks lie within the cavitation zone, which potentially could keep concentrations high at roof level.

3. Velocity vector plot downstream from 2., showing the effect of the parapet. Although it extends the reattachment point further downstream, its effect upstream on the air-handling unit is almost negligible.

4. Contour plot showing variation of boiler flue gas concentration at the building's leading edge. Although concentration levels are initially high at the point of emission, the high dilution rate ensures this is not a problem.

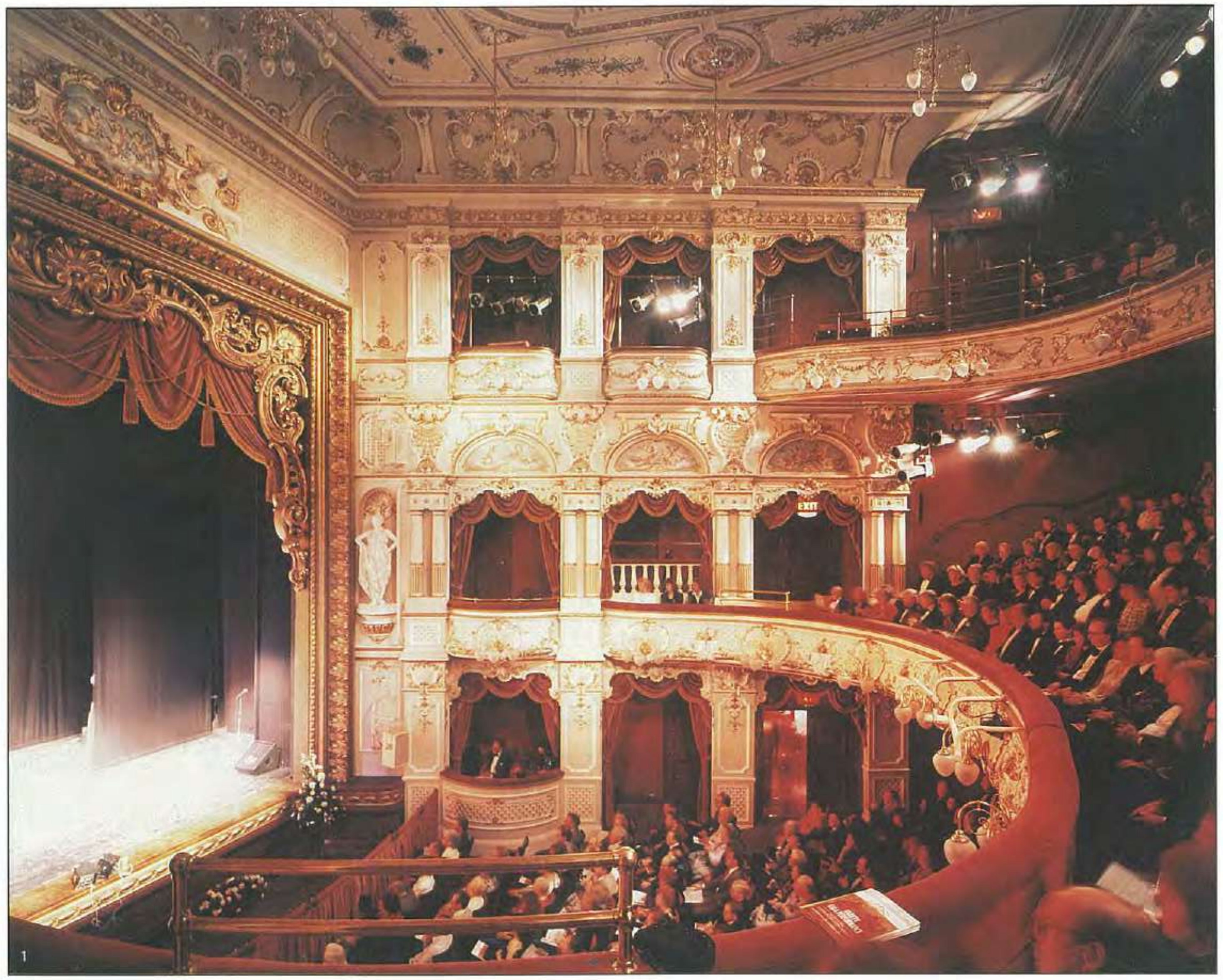
Prostar 2.0 Velocity magnitude



3

Prostar 2.0 Index: 100 = outlet concentration

4



1

Sheffield Lyceum Theatre

Architect: Renton Howard Wood Levin Partnership

Andrew Jarvis

Introduction

In 1969, after a history of over 70 years as a venue for drama, opera and variety, the Sheffield Lyceum was closed as a theatre with a final performance of *The Gondoliers* by the D'Oyly Carte Opera Company. On 10 December 1990 Gilbert and Sullivan returned to the Theatre after an absence of over 21 years. Following the completion, on time and within budget, of a complex rebuilding and restoration programme aimed at achieving a blend of modern facilities and architecture with the existing ornate Victorian decoration, the Lyceum was again open to the public with performances of *The Pirates of Penzance*.

A brief history

The Lyceum Theatre was designed by W.G.R. Sprague, a leading theatre architect at the end of the 19th century. He designed many in London including the Wyndhams, Strand, Aldwych, Ambassadors and Albery. The Lyceum opened to the public on 7 October 1897, and enjoyed a long and successful period as a major venue. It was listed Grade B as a building of architectural and historical interest in 1972, but was by then into its 20-year period of various uses and threats of demolition. In 1986, it was purchased by the Lyceum Theatre Trust, who prepared the way for the refurbishment. In 1988 Renton Howard Wood Levin prepared a successful design study for the Trust and the City Council on both the Lyceum and the surrounding square. The design team was appointed at the end of 1989, with Bovis Construction Ltd. successfully tendering as management contractors. Work started in March 1989.

The building achieved practical completion on 2 November 1990, after a construction period of 87 weeks, within the overall budget of £12M.

The theatre

The Lyceum refurbishment has been designed very much to retain the building's individuality. Whilst we were able to draw on past experiences of the refurbishment of Victorian theatres, including the Bradford Alhambra, Newcastle Theatre Royal, and London's Old Vic, this one posed many challenges not found in more conventional buildings, not least the integration of new systems into an old architectural design. For the Lyceum Theatre, all of the existing stage house and back of house facilities were demolished, to be replaced by new building. The auditorium has been restored, along with two of the existing facades. The bars, staircases and toilets are again all new.

Back of house areas

The new back of house has been constructed using an in situ concrete frame. A new rehearsal space and education and administration office are located at the upper levels, complete with an exposed steel-framed roof. The rehearsal room, together with a steel staircase at back of house, has given the opportunity for more adventurous lighting and structural systems.

In the main the back of house is conventionally serviced with radiator heating and domestic scale lighting and power systems, but includes extensive water services, drainage and mechanical ventilation systems to the many showers and toilets. Most of the new plant, tank, transformer and switchrooms are located at back of house, where they are supported from the new in situ frame. There are two new traction lifts serving all levels, and the shafts, in conjunction with the new concrete staircases, provide lateral stability to the building.

The new stage house

The new stage house is formed in reinforced concrete walls and deep beams with brickwork cladding on three faces.

The fourth face is constructed from double skin brick and reinforced blockwork, and is supported on a new proscenium steel truss spanning the whole stage width. The lightweight roof of the flytower is supported by the upper steel grid, while the lower steel grid spans from the proscenium truss to the rear stage wall.

The stage house is an interface of many functions requiring an extensive infrastructure of wiring systems. The successful integration of sound, lighting, communications and power systems demanded an intimate knowledge of theatre operation and close co-operation between our electrical engineers and Techplan, the theatre consultants. The substantial live loads imposed on the structure also required a detailed knowledge of how the stage would be operating during a performance.

2

3

The auditorium

The three-tiered horseshoe auditorium has been refurbished to its original cream, gold, and burgundy, and the rococo plasterwork on the proscenium and box fronts has been carefully restored, but beneath all the ornamentation there are engineering systems which were not available during its Victorian life. A network of electrical services and new mechanical ventilation has been integrated into the existing fabric. A single air-handling unit serves the auditorium and provides fresh air to all levels of the occupied space. This is achieved under the tiers and at rear gallery by using linear diffusers in the ceiling connected to supply headers. The front of stalls is served by a number of jetflow diffusers designed to throw air the full height of the auditorium, while maintaining a controlled air movement in the occupied zone. The exhaust air is extracted from two positions, firstly across the stage to extract plant located on top of the flytower, and secondly at rear gallery above the rear lighting bars to the main air-handling unit.

A major problem was to find appropriate routes for the substantial ventilation ducts. For example large supply and extract ducts penetrate the 90-year-old 600mm thick brick auditorium wall, which on one level is adjacent to a new double-width door opening. Steelwork ring beams, placed in an operation similar to underpinning, were used to overcome the problem, providing stability both in the temporary and permanent conditions.

To meet the requirements of Building Control, the existing steel and clinker concrete tiers have been substantially strengthened, but the existing steelwork trusses at roof level required only minor strengthening to support the network of rooftop ductwork, and the winch equipment for the period chandeliers.

Front of house

The original stairs, providing separate access to the various levels as required in the 19th century, have been completely removed to provide new open foyers with bars, circulation, and toilet spaces.

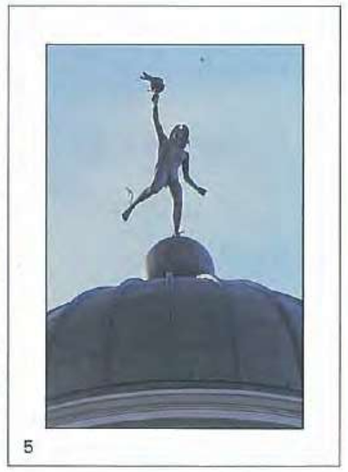
Between the existing facades and the auditorium, steel and *Holorib* slabs form new floors for toilets and foyers. At circle foyer, the load from an existing wall is transferred through the steel frame allowing the removal of the lower section of wall. To one side of the auditorium, a new concrete frame supports bars and a grand staircase. This frame also provides lateral support to the existing auditorium wall while allowing vertical differential movement.

The plantroom at rear gallery level has a steel-framed *Holorib* roof, designed to carry chiller equipment, should cooling be required at a future date. The new bars and foyers have again given the opportunity for more imaginative lighting design, to provide a contrast with the period approach to the auditorium. This culminates with the chandelier over the grand staircase, designed by the architect, incorporating a low voltage dichroic lighting system. One of the more enthusiastic pieces of design however, remains the design of the support steelwork for the Mercury statue.

In many ways the Lyceum has been a difficult building to design and construct, with numerous problems associated with the integration and interfacing of the many systems involved in such extensive restoration.

It is, however, now attracting the crowds again, with both *The Pirates of Penzance* and *Showboat*, the following show, sold out.

4



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6

7

1. Auditorium interior.
2. The auditorium from the stage.
3. Front elevation, with the Crucible Theatre (1970 Arup Job 2798) in left foreground.
4. Grand Crush Room.
5. Statue of Mercury on rotunda above Grand Crush Room.
6. Stained glass over stage door entrance, back of house.
7. New bar with grand staircase.
8. Back of house elevation.

8

Credits

Client:
Sheffield Theatres Ltd. in conjunction with Sheffield City Council

Architect:
Renton Howard Wood Levin Partnership

Structural and services engineers:
Ove Arup & Partners

Theatre consultants:
Technical Planning Ltd.

Quantity surveyors:
Bucknall Austin plc

Management contractor:
Bovis Construction Ltd.

Photos:
1, 2, 4, 7: John Walsom, courtesy RHWL
3, 5, 6, 8: RHWL.

Andrew Allsop

Ove Arup and Partners have been designing wind-sensitive structures since the early days and have often taken a leading role in applying state-of-the-art wind engineering knowledge, notably for such structures as the Emley Moor television tower and the Hongkong Bank. However, these examples tend to obscure the large number of more limited wind tunnel studies that are carried out at a current rate of perhaps 10 a year, as well as the much larger number of structures we design which lie outside the scope of existing codes of practice but for which wind tunnel testing cannot be justified.

In this article I do not intend to go over again ground already covered in previous *Arup Journals* but rather to look ahead to the developments we expect and wish to encourage in wind engineering: the formation of the UK Wind Engineering Society, the coming of the new UK wind code, the establishment of codes of practice for wind tunnel testing, and the application of numerical methods and theory to what has been, up to now, a very practical science.

The last 20 years

These decades have seen great developments in wind engineering practice, marked in particular in the UK by the Construction Industry Research and Information Association (CIRIA) conferences 'Modern design of wind sensitive structures' in the '70s and 'Wind engineering in the '80s'. However it is instructive to consider that many of the recent advances in practice have come from the ability to process data cheaply and easily rather than from great changes in fundamental understanding of wind behaviour. Eiffel and the early long-span bridge builders were certainly aware that real wind behaviour was dynamic, but ultimately they had to chose some kind of static gust force for design.

In the early days most wind tunnel testing was carried out in smooth (low turbulence) flow and no data could be obtained on any dynamic load that depended on wind turbulence or buffeting. Smooth flow tunnels are still used for aeroelastic instability problems, especially of bridges, as such testing has been shown to be generally conservative in this case. Testing for buildings is now normally carried out in boundary layer wind tunnels which are capable of reproducing the main features of atmospheric wind, including the variation of wind speed with height and the intensity and spectrum of wind turbulence. These features of the real wind have been shown to affect even the mean patterns of wind and certainly have a strong effect on local pressures.

Alan Davenport has been one of the main pioneers in applying increasing knowledge of wind behaviour to calculating and measuring real dynamic wind response of structures. The gust factor method that he has successfully advocated and other similar approaches are now widely used for the design of tall buildings and long-span bridges. In parallel, and notably in the UK under the direction of the Building Research Establishment, static gust methods, similar to those used in the current UK wind code, have reached very high levels of sophistication for use for low-rise structures.

These two approaches are complementary and both are now based on a similar understanding of wind behaviour. They should produce similar loads for appropriately shaped structures that are not strongly dynamic. Unfortunately the different methods are often presented in codified form with very different presentations of the wind pressures. Forms commonly encountered include gust speeds with various averaging times or none (*CP3*), fastest mile (USA) and 10m and hourly means. There are relationships between all these wind speeds and corrections can be made, but they do not explain all the differences between different code methods.

Formation of the Wind Engineering Society

The increasing use of wind tunnel results for structural design has prompted the formation of more formal wind engineering groups involving both researchers and structural engineers, with the purposes of sharing experience, establishing standards for wind loading and testing and identifying areas that need further research. These groups extend initiatives begun by Alan Davenport and the late Ken Anthony.

In addition to the forum provided by the International Association of Wind Engineering, there are US, Canadian and Australian groups. In Europe a German-speaking society has been formed, a French equivalent has been proposed and there is the recently-formed Wind Engineering Society in the UK, now an affiliated learned society of the ICE.

The Wind Engineering Society[1] is intended to provide a focus for research and application in the UK. It covers wind engineering activities ranging from meteorology and statistical analysis through wind loads, wind environment and ventilation.

Clearly these involve a great range of people and activities and it is anticipated that specialist groups will form but continue to share common interests. The society is looking for a wider involvement of practising engineers (and architects?), especially in such subjects as environmental winds, ventilation and fire engineering.

The new UK code of practice for wind loads

BS6399: Part 2 is shortly to be issued as a draft for comment. When *CP3: Chapter V: Part 2 (1972)* was first published it was probably the most advanced static gust wind design code in the world and widely copied in other countries. In use, however, it has limitations and different clauses can produce widely varying answers — one clause is now known to be wrong and can have large consequences for cladding. The new code is intended to be much more precise than *CP3* and covers a much wider range of typical building structures. In consequence it will be a thicker and more complex document, requiring software aids; Arups are already preparing these.

Wind tunnel testing

It is clearly impossible to cover all structures with accurate codified design rules and there are other influences on design such as environmental winds, smoke extraction, and ventilation, for which there is even less guidance. The main influence on wind effects is the basic building massing together with the arrangement of surrounding buildings. Add-on wind deflectors or dissipators can sometimes improve local conditions if space and other criteria allow, but the best solutions are often only available during initial scheme design. Good advice on avoiding environmental wind problems and on natural ventilation can be obtained without wind tunnel testing, but these assessments are very difficult to quantify reliably.

Wind tunnel studies may be needed for various purposes through the period of a project and these should be co-ordinated to obtain the best value. A fairly normal large development might require wind tunnel studies at the planning stage for the effects on the wind environment for pedestrians, and on ventilation and natural smoke extraction, all of which can have a strong effect on overall building cost or even feasibility. Particular care is needed for natural ventilation of partially covered areas where the hoped-for effects of mechanical ventilation can be swamped by natural air movements. During detail design, wind tunnel studies may be useful to establish wind loads on cladding and for structural loading. For structures where the wind is a major part of the load, such as lightweight roofs or slender buildings, the wind loads or building movements can strongly affect the suitability of the structural arrangement and consequently feasibility and wind tunnel testing may be required at an early stage.

Overall wind loads on essentially static structures can be determined from pressure tests, but the required distribution of pressure measurement points is different from that required for cladding loads. The overall forces on taller or more flexible and slender structures should be obtained from measurements taken from a force balance on which a 'rigid' building model is mounted, from which the spectra of wind forces can be obtained. These spectra can be transformed to estimate the dynamic response at the natural frequencies of the building. In cases where the movements of the structure will affect the wind forces (aeroelastic problems), a flexible model is required with correctly scaled frequencies, vibration shapes and damping.

We regularly carry out wind tunnel testing in a number of different tunnels and have knowledge of the capabilities of a large number of others. In addition to several laboratories in the UK, work is regularly carried out in Canada and Australia. In Europe, major projects are being undertaken in France and Germany and there are good boundary layer tunnels in most of the other European countries. We have the experience to evaluate capability but it is best to do this before the testing is commissioned.

Despite other developments, wind tunnel testing will remain a useful tool for the foreseeable future. Indeed, the costs and time to obtain test results are still falling, at least in real terms. In North America there are now several commercial wind tunnels competing on both price and service with university research facilities. The technical skill of some of these companies is high and they can provide a well-managed service, but if you are looking for competition on price and programme you had better be sure what you are buying.

Standards and specifications for wind tunnel testing

Testing from different laboratories rarely gives completely equivalent information, nor data that can be directly referred back to codes of practice. In most cases, the measurements will be rather more accurate than application of the code, although until codes of practice are built around wind tunnel testing procedures there will be some doubt about the exact correspondance. A major concern of the Wind Engineering Society is the development of standards for wind tunnel testing that will help ensure that results from different wind tunnel establishments are compatible and can be compared to code values.

In the modern economic climate, wind tunnel testing is increasingly seen by our clients as something that can be subjected to the disciplines of competitive tendering. However none of the testing establishments carry insurance capable of covering consequential risks. In these circumstances, if we do not specify precisely what we want and ensure that appropriate quality assurance is carried out, it is we who may be at risk.

Wind climate models

Some things are impossible or very difficult to scale in wind tunnels. Atmospheric stability, the effects of the earth's rotation, ground topography, and changes in the effective ground roughness all have some significant effect on the variation of wind speed with height and the gustiness of the wind. These factors can be assessed using simplified theory based on measurements and theoretical and Computer Fluid Dynamics (CFD) analyses. The Engineering Sciences Data Unit (ESDU[2]) has produced the most internally consistent wind model capable of handling these variables but there are still a few gaps, notably on the effects of atmospheric stability and on wind spectra in non-equilibrium conditions.

Some predictions of the model are controversial and it needs further testing against a wider range of full-scale data. However, ESDU's application of aerodynamic theory to problems of the atmospheric boundary layer has enabled full-scale information to be evaluated more reliably and to a sufficient degree of confidence for a simplified version of the model to be used in the proposals for the new UK wind code. The assessment of the appropriate design wind speed is probably the largest source of error in assessing wind loads and the additional effort required by the new approach is well justified by improved accuracy.

Full-scale measurements

These are essential to calibrate the work that has been done in both wind tunnels and theory. The limited amount of full-scale data is becoming a significant bottleneck in achieving confident advances in the use of wind engineering methods. There are scaling effects that ensure that details of wind flow around wind tunnel models are not quite the same as full-scale and certainly problems of atmospheric boundary layer physics will not be solved to everyone's satisfaction without more measurements.

We have occasionally been involved in full-scale measurements and valuable results have been obtained despite the difficulties. Recent developments in micro-computers and remote control using telephone lines enable many of the main organization problems to be avoided, and certainly costs are falling.

Unfortunately there is rarely any value to our direct clients in taking such measurements unless they are prepared to take a very long-term view. However, we should continue to encourage clients and researchers and we are prepared to help in appropriate cases.

Changes in world climate

There has been much talk in recent years of changes in the world climate and global warming and whether climatic change may make some difference to design wind speeds as evidenced by recent winter storms. The BRE has a Government project to look at this subject and determine if there is any way of detecting such changes.

The main difficulty is that the wind speeds and directions are subject to many conflicting variables and simply to random fluctuations, some of which occur over a very long time.

Design wind speeds have therefore to be presented in statistical form with a certain probability attached to them such as the usual UK design criteria of 50-year return or once on average in a 50-year period.

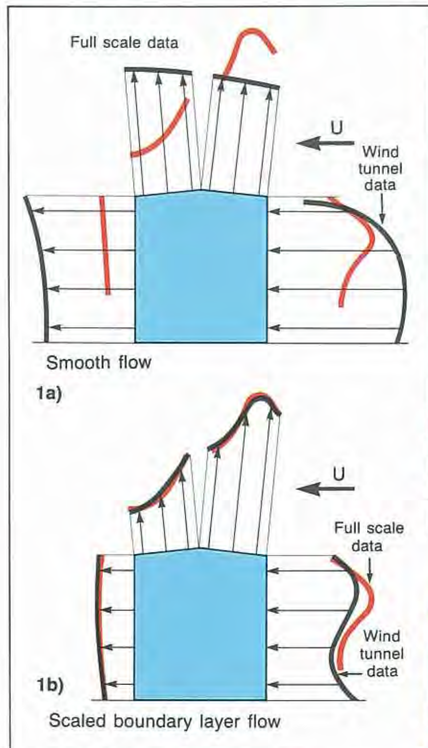
Obviously exceptional wind speeds are caused by exceptional atmospheric conditions. The question is whether the normal atmospheric conditions are changing in such a way as to make currently exceptional events more likely.

The driving force for the weather ultimately comes from the sun but, as in any heat engine, it requires a temperature difference to generate wind; simplistically, the higher the temperature difference, the stronger the wind. The temperate zones of the world are where the cold air from the north mixes almost continually with the warm air from the topics in the series of warm and cold fronts that pass the British Isles. The ocean temperatures make a large difference to this process as the Gulf Stream brings additional heat energy north from the tropics. The strength and position of the Gulf Stream also fluctuates. On the cold side the snow cover in the north is important as it provides a low temperature 'reservoir' in the form of the latent heat of melting, in addition to reflecting some of the sun's radiation energy back into space.

The interaction of all these factors plus the effects of cloud cover on radiation to and from space are very complicated and some of the world's largest supercomputers spend most of their time trying to extrapolate ahead for weather prediction, with a success rate judged in days at best. When a small storm of only 100-200km diameter such as a hurricane in the Gulf of Mexico can significantly affect the rain and wind in the UK, perhaps a few days is the best we will ever do.

Whether man's activities have any effect on these processes is even more difficult to determine. Certainly pollutants which change the amount of solar radiation absorbed or re-emitted could have effects such as those thought to be caused by major volcanic eruptions such as the Krakatoa explosion (1883). Certainly there have been large changes in world climate over the millions of years that the earth has existed, including many ice ages and warmer periods than at present. The most recent ice age only receded a few thousand years ago. Climate change in more recent times is demonstrated by historical data on regular wine making and grape harvests in the English monasteries of the Middle Ages, whilst winters are now clearly much warmer than some of those in Victorian times.

The prediction of 50-year storms has not significantly changed due to the recent weather. Indeed, until the storms of the last few years arrived, the UK had been extremely lucky, with rather fewer very large storms than estimated on average. Despite the apparent number of large storms in recent years, it should also be noted that the areas of the UK experiencing the worst effects did not overlap significantly. Most areas have had only one storm that approached or exceeded the 50 year return event.

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3


4

1 a), b): Effect of correct simulation of wind behaviour (after Jensen)

2. La Tour Sans Fin: Architect's model

3. Are tie-downs really necessary?

4. La Tour Sans Fin: Wind tunnel model (CSTB)

References

(1) Andrew Allsop is Vice-Chairman of the Wind Engineering Society.

(2) ESDU International, Wind Engineering Series of Data Items. These are updated continually. Andrew Allsop is on the ESDU wind engineering committee.

ITN headquarters, 200 Grays Inn Road, London

Architect: Foster Associates

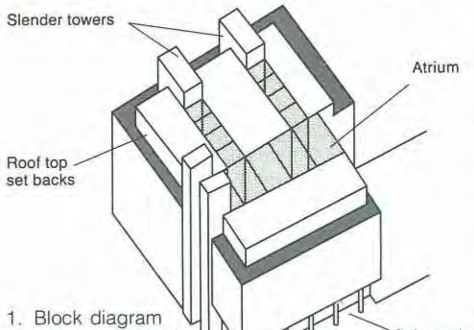
John Hirst

Introduction

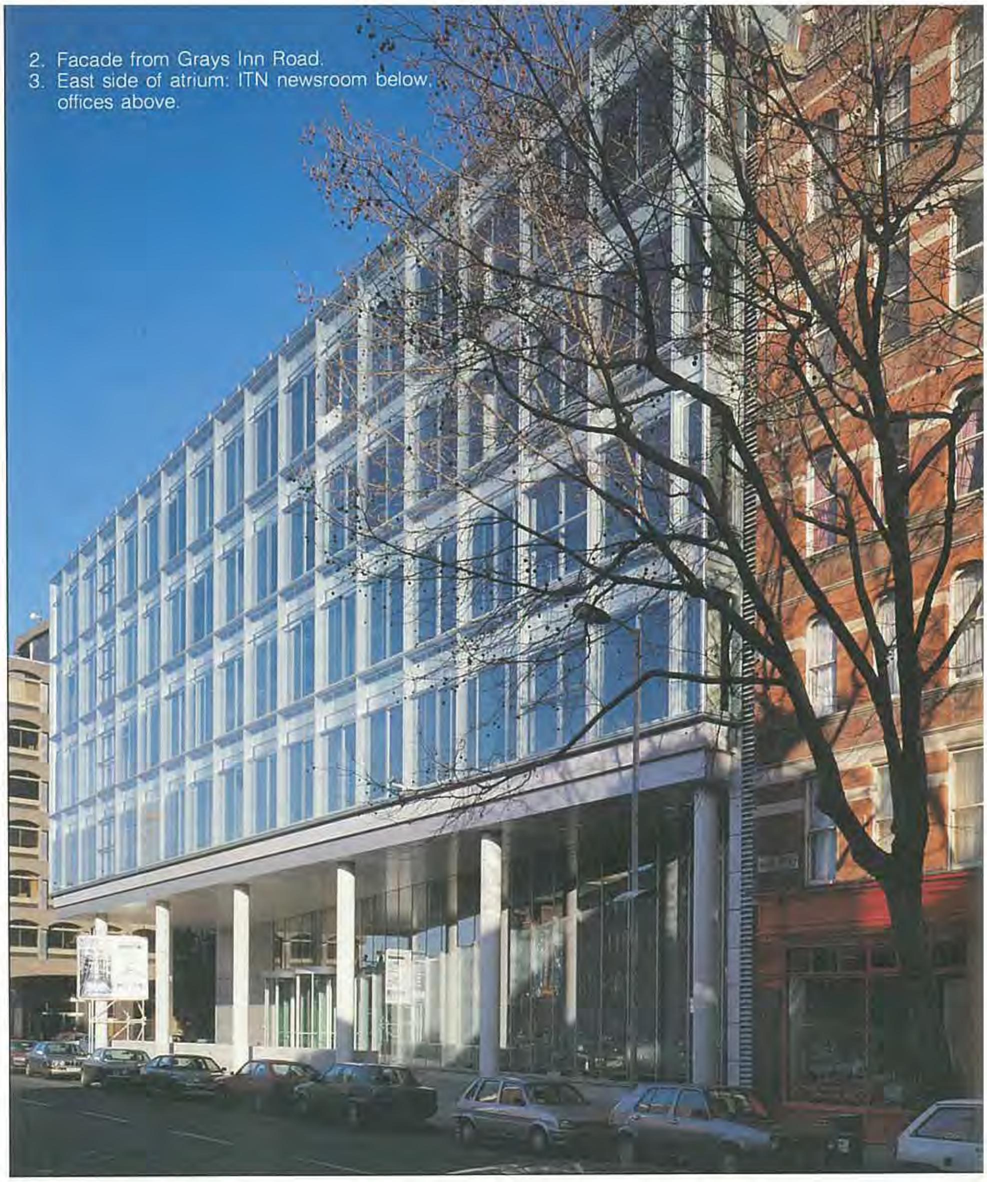
On 17 December 1990, 'Channel Four Daily' was the first programme to be transmitted from Independent Television News' new London headquarters in Grays Inn Road. This occasion marked the culmination of two years' work on the project.

ITN had occupied their previous premises, ITN House in Wells Street, since 1969. From then their output of news broadcasts had increased from 40 minutes to nine hours per day, and as a consequence they had simply outgrown the accommodation. The new site was previously occupied by the old *Sunday Times* building, Thomson House, which became available after News International moved their operations to Wapping.

The main building comprises some 33 500m^2 over 10 levels, including two basement levels.



1. Block diagram based on architect's concept sketch.



2. Facade from Grays Inn Road.
3. East side of atrium: ITN newsroom below, offices above.

ITN occupy the lower five storeys with the remainder let for general office purposes. Part of the adjacent premises, 214 Grays Inn Road, has been adapted for use as news archives, office storage and car parking, with a limited amount of office space. This additional floor area is in the order of 9300m^2.

The site

The trapezoidal site is bounded by roads on three sides: Grays Inn Road to the west, Gough Street to the east and Coley Street to the north. Elm House, an existing office building, is located to the south, between the main site and Elm Street. Below ground, Thomson House had been split into two main areas. The printing presses were housed in a double storey-height basement on the west of the site, while on the east there were two levels of office accommodation. The building and the printing presses were founded on pad footings.

Retaining walls of various types surrounded the site; however, the wall along the northern boundary effectively became redundant when 214 Grays Inn Road and the elevated road deck for Coley Street were constructed in 1973.

The building design

One of the key design objectives was to incorporate a degree of 'openness' which would also provide a central focus for both the occupants and the visitors. This objective has been realized by an atrium which splits the main building into two parts. Most of the atrium is full height and in the form of a long trapezium with sides parallel to the east and west boundaries. The south wall and the roof are completely glazed. The overall effect is to allow natural light into all levels and to allow each of the east and west areas to be planned on rectilinear grids. The arrangement is so successful that it is difficult to believe that the two lowest levels are below ground.

The block to the west of the main atrium is approximately 50m by 18m, the latter being normal office depth. Cores at each end provide vertical circulation, whilst areas at the north of each floor are used for plant. Part of the first floor slab is set back to provide a double storey area with a colonnade for the main entrance along Grays Inn Road.

The block to the east, along Gough Street, is approximately 63m by 36m, with the two main double storey-height studios located in its basement. At second floor level and below, the deep floor plan was quite advantageous to ITN due to the inter-relationships between the various departments involved in news gathering and programme production. Above second floor level, however, a planning arrangement more appropriate to conventional offices was needed.

This has been achieved by the introduction of two terraced sections of the atrium perpendicular to the main axis, with the effect of creating floor areas of conventional depth and increasing the amount of natural light entering both the atrium and the offices.

The eastern block is served by four staircases; one at each end on the western edge and two located more centrally on the eastern edge.

Six main glass-sided lifts are placed at the north end of the atrium with four more in pairs within the cores on the east side. These additional lifts were incorporated at the request of ITN to assist in communication between departments. An extension to the lift lobby acts as a link between the two blocks at the north. A further link is provided at the south end of the atrium adjacent to but separate from the glazed wall.

The lowest four floors are substantially open to the atrium. The perimeter of the remaining floors facing it are enclosed in floor to ceiling glazing.

At the level of the second basement, the main building is linked to the ancillary area in 214 Grays Inn Road below the elevated Coley Street road deck. The remainder of the area below Coley Street is used for mechanical plant, water storage, etc.

The main building structure

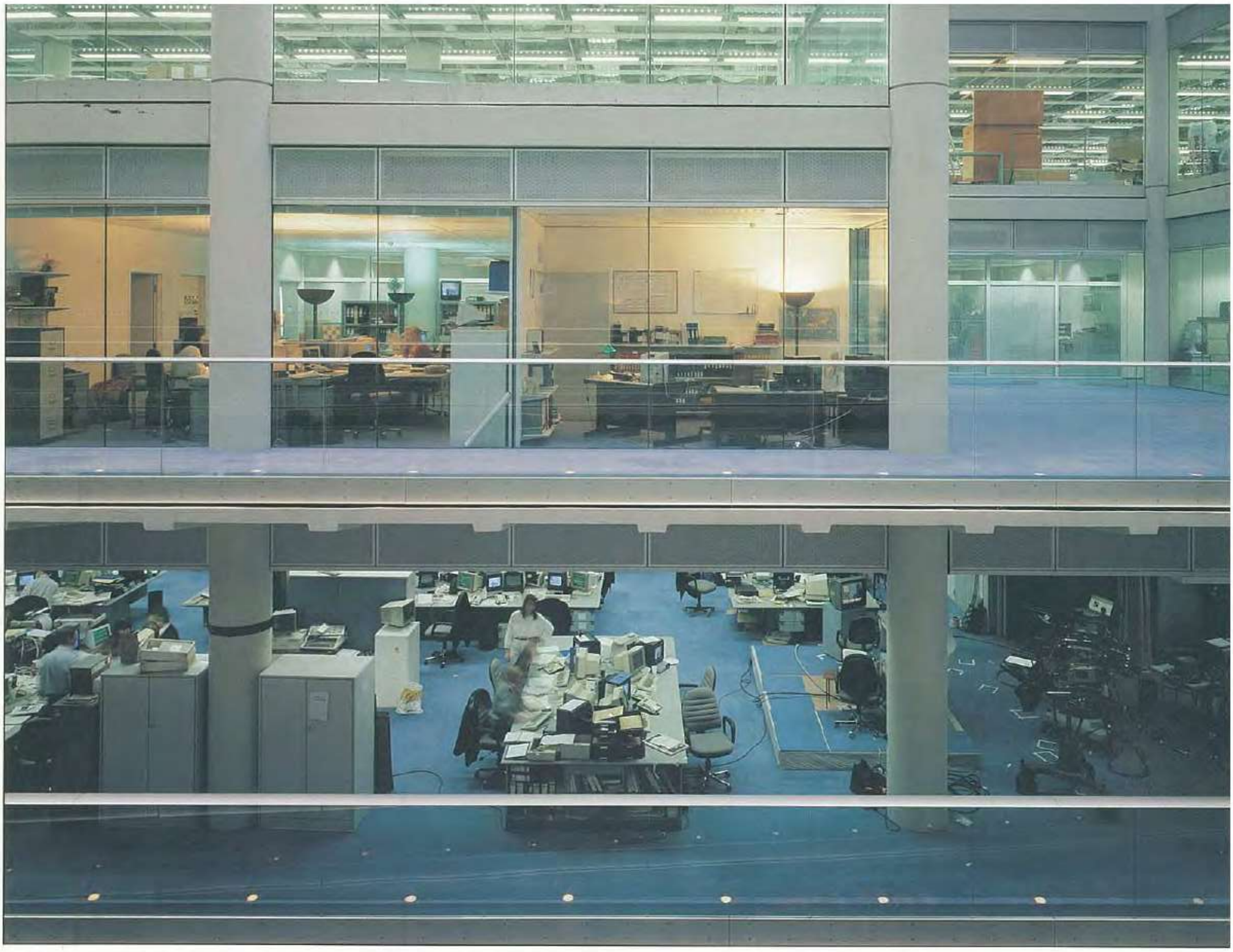
The principal criteria for the selection of the structural form were cost, speed and flexibility.

Based on the overall building concept, various alternative options for both materials and grids were considered. For the west block a combination of 9m × 9m and 9m × 8.25m grids was selected, whereas for the east a mixture including 9m × 9m, 9m × 7.5m and 9m × 6.75m was adopted. A range of concrete and structural steel floor systems was considered during the development of the floor grids. ITN expressed a strong preference for a structural floor system without drops or downstands, which would also allow the formation, if necessary, of additional holes through the floors after completion of the structure. They wanted a structure that was 'flexible' in terms of the extensive technical fit-out that they would have to carry out.

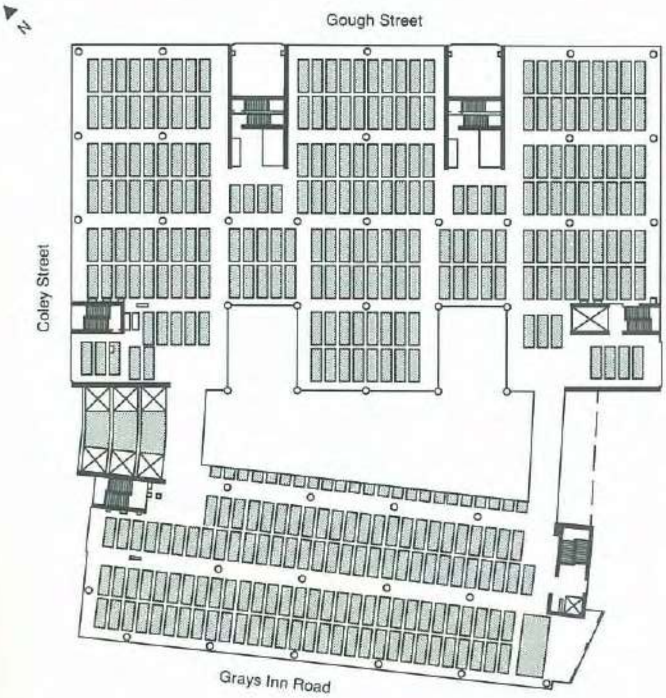
An in situ one-way ribbed slab spanning onto spine beams of the same depth was finally selected. Generally a floor depth of 425mm was used with a 125mm thick topping and ribs spaced at 1.5m centres. The ribs were designed as L-sections to allow holes to be formed in the topping between ribs at any point of their span.

Tapered ribs are extended into the atrium on the west side and up to second floor on the east side to form internal circulation walkways.

The ground floor slab acts as a prop to the basement retaining walls. Due to the presence of the atrium void and changes in level, the behaviour of the slab as a diaphragm is quite complex.

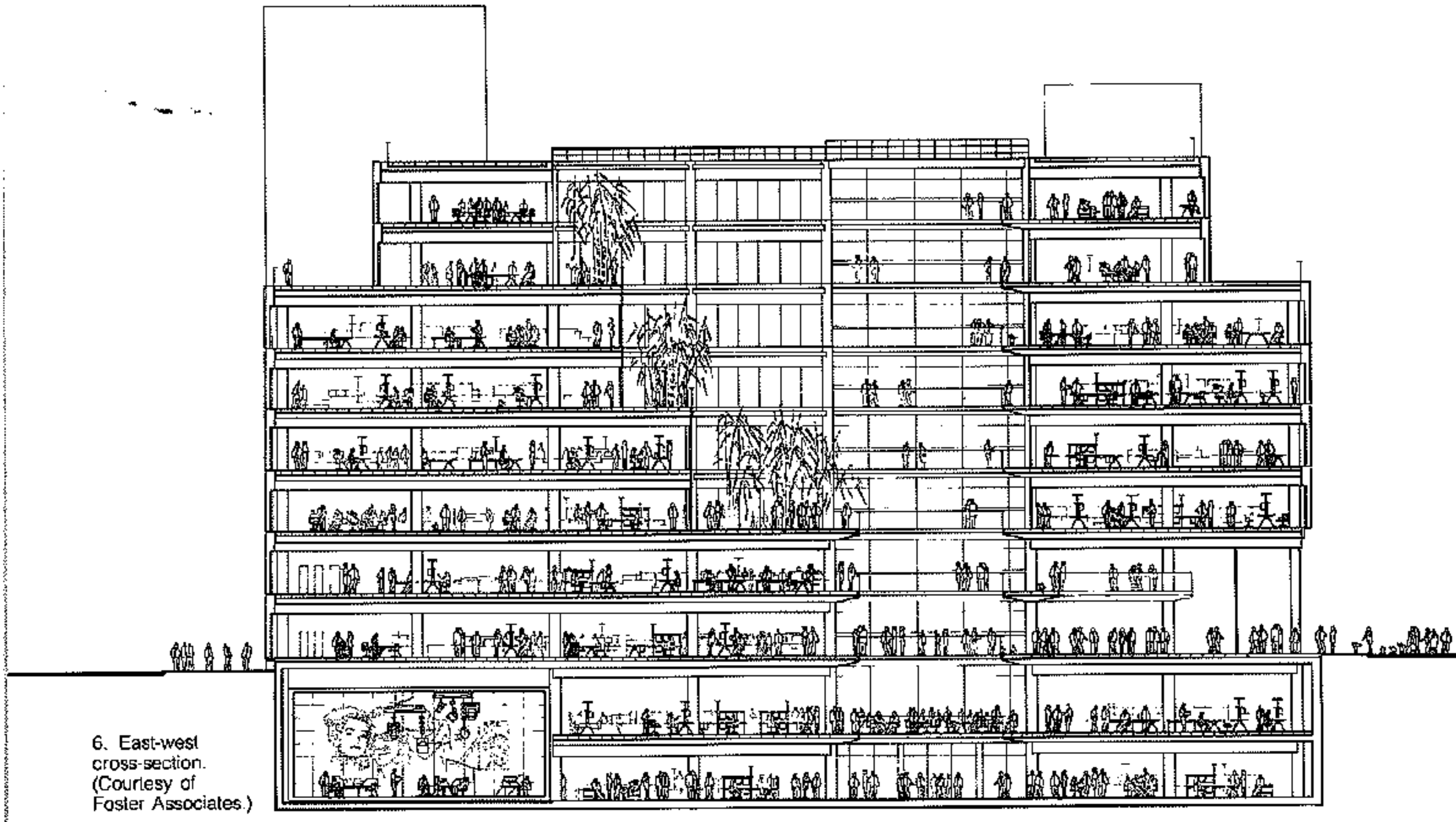


A detailed investigation of its behaviour was carried out and as a consequence the depth of the ground floor slab was increased to 450mm with an increased topping thickness of 150mm. The two main studios located at the lowest basement level each required a column-free area measuring approximately 15m × 17m, with a clear height of some 7.5m. This was achieved by introducing two transfer beams at ground floor level. Each of these spans 18m between the perimeter retaining wall and an internal column and supports the loads from columns extending from ground floor to roof at Level 8. Lateral stability of the building is provided by shear walls located around stairs and lifts. The construction of these shear walls was identified as one of the critical activities affecting the construction programme for the structural and therefore following works. In order to facilitate rapid construction, the wall arrangements were kept as simple as possible; nibs and return walls were eliminated and a minimum of cross-walls were used. The concrete works contractor elected to construct the walls in advance of the



4. Floor plan.

5. Atrium roof details.



6. East-west cross-section. (Courtesy of Foster Associates.)

floor slabs. The walls in the two east cores were poured in double storey heights and for these prefabricated reinforcement was utilized. This construction approach necessitated the extensive use of bent-out bars and cast-in, screwed couplers.

Above the ground floor all stair flights were precast with in situ landings, installed as the frame progressed to provide construction access. The concrete walls within the staircases have simply been painted.

Walkways at each level, each with a clear span slightly more than 14m, provide the link across the south of the atrium. A trapezoidal section, 425mm deep, was adopted to harmonize with the tapered, cantilevered walkways around the remainder of the atrium. The design developed for these post-tensioned link bridges allowed them to be constructed after the main floor areas and therefore removed them from the critical construction path.

The roof of the atrium comprises clear glazing supported on rectangular hollow section (RHS) purlins, designed as two pin-arches. These span onto fabricated main beams, which are supported by the concrete frame. The horizontal thrust from the purlins is resisted by in plane bracing located at the north and south ends. The main beams were fabricated from circular hollow sections, curved side plates and a closing top plate into the form of an aerofoil. This theme of the aerofoil slope occurs in a number of locations, including the atrium handrail and the mullions to the external cladding.

The south atrium wall comprises translucent glazing supported on RHS transoms and mullions. The load-bearing mullions are restrained by aerofoil profile beams spanning horizontally between concrete shear walls located on each side of the atrium.

The size and geometry of the basements to the *Sunday Times* building were such that an envelope was available within which it was possible to plan, design and subsequently construct the two new basement levels of the new building. A raft foundation varying from 800mm to 1350mm thick was used. Essentially this is made up of a series of pads and strips linked by the shallower 800mm sections. Below these shallow areas a sub-slab drainage layer, linked to pumped sumps, has been provided. This external drainage system in combination with a double layer external membrane provides security against water ingress into the basement areas. An interactive structure-soils analysis was used during the design of the raft. Detailed investigation revealed some areas where either differential settlements or bearing pressures were slightly greater than those acceptable. In these areas the raft was augmented by groups of mini-piles.

The studios

The two main studios are acoustically isolated from the remainder of the building. Each roof is made up of a dense concrete composite slab supported on steel trusses which span parallel to, but are separate from, the transfer beam at ground floor level. These trusses are supported by braced stanchions which restrain the inner skin of dense blockwork of the walls. The whole is supported by a concrete ring beam constructed over rubber isolation pads. The floor slab, which consists of two debonded layers, was also constructed over isolation pads.

214 Grays Inn Road

Ove Arup & Partners were involved in the design of this building, also known as New Printing House Square, which was constructed during 1973 and 1974[1]. It has a deep basement, part of which contained four levels of car parking and the remainder, although of similar depth, without intermediate floors. This latter area was to provide space for the possible future expansion of, and newspaper print storage for, the printing works in the adjacent Thomson House.

The original design had also made provision for the later addition of two additional floors within that part of the basement area that lacked intermediate floors. The construction of simple flat slabs to provide car parking and office storage areas was therefore exceptionally easy. The formation of a ramp from ground floor and adapting stairs and smoke vents were the only slight complications.

The ground floor of the building had been designed to allow the delivery of newsprint. As a result there was sufficient load capacity and headroom to introduce a mezzanine floor between ground and first floors. The area created has been utilized as office areas by ITN, thereby releasing space within the main building.

Programme

An article about ITN would not be complete without some reference to the speed of the design and construction of the building. Some of the key dates are therefore given below.

July 1988:
First discussions

September 1988:
Planning application

October 1988:
Demolition of Thomson House commenced

March 1989:
Planning approval and provisional acceptance of atrium by District Surveyor

May 1989:
Concrete works contractor appointed

June 1989:
First concrete to the raft placed

October 1989:
Substructure area released for construction of studios as part of fit-out

January 1990:
Topping out

March 1990:
Technical fit-out commenced

June 1990:
Practical completion, shell and core

December 1990:
Practical completion, fit-out. ITN complete transfer of operations to the new building.

Credits

Client, shell & core:
Stanhope Properties

Client, fit out:
Independent Television News

Architect:
Foster Associates

Structural engineer:
Ove Arup & Partners

Mechanical and electrical engineers:
J. Roger Preston/Sandy Brown Associates

Construction manager:
Bovis Construction Ltd.

Concrete works trade contractor:
R. O'Rourke & Son Ltd.

Photos:
Peter Mackinven

Reference

(1) JENKINS, M. *and* SMITH, K. *The Times* New Printing House Square. *The Arup Journal, 12*(3), pp.10-15, September 1977.

7. The atrium looking north.