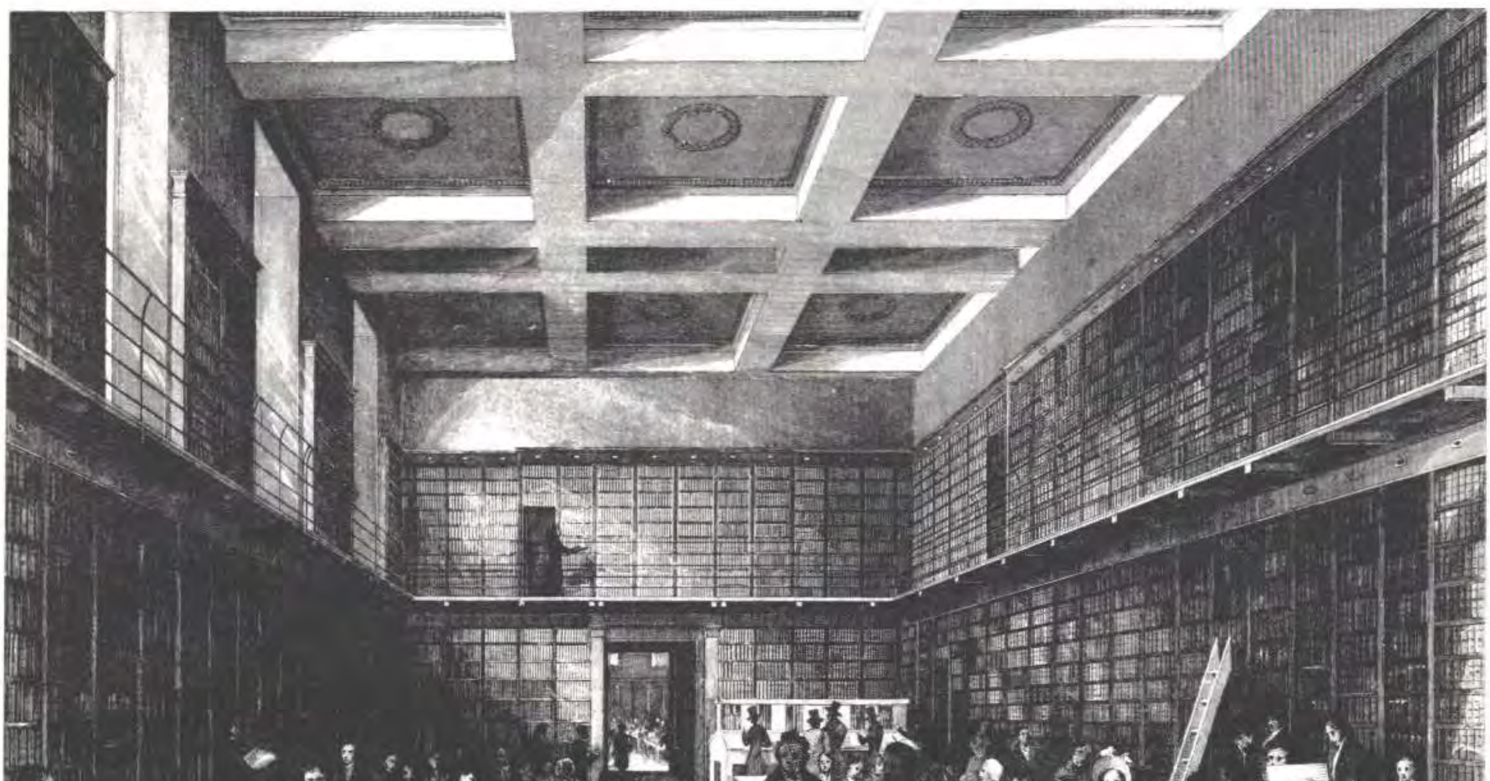
THE ARUP JOURNAL

DECEMBER 1978



T. H. Shepherd.

H. Melville.

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Back cover: Guildhall School of Music, Thames Embankment, Blackfriars, 1886 (Illustration by courtesy of Mary Evans Picture Library)

The British Library

David Croft
Peter Ryalls

Introduction

In March 1978 the Government announced their intention to start construction of the new British Library. In a written reply to a question from Mrs Lena Jeger, M.P. for Camden, Holborn and St. Pancras South, the Minister of Education and Science, Mrs Williams, said:

'The Government intend to start the construction of a substantial first stage of the new British Library building in the Euston Road in 1979-80, subject to the clearance of details with the planning authority. Preparation of the site will be put in hand as soon as possible.

The building has been designed to the requirements of the Library for the Department of the Environment by the nominated architects, Colin St. John Wilson and Partners. The first stage is expected to be occupied towards the end of the 1980s, enabling a substantial part of the Library's unique collections to be housed in satisfactory conditions and providing a greatly improved service to readers.

The estimated cost of the first stage, which will be spread over 10 years, is £74m. (at June 1977 prices); it will provide considerable employment in the construction industry and when it is occupied there will be substantial savings from the vacation of much of the Library's present scattered accommodation.

Decisions on the remaining stages of the building will be taken later.'

At that time development of the design had reached RIBA Stage D for the whole of the project (Fig. 1). Detailed design is now continuing on the first stage.

The purpose of this article is to describe the project in general terms. Future articles in *The Arup Journal* will cover some of the more interesting aspects in greater detail.

History of the new building

The British Library was formed in July 1973 (under the British Library Act 1972) from the library departments of the British Museum and other national libraries and receives an annual grant-in-aid from the Department of Education and Science. Control and management of the Library is the responsibility of the British Library Board.

The British Museum Library has long suffered from inadequate accommodation and the situation is becoming worse. Already some millions of books have to be housed 10 miles away at Woolwich, and the stock increases at the rate of just under two miles of shelving a year. Storage conditions are also generally unsatisfactory, lacking the controlled atmosphere and better environment that will be provided in the new building. The requirement for a new building was first formulated in principle in the County of London Development Plan in 1951. A site was designated for it in Bloomsbury and design proposals were put forward for new buildings to house the British Museum Library in 1964. Later, a larger building was designed to incorporate the National Reference Library of Science and Invention (now the Science Reference Library) and to provide a centre for the reference and bibliographic services of the newly-formed British Library. However, increasing difficulties arose in the way of realizing the plan to build in Bloomsbury because of the amount of redevelopment and disturbance involved and, following a feasibility study, the decision was made in 1975 to build the Library on the Euston Road site.

The site

The site comprises about 3.8 ha at the southern end of the block bounded by Euston Road on the south, Ossulston Street on the west, Phoenix Road on the north and Midland Road on the east, presenting a main frontage to Euston Road between the Shaw Theatre and St. Pancras Station. It is within walking distance of the British Museum and directly accessible from Euston, King's Cross and St. Pancras main line railway stations and their related Underground lines and bus routes.

The elevations of the boundary roads vary between 21m OD and 17.5m OD and the site is almost level. The buildings on the site were constructed for the Somers Town Goods Depot and the structures consist of heavy wrought iron frames supported on massive brick and concrete foundations. The buildings are currently used as warehouses and car parks.

Underground tunnels

A number of tunnels are located beneath or adjacent to the site as shown in Figs. 2 and 3.

The British Rail 'Midland Curve' tunnel passes the north-east corner of the site. The tunnel is an elliptical brick arch, with its crown at about + 14m OD.

The Circle and Metropolitan Lines run beneath Euston Road on the southern boundary of the site. At the south-west corner of the site the tunnel is an elliptical brick arch with its crown at about + 18m OD. Further east the construction consists of cast iron roof beams resting on brick columns where the tunnel widens into King's Cross tube station close to the south-east corner of the site.

The Northern Line passes east-west across the centre of the site. The two running tunnels are 4m in diameter, built in cast iron segments, with a crown level of about – 4m OD. The 'Piccadilly Switch,' under the eastern boundary of the site, connects the northbound tunnel to the Piccadilly Line. Before construction of the Victoria Line, alterations were made to the route of the northbound tunnel. This required a new section of tunnel which connects to the existing tunnel via a step plate junction under the

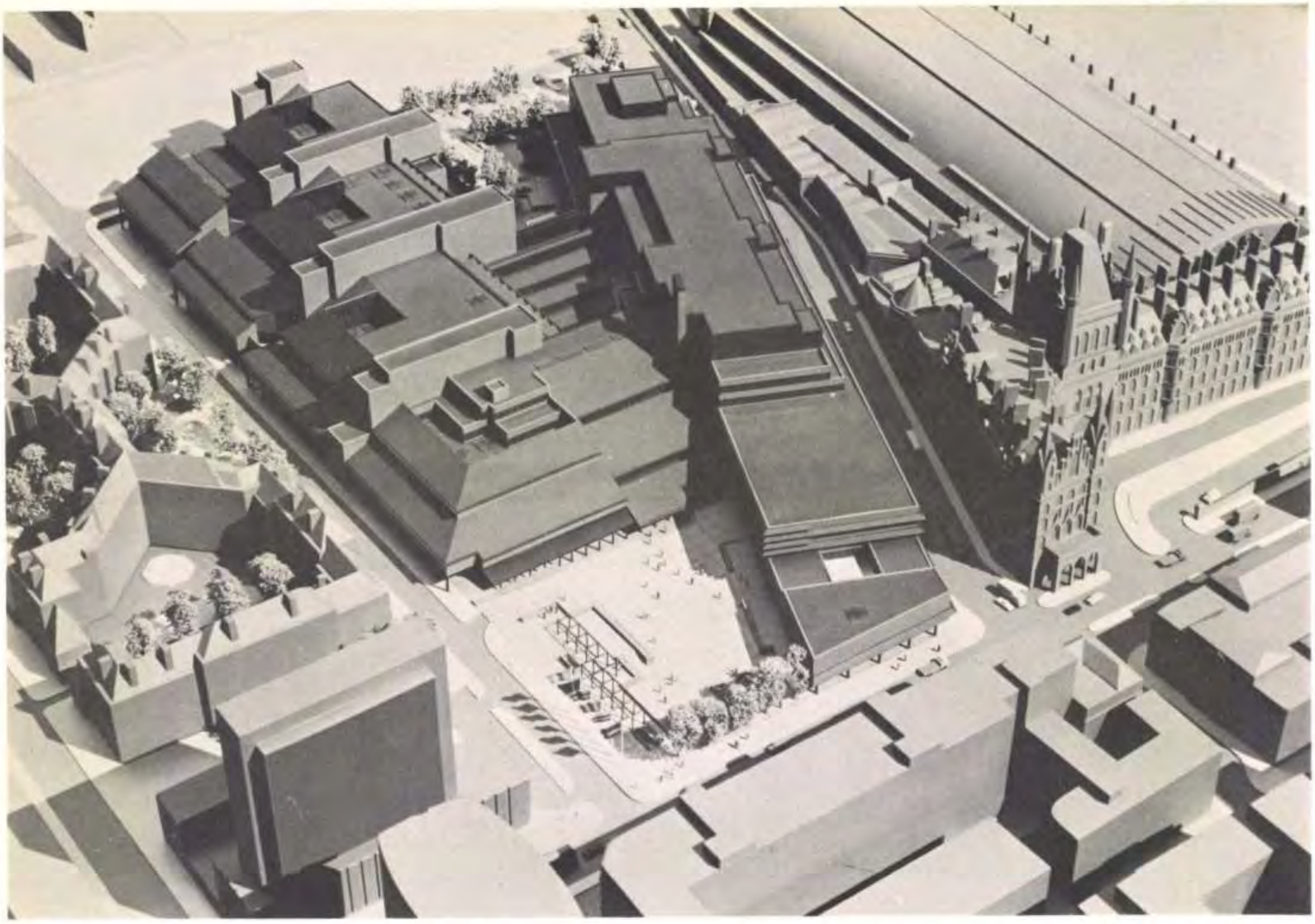


Fig. 1
Photograph of architect's model
(Copyright of Colin St. John Wilson and Partners)

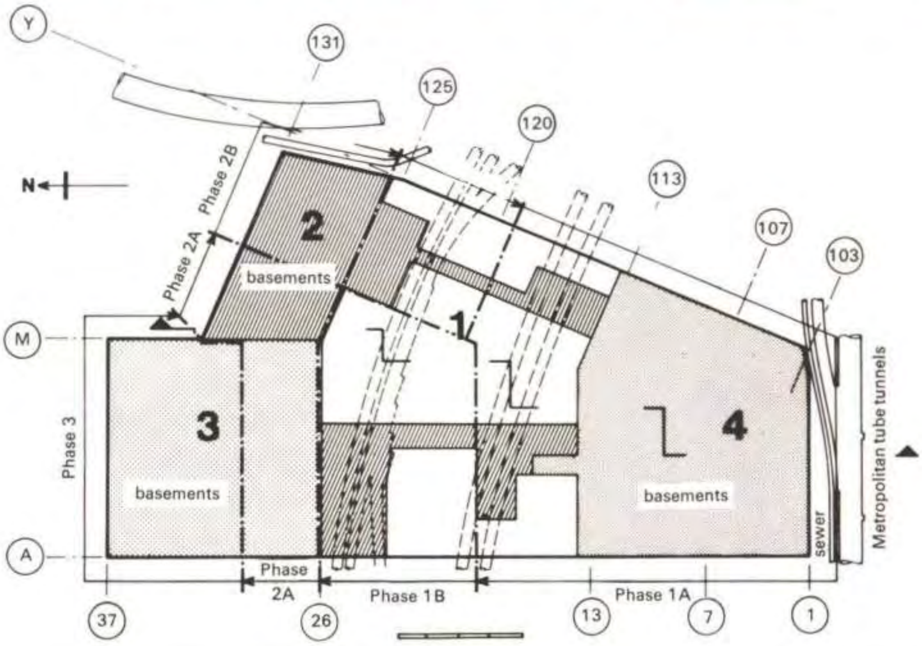


Fig. 2 right
Site plan

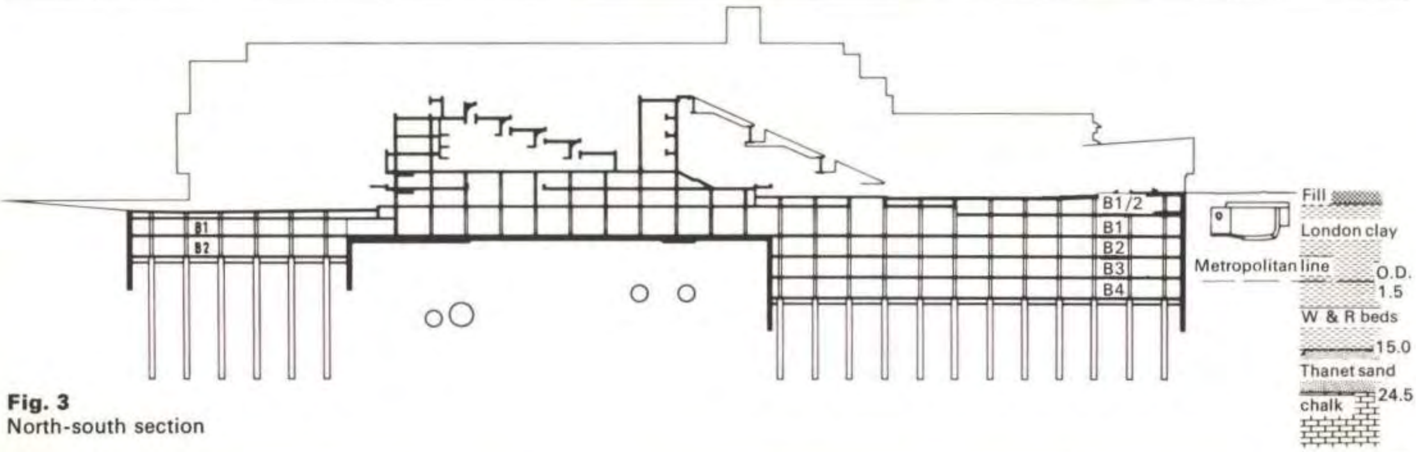


Fig. 3
North-south section

site. During construction of this junction, water bearing silty sand was encountered and it was necessary to stabilize the ground by grouting.

The Victoria Line also passes in an east-west direction beneath the southern part of the site. The two running tunnels are of 4m diameter with a crown level of about OD. The lining of the southbound tunnel is of cast iron, and that of the northbound tunnel is precast concrete. Silty sand was again encountered during construction.

Accommodation

The total floor area will be roughly 200,000m² and will ultimately house about 3,500 readers, 2,500 staff and 25 million books. In addition there will be large numbers of public visitors to exhibitions and restaurant areas as well as groups using the lecture and meeting room facilities.

Books will be stored either in open-access shelving which will be accessible to readers, or in closed-access areas to which only staff have access. All the closed-access storage, which amounts to roughly 30% of the total floor area, as well as most of the mechanical plant, is located below ground, thereby reducing the size of the superstructure to manageable proportions. Much of the closed-access storage will be in the form of mobile shelving to reduce the space required. Books will be delivered to the areas above by means of a mechanical handling system.

Above ground, the Science Reference Library, and the principal office areas, are located on the east side facing Midland Road. On the west side are the various British Museum Library reading rooms which are triple height with reading terraces stepping down beneath the sloping roofs above. The latter effectively reduce the scale of the building along the Ossulston Street frontage in keeping with the primarily residential nature of the area. Shared facilities including the entrance and catalogue halls are located in the centre of the building. A typical floor plan is shown in Fig. 4.

The main entrance will be from Euston Road across an open public forecourt (Fig. 5). The resulting set-back of the building will preserve the views of St. Pancras Station along Euston Road from the west. Access to the service areas and the small staff car park will be from the northern end, via Midland Road.

The building has been designed so that it can be constructed and occupied in phases as shown in Fig. 2. The first stage will be roughly 83,000m² of which about half will be below ground.

Building materials have been chosen to weather well and to relate to St. Pancras Station. The exterior will be largely brick with the window framing and cladding to ground level columns and roof level superstructures in bronze-coloured metal trim. The paving will be brick and stone and the sloping roofs will be natural slate. Materials on the interior will be somewhat softer, with carpeted floors and a certain amount of timber panelling in the reading rooms.

The building will be fully air-conditioned throughout in order to meet the very stringent environmental requirements for the conservation of the book material.

The structure

Above ground the structure generally consists of 400mm coffered slabs supported by columns on a 7.8m square grid. The columns are either reinforced concrete or cased steel sections. The latter enables the column sizes to be reduced to a size compatible with the bookstack widths, thereby significantly increasing the efficiency of the bookstack layout. Lateral stability is provided by the reinforced concrete walls around staircases and lifts. The entrance and catalogue hall roofs involve large spans, and prestressed concrete and steel truss schemes are currently being investigated.

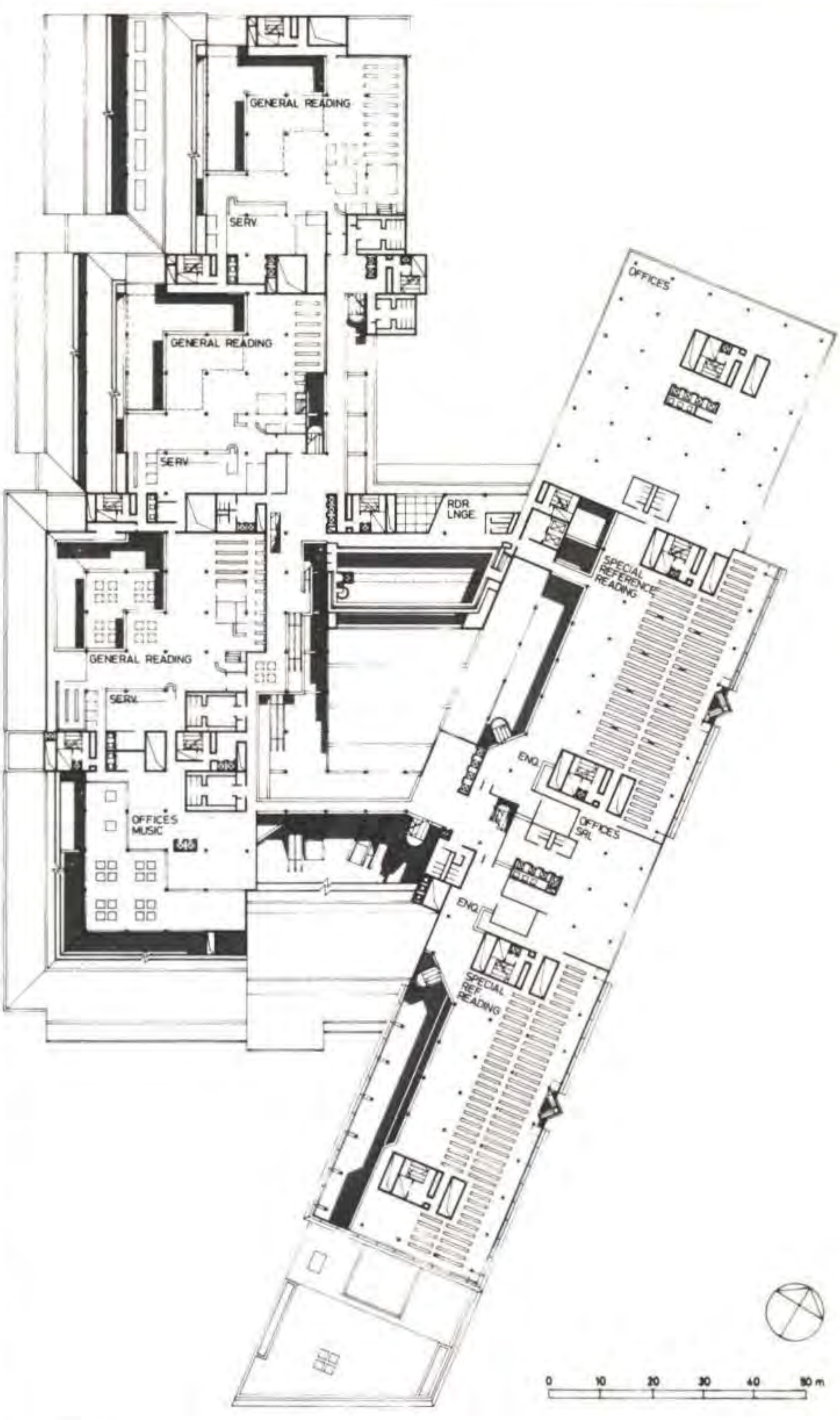


Fig. 4
3rd Floor plan

Below ground the floors are 400 mm thick solid slabs in order to carry the heavy loading from the mobile bookstacks together with the strutting forces to the retaining walls. In the deeper basements the columns are steel sections as required by the construction method described below.

In the south area there are four basements, B4 level being approximately 23 m below ground level and the columns are supported by bored piles founded in the Thanet Sands. The alternative of founding in the Woolwich and Reading Beds was also considered, but it was found that large underreams would be required to provide the required bearing capacity and, in view of the presence of fissures and possibly water-bearing sand lenses, it is considered that such underreams cannot be constructed safely and reliably in this stratum.

The B4 slab is suspended and underneath there is an undercroft which will allow heave of the soil below to take place. Excavation is therefore required down to approximately 24 m below ground level (−5 m OD) and 4 m into the Woolwich and Reading Beds. The retaining walls are 1 m thick diaphragm walls which are supported horizontally by the 400 mm thick solid floor slabs.

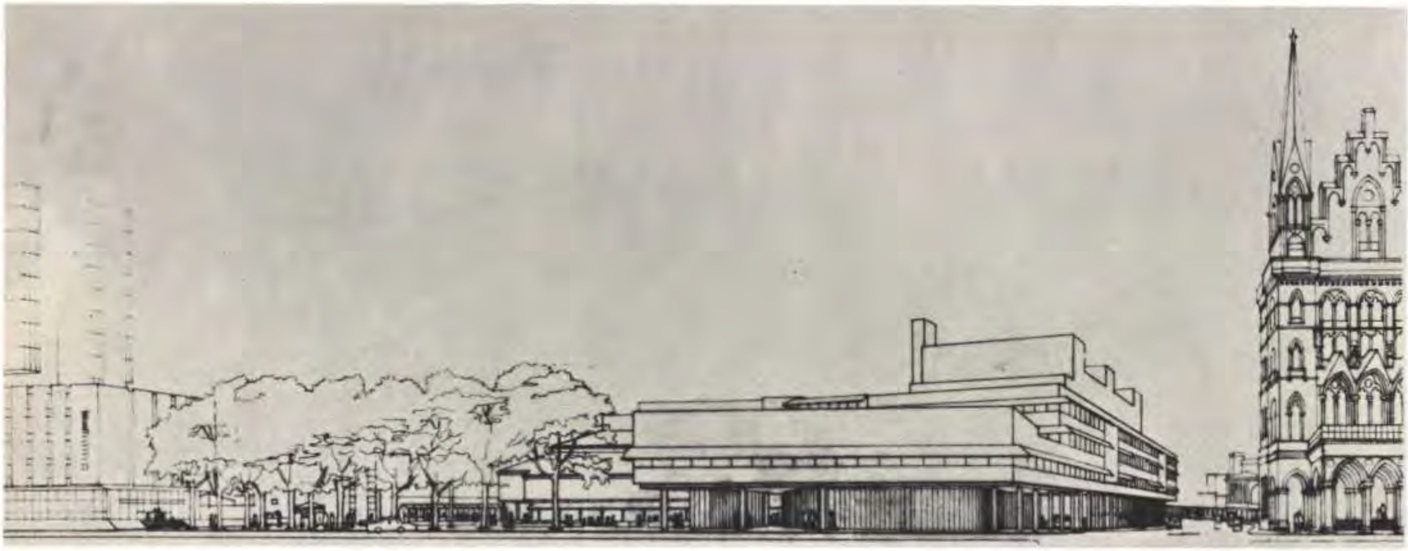


Fig. 5
View from south (Courtesy of Colin St. John Wilson and Partners)

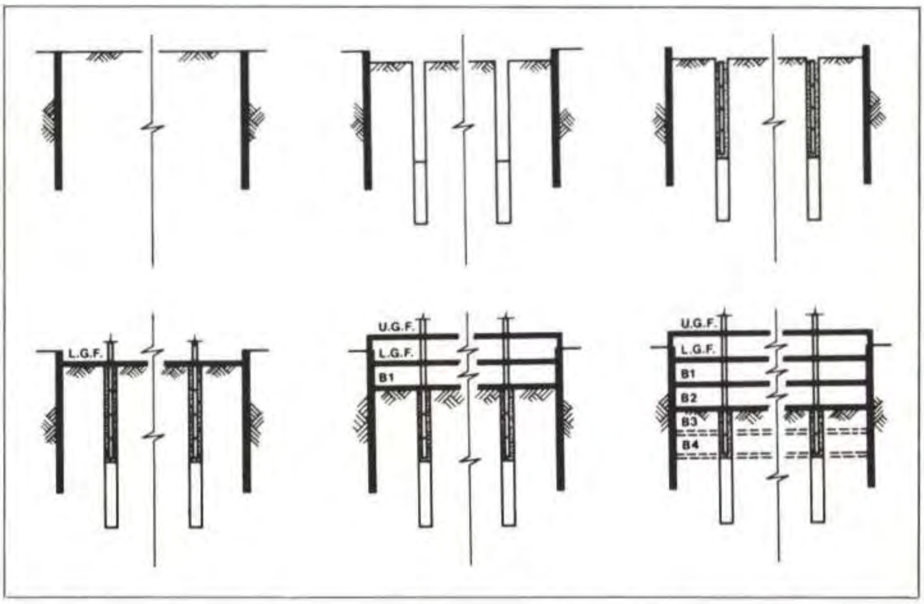
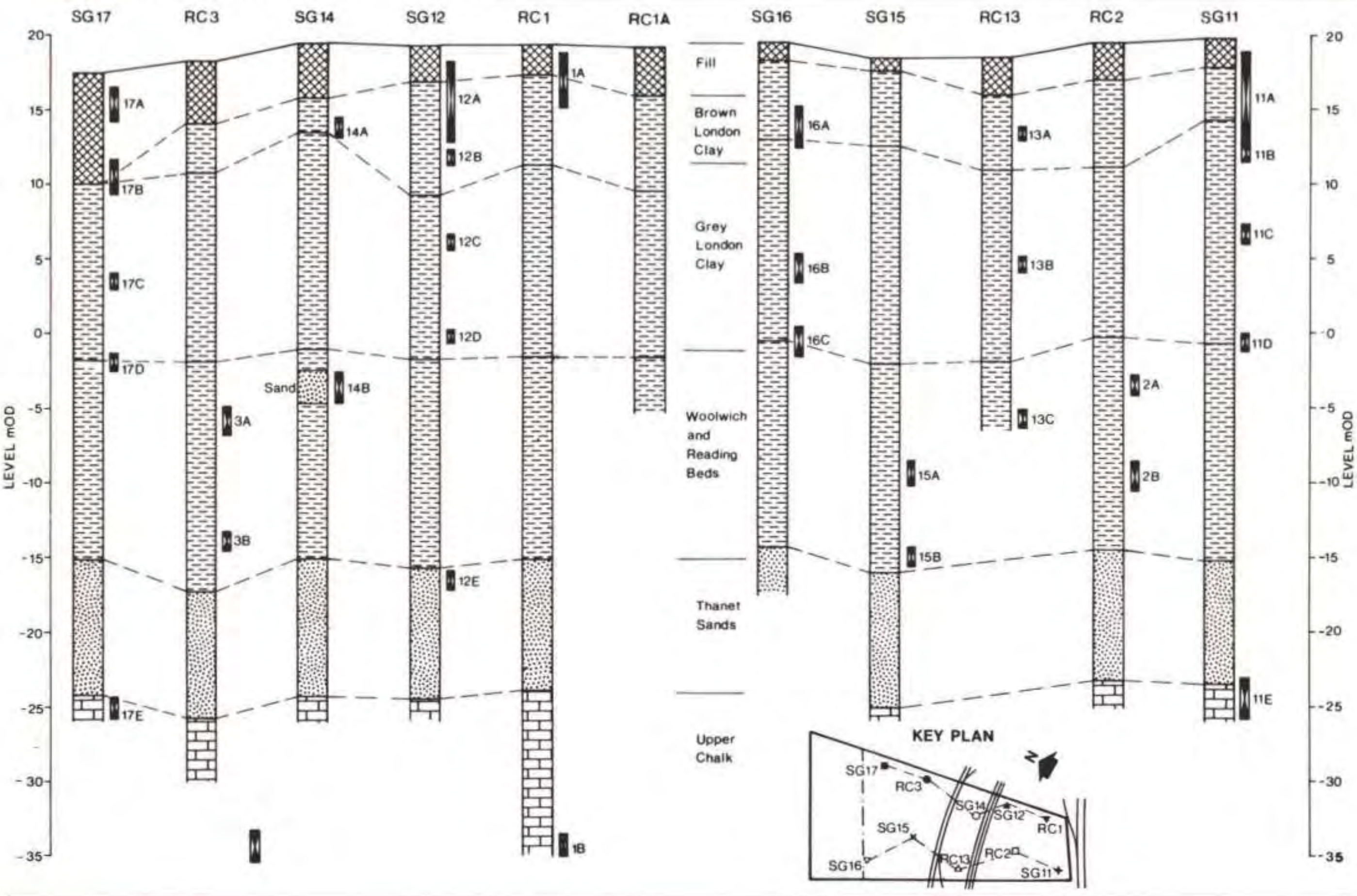


Fig. 6
Construction sequence for deep basements

Fig. 7
Geological succession in boreholes



The proposed construction method for the basements (Fig. 6) which has been developed in order to minimize ground movements, is as follows:

(i) Temporary walls will first be installed to allow the removal of the existing foundations.

(ii) The diaphragm walls and bored piling will then be installed.

(iii) Steel columns will be erected in the pile shafts.

(iv) B½ slab will be constructed. This will be designed to span horizontally E-W across the site to provide support to the south wall.

(v) Excavation will be carried out to B1 level, and B1 slab cast.

(vi) Excavation will be carried out to B2 level, and B2 slab cast. This process will be repeated for levels B3 and B4.

(vii) Once B1 level is complete, construction of the lower ground floor and superstructure will proceed in parallel with the operations below ground.

An alternative sequence, in which the lower ground floor is cast after the B½ slab but before B1, is also currently being investigated.

In the central area, the lowest level is generally at B1 level, but goes down to B2 level in places and the foundation in this area is a continuous raft. The diaphragm wall is supported laterally by the lower ground floor and by the raft. The proposed construction method involves open excavation down to Basement 1 level. However, an alternative method, whereby the lower ground floor will be constructed first, is also being investigated.

At the north end the lowest basement levels are at either B2 or B3 level. The structural system is as described for the South Area and it is currently proposed that the construction method should be similar. However, it is possible that this might be modified in the light of experience gained from the construction of the South Area.

Site investigation

A fairly conventional preliminary site investigation was carried out during 1975 and 1976. Six boreholes were sunk using soft ground boring equipment to depths between 37 and 45 m. Undisturbed and disturbed samples were obtained and standard penetration tests were carried out. Five boreholes were also sunk using wireline rotary core techniques to obtain 75 mm continuous cores to depths of 24.5 to 53.4 m and a further 17 boreholes were sunk to permit installation of 27 piezometers and four standpipes.

The levels of the various strata are reasonably constant across the site (Fig. 7) except where one of the boreholes intersected fill alongside the Midland Curve tunnel. The lens of silty sand which was encountered during construction of the Northern and Victoria Line tunnels was also found in one borehole.

The ground water level is close to the top of the London Clay and the water pressure in the London Clay and Woolwich and Reading Beds is about 60% of the hydrostatic pressure from a level of +17 m OD decreasing rather abruptly to zero close to the interface with the Thanet Sands. This piezometric profile is typical of sites in London and is caused by the pumping of water from the chalk. It is likely that the water table in the chalk is about 45 m below the top of the chalk and is unlikely to change significantly in the foreseeable future.

Construction programme

The overall programme for the first stage is as follows:

Dec 1978 – Mar 1979	Trial pile installation and test
Apr 1979 – Dec 1979	Demolition
Oct 1979 – Mar 1981	Diaphragm walls and piling
July 1981 – Oct 1984	Excavation, basement and superstructure
July 1984 – June 1989	Services and finishes

Conclusion

The British Library project has been described in general terms. It is intended that future articles will cover various aspects in greater detail.

Credits

Client:
The Department of the Environment

Architect:
Colin St. John Wilson and Partners

Services engineer:
Steensen, Varming, Mulcahy and Partners

Quantity surveyor:
Davis Belfield and Everest

Guildhall School of Music and Drama

Alan Steele

The Guildhall School of Music and Drama had always been included in that part of the Barbican redevelopment designated the Arts Centre. In the original design it was relatively small and, with the glass dome from the Coal Exchange as a centre piece on its roof, was positioned at the focus of the C-shaped terraced block of flats.

During the evolution of the design of the Arts Centre it became apparent that the GSMD too would have to be considerably increased in volume and, therefore, relocated. The space available for the Arts Centre and the remaining parts of Phase 5 of the development was very limited and the School was eventually placed at the eastern end of this final Phase. It is bounded by the Theatre to the west; the realigned railway tunnel to the south; the main access road to the Arts Centre – Silk Street – to the north; and to the east, by Speed House (one of the

1

2

blocks of flats constructed in Phase 3) and an underground car park to the south of that block.

As its name implies the School is in two parts, a Music School and a Drama School. Although some facilities are shared the design recognizes these two distinctly different functions.

The Music School consists of a number of individual practice studios on three levels spaced around three sides of the Music Hall which is about 24m × 14m × 9m high and there are recording studios under the Hall. An east/west expansion joint separates the Drama School from the rest of the building and the space between this joint and the Music School has an access road (to the car park east of the school); main foyers and, at the upper levels, the library.

Under the Drama School there is a plant room (about 10 m below street level) and this part of the building consists of a theatre, (32m × 21m) with six large (10.5m × 7m) movement studios above the auditorium; a lecture theatre and changing rooms to the west; administration offices to the east; and gymnasium, scenery dock and other specialist rooms to the north.

Acoustics

An essential feature of the music studios is the need to isolate them acoustically from the structure and each other. To achieve this they were designed as boxes with concrete floors, stabilized brick walls and, because of constructional problems, the roofs were formed of a lightweight decking with an acoustic blanket. Each box is supported on four Andre rubber bearings on the main frames.

On the south elevation the studios are located over the railway tunnel which was not able to support the additional load so twin beams which cantilevered from the Hall floor were separated from the railway structure to avoid the transmission of noise and vibration. The Hall floor is of very heavy construction because the Hall itself could not, during construction, provide the necessary counterbalancing weight for the cantilevered studios. The Hall is formed of six portal frames between which services and roof lights are located.

The theatre and movement studios are in a concrete box supported on six columns and here the most interesting structural problems stemmed from the need to have openings into the various parts at positions which lead to very high tensile forces and shear stresses being developed.

Fig. 1
The south elevation. The barrel vaults of the library roof with pergolas can be seen. The lakeside terrace is very popular with the students for lunchtime picnics
(Photo: Harry Sowden)

Fig. 2
Looking south from under Speed House. The administrative offices are on the right. The entrance to the link between the two parts of the School is at the centre
(Photo: Harry Sowden)

Fig. 3
Looking east at the gardens south of Speed House. The stained glass is an important part of the treatment of all the glass elevations
(Photo: Harry Sowden)

3

5

Fig. 5
Looking up at the grid in the flytower. Some of the counter balance weights (the stacks of small blocks) can be seen at the right centre (Photo: Harry Sowden)

4

Fig. 4
The link between the two parts. The Music Hall is on the right, the Theatre on the left, but,probably more important,the bar is down the stairs. The cross-over at the far end is from where Fig. 3 was taken (Photo: Harry Sowden)

7

6

Fig. 6
The barrel vaulted library extension. (The student is making notes in Mandarin whilst listening to Mozart's Piano Concerto No. 23 K488) (Photo: Harry Sowden)

Fig. 7
Detail of Music Practice Studios. The spaces for the Andre rubber bearings are clearly shown (Photo: Harry Sowden)

8

Fig. 8
Detail of Music Practice Studios (Photo: Harry Sowden)

Fig. 9
Outside the Music Hall (Lakeside Terrace level). The back of a music practice studio is at the top. The seats, of reinforced concrete, contain heating coils (Photo: Harry Sowden)

Fig. 10
Inside the Music Hall. The profile of the portal frame can be seen (Photo: Harry Sowden)

Fig. 11
Inside the Music Hall. A portal frame with the spaces for services each side (Photo: Harry Sowden)

Fig. 12
Outside the Music Hall (upper level). The finely detailed and built brick walls of the Practice Studios are shown.
The surface of the wall to the Music Hall is bush hammered (Photo: Harry Sowden)

11

9

10

12

The podium to the north of the flytower is to be part of the general circulation space of the development as a whole, whilst that to the south is for use of the students as is the lakeside terrace over the railway tunnel. The conservatory which surrounds the Arts Centre Theatre is extended eastwards to link with the small conservatory over the GSMD flytower.

During the building of the school it became apparent that the library was too small so an area of the original roof was allocated for an extension and barrel vault roofs were added which reflected the penthouse roof design in the earlier terrace blocks. The reanalysis of the forces induced in the existing structure to ensure its capability to carry the extra loads fortunately showed that little modification was necessary.

The concrete wall surfaces are generally bush-hammered; brick is used extensively on the external elevations; brick paviors in the circulation areas; wood block floors in the larger rooms and halls, but the music practice studios have carpeted floors.

As a structure I believe it to be successful. Comment on its success as a building will have to await the occupation of the Concert Hall and the Theatre in the Arts Centre but, if the music I heard being performed there the other day, and the drama students with whom I spoke, are typical of the quality being produced at the Guildhall then, at least, it can be said that it has all been worthwhile.

Credits

Client:
The Mayor and Commonalty and Citizens of the City of London
Architect:
Chamberlin, Powell & Bon
Quantity surveyor:
Davis, Belfield and Everest
Services engineer:
G. H. Buckle & Partners
Main contractor:
John Laing Construction Ltd.

Coventry Point

Ernest Irwin
Malcolm Jordan

Introduction

The completion of the twin towers of Coventry Point marked the final step in the redevelopment of the shopping precinct area of central Coventry which was first conceived some 30 years earlier out of the blitzed ruins of the mediaeval city. The developer won the competition to erect office buildings on this remaining prime site through the bold solution of setting one of the towers in the centre of the pedestrian precinct itself.

We first learned about the job when Derek Davis of the John Madin Design Group casually asked what was the minimum number of columns necessary to support a 13 storey building some 10 m above ground. We were tempted to say 'four' or even 'three'. Only after looking in detail at the problem did we concede that two columns in the shape of two single lift shafts would be sufficient.

We heard nothing more for a long time until one day the telephone rang and we were asked to design the structure of this office development by standing one of the blocks on a single line of supports along the axis of the precinct.

There were in fact more than the usual number of interesting problems for a job of this kind. The foundations were unusual, the site location greatly influenced the choice of structure, there was to be a wind canopy around the building to eliminate downdraughts, and the construction was to overhang the pedestrians and the vehicular access in the precinct without impediment to either.

For clarity, we have chosen to describe separately the following interacting aspects of the job:

Site location, structure foundations, construction, and wind canopy.

Site location

In the rebuilding of Coventry following the enormous devastation in the Blitz, a new pedestrian shopping precinct was planned in the centre of the city It has two principal shopping avenues intersecting at right angles in the form of a cross Behind the shops in the quadrants of the cross are multi-storey car parks and a retail market Rear access service roads are fed from the surrounding road system.

As part of this plan, the spire of the bombed-out cathedral forms a landmark on the east end of the major axis of the cross. Modern tower blocks have been positioned at the other end of this major axis and at the north end of the minor axis Consequently, a site remained at the south end of this axis, where a further tower block was planned to complete the disposition of tall new buildings in central Coventry.

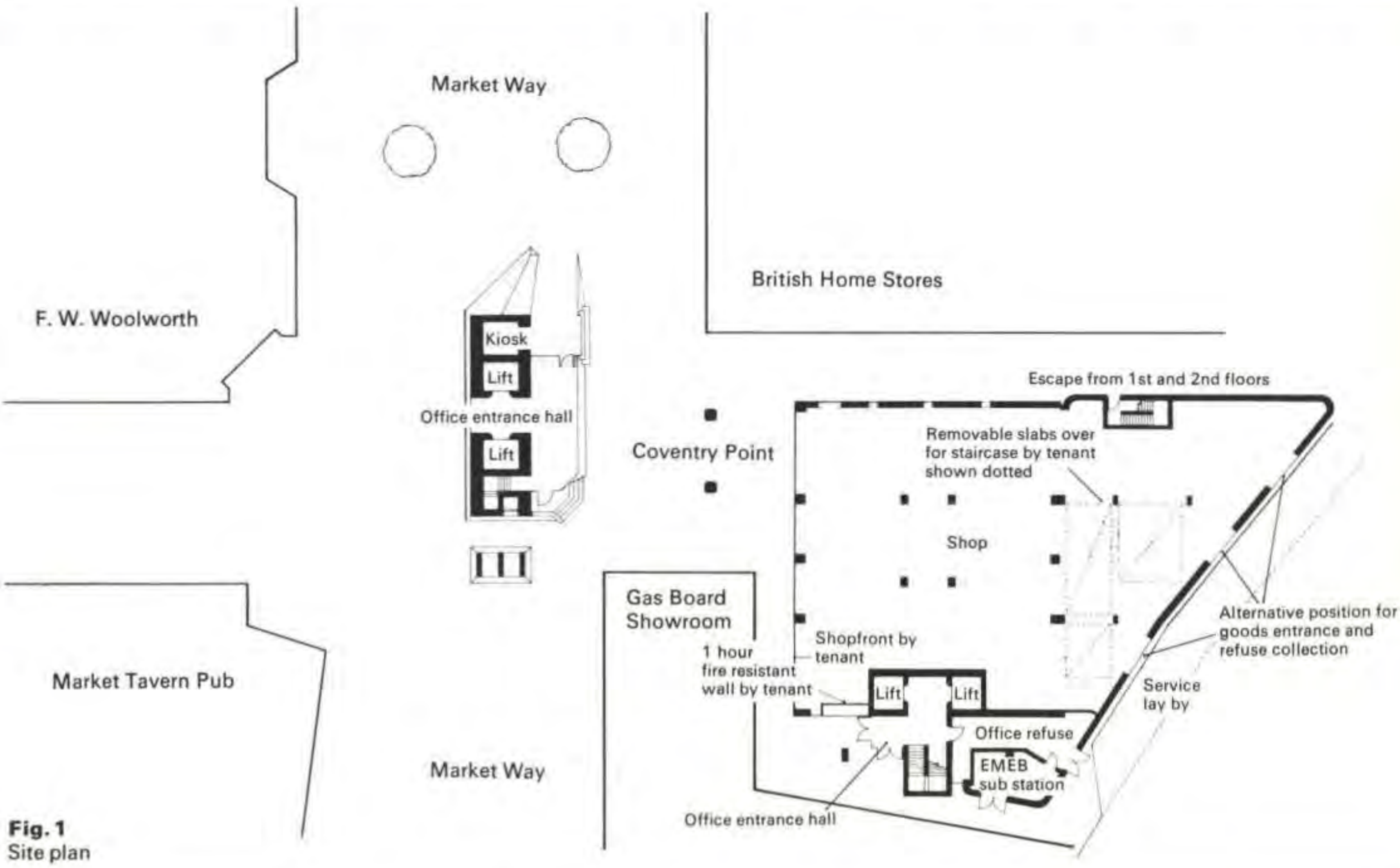


Fig. 1
Site plan

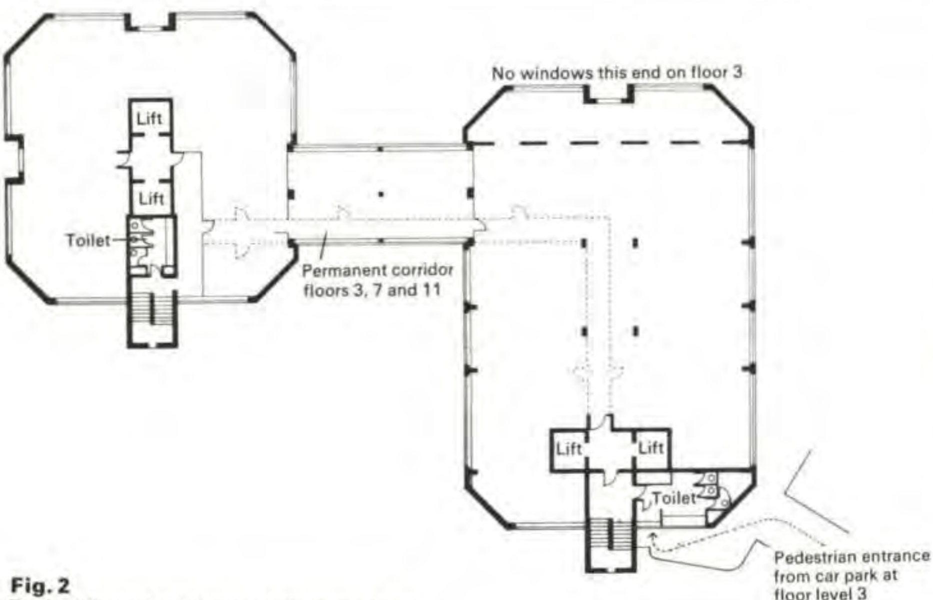


Fig. 2
Typical floor plan in precast superstructure

Adjoining the vacant position in the centre of the pedestrian way was a small cleared site for shops and the City Council invited developers to put forward solutions for mixed shops and offices which would provide a suitable landmark on this minor axis. They made it clear that the pedestrian way could not be blocked because access was required for emergency vehicles at all times.

Access for construction was confined to the rear of the site on a narrow street bounded on its far side by a multi-storey car park to which the new office development would be linked.

Clearly, considerations of cranage, lack of storage area, the need for shoppers and emergency vehicles to continue using the pedestrian way, and the narrow street rear access, were all major constraints on the design.

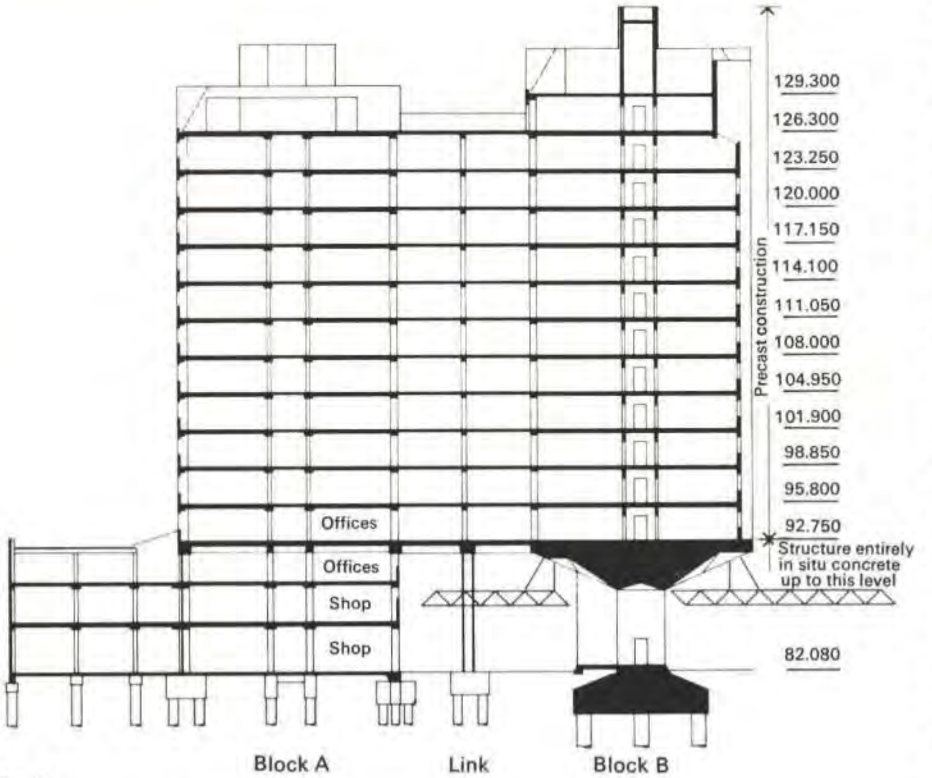


Fig. 3
Building cross-section

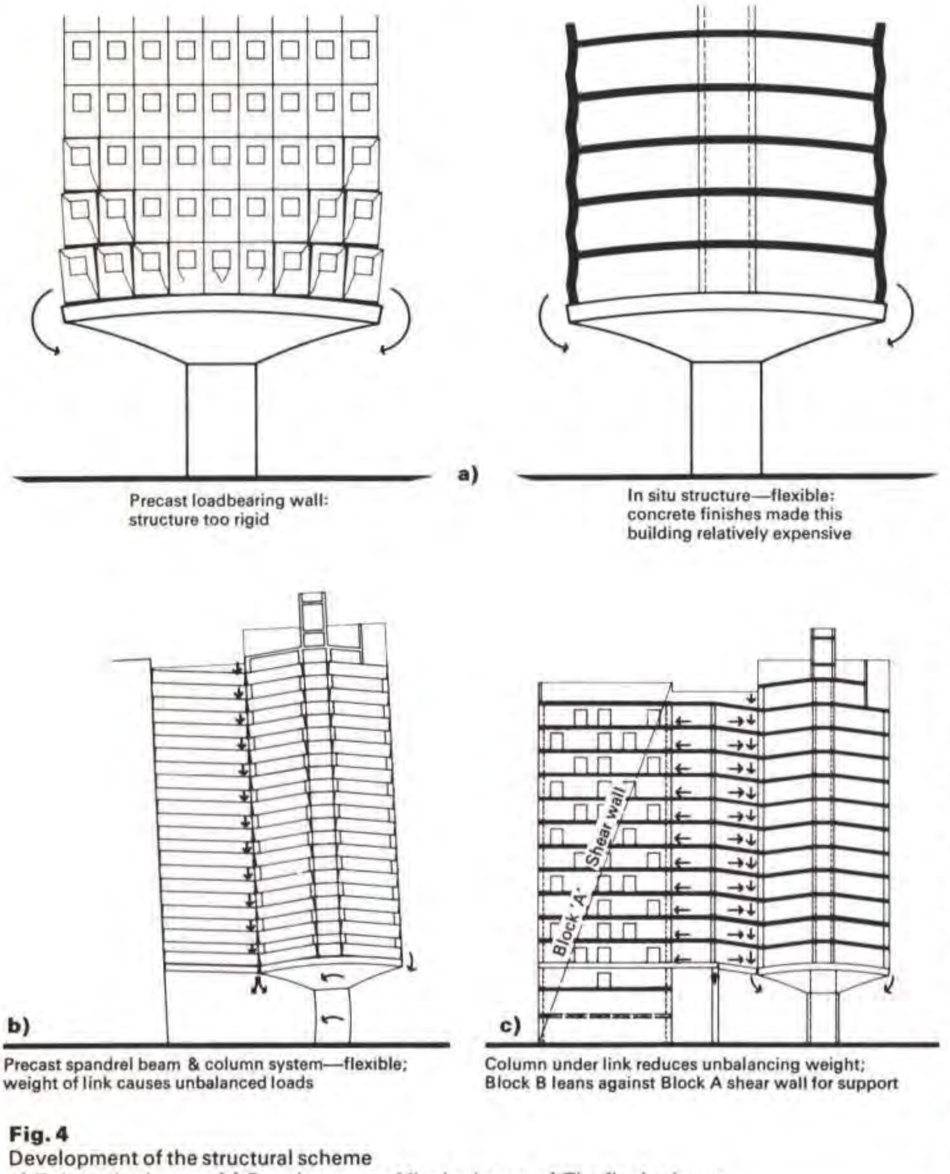


Fig. 4
Development of the structural scheme
a) Rejected schemes. **b)** Development of final scheme. **c)** The final scheme

The structure

Coventry City Council had produced a detailed design and planning specification for the development which included a height requirement, a white external finish, an 'area of brightness' in the precinct beneath the building, a deflector for wind downdraughts, a minimal restriction on pedestrian and service vehicle access and a minimal restriction on the visual aspect along the precinct to the shop fronts. In addition to offices, the development had to include an entertainment centre at high level and two-storey shop units at ground level (see Figs. 1 and 2).

The architect, seeing the need to reduce the apparent mass of a tower building in this location while optimizing the floor area achievable on the site, chose twin towers connected by a 'light' glass-curtain walled link-block. He further reduced the apparent width of the towers by chamfering the corners, which also had structural advantages. The overall width of each tower block was 19 m.

The complete development, therefore, took the form of twin linked towers, one of which comprised 11 storeys of offices above three storeys of shops (Block A) and the other 12 storeys of offices with the lowest storey commencing at Level 4 (Block B). The towers are linked at each level above Level 4 (see Fig. 3).

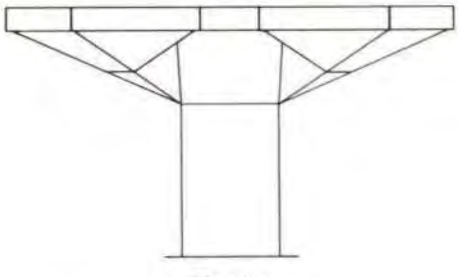
To minimize any restriction to pedestrian flow in the shopping precinct, the higher of the two blocks, known as Block B, is balanced on its centrally placed lift cores alone (see Fig. 3). The weight of Block B is transferred from the outside walls to the lift core walls at third floor level by a 2000 tonne reinforced concrete double cantilever slab in the form of a mushroom head tapering from 4 m deep at the root to 1 m deep at the perimeter. A pile cap of similar proportions exists below ground to transfer the weight of the building from the central cores to the piled foundations. The lift shafts dividing these two elements are only 4 m wide, resulting in a height to width proportion of about 14 to 1. Lateral stability had, therefore, to be achieved by tying the two tower blocks together through the floors of the link block. The relative position of the lift cores meant that there were no difficulties in achieving stability along the axis of the precinct.

Precast system

Because of the difficulties of access, the contractor/developer was very keen to use a precast concrete superstructure. This would mean that all structure could be manufactured off site, and a crane located between Blocks A & B could lift all materials from trucks in the service road directly to their final location. The contractor proposed a Bison wall frame system but we were apprehensive that the rigidity of such a construction could not satisfactorily accommodate the differential deflections anticipated in the double cantilever (see Fig. 4a). The architect's decision to make a 3 m chamfer at the four corners was nevertheless of great benefit in reducing the relative vertical deflections.

Our preference was for an in situ concrete frame having the necessary flexibility and strength and which we felt would also suit the fairly irregular plan shape. However, the construction difficulties were such that our client instructed us to find some way of using a precast system. With the co-operation of Concrete (Midlands) Ltd. we finally arrived at a solution utilizing wall columns and spandrel beams, with the connections designed to accommodate the anticipated deflections of the double cantilevers (see Figs. 4b and 4c).

The link block connecting the towers was not merely a passage, but provided office floor space in its own right. It is 11 m long × 6 m wide, and consequently, without intermediate



Elevation

Fig. 5
Facets of the double cantilever support

Fig. 6 right
The completed double cantilever with falsework partly removed (Photo: The John Madin Design Group)

support, it caused a huge unbalancing load on one edge of the double cantilever (see Fig. 4b).

The planning authorities were unwilling to allow intermediate support but we consistently made the point to the architect and our client that two columns under the link would be outside the main pedestrian way and would make the structure much more economical and sensible. Finally, we won the day and two columns were permitted in the position shown in Figs. 1 and 4c.

Block A, which was to provide the transverse stiffness both for itself and Block B, was not without its problems. An open plan office space was required, yet at the ends of the building it cantilevered 3.5 m beyond the three storeys of shopping space. Obviously, shear walls at the ends of the cantilevers would be ineffective. The lift core and staircase positioned at the end of the building remote from the link block were, of course, inadequate because of their size and position. After much debate, it was found possible to introduce two coupled shear walls on the line of one side of the link block with additional doorways staggered on alternate storeys (see Fig. 4c).

Stability

Wind resistance and stability of the buildings were achieved by making use of these coupled shear walls together with the core at the other end of Block A. However, analysis showed that under certain conditions vertical tensions would develop near the top of the building in some of the walls. This was catered for by continuous reinforcement in the in situ stitches between wall units. Continuous reinforcement in the in situ joints of the floor units was also necessary to cope with the forces developed between the cores of the coupled shear wall systems.

While the analysis and design was made complex in order to achieve stability without increasing the number of walls, the additional site work in providing continuous reinforcement in the in situ stitches and joints had no significant effect upon erection time or cost.

The 'white' finish to the building was achieved by using a 40 mm size calcined flint aggregate exposed for a depth of 12 mm, which was as deep as could be achieved without risk of some stones becoming dislodged.

Precast construction was clearly not possible for the double cantilever and its supports beneath Block B. Consequently, all construction up to Level 4 in both buildings was designed in in situ reinforced concrete. This was seen as the best means of catering for the large horizontal thrusts developed near the base of the shear walls and was also suitable construction for shop loading.

The project architect, Douglas Hickman, envisaged a sculptured form to the massive double cantilevers and in the early days various alterations to the shape of the cantilever, together with slight adjustments to the plan form, took place as the triangular faceted surface evolved. The cantilevers as a major exposed structural element had to be integrated within the total visual concept. Relating their slope to the chamfered and notched floor plan led to a system of triangular planes giving the many-faceted surface which concentrated the depth of structure, and hence the strength and stiffness, where it was most needed (see Figs. 5, 6 and 7).

With such a vast volume of concrete within the double cantilever and its supporting core and foundations, the use of lightweight concrete was examined. Unfortunately its price in the Midlands at that time was much greater than ordinary concrete and, as no saving could be seen, the idea was abandoned.

In order to match the massive scale of the structural elements at low level, a system of large recessed joints and bevelled corners was evolved for the structural details and these have been very successful in ease of construction and appearance (see Figs. 8 & 9). Perhaps such treatment of construction joints and arrises in exposed structural concrete would be more successful if it were always as bold as in this case.

The architect chose a bush-hammered finish for the exposed in situ concrete even though the use of readymixed concrete increased the risk of inconsistency. The problems were fully discussed with the readymix supplier and the results show very good quality concrete.

The need to minimize costs was a continual problem because obviously the double cantilever supporting structure was expensive.

Fig. 7
The completed double cantilever showing access beneath for pedestrians and service vehicles
(Photo: The John Madin Design Group)

Superimposed loadings on floors were, therefore, kept to the British Standard for office spaces except on certain storeys where computers or other activities might require larger loadings.

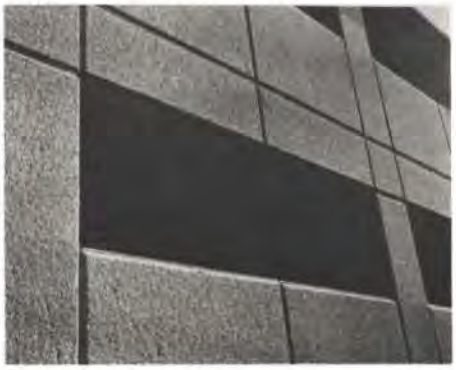


Fig. 8
Bush hammered in situ concrete on the shop face of Block A, showing the deeply recessed feature joints
(Photo: ICA Studios Ltd.)

Fig. 9
Bush hammered in situ concrete of the link support columns. Recessed feature joints correspond to the levels of those of Block A and B structures (Photo: ICA Studios Ltd.)

Foundations

Block B presented us with the problem of how to support 5000 tonnes of building on the variable ground at Coventry within the restricted area of the precinct while still allowing the public and service vehicles through it. From examination of records we knew we were very close to a filled stream bed. Investigations revealed debris and fill down to a 3 m depth, below which there were sands and silty clays to 5.5 m. Below this were bands of weak to moderately strong sandstone. Piles bored to refusal on the strong sandstones, and carrying no more than 2000 kN/m² in end bearing, had been a frequent solution in Coventry. However, such piles had been known to fail when stopped on top of very hard but very thin bands of sandstone. In any case there was insufficient space for piles of such capacity.

The solution proposed by our Geotechnical group was 'rock sockets' at 5000 kN/m² – the concrete capacity. These bored piles were carried to a predetermined depth into the rock strata in order to distribute the load (see Fig. 10).

The usual contract clause offered by piling contractors of boring to the design depth or 'to refusal' could not be accepted, and some laborious chiselling was needed before the hardest of the sandstone strata could be penetrated. Once the contractor fully appreciated the importance of achieving the designed depth into the sandstones, the construction of the piles proceeded smoothly if somewhat noisily.

Construction

The contractor chose to use a single tower crane located between Blocks A & B to serve the construction of the two buildings. This crane could pick up materials from trucks on the service road and deliver to the furthest extremity of the site. However, it was only possible to operate one crane and this determined the speed at which the building could be constructed. Very careful scheduling of the arrival of materials, including the many precast units, was therefore necessary.

The space available for the crane base was extremely limited so this too was placed on 'rock socket' piles similar to those used for the building foundations.

The construction of the in situ double cantilever slab presented the contractor with special problems. While he could restrict the pedestrian way to some extent, he nevertheless had to allow access for service vehicles, particularly in case of emergency at all times. He considered constructing a special scaffolding with openings through for pedestrians and for service vehicles, and one day, while discussing the difficulties with him, we suggested hanging formwork from a steel structure which might be encased within the concrete cantilevers. Shortly he returned to this idea and asked us to design this falsework.

We found it impracticable, however, to design steelwork which would be later encased due to the restrictions of the faceted cantilever. As an alternative, we suggested steel suspension towers built on top of the liftshafts from which would hang trusses supporting a level timber deck on steel joist bearers. The formwork would then be built up from this deck. The contractor grasped the idea enthusiastically and we very quickly designed the structural steel framework shown in Figs. 11 & 12.

When this was erected and the bottom platform constructed, completely free access for pedestrians and vehicles was available underneath. The plywood formwork was supported on a subsidiary timber falsework system propped from the level deck. When this formwork was being constructed, it looked like an inverted hip roof as can be seen in Fig. 13.

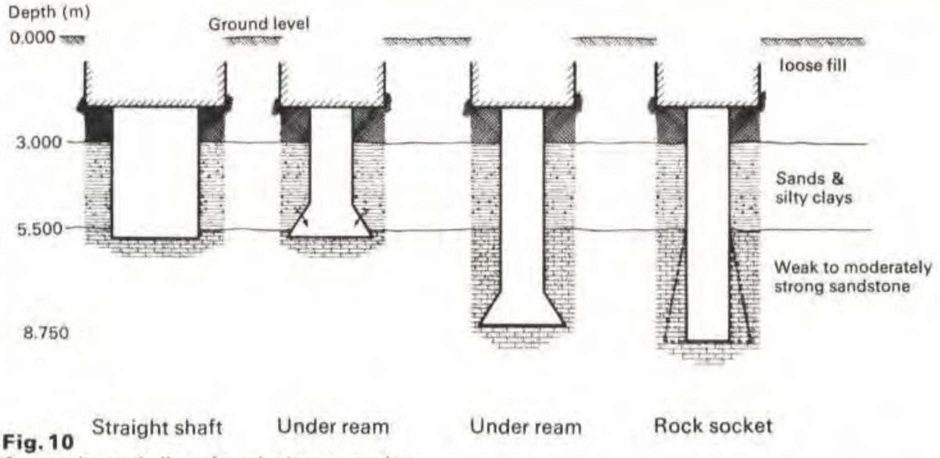


Fig. 10
Comparison of piles of equivalent capacity

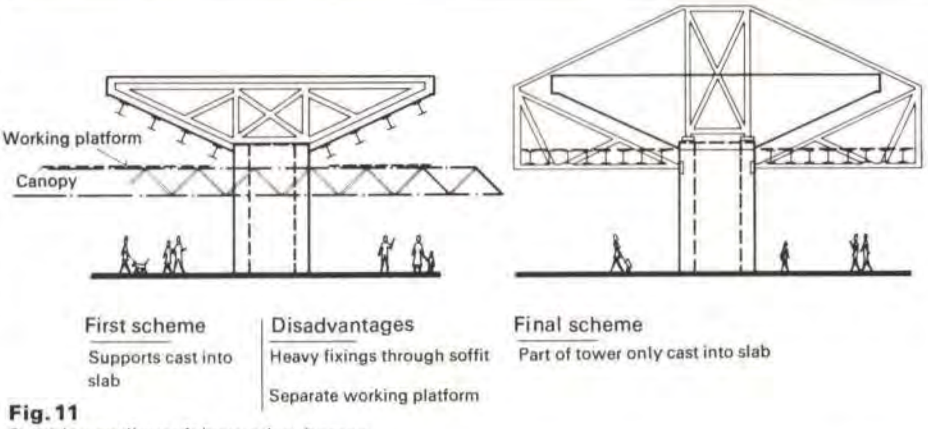


Fig. 11
Double cantilever falsework schemes

Fig. 12
Aerial view of falsework with working platform under construction (Photo: ICA Studios Ltd.)

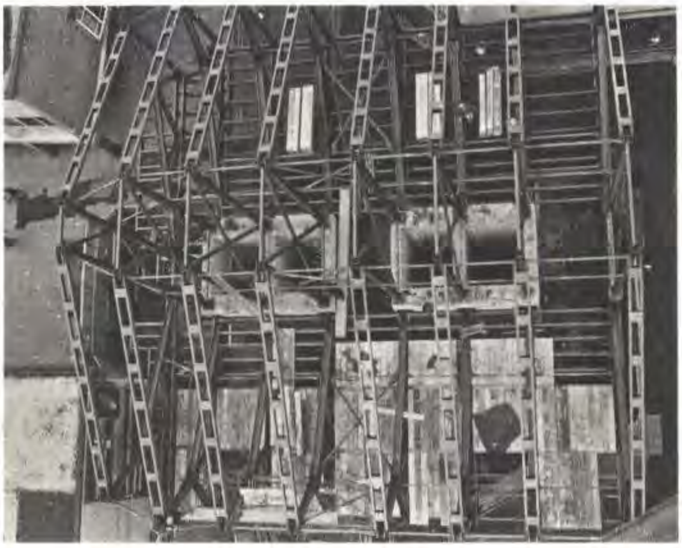


Fig. 13
Aerial view of falsework ready to accept plywood forms. The inverted 'hip roof' form can be clearly seen (Photo: ICA Studios Ltd.)

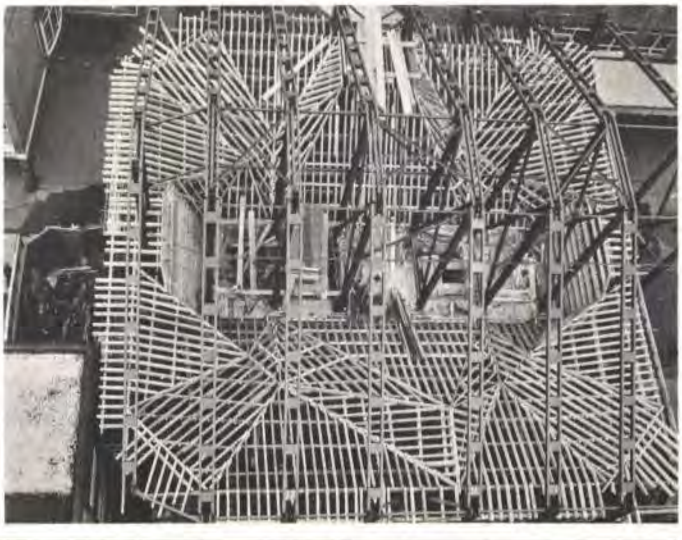


Fig. 14
Erection of precast superstructure
(Photo: ICA Studios Ltd.)

Concrete elements of the size of the pilecap underneath Block B and the double cantilever slab constitute large concrete pours. We, therefore, consulted Michael Fitzgibbon of the Cement and Concrete Association and Dr. Barry Hughes of Birmingham University who advised us on the pour sizes and the use of anti-cracking reinforcement, bearing in mind the mix proportions and the degree of insulation offered by the formwork. The double cantilever was placed in three stages, the initial pour being halfway up the sloping soffit where a feature recess was introduced to mask the joint.

The second pour completed the sloping soffit and the third pour, containing most of the reinforcement, completed the top 800 mm depth. While this work was going on, precast concrete units were being produced by Concrete (Midlands) Ltd. at their works in Lichfield for the entirely precast construction above the third floor.

Once the in situ podium slab had been completed, erection of the precast structure proceeded rapidly using the single tower crane (see Fig. 14). The site is entirely filled by the buildings and so precast units had to be delivered on the adjacent service road and lifted directly into place. There was no room for on-site storage.

All exposed aggregate precast units were examined by covermeter at the manufacturer's yard in order to ensure in advance that we got the required cover to reinforcement. After erection they were in a most inaccessible position.

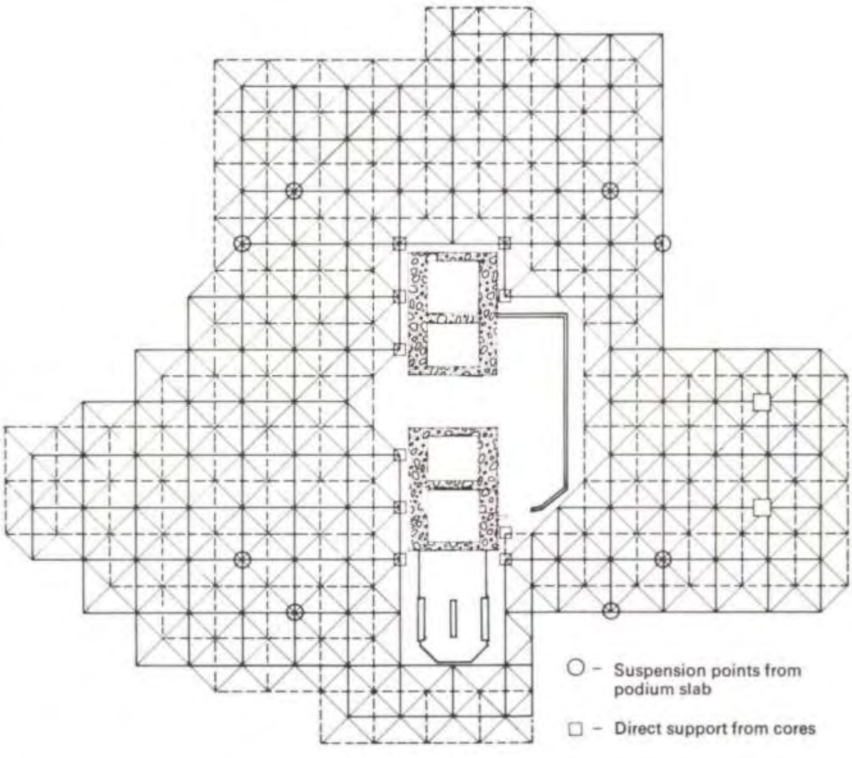


Fig. 15
Plan of wind canopy space frame structure

Fig. 16
Wind canopy supported on inclined hangers and fitting closely to the surrounding buildings (Photo: BSC Tubes Ltd.)

Fig. 17
Tubular hanger casings surround *Macalloy* rods (Photo: Ove Arup & Partners)

Fig. 18
Inclined hangers support the space frame (Photo: Ove Arup & Partners)

Fig. 19 below
The unclad soffit of the wind canopy (Photo: BSC Tubes Ltd.)

The wind canopy

The problems of downdraughts from tower blocks adjacent to low rise buildings, creating high local winds and turbulence at ground level, are well-known. The purpose of the canopy was to reduce these effects by fitting the canopy closely to the surrounding buildings. At the same time sufficient space was to be left for the geometric form of the mushroom head double cantilever to be appreciated from the entrance hall of the new building. Consequently, the plan shape of the wind canopy is complex as can be seen from Fig. 15.

The structural complexity of the canopy was further compounded by the Local Authority's insistence that no supports could be situated in the pedestrian precinct below it. Structural support was, therefore, provided by the concrete cores and the two link columns and by splayed tie rods suspended from the four corners of the podium slab. These can be seen in Figs. 16, 17, 18.

In addition, the soffit of the structure was to remain unclad and it was essential therefore to arrive at a solution with clean and simple lines (Fig. 19). A space frame of uniform biaxial pattern, which can distribute loads in all directions to concentrated supports, was clearly the only choice.

Systems involving prefabricated pyramids connected by tie rods could not cope with both the positive and negative bending induced by the unusual support system and reversals of wind loading. The choice, therefore, lay with the various space frame jointing systems which allowed us total flexibility in the design of member elements to accommodate these complex stress patterns.

The Nodus joint was chosen because it allowed flexibility in member design, it had a neat appearance and it allowed the decking to be fixed directly to the structure without a subsidiary purlin system. Also the Nodus joint allowed the cambers to be generated automatically during erection, when due allowance was made to the relative member lengths. However, the large hole in the centre of the canopy for the lift cores prevented full advantage being taken of this aspect of the system.

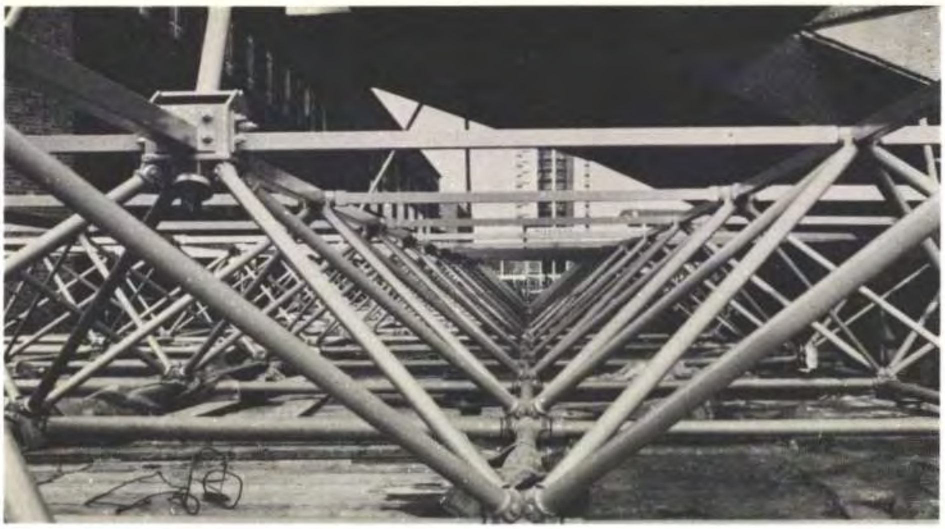


Fig. 20 left
Canopy space frame being assembled in its final position
(Photo: Ove Arup & Partners)

Fig. 22 right
Coventry Point from the pedestrian precinct
(Photo: Henk Snoek)

Fig. 21 below
White finish on the precast splay-corner walls
(Photo: Henk Snoek)

The overall depth of the canopy was limited by the needs to fit it below the springing of the podium slab at the lift cores on the one hand and to be clear above the canopies of adjacent buildings on the other. The necessary shallow depth resulted in anticipated total deflections of the order of 150 mm in the worst cases.

Design wind loadings were assessed from a series of wind tunnel tests of a 1:100 scale model of the building, including the canopy and the surrounding buildings. The model was subjected to a wind velocity spectrum equivalent to a basic wind speed (CP3: Chapter 5, Part 2) for Coventry of 44 m/s. The maximum indicated average design load downwards on the canopy turned out to be about 1 kN/m^2.

Steelwork

The structural form of the space frame was a square on offset square grid of 2.4 m module and a depth of 1.2 m, using square hollow section members in the top plane to facilitate fixing of the steel decking and circular hollow sections for all the other members (see Fig. 20). We tried hard to use Grade 43C steel throughout but inevitably Grade 50C had to be used for a few of the most highly stressed members after analysis. The frame was analyzed by a computer program modified by BSC Tubes Division to take account of the load eccentricities developed in the Nodus joint. The final structure covered a nett area of 680 m^2 for a structural frame weight of 19 tonnes at an average height of 5.4 m above the precinct.

Method of assembly

Assembly of the frame was carried out in its final position on a scaffold platform (see Fig. 20) rather than at ground level (which would have been much less costly) because emergency vehicle access was required 24 hours a day through the precinct.

Summary

This was a development with so many unusual problems that the successful resolution of each, together with their eventual realization on site, was a source of real satisfaction to all those in the design and construction teams.

The following five aspects were of special interest:

Piling of such a capacity had never before been achieved to our knowledge in the Coventry area and we had several subsequent calls from piling contractors enquiring as to the success of the design. It has also formed the subject of a paper by Ken Cole and Martyn Stroud to an international symposium on foundations in soft rocks.

Despite concern about the cost of the double cantilever suspended falsework, the contractor was subsequently so convinced of its success that he would have liked to have used it on a similar project elsewhere for which he had used birdcage scaffolding only 12 months earlier.

The irregular pattern and support system of the wind canopy has stretched the versatility of the Nodus joint system further than we have come across before and it has been found to perform satisfactorily.

The in situ cantilever slab was a successful solution to supporting 12 storeys over a pedestrian precinct without appearing like a bulky mass of concrete.

Finally, the architect has managed to adapt a fairly standardized precast building system into an unusual and attractive building which with its bold white walls forms a dominant landmark in an already pleasing city centre. (Figs. 21 & 22).

Credits

Client:
Bryant Samuel (Holdings) Ltd.

Architect:
John Madin Design Group
Partner – Derek Davis
Project Architect – Douglas Hickman

Quantity surveyor:
Francis Graves & Partners

Main contractor:
C. Bryant & Sons Ltd.

Precast concrete:
Concrete (Midlands) Ltd.

Foundations:
Piggott Foundations Ltd.

Wind canopy:
Sanders Tubercrafts Ltd.

Aseismic design in Iran

David Croft

Introduction

A considerable amount of literature exists concerning earthquakes and methods of designing structures to withstand them. In particular the book by David Dowrick[1] is recommended for general guidance on aseismic design and for a discussion of the various methods of establishing design parameters. However, it does not attempt to give specific advice on any particular country and, in view of the number of projects currently being carried out by the firm in Iran, it was considered that a common approach would be beneficial. An Arup Design Guide[2] has therefore been produced and was circulated in late 1977.

This article is an abridged version of that document and describes how the design recommendations have been derived. Details that have necessarily been omitted and a full list of the various references that have been used can be found in the Design Guide.

Geology

In describing the geology of Iran as a whole, it must be remembered that the country is vast (Fig. 1). The total area is roughly 1.6 million km^2 and the distance between the north west and south east corners of Iran is almost as far as from London to Athens. The geology is highly complex reflecting the tortured tectonic history of the area and the following geological history is necessarily greatly simplified.

The bedrock strata were formed during the pre-Cambrian, Palaeozoic, Mesozoic Eras and up until the Miocene Epoch. Interspersed with the formation of these strata were tectonic movements which created the various mountain ranges. More recently in the Pliocene and Quarternary Periods (during the last seven million years) material has been washed out of the mountains and deposited as alluvial plains in between, although significant tectonic movements still continued during this time. Thus the depth to bedrock from the surface varies from zero at the mountains to thousands of metres in the plains.

Generally towns have tended to develop on the plains, where the ground is relatively level and fertile, but close to the mountains which provide a source of water. It is therefore these areas that are of most interest in the aseismic design of structures.

Seismicity of Iran

Iran forms part of the Alpine-Himalaya seismic belt and is enclosed by three main tectonic plates: the Euroasian plate to the north which is stationary, the Arabian plate to the south and west which is moving north-eastwards, and the Indian plate to the east which is moving northwards. Associated with the boundary zones of these three plates are seismically active mountain belts as follows:

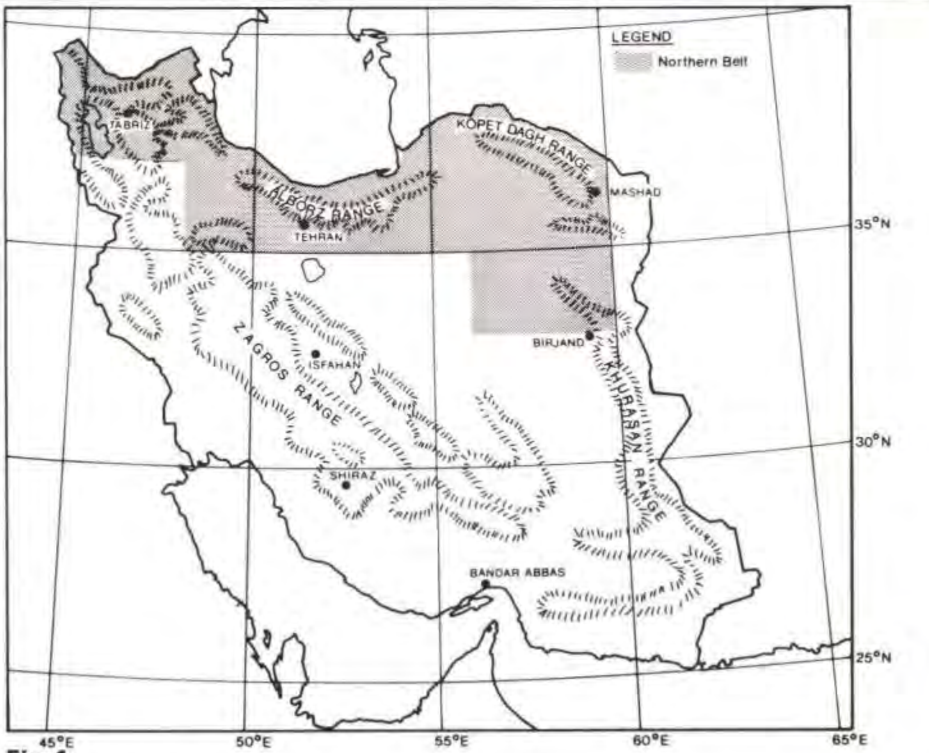


Fig. 1
Map of Iran

(i) In the north, the Alborz Mountains stretch from Tabriz in the north-west past Tehran in the centre, to Mashad in the east. Also included in this belt is the Kopet Dagh range stretching from Shirvan in the east under the Caspian Sea to Baku in the west.

(ii) In the south and west, the Zagros Mountains join the Alborz Mountain belt around Tabriz in the north, extend through Shiraz in the centre to beyond Bandar Abbas in the south.

(iii) In the east, along the frontier with Afghanistan and Pakistan, the Khurasan Mountains join the Alborz Mountains in the north around Mashad and extend through Birjand in the centre to join the Zagros Mountains in the south.

The central area of Iran, bounded by these mountain ranges, is a high plateau region which is generally of relatively low, though still significant, seismic activity.

The average rate of release of seismic energy in Iran is comparable to, though lower than, that of California. In California the maximum magnitude of shocks is greater ($M > 8$) and events of a given magnitude are felt over a greater area than in Iran.

In the north of Iran, occasional shallow high magnitude shocks ($6.5 < M < 7.5$) occur causing large-scale devastation. Recent high magnitude shocks in this belt have been: Ashkhadbad (north-east 1948 $M = 7.6$), Basht-e-Bayaz (east 1968 $M = 7.2$), Boyin Zara (west 1962 $M = 7.3$), Abegharm (centre 1957 $M = 7.1$). Historical evidence, however, suggests that areas which are currently quiescent have been severely devastated during several periods in the past. For example, the Tehran area was devastated during the 10th and 18th centuries but is currently quiescent. Therefore, when evaluating seismic risk, the activity of the whole area must be considered.

In the south and west the dissipation of energy is more continuous and high magnitude shocks are relatively rare. In some areas, particularly to the south of Lake Rezaihey, the dissipation of energy is so gentle and continuous that, until the recent installation of sensitive seismic measuring instruments, the area was considered to be earthquake-free.

Recurrence relationships

The seismicity of an area can be expressed in the form of a recurrence relationship as follows:

$$\log_{10} N = a - bM \qquad (1)$$

where N is the average number of earthquakes of magnitude greater than M occurring in an area of 1000 km² per year.

M is the magnitude

and a and b are constants for the area under consideration.

The values of the constants a and b for Iran are given below. Also included for comparison are values for Average California and South California.

	a	b
Average Iran	2.69	1.0
Northern Iran	1.45	0.72
Average California	3.59	1.0
South California	2.25	0.75.

These recurrence relationships are shown in Fig. 2 and the area referred to as Northern Iran is defined in Fig. 1.

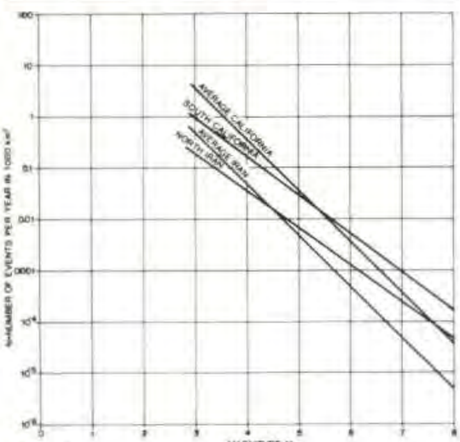


Fig. 2
Recurrence relationships

As an example these relationships predict on average every 50 years 710 earthquakes with M>5 in the whole of Iran (120 of them in the north) and seven earthquakes with M>7 (four of them in the north). It will be seen that the proportion that occur in the north compared to the country as a whole increases with the magnitude.

Theoretically, for areas outside the northern belt, values averaged for the rest of Iran should be derived rather than the average for the whole of Iran. However, some areas in the rest of Iran are more seismic than others and to take the overall average makes some allowance for this.

Equation (1) can be expressed in the form:

$$N = 10^{a-bM} \qquad (2)$$

This is a Cumulative Probability Function and the corresponding Probability Density Function can be found by differentiating, i.e.:

$$P_1 = -\frac{dN}{dM} = 2.302\, b\, 10^{a-bM} \qquad (3)$$

Attenuation formulae

In an earthquake, ground motions are naturally greatest close to the epicentre and reduce with distance away. Many attenuation formulae relating ground motions to earthquake magnitude and distance have been proposed and the following which have been used here are by Esteva.

$$a = \frac{5600\, e^{0.8M}}{(R+40)^2} \qquad (4)$$

$$v = \frac{32\, e^{M}}{(R+25)^{1.7}} \qquad (5)$$

where a = Peak Ground Acceleration (cm/sec²)
v = Peak Ground Velocity (cm/sec)
R = Focal Distance (km) (see Fig. 3)
M = Magnitude of Earthquake

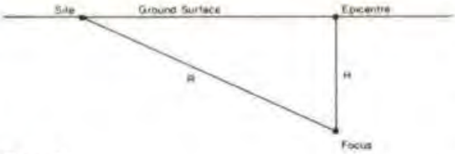


Fig. 3
Geometric relationship between focus and site

The maximum distance away from the site at which an earthquake of magnitude M can occur and still produce a peak ground acceleration greater than a given value a is therefore given by:

$$R = \left[\frac{5600}{a}\, e^{0.8M}\right]^{\frac{1}{2}} - 40 \text{ km} \qquad (6)$$

And the probability P_2 that an earthquake predicted by Equation (3) will occur within this distance is:

$$P_2 = \frac{\pi\,(R^2 - H^2)}{1000} \qquad (7)$$

where H is the Focal Depth in km and for Iran has been taken as 20 km.

The 'threshold' value M_0 which is the magnitude of the smallest earthquake that will just cause the value a corresponds to the value R = H and is given by:

$$M_0 = \frac{1}{0.8} \log_e \left(\frac{a}{1.555}\right) \qquad (8)$$

Peak ground accelerations and velocities

Using equations (3), (6), (7) and (8) and assuming a maximum credible earthquake of M = 8, the probability of any given peak ground acceleration being exceeded can be found, i.e.

$$P = \int_{M_0}^{8} p_1 p_2 dM \qquad (9)$$

where P is the probability of a peak ground acceleration of a cm/sec² being exceeded in any one year. The probability of attaining a given peak ground velocity can be found in a similar manner using Equation (5) instead of (4).

The corresponding return period T is given by:

$$T = \frac{1}{P} \qquad (10)$$

and the probability P_t that a given value will be exceeded in the design life of the structure (t years say) is given by:

$$P_t = 1 - (1 - P)^t \qquad (11)$$

Values for peak ground accelerations and velocities corresponding to various return periods have been derived for both North and Average Iran and are shown in Figs. 4 and 5. The corresponding values for South and Average California have also been plotted for comparison and it is apparent that for the range of return periods from 50 to 500 years the values for Northern Iran vary from 50% to 66% of those for Southern California and that the values for Average Iran vary from 55% to 70% of those for Northern Iran.

Choice of return period

It is common practice to treat seismic loads in the same way as wind loads and to apply similar load factors in design. However, it can be seen from Figs. 4 and 5 that, for example, the lateral loads corresponding to the 500 year (assuming that they are proportional to the peak ground motions) are between 2.4 and 2.8 times as large as those for the 50 year period. On the other hand, for wind loads this ratio is generally much lower (e.g. for the UK the ratio is 1.4). Thus, if the design is based on, say, a 50 year return period characteristic load, the probability of the ultimate earthquake occurring is very much larger than that of the ultimate wind.

One solution to this problem is to use a 50 year earthquake for the determination of the characteristic load and to use higher load factors. This approach may be suitable for some special projects but for general use may lead to confusion.

An alternative approach and the one that has been adopted here, is to consider the 500 year return period earthquake as corresponding to the limit state of collapse. An *equivalent* 'characteristic' load can then be derived by dividing by a global safety factor which for this purpose has been taken as 1.7.

From Figs. 4 and 5 and using the simple harmonic relationship $a = 2\pi V/T$ the peak ground acceleration for Northern Iran for a return period of 500 years is given by:

$$a = \frac{2.01}{T} \not> 2.5 \text{m/sec}^2 \qquad (12)$$

where T is the period in seconds.

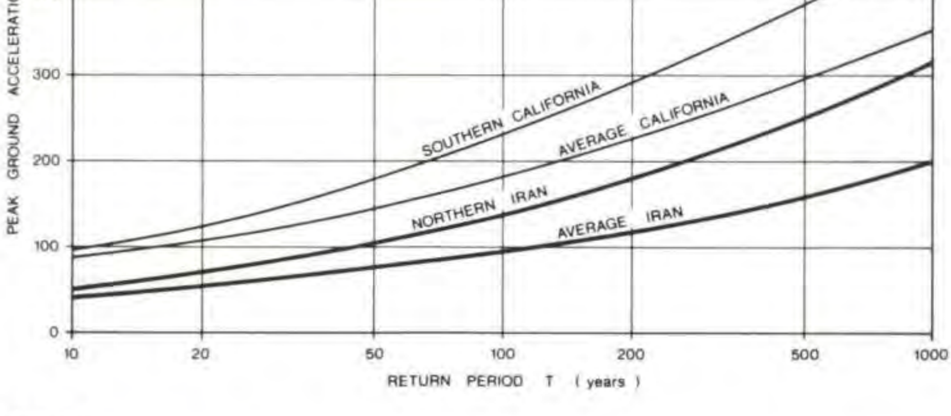


Fig. 4
Peak ground acceleration

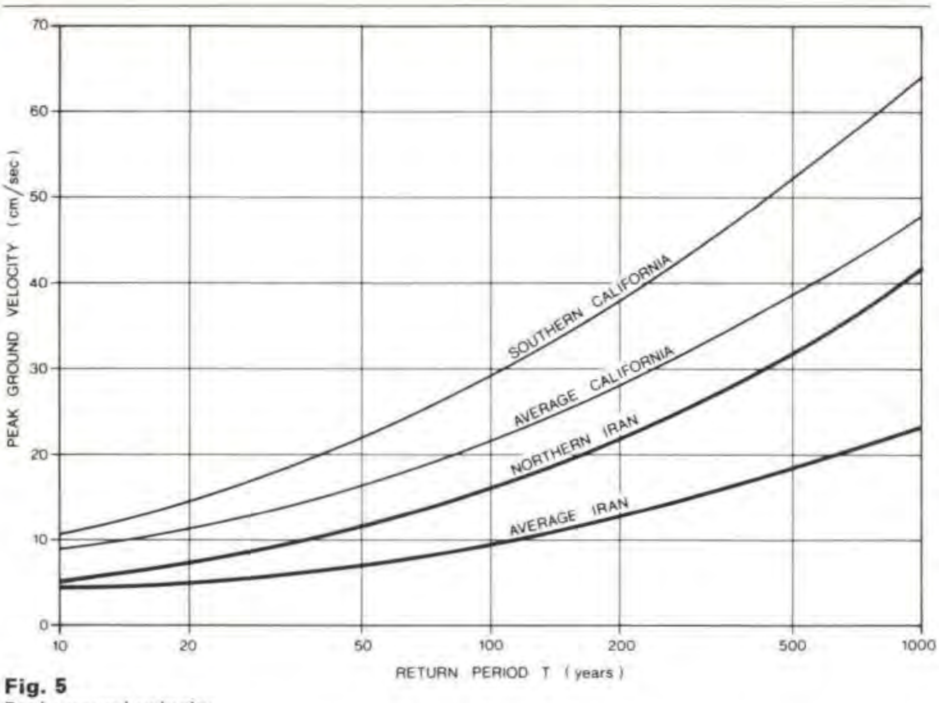


Fig. 5
Peak ground velocity

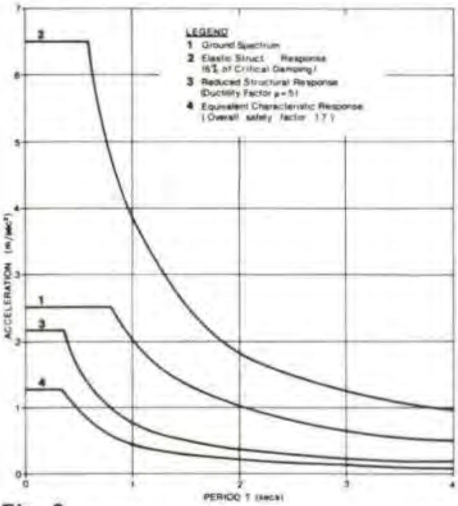


Fig. 6
Response spectra for Northern Iran (500 year return period)

Structural response

The calculation of design loads from the peak ground motions involves first the construction of the elastic response spectrum for the structure and then making allowance for the energy absorbed by plastic deformation.

The loads obtained in this way are therefore not characteristic loads in the true sense as they imply significant permanent damage to the structure. However, if these loads are treated as wind loads and the normal wind load factors applied, then an appropriate probability of collapse will be attained.

The elastic structural response spectrum has been constructed using the method described by Blume *et al* with the magnification factors suggested by Newmark and Rosenblueth which are given below:

% Crit Damping	Acceleration	Velocity
1	5.2	3.2
2	4.3	2.8
5	2.6	1.9
10	1.5	1.3

The Elastic Structural Response assuming 5% of critical damping is therefore given by:

$$S_{ae} = \frac{3.82}{T} \not> 6.5\text{m/sec}^2 \qquad (13)$$

To allow for the energy absorbed by plastic deformation, reduction factors have been used as described in Reference 1 as follows:

For accelerations $\dfrac{1}{\sqrt{2\mu - 1}}$

For velocities $\dfrac{1}{\mu}$

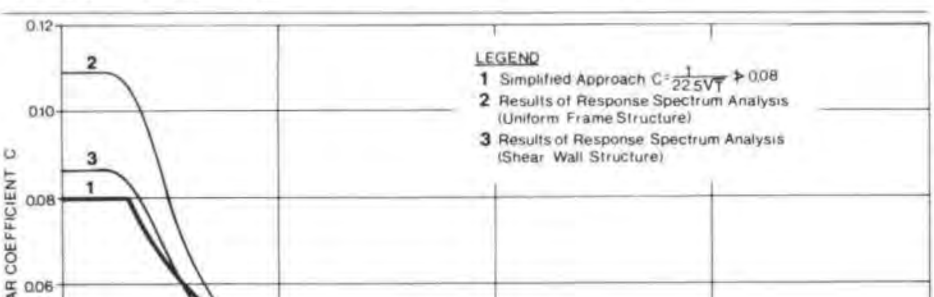


Fig. 7
Base shear comparison between alternative methods

where ductility factor μ is defined in generalized terms by

$$\mu = \frac{\text{Deflection at Collapse}}{\text{Deflection at First Yield}}$$

Assuming an average ductility ($\mu = 5$) the response becomes

$$S_{a(ult)} = \frac{0.764}{T} \not> 2.17\text{m/sec}^2 \qquad (14)$$

and the equivalent characteristic spectrum assuming an overall safety factor of 1.7 is therefore:

$$S_a = \frac{0.449}{T} \not> 1.27\text{m/sec}^2 \qquad (15)$$

These curves are plotted in Fig. 6.

Design recommendations

If a dynamic analysis is to be carried out as part of the design, then the response spectrum given by Equation (15) can be used directly in a response spectrum analysis. This spectrum for North Iran is now a standard option within the GLADYS Cantilever Dynamic Analysis program.

In most cases, however, a full dynamic analysis is neither necessary nor justified and a simplified approach is suggested. As noted above, North Iran is approximately two thirds as seismic as California and it is therefore reasonable to design to the Californian Code with reduced factors. Further details are given in the Design Guide[2].

As a comparison between the two approaches, response spectrum analyses have been carried out on a range of hypothetical buildings with varying fundamental periods of vibration and with either shear wall or frame systems resisting lateral forces as these represent the extremes in terms of modes of response in conventional structures. The values of the base shear are compared with those given by the simplified approach in Fig. 7. The results show reasonable agreement and indicate that the simplified approach is conservative compared to the response spectrum analysis method except for structures with periods of less than 0.8 secs. As the formula for C is factored down from the Californian Code, this probably reflects the traditional optimism of the Californians regarding ductility in that the code requirements cannot be substantiated theoretically with realistic values of μ at the present time. There is, however, some physical evidence supporting the view that well designed low-rise structures perform better than present theory would suggest, so that the values given by curve 1 for low period structures are probably justified.

Iranian statutory requirements

Seismic loading is covered by the Iranian Standard ISIRI 519. This document gives lower forces than the methods described above except for tall buildings for which it is unduly conservative. There are also a number of other documents which are issued by various government ministries and agencies, but it is not always clear when and how they should be applied. As the situation is continuously changing, it is recommended that advice is sought from the Tehran Office on current requirements.

Acknowledgements

Much of the data used in the Design Guide was researched and produced by David Dowrick and Mike Glover. Their advice and comments on the way this material has been subsequently used have also been most helpful.

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Freeman Hospital, Newcastle-upon-Tyne

Peter Ross

INTRODUCTION

In the post-war years, the newly-established National Health Service reviewed the existing centres of urban medicine. Newcastle General Hospital at that time occupied what had been the old Workhouse, and infill development of the long, narrow site by one and two-storey buildings was virtually complete.

A study showed that any fundamental redevelopment of the site would produce disruption on such a scale as to be unacceptable, and the decision was eventually made to build anew at Freeman Road, which, while somewhat to the east of the city, was a greenfield site and large enough to allow for considerable future development.

In 1967 the (then) Newcastle Regional Hospital Board appointed Arups as structural consultants, to work with the Board's own architect and engineer.

The contract work commenced on site in August 1971, and was complete in May 1978.

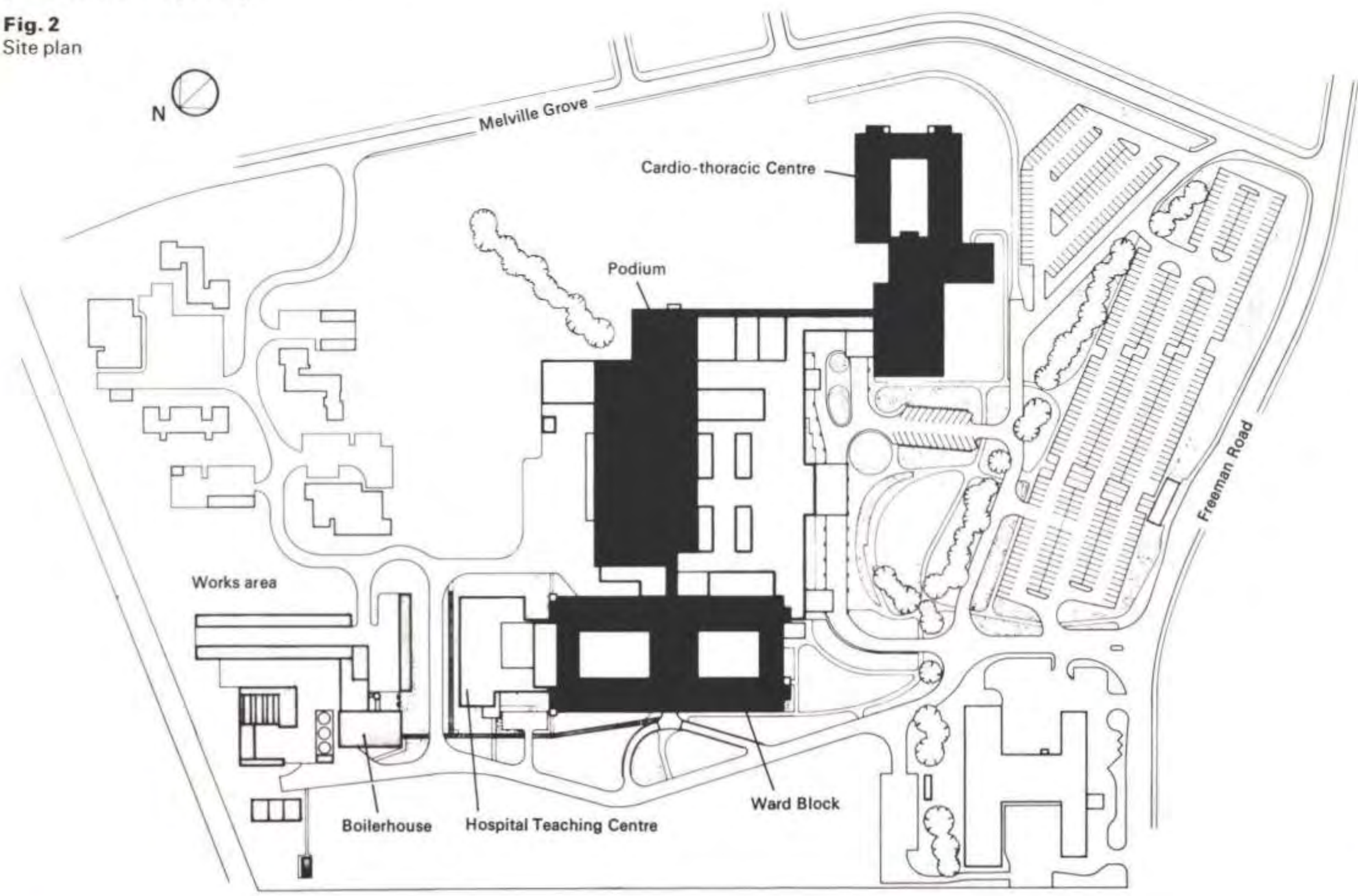
GENERAL DESCRIPTION

The project consists of four main blocks; Ward, Podium, Cardiothoracic and Works Area.

The other two groups of buildings on the site – the Artificial Limb Centre, and the residential blocks – were the subject of separate contracts.

Fig. 1
Entrance to the out-patients department
(Photo: Photo-Mayo Ltd.)

Fig. 2
Site plan



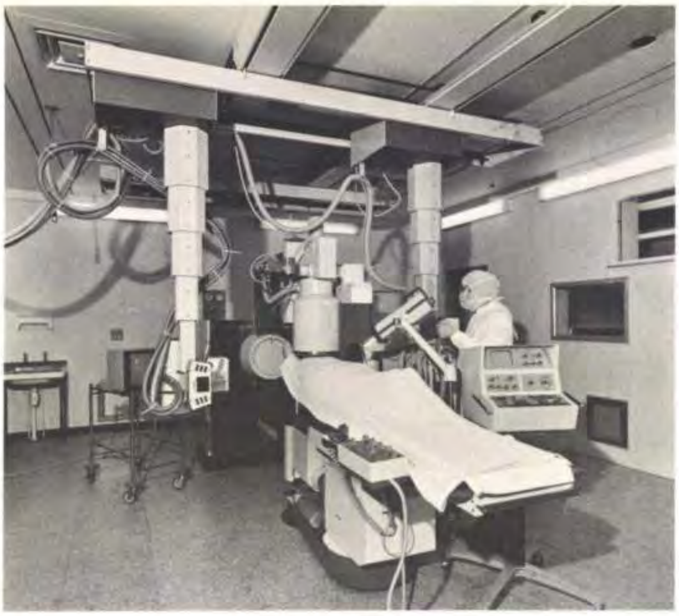


Fig. 3 above
The X-ray department (Photo: Allan Glenwright)

Fig. 4 right
The hydro-therapy pool
(Photo: John Laing & Son Ltd.)

Fig. 5
An operating theatre
(Photo: John Laing & Son Ltd.)

Fig. 6
The Ward Block under construction
(Photo: Turners (Photography) Ltd.)

Podium Block

The Podium contains 11 suites which provide for general, orthopaedic and urological surgery. Each consists of theatre, anaesthetic, and preparation rooms, and the suites are linked to a 12-bed recovery area.

The X-ray department, which serves both in- and out-patients, is now the foremost centre of radiology in the North of England.

At ground floor level there is an extensive Out-Patients Department and Rehabilitation Department, with two gymnasia and a hydro-therapy pool.

At Level 1 are the main staff rooms and kitchens, and central stores area.

Cardiothoracic Centre

Being one of the first purpose-built heart and chest units in Europe, the Centre has brought together under one roof several established units of cardiothoracic medicine and surgery. The Centre contains five suites of operating theatres and two radio-diagnostic theatres. For the post-operative care of patients who have undergone major cardiac surgery, there is a 10-bed Intensive Care Unit, with equipment to monitor various physiological parameters (such as temperature, blood pressure, pulse rate) linked to a central station.

Ward Block

The Ward Block houses 583 beds for acute cases in the uppermost five levels, predominantly in six-bed bays and single rooms, each unit having its own toilet and shower facilities.

Below is the Geriatric Day Unit and general administration area, and at ground level, the Teaching Centre. This will be the largest local centre for the training of nursing personnel, and will form a new base for the Area School of Nursing. As well as the normal library,

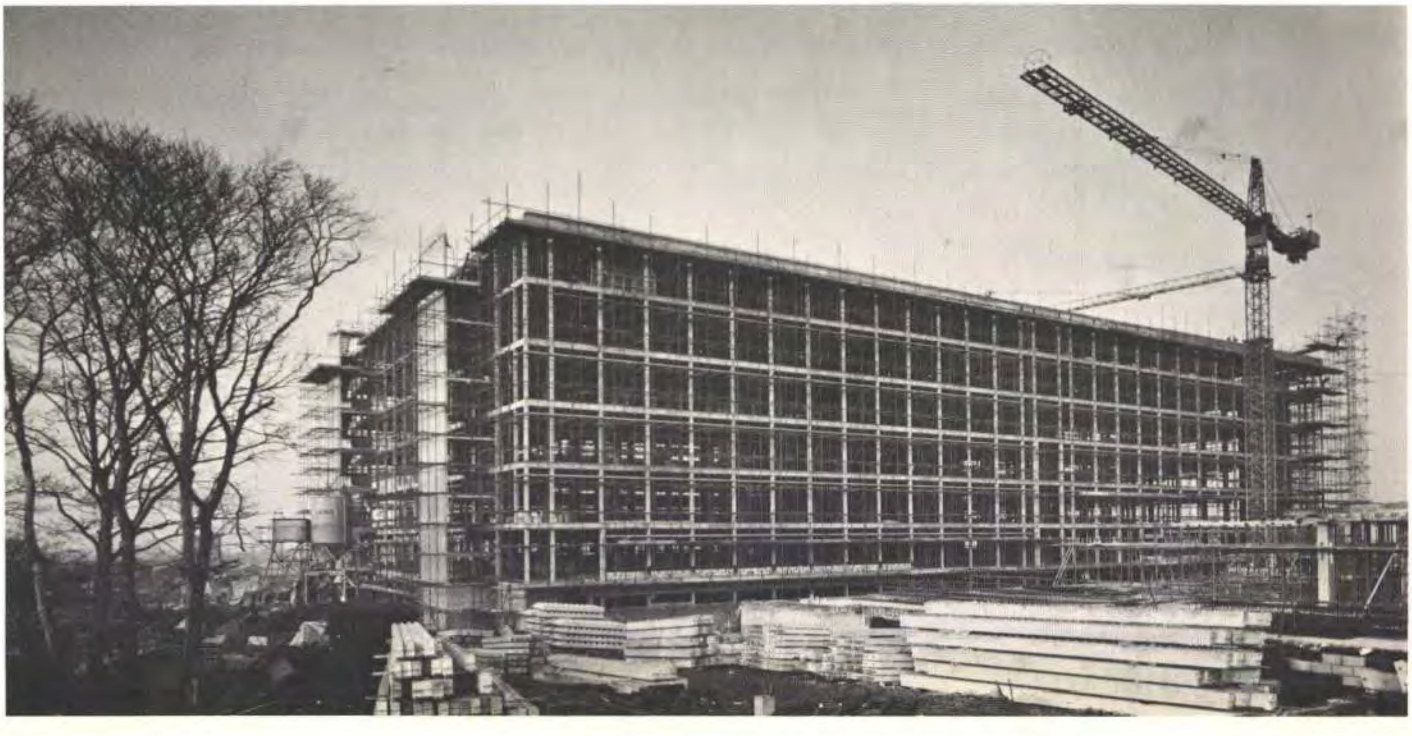
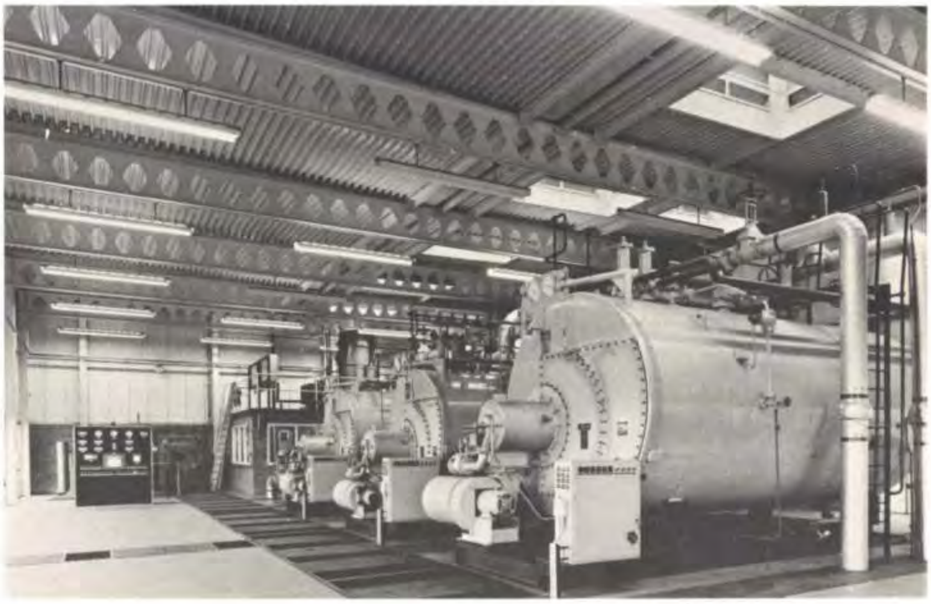


Fig. 7
The Ward Block, with the Hospital Teaching Centre to the right. The curved roof to the twin lecture theatres is sheathed with copper-faced felt
(Photo: Photo-Mayo Ltd.)

Fig. 8 left
Open-heart surgery
(Photo:
Allan Glenwright)

Fig. 9 below
The link corridor between the Podium and Cardio Blocks clad with *Profilit* glazing
(Photo: Photo-Mayo Ltd.)

Fig. 10 below
The Boiler House
(Photo: Allan Glenwright)



classrooms and practice suites, additional facilities are available in certain wards, and the lecture rooms are linked by CCTV to the operating theatres.

Works Area

Set a little apart from the main building complex, the area contains the boiler house and standby generator, incinerator, workshops, water storage tanks and flammable gas stores.

INTERNAL ARRANGEMENT

Circulation

As hospitals have a great amount of wheeled traffic, the floor levels are constant throughout the three blocks. The primary horizontal circulation takes place at Level 2, although the slope of the site made it economic to build a lower floor in Ward and part of Podium. These areas, however, are only occupied by staff. Ramps were considered to be too wasteful of time and space, and all vertical circulation is by means of bed/passenger lifts, grouped in each block.

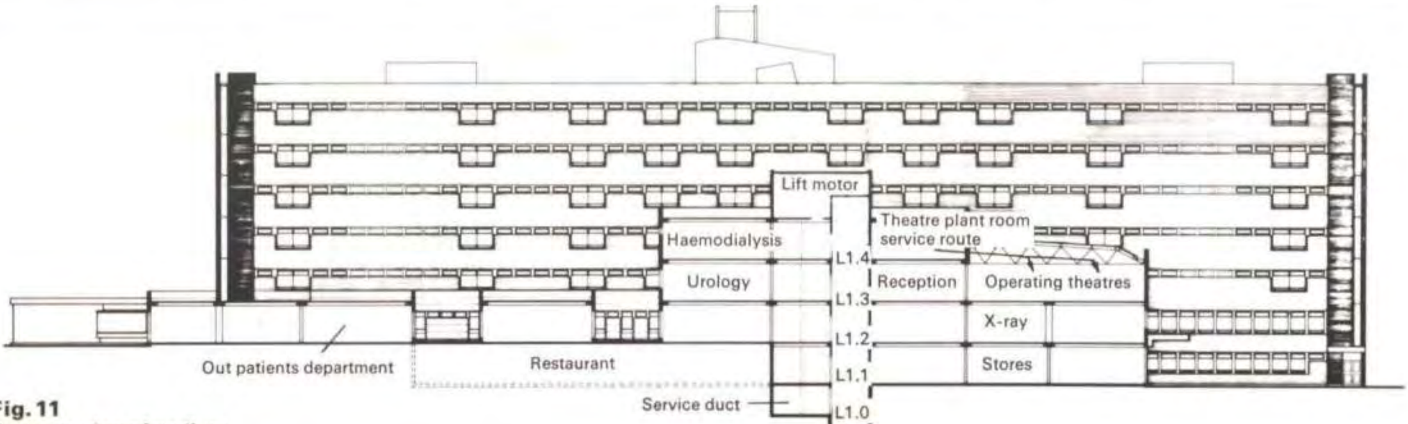


Fig. 11
Cross-section of podium

Fig. 12
A nurses station in the corridor of the Ward Block (Photo: John Laing & Son Ltd.)

Ward layout

The ward accommodation is based upon the six-bed unit, with single beds for isolation and intensive care nursing. While this smaller ward unit is generally preferred by the medical authorities in preference to the Nightingale arrangement (a long, open ward with beds down each side), much more consideration must be given in the layout to patient supervision.

A section of the Ward Block is shown in Fig. 14 where two six-bed bays and two single rooms are supervised from a central nurses station, set off the main corridor. There is in addition a nurse call facility at each bed and in the toilets and showers linked to the station. The call is transferred to the next station if the nurse is not in attendance.

The single rooms may be used for barrier nursing (i.e. isolation of a patient who is infectious, or who may be easily infected). The direct connection to the corridor is locked, and all contact with the patient is made through the vestibule, where sterilization procedures may be carried out.

SERVICES

Whilst the design of the mechanical and electrical services was not our responsibility, it is necessary to include them in this description of the hospital, both for their significance within the project, and for the influence of their requirements on the structural design.

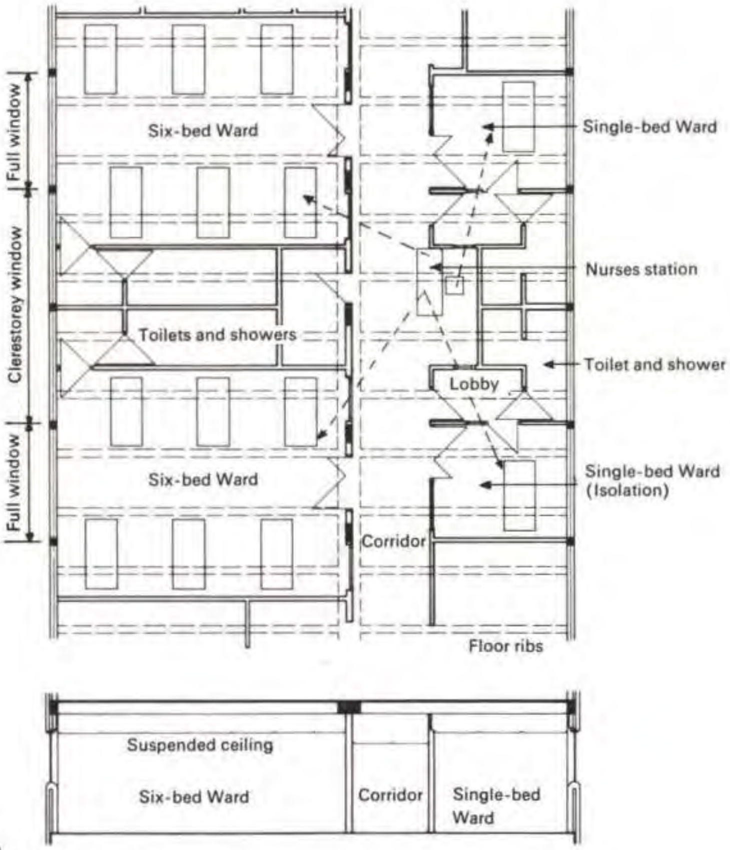


Fig. 14
Section of Ward Block

The whole complex is heated by means of medium pressure hot water supplied from the boiler house. As a major consumer, the Board had decided to design an installation which was not confined to a single type of fuel. The boilers operate on gas, but can be changed instantly to burn light oil, held in reserve tanks. This enables gas to be purchased on the basis of an interruptible supply, with considerable reduction in tariff. It is also possible to fit pre-heaters so that heavy oil may be burnt, should this ever be financially worthwhile.

As there was a possibility of burning oil it was necessary to provide a flue complying with the requirements of the Clean Air Act and so we received a commission to design the chimney, combining into the stack an additional flue to serve the incinerator.

Early designs were based on separate winter and summer flues for the boiler, but this layout, with a total of three flues, produces a potentially large amount of 'cold' lining, together with a complex shell form. The final profile (Fig. 15) with one flue for the boiler, allowed the use of a simple circular shell.

The boiler flue is designed for almost twice the loading from the present buildings, to

Fig. 13
The works area (Photo: John Laing & Son Ltd.)

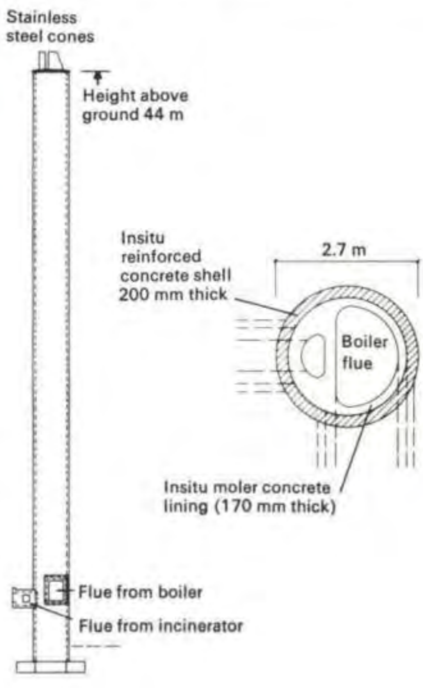


Fig. 15
Details of chimney

allow for expansion of the hospital. In order to achieve satisfactory exit velocities on the present load, a stainless steel reducing cone is fitted to the top of the boiler flue, with a matching cylinder on the incinerator flue. These also help to lift flue gases clear of the concrete shell and reduce the possibility of staining. The cone can thus be either truncated or removed, in accordance with increased flue loading. A further increase in capacity could still be achieved by diverting the flue gases through a booster fan at the base of the chimney.

The contract was let on a design and build basis with certain parameters determined, such as the wind pressure curve and the basic stack configuration. Tenders were invited from both precast and in situ specialists. The successful tenderer constructed the chimney in in situ concrete, with an in situ moler concrete lining.

The boiler house is linked to a service spine duct, which passes under all the blocks at the lowest level. Vertical shafts rise at intervals around the blocks, and distribution of services at each floor is in the ceiling over the corridor. The hospital is fitted throughout with heated ceilings, chosen in preference to a system of radiators for appearance and hygiene, and to ducted air because of the demands made by that system on ceiling space.

Electricity is supplied through a sub-station in each block, with a standby generator installed in the works area to provide over 50% of the running loads in the event of a mains failure. Areas of high priority, such as the operating theatres, are served by a battery inverter to bridge the generator start-up time.

STRUCTURE

Foundations

The subsoil is generally boulder clay overlaying sandstone at various depth up to 7 m. The foundation design was thus straightforward, with the Ward and Podium Blocks founded on rock and the Cardiothoracic on the clay.

Suspended floors

In addition to the normal design criteria of stability and fire resistance, a hospital floor must be able to support, and be penetrated by, the many service elements contained in the building. The structure which evolved was basically a ribbed floor with a thin topping. The ribs are precast, of constant cross-section, and the in situ topping is cast on permanent form work of *Holorib* which spans between the ribs and is located in position by the projecting reinforcement (Fig. 16).

The topping is 75mm thick which, with the 25mm screed, gives a fire resistance of one hour. The Fire Research Station confirmed that the indentation in the *Holorib* would not reduce this period. The *Holorib*, while acting compositely with the slab, is unprotected against fire, but the mesh, incorporated into the topping to give overall continuity to the structure, provides an adequate reserve of strength.

The ends of the precast units are supported by in situ spine beams of the same depth, which span between precast columns. The in situ concrete thus joins the precast units together, avoiding the problems of fit associated with a totally precast structure, and providing the continuity required when considering explosive forces.

The structure is contained within an overall depth of 375mm, which allows a free zone of 400mm for services above the suspended ceiling, with an additional 'hop-over' facility between the ribs.

Services are supported from the floor by means of 6mm drop-rods, screwed into solid wedges (Fig. 17). The wedges are introduced into the groove sideways, and then turned into position. Load tests were carried out on site, which showed that the 6mm rods could be used to their maximum working load. The system worked successfully as the wedges were cheap enough to be used for the lightest loads, while being strong enough to carry the heaviest.

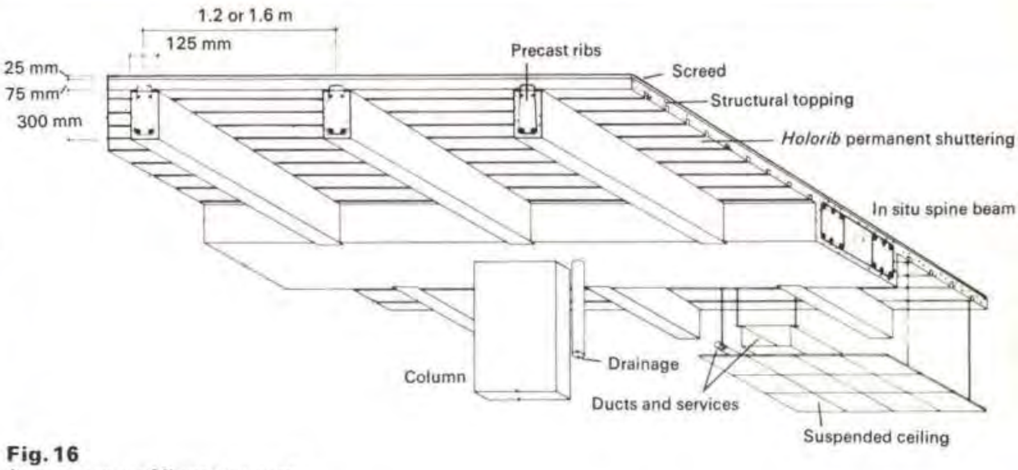


Fig. 16
Arrangement of floor structure

Fig. 21 details a part of the Ward Block, and shows the linear development of the structure and its relationship to the partition layout. An examination of the services which penetrated the floor showed that they were either inside a partition (water supply, electric cables) or within 200mm from the face (w.c. outlets). Displacing the rib module from the partition module by 400mm ensured that the builder's work holes were in the topping.

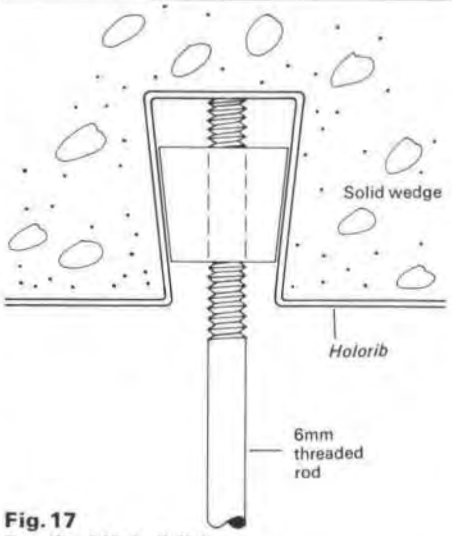


Fig. 17
Details of *Holorib* fixing

The drainage stacks and the centre line of columns are contained within the corridor partition. Since these columns lie under an in situ beam, it is possible to allow flexibility in their spacing, as long as the position is repeated at each floor. The centre zone of the beams is free of longitudinal reinforcement, so that the holes required for the drainage stacks may be formed with only minor displacement of links.

Theatre areas

The system was sufficiently flexible to apply to all parts of the structure, with the exception of the operating theatres, where the rooms are generally 3.3m high, with full air conditioning.

This area was placed on the top floor of the Podium Block and roofed in a clear span of 17m by steel trusses. The roof void thus created allowed air ducting and other service connections to be made between the theatres and the plant room, and relamping of the theatres from above.

Fig. 18
The central stores, with the basic floor structure exposed (Photo: Allan Glenwright)

Fig. 19
An aerial view of the hospital
(Photo: courtesy John Laing & Son Ltd.)

Fig. 20
The main entrance
(Photo: John Laing & Son Ltd.)

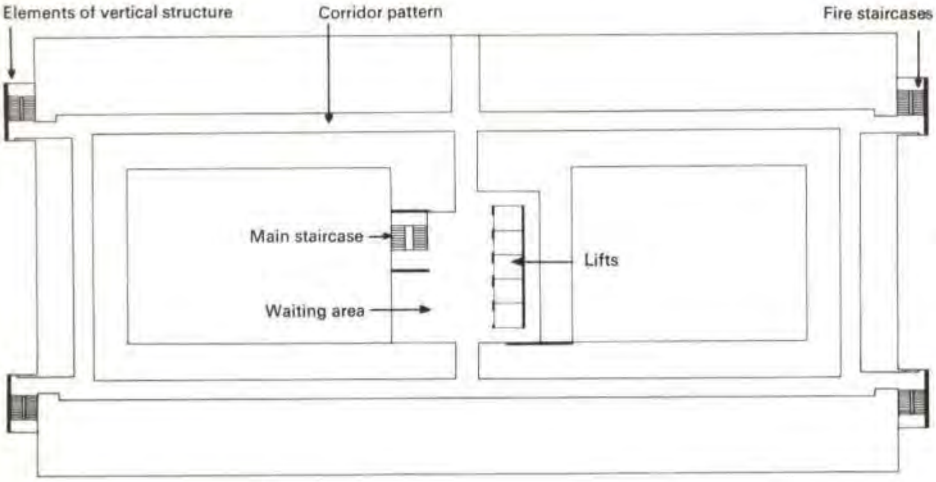


Fig. 21
General layout of the Ward Block, showing the corridor areas, escape routes and the vertical structure for wind stability

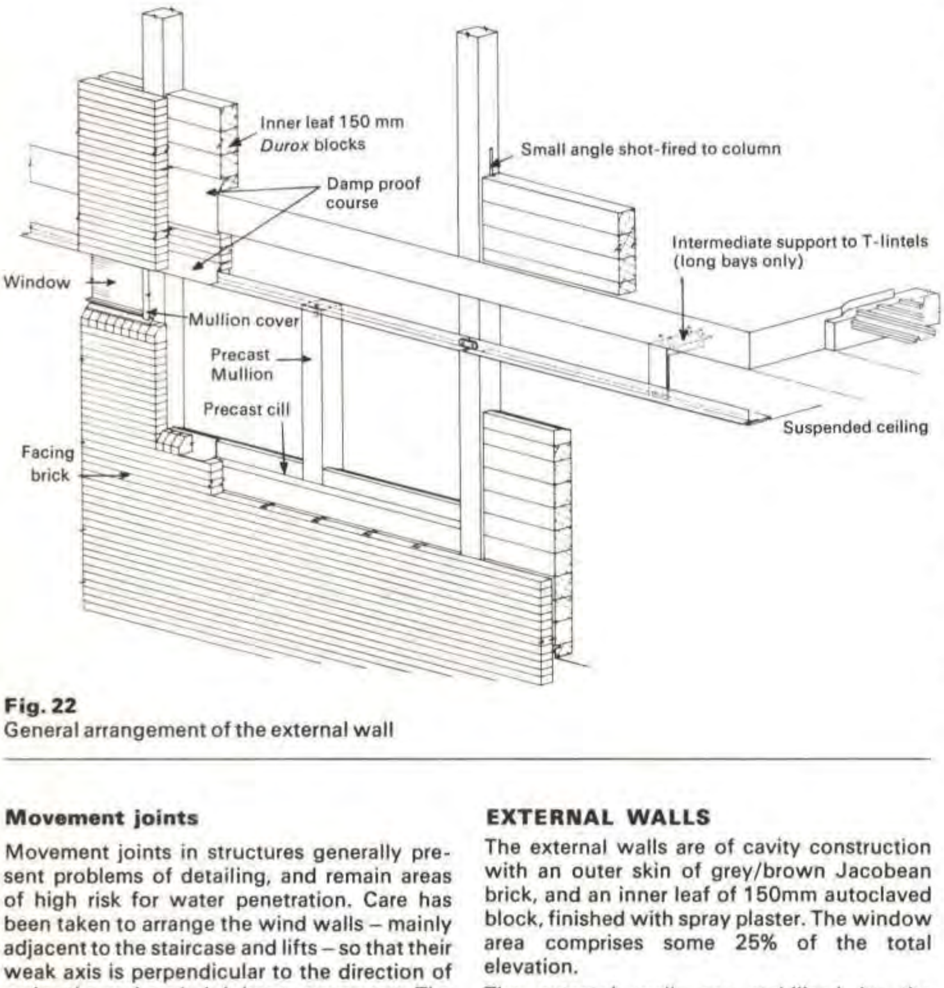


Fig. 22
General arrangement of the external wall

Movement joints

Movement joints in structures generally present problems of detailing, and remain areas of high risk for water penetration. Care has been taken to arrange the wind walls – mainly adjacent to the staircase and lifts – so that their weak axis is perpendicular to the direction of major thermal and shrinkage movement. The blocks themselves are constructed without movement joints; these only occur at the junctions of the blocks.

EXTERNAL WALLS

The external walls are of cavity construction with an outer skin of grey/brown Jacobean brick, and an inner leaf of 150mm autoclaved block, finished with spray plaster. The window area comprises some 25% of the total elevation.

The external walls are stabilized by the building-in of the inner leaf to the frame formed by the floor and the façade columns (see Fig. 22). In the Podium Block there are a few areas where the column spacing is increased, and here additional stabilizing angles are introduced at cill level. The outer leaf is in turn supported by brick ties from the inner leaf, forming a continuous cavity. The windows are set directly over this cavity, creating a single vertical plane at which water is excluded from the building, and simplifying the damp proof course detailing.

Fig. 23
A typical six-bed block
(Photo: John Laing & Son Ltd.)

The lintel at the window head is an inverted T section of galvanized steel spanning directly between columns at 3.2m centres, or intermediate supports from the floor above if the spans are larger.

The form of the windows is determined by the requirements of the various internal areas. In order to provide natural light to as many internal rooms as possible, there is a continuous 'clerestory' band of windows at each floor, from 2.1m above floor to ceiling level (2.7m). These give adequate light to areas such as stores, toilets and treatment rooms, while still allowing articles of furniture or equipment to be placed against the external wall. In the six-bed and single bed wards there is an additional requirement to see out, and here the band is deepened to a cill height of 1.2m. By limiting this to half the room width, undue solar gain at the outer bed positions is avoided (Fig. 23). All the windows are designed to blow out at lower pressures than the façade structure, and thus act as explosion vent panels.

The staircase towers, as elements of vertical structure, are correspondingly lit by vertical slit windows.

DRAINAGE AND EXTERNAL WORKS

We were commissioned to design the drainage of the whole complex, linking to a new outfall in the north corner of the site. We therefore had responsibility for all construction in the ground, and could co-ordinate the drainage with the foundations.

Our brief also included the design of the external works, including the roads, car parks and footpaths, and co-ordinating these with the external service runs and landscaping.

Credits

Client and architect:
Newcastle-upon-Tyne Regional Hospital Board

Main contractor:
John Laing & Son Ltd.

Quantity surveyor:
J. W. Summers & Partners

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