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Front cover: Re-entrant corner detail. IBM Cosham Phases 1 and 2 (*Photo:* Arup Associates)
Back cover: The new grandstand at the Crystal Palace National Sports Centre: support pyramid (*Photo:* Harry Sowden)

IBM Cosham

David Thomas

History

In August 1969 IBM United Kingdom Ltd. appointed Arup Associates to investigate the feasibility of developing part of the mud flats along the foreshore at Cosham near Portsmouth as a site for a new administrative headquarters to replace their existing headquarters at Chiswick.

A master plan was prepared in 1970 fully documenting all the factors influencing the use of the site and investigating possible alternative solutions for the site zoning. The three major factors influencing the use of the site – reclamation and polder lake, the siting of the building developments and the siting of the car parking areas and internal roads, were zoned to provide a framework within which the detailed planning of buildings on the site would fit.

At the same time as the master plan was being prepared, a preliminary concept for the administrative headquarters, accommodating some 3000 people, was commissioned. The studies for both overlapped in several respects and were carried out concurrently between May and September 1970, culminating in the presentation to IBM United Kingdom Ltd. in September 1970. Work then immediately proceeded on the final concept for the administrative headquarters (Scheme 1).

In January 1971 the final concept programme was interrupted at IBM's request so that several alternative plan forms, including a completely cellular building (Scheme 2) could be investigated and to enable a reappraisal of the brief requirements to take place.

Between January and June 1971 alternative designs were examined and in October 1971 agreement was reached on a partially deep and partially cellular plan form which was incorporated in Scheme 3.

The site

The site is 50 ha of reclaimed land in the north east corner of Portsmouth Harbour at Cosham and is within an overall area of reclamation of 156 ha promoted by the Portsmouth City Council. The reclamation of the site was based on the Dutch Polder system under the direction of the Dutch consultants Grontmij NV and the surrounding dykes are formed by the new M27 motorway embankments. A lake of 6 ha was formed in the area of the site where the chalk underlay is deepest and is used as a balancing tank for the polder drainage. At the eastern extremity of the lake a polder pump house discharges surplus water from the site into the sea.

Before reclamation started the site consisted mainly of low lying mud flats intersected by natural drainage channels. The chalk stratum of Portsdown Hill, to the north of the site, is part of the southerly dipping limb of the Portsmouth anticline and this gives rise to an artesian condition on the site with a fractured chalk layer acting as the principal aquifer. A

Fig. 1
The site prior to reclamation (*Photo:* Handford Photography)

perimeter canal, connecting the eastern and western ends of the lake has been constructed to take the discharge of land drains laid throughout the site. Excavated material from the lake was spouted up on to the site to form a roughly level site at 1.2 m above Ordnance Datum. Over a period of 10 years the site is expected to settle to a level of 1 m above Ordnance Datum. In 1971, during the reclamation process, IBM decided to fill a large section of the western end of the building zone with stiff clay and chalk nodules to provide a hardstanding for a possible additional building project. This project did not materialize and it became necessary to consider how this hardstanding area could be suitably landscaped. The recommendations of the Dutch Consultants were accepted and, by creating water channels through the hardstanding and spouting up silt from these channels a water-rich area was formed. Part of this water-rich area was subsequently taken over for the site of the Phase 3 computer building.

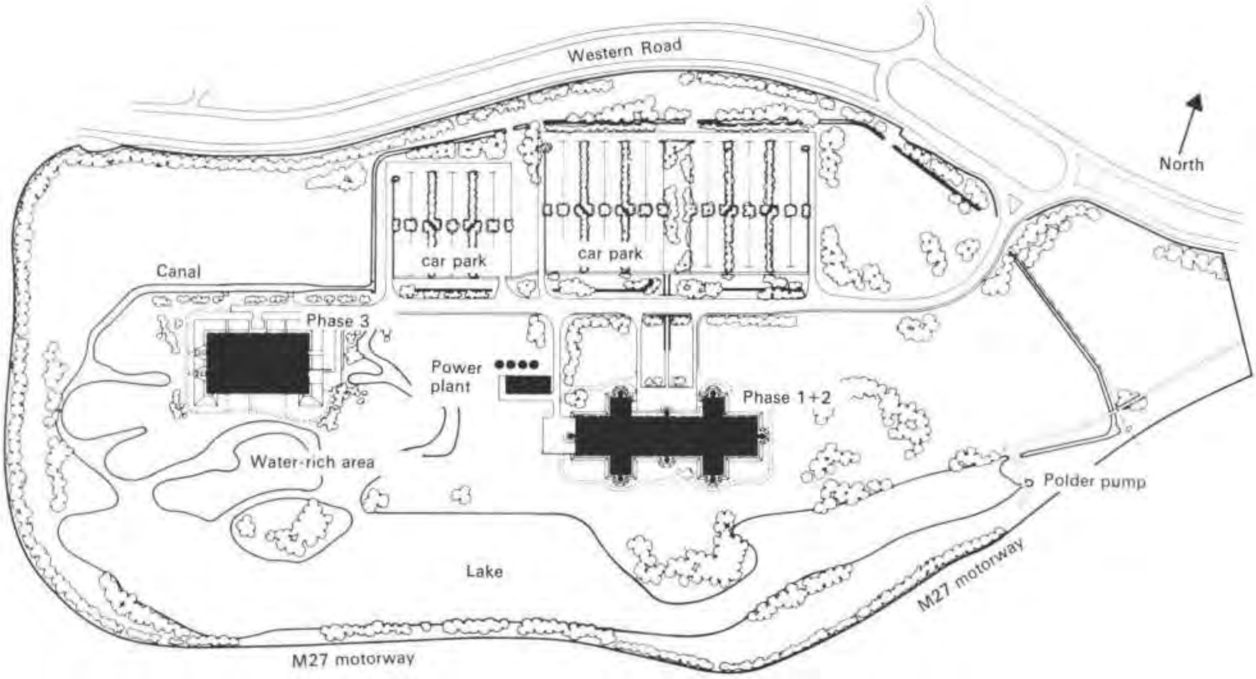


Fig. 2
Site plan

THE MASTER PLAN

Existing site zoning

In the 1970 master plan three broad zones were established for:

(1) Car parking and roads

(2) Building development

(3) Landscaping/polder lake.

The position and shape of the polder lake was determined by the contours of the strata of chalk beneath the overlying silt. The lake thus formed a natural barrier between the possible building development zone and the M27 motorway and it was proposed that this southern part of the site should be treated solely as a landscaped lake zone providing a valuable amenity out of an engineering necessity.

After considering several alternatives for the location of the car parking area and road systems serving them, a zone to the north of the site was proposed. This siting related well to the available site access points; it placed the cars to the north of any building development, providing a buffer between buildings and Western Road and the industrial estate to the north, and offered a free building development zone overlooking the landscape lake. This siting also allowed the building development and car park/road developments to grow in step with one another.

Within the car parking/road zone a detailed proposal relating to the ultimate development was made for a one-way road system serving four car park units on two levels. This road system connected with the access off Western Road and also with an access under the M27 motorway to Portsbridge roundabout. It was demonstrated that, with the car population forecast for the development, on-grade parking would reduce the building development zone by 50% and produce very long walking distances from cars to work places. In addition, the parking areas would be so large and

Key
1 Power house
2 Main entrance
3 Staff entrance
4 Mail room
5 Reprographics
6 Microfilm
7 Store
8 Refuse
9 Offices
10 Kitchen stores
11 Kitchen
12 Servery
13 Dining room
14 Wash up
15 Interview rooms
16 Electrical plant
17 Telex
18 Telephones
19 Medical suite
20 Education department
M Men's lavatory
W Women's lavatory

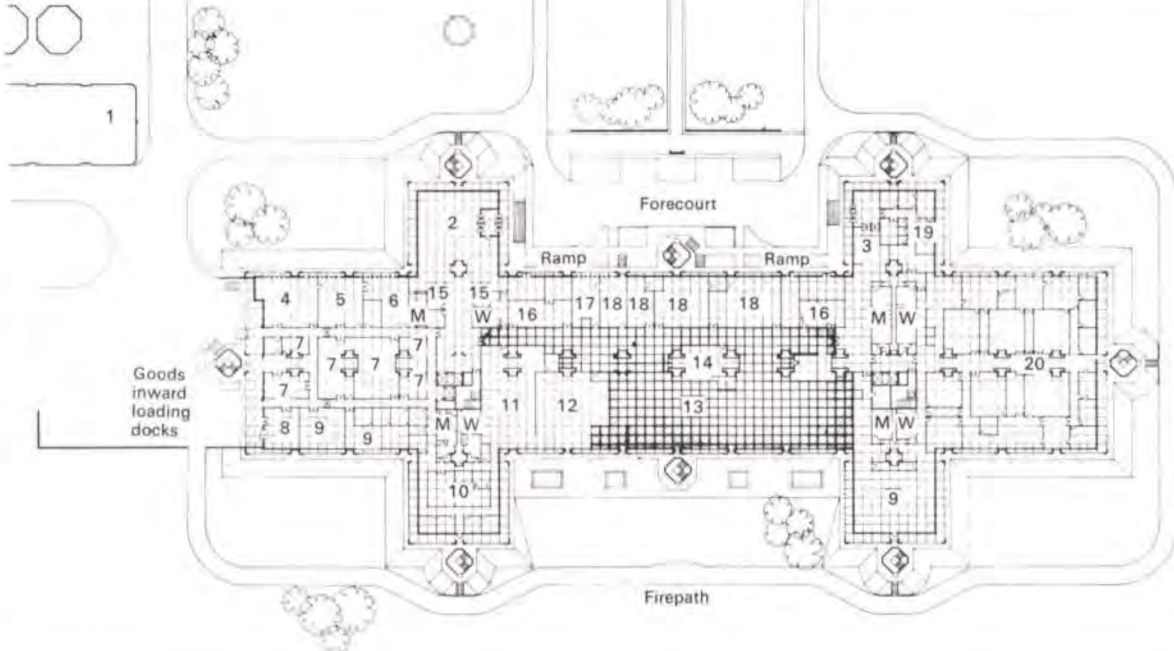


Fig. 3
Phases 1 and 2:
ground floor plan

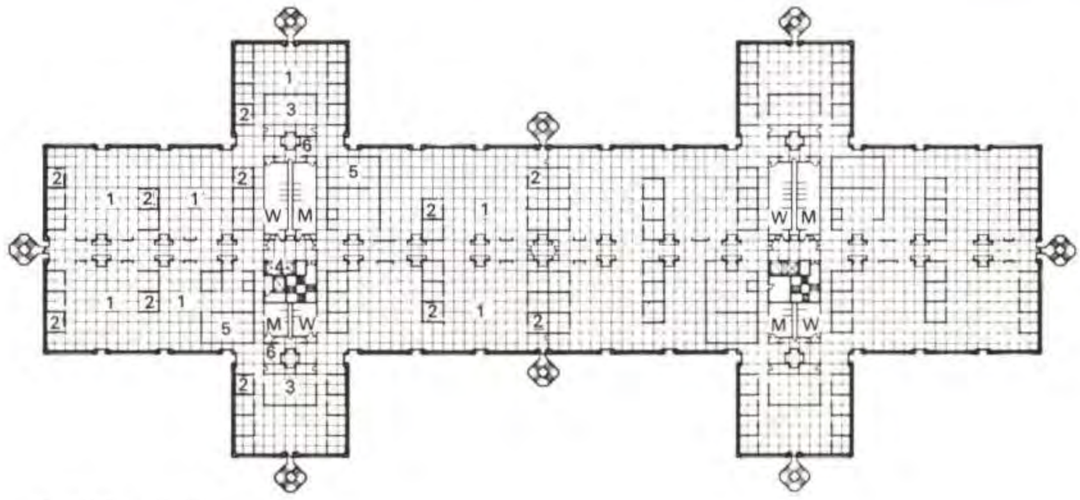


Fig. 4
Phases 1 and 2:
first floor plan

the available space for landscaping so limited that the huge expanse of cars would not successfully be hidden from surrounding roads and the upper floors of buildings. The visual field would, therefore, be dominated by the car parks and the resulting development would be compromised.

The building zone proposed in the 1970 master plan was a broad east-west zone in the centre of the site between the lake and car parks. It was recommended that the major building development should be related to the car parks and this led to a zone to the east of the site being reserved for special buildings with low population densities. One of the main factors in determining the position of the building zone was the noise levels generated by the traffic on the surrounding roads and motorways. Hence the building zone stopped short at the western site boundary which is adjacent to a spur road leading onto the M27 motorway.

Landscaping

The landscaping of this site has always been considered as a very important and integral part of the design of the development.

The landscaping concept is intended to create over a period of time a park-like natural landscape in the manner of the 18th century English tradition.

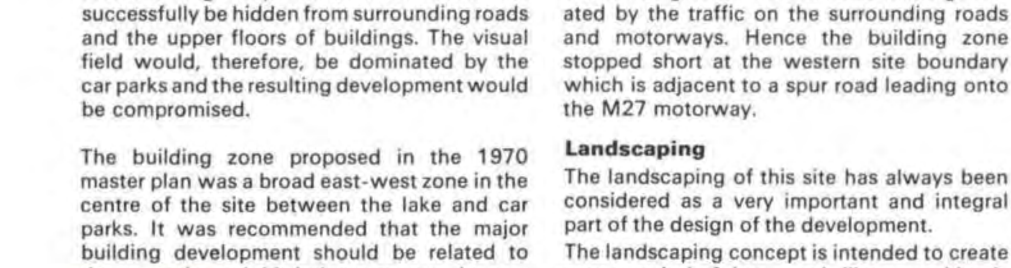


Fig. 5 (above)
Phases 1 and 2: view of entrance wing
(*Photo:* Arup Associates)

Fig. 6
Scheme 3
(*Photo:* Arup Associates)

Planting generally has begun but adverse weather conditions over the past two years, combined with the natural hazards of a site reclaimed from the sea without imported topsoil, have produced problems which have retarded progress. Even so it has always been recognized that it will be many years before the trees which have been planted will fulfil their roles of softening the motorway embankments, of adding perspective, of acting as a foil to the buildings and creating a backdrop to the lake.

The Development

In 1972 a complete scheme (Scheme 3) was designed for some 82,300 m² of building which included a separate IBM function (RESPOND) with the administrative headquarters. It incorporated a central services building linked to two four-storey office buildings, one for the administrative headquarters and one for RESPOND with two separate computer machine room buildings serving each function.

The existing development on the site consists of three phases of Scheme 3.

The first two phases were completed in 1975 and 1976 respectively and together provide 33,400 m² gross outside area in the form of a four-storey office building with temporary locations for central services, cafeteria and education facilities on the ground floor. Phase 1 also included a separate power house designed as part of the central services block proposed in Scheme 3. Phase 3 of the development was also completed in 1976 and provides 9940 m² of computer machine space with back-up facilities for the RESPOND function. Although larger in area than envisaged in the Scheme 3 proposal, this two-storey building is sited according to the parameters set out in both the 1970 master plan and the design of Scheme 3.

The existing road system is the southern leg of the proposed one-way loop road and is currently used as a two-way road operating on a tidal flow basis. This road serves three grade level car parks with access roads leading to the three phases of the building. The two car parks to the west are sited in accordance with the 1970 master plan proposals for two level car parks but the eastern car park is not. The latter was constructed at IBM's request (to bring parking spaces closer to the buildings) on the basis that its continued existence would be reviewed when two-level car parks became a necessity. In order to allow a future second access onto the site, two bridges, designed by Ove Arup and Partners and paid for by IBM, have been built on the M27 motorway. The roads giving access to Portsbridge roundabout will be built at a later stage when the car population exceeds 2500.

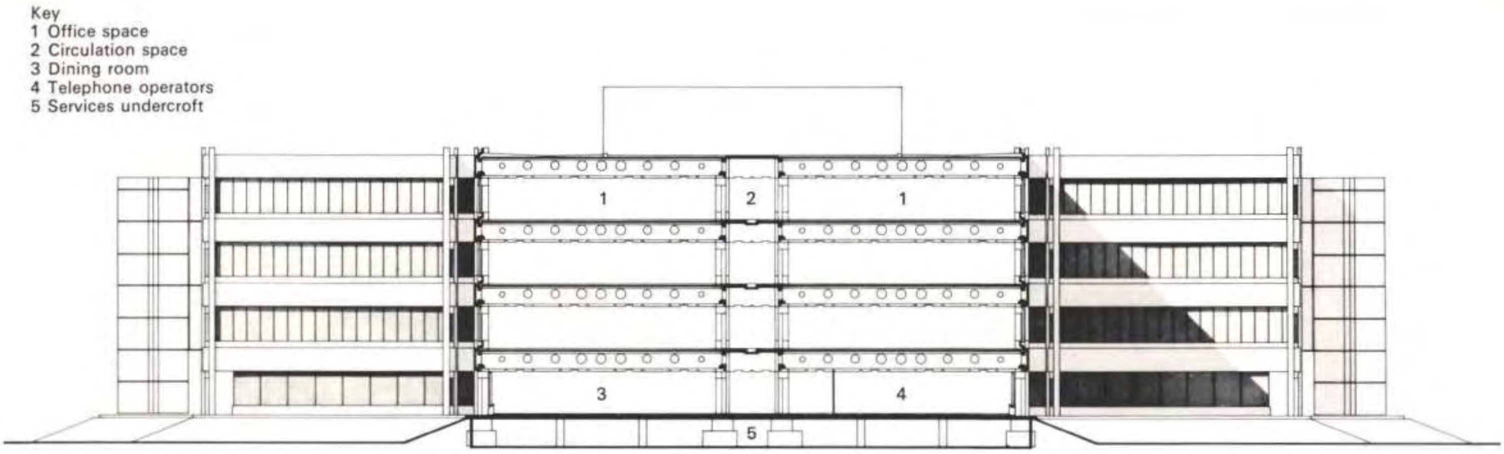


Fig. 7
Phases 1 and 2: section

Fig. 8
Phases 1 and 2: south elevation
(*Photo:* Arup Associates)

Fig. 9
Aerial view of site in 1977
(*Photo:* Handford Photography)

Phases 1 and 2

IBM were somewhat undecided about the type of office space needed to satisfy their organizational requirements. Within their organization there is a need for both group space and managers' offices. The relationship between those who work in group spaces and their managers is very close and these departments vary in size so that in some cases the ratio between manager and staff may be 1 : 10 or higher and in other cases as low as 1 : 2 or even 1 : 1.

One way of satisfying this organizational requirement is to provide totally cellular space and in recent IBM headquarters on the continent, this type of planning is being implemented. Another way is to design totally landscaped open offices as in the IBM Italian Headquarters but IBM United Kingdom Ltd. categorically ruled out this alternative.

At the design stage of North Harbour many planning alternatives were fully investigated and in 1972 IBM had an independent study carried out, the results of which suggested that their needs could best be fufilled by partially open and partially cellular space. The brief to Arup Associates required a fairly deep plan building with maximum flexibility and adaptability to allow a wide range of internal planning possibilities for the small 3.6 × 3.6 m managers' offices. Such a brief not only demanded a highly organized building anatomy but also demanded an awareness and observance of internal space planning rules in order not to erode the design intentions. A further complication to the brief was the need for some more senior managers to have individual offices with windows to the outside.

Thus a hierarchical pattern of space was demanded – managers' offices with outside windows, managers' offices related to group space and group spaces. The requirement therefore was for a building with large floor

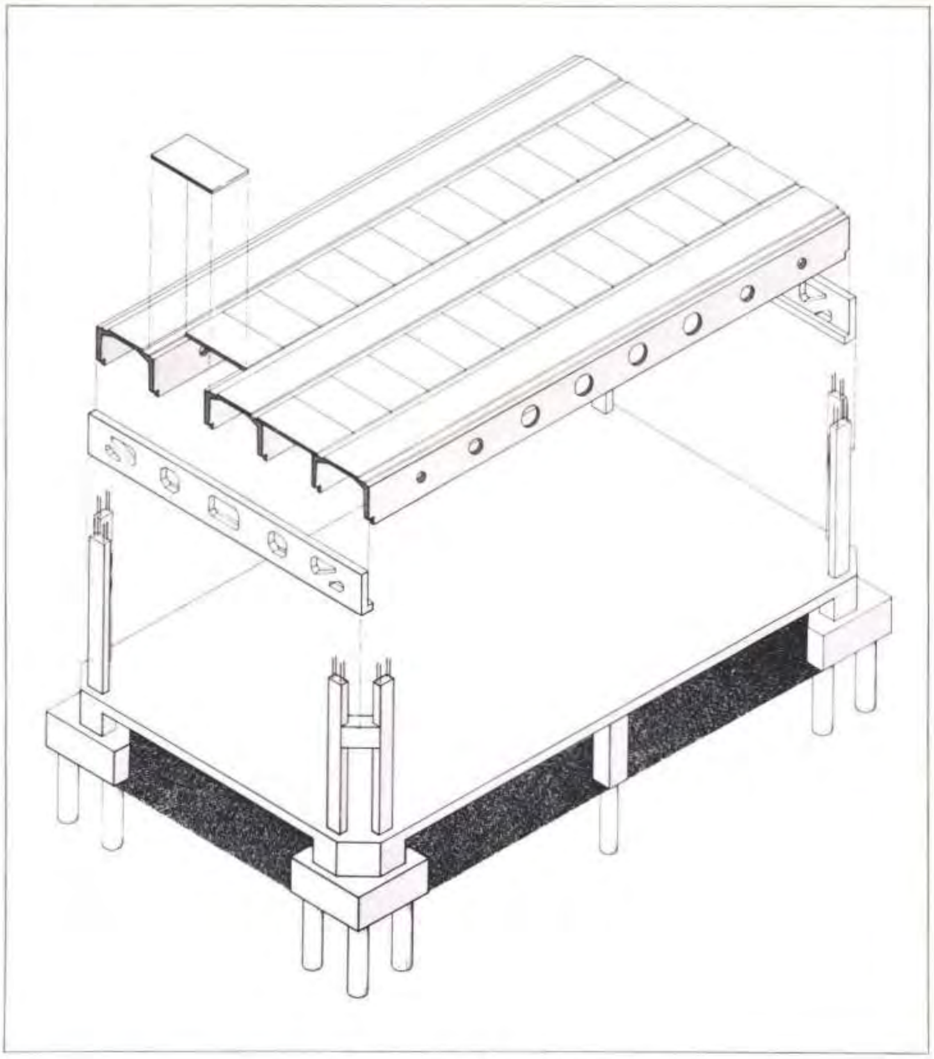


Fig. 10 (above)
Phases 1, 2 and 3: structural bay

Fig. 11 (left)
Phases 1 and 2 under construction
(*Photo:* Arup Associates)

Fig. 12
Phases 1 and 2: east end elevation
(*Photo:* Arup Associates)

areas, instant flexibility and adaptability, and an extended perimeter.

The building is a classical Arup Associates precast concrete frame building with long spans, minimum fixed vertical internal obstructions and an integrated services distribution system.

The frame, the main element of the architectural expression, is totally precast concrete with pairs of columns supporting primary beams spanning 9 m which in turn support inverted prestressed U-beams spanning 16.2 m. These 16.2 m by 9 m bays are separated from each other by minor bays of 1.8 m widths and 3.6 m widths through which the vertical services rise and into which are plugged the external fire escape stair towers. The finish to both the columns and cladding panels is exposed Portland Capstone aggregate in white cement, the columns being tooled and the cladding panels being grit-blasted. To give maximum outside awareness to all parts of the building the external wall of bronze-tinted glass runs between spandrels and behind columns on all floors and on all façades.

Apart from white concrete and bronze-tinted glass the only other material used on the site is brown brindle brickwork. This is used for the ground floor walls, external pavings and steps, and for the power house complex.

The building is fully air-conditioned and the horizontal services distribution is within the depth of the inverted U-beams. To meet the need for flexibility of planning, the 1.8 m × 1.8 m planning grid is evident in the suspended ceiling which is designed to take the variable air volume air-conditioning outlets and lighting fittings on a modular basis. This ceiling, which consists of removable acoustic panels on a specially extruded aluminium track, allows the demountable partition system to be placed on any grid line.

Fig. 13 (above)
Phases 1 and 2: ground floor detail
(*Photo:* Arup Associates)

Fig. 14
Phases 1 and 2: escape stair towers
(*Photo:* Arup Associates)

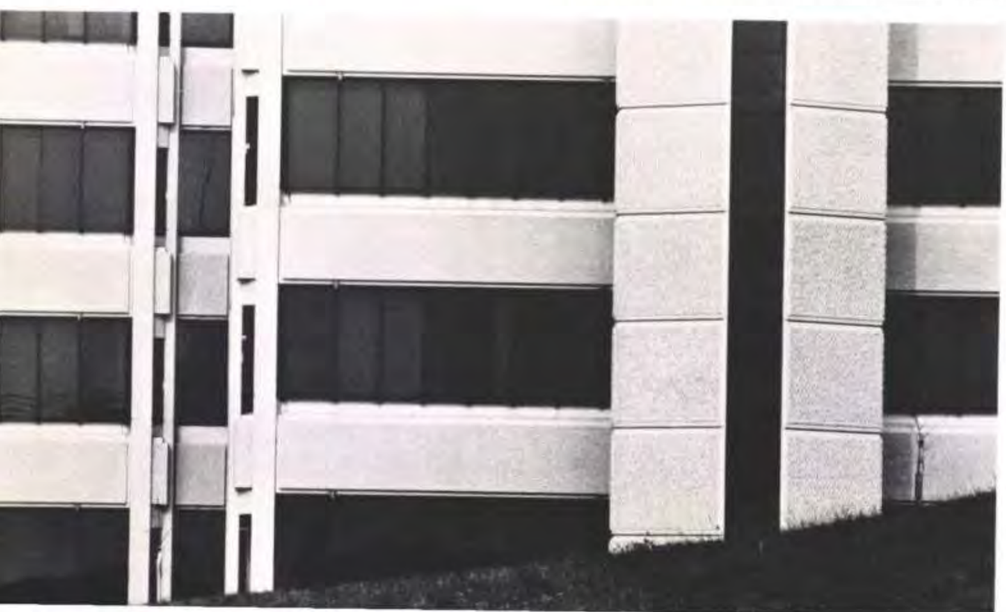


Fig. 15
Phases 1 and 2: elements of the façade
(*Photo:* Arup Associates)

Figs. 17 & 18 (above and right)
Phases 1 and 2: perimeter walkway
(*Photos:* Arup Associates)

Fig. 16
Phases 1 and 2: concrete textures
(*Photo:* Arup Associates)

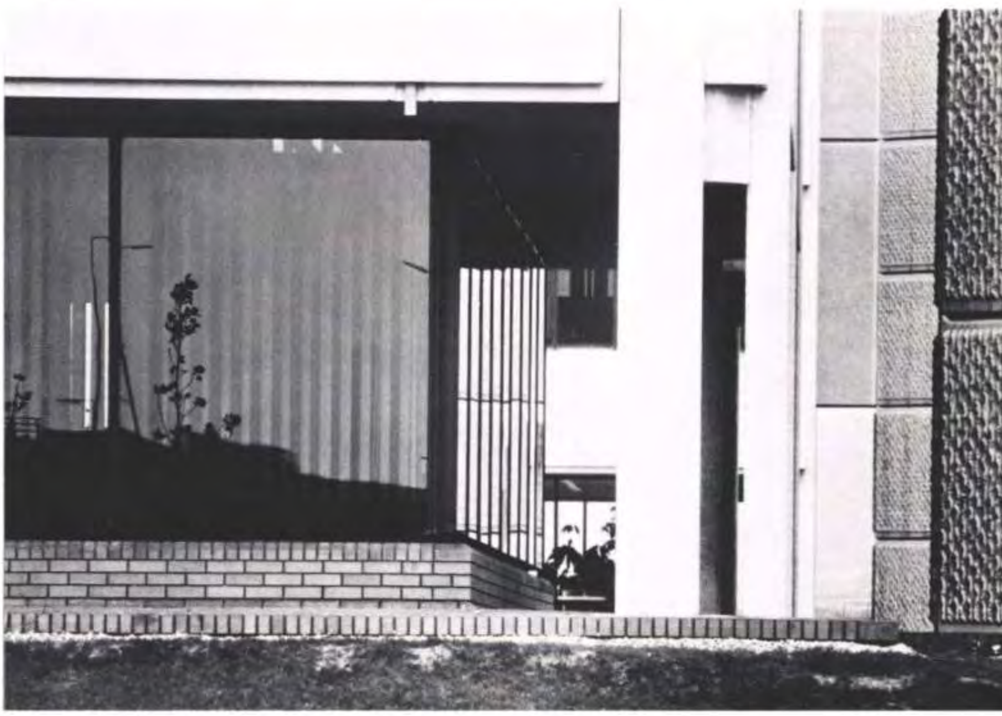


Fig. 19 (below)
Phases 1 and 2: circulation area
(*Photo:* Arup Associates)

Fig. 20 (bottom right)
Phases 1 and 2: ground floor lift lobby
(*Photo:* Arup Associates)

Fig. 21 (right)
Phases 1 and 2: staircase detail
(*Photo:* Arup Associates)

Fig. 22
Phases 1 and 2: cafeteria
(*Photo:* Arup Associates)

The nature of the settling site demanding a suspended ground floor slab, the high water table and the flood risk, all combined to make it reasonable to design the ground floor at 1.8 m above the site level giving an undercroft space for ground floor main services distribution which includes a document handling system.

The theories of flexible and adaptable buildings first exemplified in the University of Loughborough developments depended upon the sensitive use of the freedom which this flexibility and adaptability gave to the space planning of buildings. If this freedom is abused by insensitive planning then the advantages of adaptability and flexibility are outweighed by the disadvantages of poor internal environments. Inherent in the design concept for this building was the need to plan the spaces with great care and at all times preserve a sense of spaciousness and place. The building was not designed as, nor intended to be used as, cellular space except in those areas designated as such. Arup Associates, whilst having only partial responsibility for the design of the interior, made positive recommendations for the internal space planning and furnishing – recommendations which would preserve the design intentions. Having largely ignored these recommendations in fitting out the building and now realizing that both the quality and effectiveness of the space is impaired, IBM have decided to implement the original proposals although this can now only be done over a fairly long period of time.

Fig. 23
Phases 1 and 2: third floor office space
(*Photo: copyright* Fram International Ltd.)

Fig. 24
Phases 1 and 2: waiting area in office space
(*Photo: copyright* Fram International Ltd.)

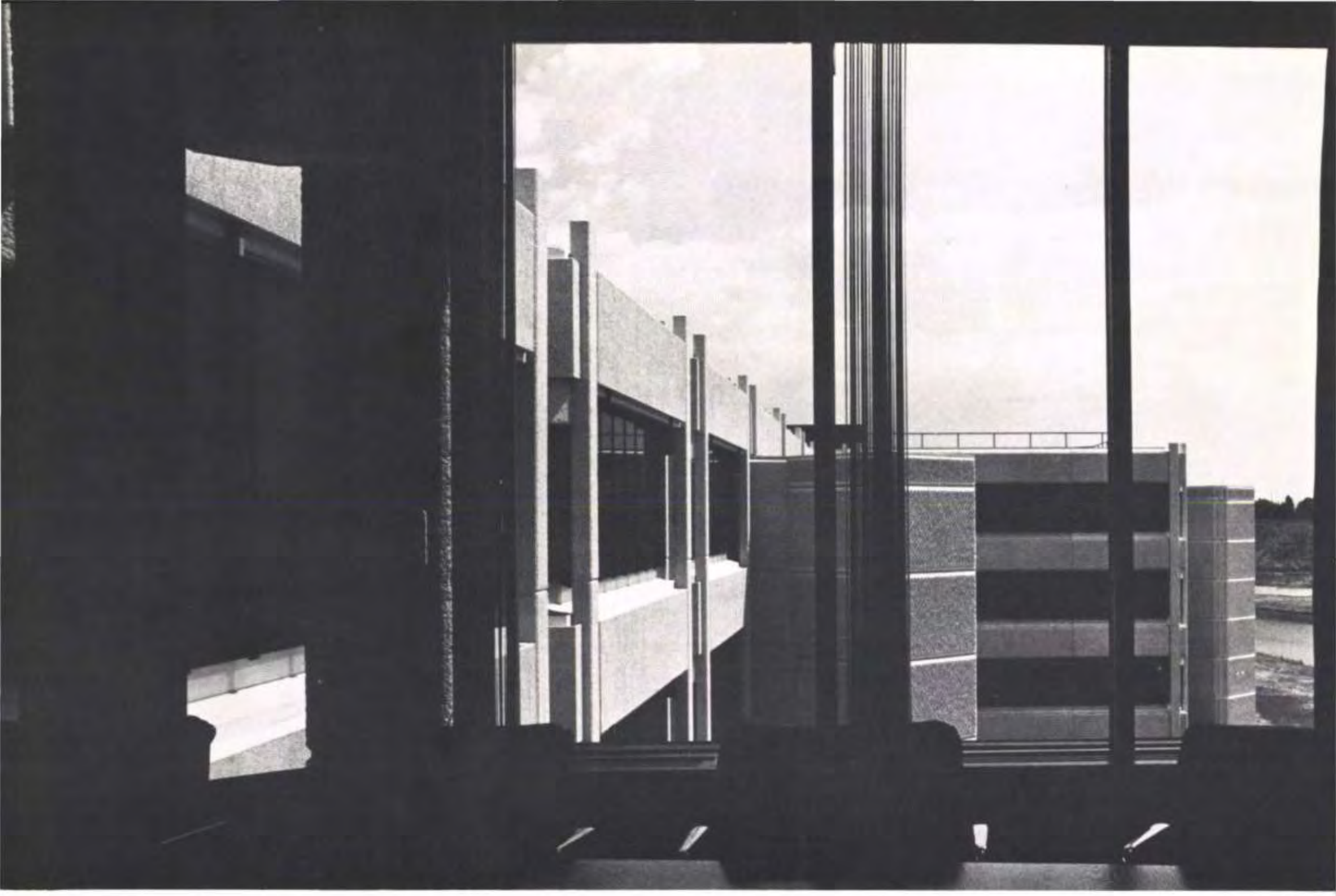
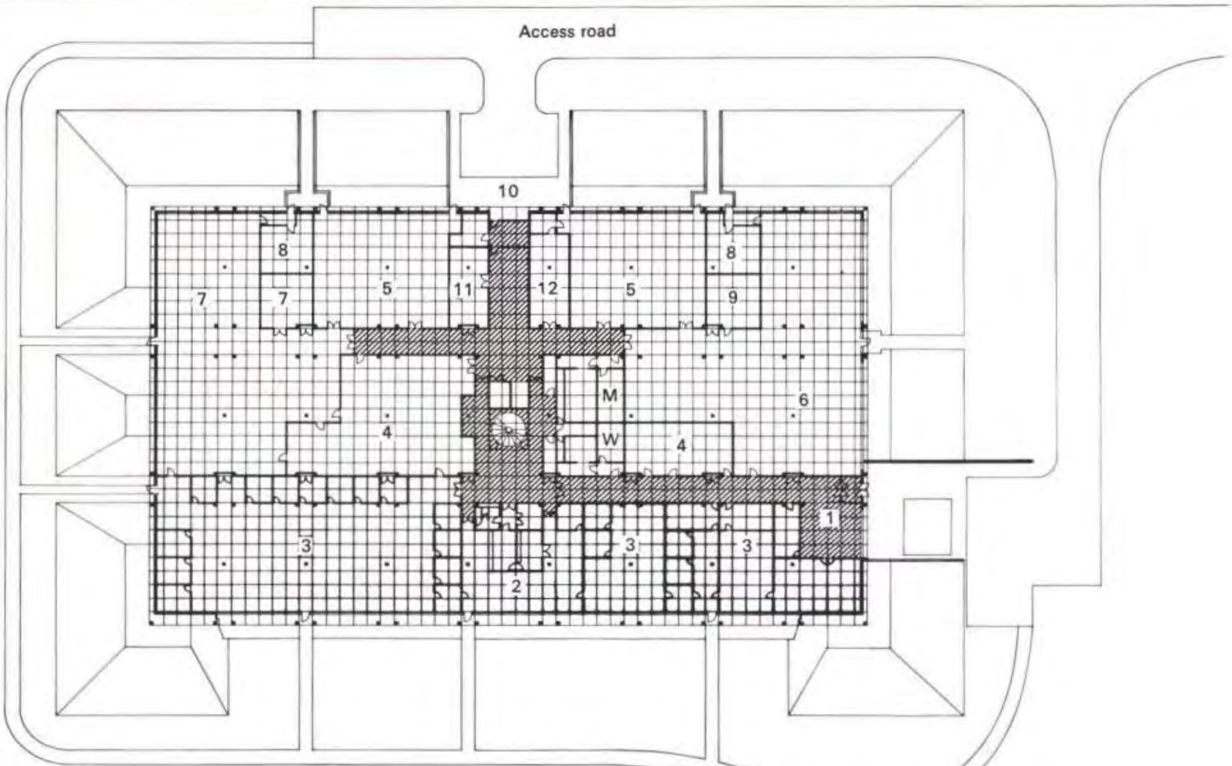


Fig. 25 (above)
Phases 1 and 2: view from third floor conference room
(*Photo:* Arup Associates)

Fig. 26
Phases 1 and 2: corner office
(*Photo:* Arup Associates)



Key
1 Entrance
2 Cafeteria
3 Office space
4 Workshop & stores
5 AC Plant
6 Generators
7 Electrical plant
8 Sprinkler pumps
9 Oil store
10 Goods entrance
11 Store
12 MDF room
M Men's lavatory
W Women's lavatory

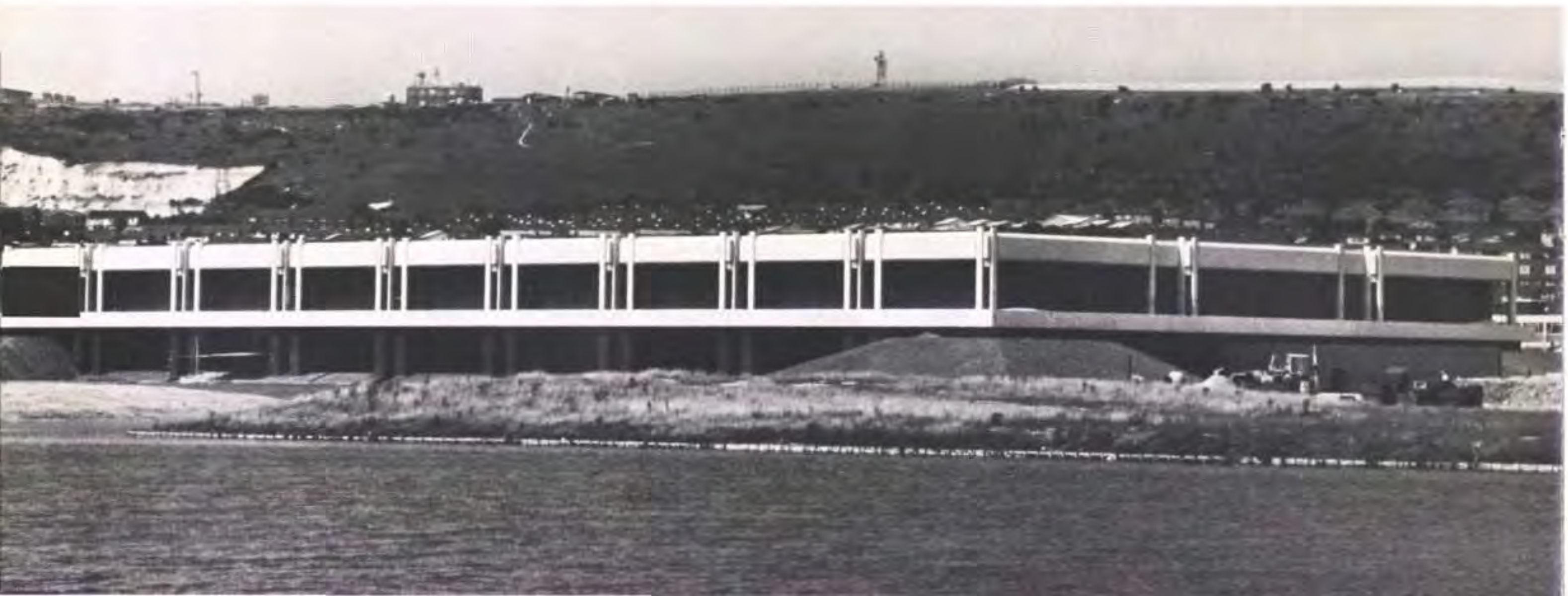
Fig. 27
Phase 3: ground floor plan

Phase 3

The third phase of the development is a computer building commissioned during the construction of Phases 1 and 2. This is a two-storey building with the two large computer rooms on the first floor and back-up office space and services plant rooms on the ground floor. Office areas have an outlook over the water-rich landscape area. The structural system developed for Phases 1 and 2 is used for the first floor superstructure but the first floor slab and support columns have been specifically designed to meet exacting fire insurance requirements. Security and environmental control problems specified in the brief for this building demanded solid external walls for the computer rooms with minimal window areas.

Fig. 28 (right)
Phase 3 under construction
(*Photo:* Handford Photography)

Fig. 29 (below)
Phase 3: general view across lake
(*Photo:* Arup Associates)



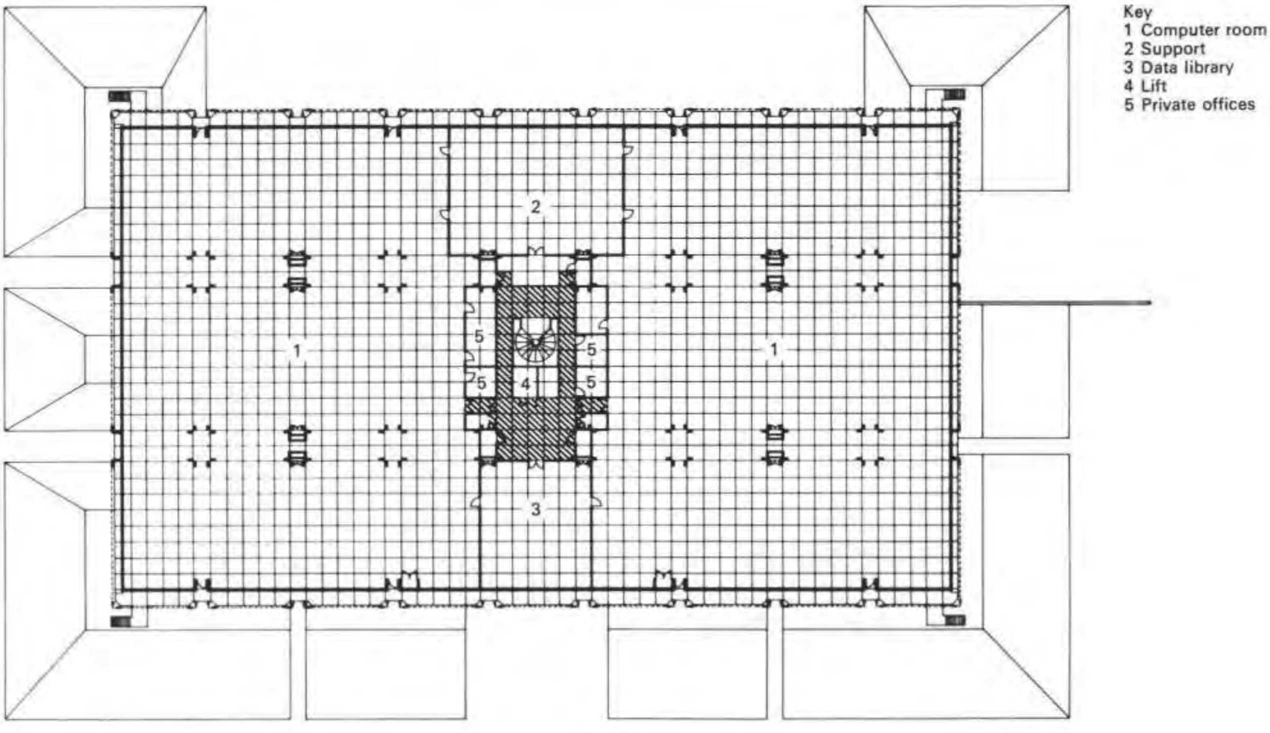


Fig. 30
Phase 3: first floor plan

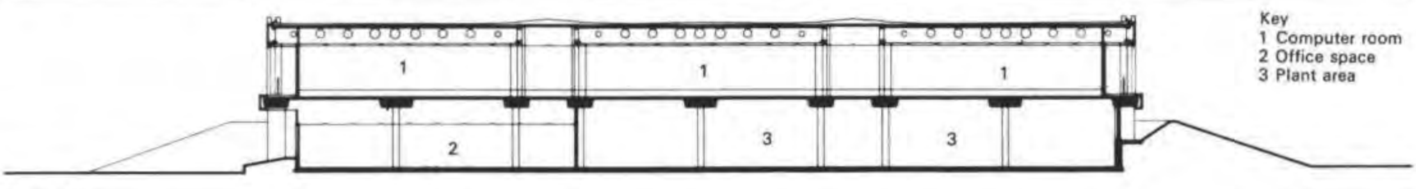


Fig. 31
Phase 3: section

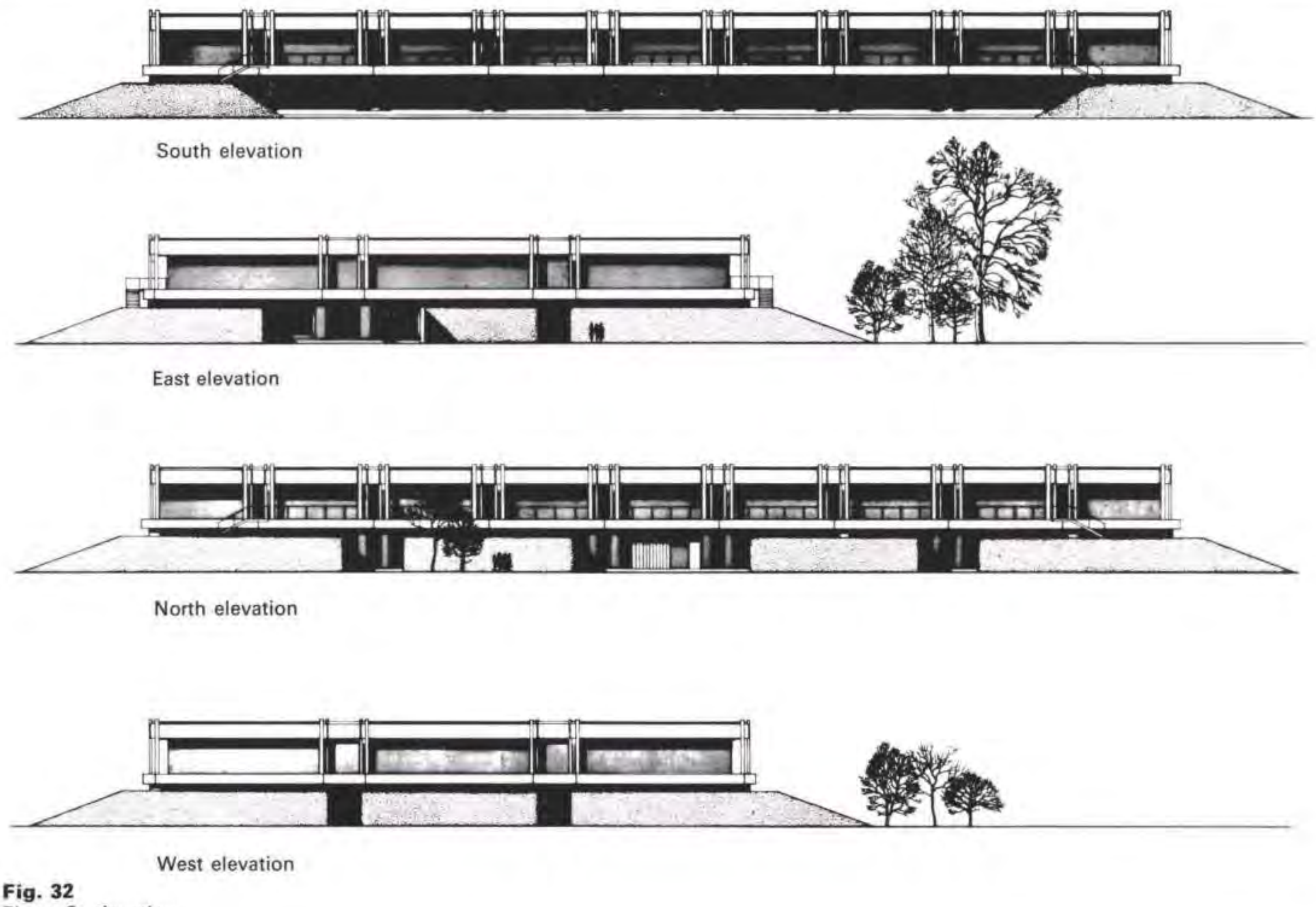


Fig. 32
Phase 3: elevation

Fig. 33
Phase 3: elevation detail
(*Photo:* Arup Associates)

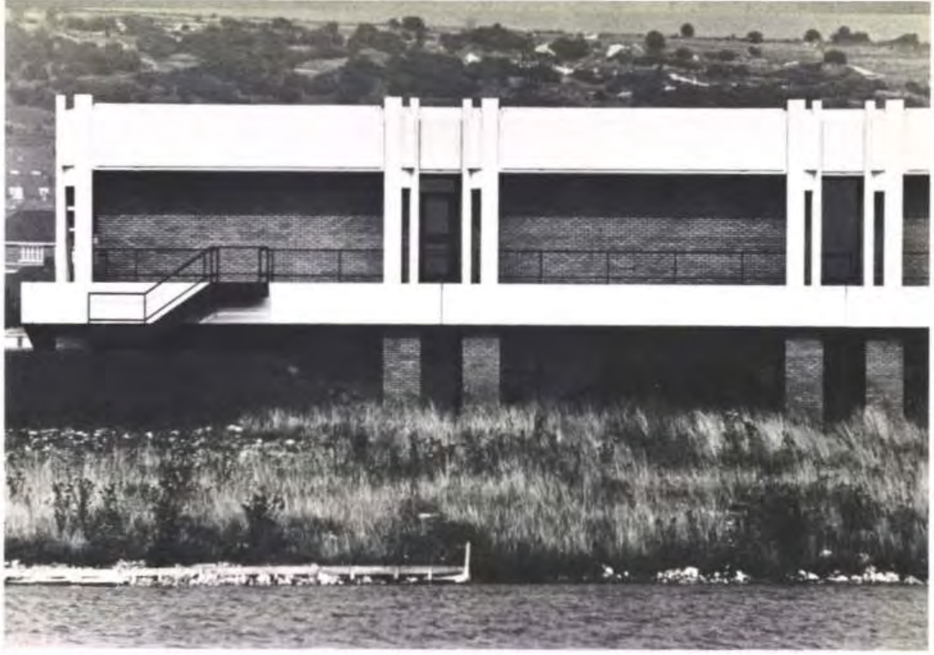


Fig. 34 (left)
Phase 3: elevation detail
(*Photo:* Handford Photography)

Fig. 35
Phase 3: stair landing at first floor
(*Photo:* John Hopkins)

Fig. 36
Phase 3: external gallery
(*Photo:* Arup Associates)

The future

A fourth phase of the development is now being planned and this will include some 27,900 m² of additional office space and 10,200 m² of central amenities, cafeteria and main entrance area. This phase will effectively complete the development although a further 23,200 m² of building will still be possible on the site.

In planning this phase we will be able to remove the temporarily planned cafeteria and service areas from Phases 1 and 2 and establish the main circulation routes throughout the building complex, linking Phases 1 and 2 with Phase 3. This phase also provides the much needed focal centre to the development. In the five years since Scheme 3 was developed many changes have taken place in IBM's requirements: we have also learned a great deal about IBM as a user of our buildings and whilst the general strategic planning will remain unaltered in principle, the detail design of this next phase will be quite different from the 1972 concept.

Credits

Client:
IBM United Kingdom Ltd.

Architects + Engineers + Quantity Surveyors:
Arup Associates

Management contractor:
Taylor Woodrow Construction Ltd.

The new grandstand at the Crystal Palace National Sports Centre

André Bartak
David Kaye
Tim George

Key
1. Existing stand
2. Sports hall
3. Hostel
4. New stand
5. New scoreboard
6. New training pool

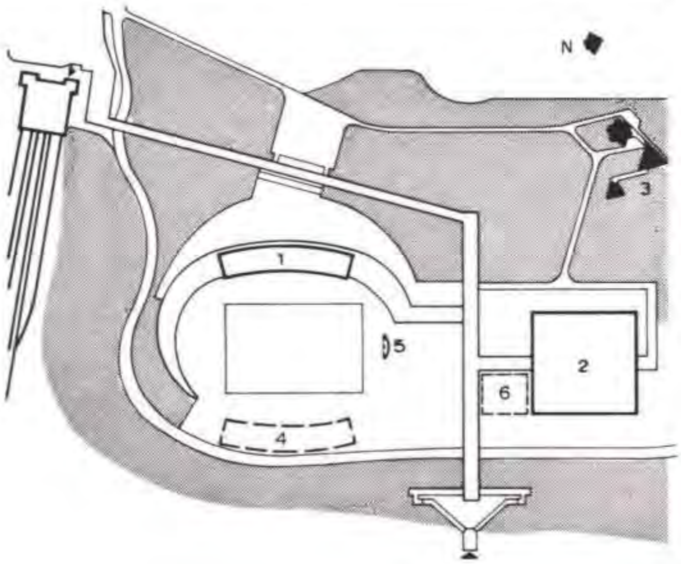


Fig. 1
Site plan

INTRODUCTION

Background

When in 1953 the decision was made to build a sports centre worthy of the nation at the Crystal Palace Site, Sir Leslie Martin, then the Chief Architect to the LCC (now GLC), was commissioned to prepare, in conjunction with the Council for Physical Recreation, a scheme for a centre capable of providing all the facilities needed for training of athletes to international and Olympic standards.

In view of the history of the site and the limited resources available this was a formidable challenge.

The appointment of Ove Arup & Partners as consulting engineers was made at an early stage, thus enabling the engineers to make a significant contribution to the design and resulting in structures which were both elegant and advanced for their time.

The scheme, which is well-known to Londoners, is represented diagrammatically in Fig. 1. The main elements were a sports hall, a stadium and an athletes' hostel. The stadium had an overall seating capacity of 12,400 of which 4,000 seats were under cover.

The project was realized in 1964 and proved an immediate success. However over the years a gradual change of use emerged, in that the participation by the local community increased rapidly to a dominant role. In order to cater for the expanded use and to encourage its future increase, it was decided to extend the facilities and in 1972 The Department of Architecture and Civic Design of the GLC was briefed to develop designs for the new training pool and the new covered grandstand. In view of their involvement in the original scheme, Ove Arup & Partners were asked to act as consulting engineers for the new development.

The training pool was subject to a separate contract and was completed in June 1976. The new grandstand was completed in October 1976, and this article, apart from a brief excursion into the colourful history of the site, describes the structure, its design and construction.

History of the site

The whole of the Crystal Palace Park at Sydenham is 81 ha in extent. Paxton's famous Crystal Palace stood on the highest point of the park from 1854 to 1936. Before this, the park was the grounds of a large mansion called Penge Place, owned by Mr. Shuster, a director of the railway from London Bridge to Brighton, which ran nearby.

The Crystal Palace was first built in Hyde Park by the Serpentine for the Great Exhibition of 1851. Shuster and Paxton were directors of the Sydenham Company formed to acquire the fabric of the building at the close of the exhibition in the autumn of that year. It was re-erected in an adapted and more grandiose form on Shuster's land at Sydenham, and opened there in 1854. Its impact on the history of architecture is well recognized, and of course documented in a number of books.

The site of the present athletics track, in a bowl below Paxton's building, was occupied at that time by one of two basins which formed part of the elaborate water display of cascades and fountains which Paxton engineered. Later on, the basins were filled in and this site became a football ground. The FA Cup Finals were held here from 1895 to 1911, watched by crowds of the order of 70,000 people. Two covered stands were built, roughly on the site of the present stand built in 1964. In 1911 the entire park, Palace and contents were auctioned because of the failure of the original company, and bought for the nation.

In 1936 the history of the Palace as a place for public entertainment came to an end when it was destroyed by fire. The reconstruction scheme for the area, projected after the war, envisaged an exhibition centre on the site of Paxton's building, and a sports centre on the site of the two basins. The former was never built; but the latter was opened in 1964, thus maintaining the sporting environment associated with the site.

Of the Crystal Palace itself, nothing remains except some of the arches supporting the terraces, but Paxton's magnificent arcade under Crystal Palace Parade, with polychrome brick fan vaulting which led from the now demolished high-level railway station directly into the Palace, still exists. The south water tower, 61 m high, built by Brunel, survived the fire, but was demolished afterwards because of fears for its stability. Interest has been generated recently by an exploratory dig to locate an experimental underground railway tunnel built in the last century but, at the time of writing, it has not been found.

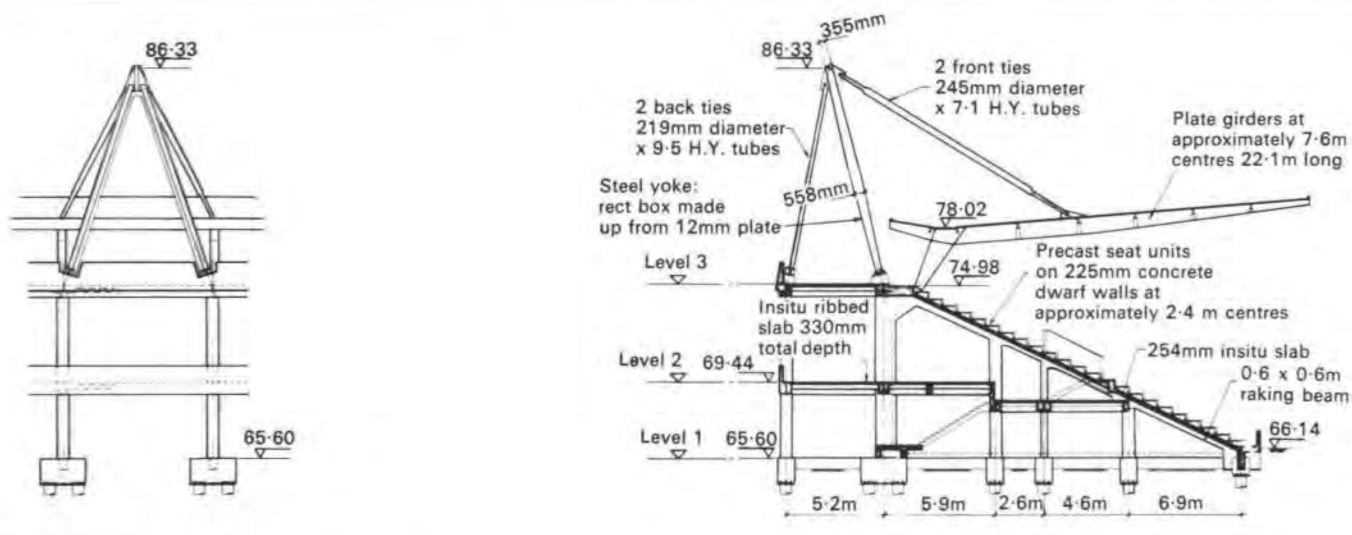


Fig. 2
Cross-section through new grandstand

Key
A PVC coated metal decking
B GRP gutter
C Tubular back ties
D Built up box strut
E Tubular front ties
F UB purlins at 3·05m centres
G Insitu concrete frames
H 254mm insitu sloping slab
J Precast concrete seat units
K Insitu reinforced concrete ribbed slab

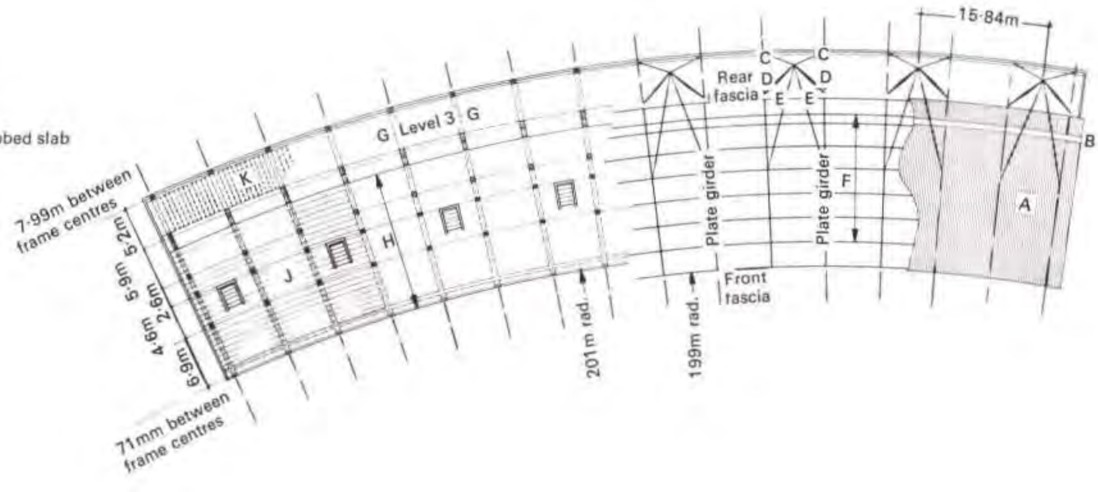


Fig. 3
Plan

DESCRIPTION

The new grandstand is set out on a large radius curve, is 122 m long and 27.4 m wide. It consists of a concrete superstructure founded on piles, and covered by a structural steel canopy (Figs. 2 and 3).

The terracing is of non-structural precast concrete seating units supported by a 254 mm thick in situ sloping suspended slab spanning between main reinforced concrete frames placed radially at 7.6 m centres. Other intermediate floors are of ribbed slab construction. The roof of the canopy incorporates 16 welded plate girders, each 22.1 m long overall and spaced at approximately 7.6 m centres. These girders are primarily cantilevers, vary in depth from 1.07 m to 0.38 m at the tip, and weigh 5000 kg each. The web plates are mainly 10 mm and 12 mm thick, and the flanges 20 mm and 22 mm thick. Universal beams spaced at 3.05 m centres span between them and carry the roof decking. The 'stressed skin' PVC coated metal trough decking is connected to the purlins by means of special watertight fasteners.

Fig. 4
Backtie-holding down detail
(*Photo:* Harry Sowden)

Eight large pyramid-shaped assemblies of steel tubes, springing from the back terrace behind the seating, offer support to the roof from above, via the inclined front ties. At their ends they are of 219 mm diameter, but the long central portion of each member is of 245 mm diameter. The transition pieces are in the form of reducer cones rolled up from plates.

The principal support element of each assembly is in the form of an A-frame. The legs of these frames are of welded box section. Each box is stiffened by internal plates. At right angles to the plane of the frame the legs have a tapered appearance, the width varying between 355 mm and 558 mm. The depth of each leg is constant at 355 mm.

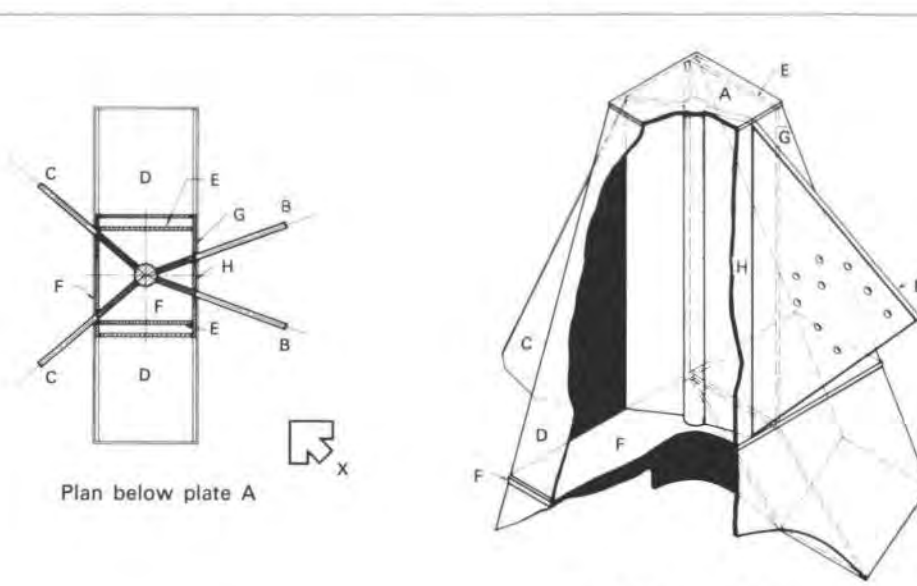
The back ties of each pyramid consist of two round tubes, 219 mm in diameter, 9.5 mm thick. These ties are anchored to the top of the concrete construction and the base details consist of a steel collar of two plates separated by six stiffeners (Fig. 4).

The capping piece of each pyramid is a complex box, shaped to form the apex of each A-frame (Figs. 5 and 6).

All the circular section tubes are of grade 50 C steel, the remainder of the steelwork being grade 43 C. All steelwork is shop-welded, the site connections being by means of high strength friction grip (HSFG) bolts.

The two rear tubes of the support pyramid have large tensile forces from anchoring the roof, which are transferred to the reinforced concrete columns beneath the rear elevation. The reinforcement in these columns is effectively continuous as the 32 mm diameter bars are joined by CCL Alpha tension splices. The tops of the reinforcing bars project out of the

Fig. 6
Capping piece
(*Photo:* Harry Sowden)



Plan below plate A

View in direction X

Fig. 5
Capping piece

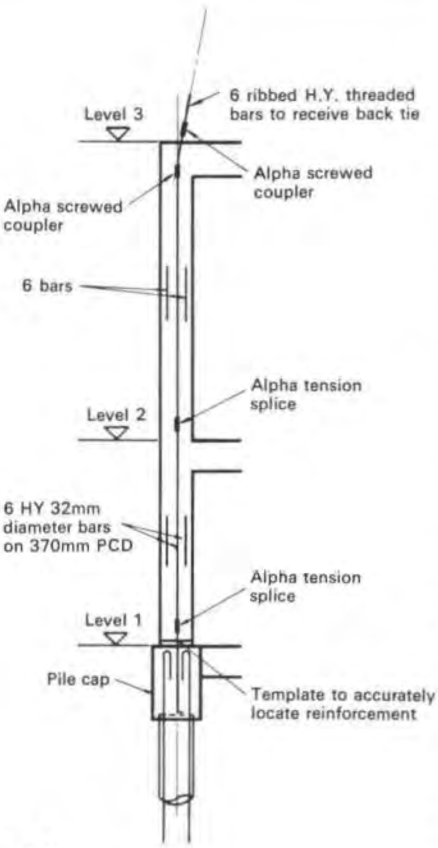


Fig. 7
Backtie-column system

concrete and are threaded; the baseplate of the back tie is fixed directly to these bars (Fig. 7).

The piling consists of in situ bored piles. There are altogether 220 piles, 610 mm in diameter, 20 m long. Out of this number, 32 are tension piles with a design load of 800 kN per pile. The remaining piles were designed to carry 1000 kN each.

The concrete superstructure is intended to house, at a later date, a restaurant, bars, and conference room, in addition to the toilets and internal circulation spaces included in the present contract.

DESIGN CONSIDERATIONS

General

The new grandstand was to be situated on the opposite side of the track *vis à vis* the existing covered seating. It was to contain 5,000 seats, thus more than doubling the number of covered seats in the stadium. The brief required the new structure to resemble the existing one as closely as possible, which was simple and effective in that it exploited the natural site topography as a basis for its structural form. The new siting did not provide the designers with a similar opportunity and a reinforced concrete superstructure was necessary in order to provide the terraced seating required. This meant, as can be seen in Fig. 8, that the back ties are anchored to the top of the concrete superstructure, rather than to a concrete counterweight in the ground, as was the case for the existing stand.

Geometrical reasons for the new stand necessitated the use of two back ties per support assembly as opposed to the single back tie employed before.

The A-frame compression members, previously constructed in concrete, were changed to steel box sections. This was considered appropriate, particularly from the point of view of erection.

These comparatively small departures, however, still enabled the basic geometry of the roof to remain.

The retention of the visual *leitmotiv* provides a link between the existing and the new stands and reflects the consistency and the continuity of the architectural design (Figs. 8 and 9).

The canopy

The roof dead weight is small. Unlike the previous roof which consisted of aluminium decking, boarding, felt and chippings, the new roof was to be PVC coated metal decking only. With the self-weight of the girders and purlins, this provided a total dead load of 55.31 kg/m². The superimposed loading used was 73.23 kg/m².

Design for wind

A frequency-mode analysis for the first five modes of vibration was carried out in order to establish the dynamic characteristics of the structure (Fig. 10). The results showed the system to be stiff and this combined with the good inertial damping characteristics, made the quasi-static approach to wind design appropriate.

It may be of interest to give here the background for this decision; however for reasons of space, only a rudimentary discussion of the problems involved is possible.

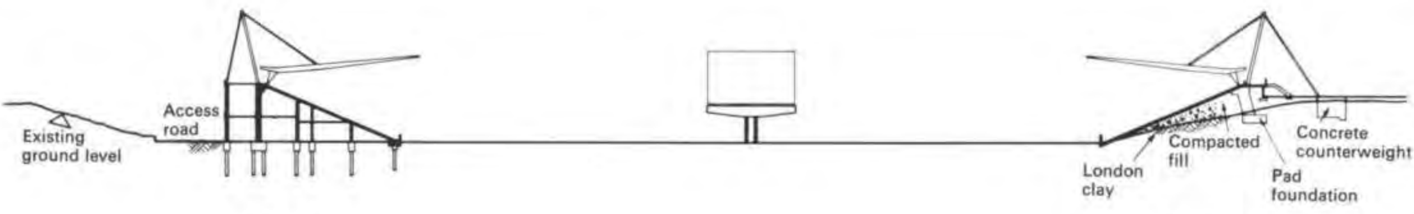


Fig. 8
Cross-section through the old and new stands

Fig. 9
Photograph of the architect's model (*Photo: copyright* Greater London Council)

The main response of a building to the action of the wind is caused by the buffeting effects of turbulence. As the terms imply, forces are imposed on the structure by turbulent wind, i.e. by the variation of wind velocity due to gusting. These forces and the movements imparted to the structure are, of course, in the plane of the wind action. Although wind gusting is a random phenomenon, having no periodicity, its structure can be represented as consisting of a number of single frequency components. A condition for dynamic excitation can exist when one of these components has a frequency similar to the natural frequency of the structure exposed to its action. The magnitude of the dynamic response of the structure will depend on the amplitude of the fluctuation of the relevant component of turbulence and the area in space over which it will be highly correlated; in other words on the intensity and the size of the relevant gusts.

This aspect is comparatively easy to visualize. However, there are two other types of dynamic excitation which can be experienced by structures even in wind of constant velocity which are less easy to understand but are potentially dangerous. For the benefit of those unfamiliar with the subject, a short discussion of these phenomena may be useful.

Vortex shedding

The first is known as excitation by vortex shedding. When a not ideally stream-lined body is subjected to a steady wind stream, a turbulent wake is formed. This consists of eddies forming on the leeward side of the body which cling momentarily and then detach themselves in a random or a regular pattern.

For a body of round cross-section, this pattern is regular in the so-called sub- and supercritical ranges and random in the transitional range (the ranges are defined by the Reynolds number). For bodies having sharp edged cross-section, regular patterns occur throughout the whole range.

The action of these eddies which form at the two leeward edges of the cross-section, is to impart forces acting at right angles to the direction of the wind, their frequency being half that of the vortex shedding frequencies – this is because the manner in which the eddies (or vortices) forming at both the leeward edges are detached (or shed) is not simultaneous but staggered – at the same time cyclic forces, with the frequency equal to that of the vortex shedding frequency, occur in the in-wind direction. These latter are, however, small when compared with the cross wind forces (Fig. 11).

It can be seen that in the régimes where regular patterns occur, transverse excitation of the structure can result, and if the natural frequency of the structure is similar to that of the frequency of the transverse forces, in some circumstances serious consequences may ensue.

Galloping instability

The remaining potentially dangerous form of excitation is commonly known as galloping instability. This may occur to aerodynamically non-symmetrical bodies subject to a steady air flow. One should mention at this point that even a geometrically symmetrical section can become non-symmetrical aerodynamically when it is tilted relative to the wind stream, at an angle which is known as the angle of attack (Fig. 12). In such a case, in addition to the drag acting in the in-wind direction, another force occurs, i.e. a lift force which has a cross-wind component. Only the circular cross-section can always remain aerodynamically symmetrical, at least in theory. In practice the symmetry can be disturbed by various agencies, e.g. by icing.

In some circumstances the action of the lift

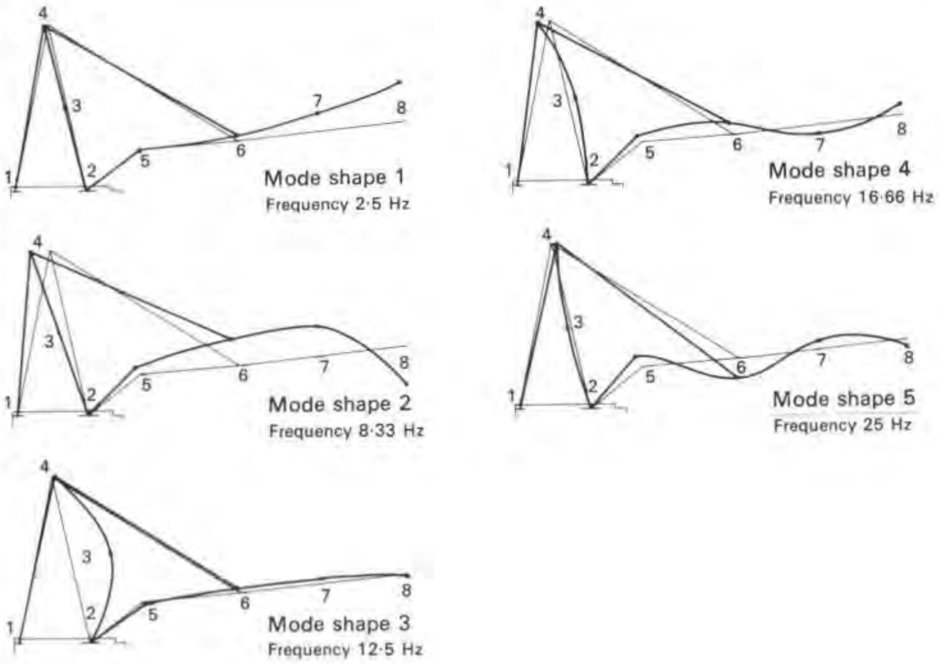


Fig. 10
Natural vibration mode shapes of the roof

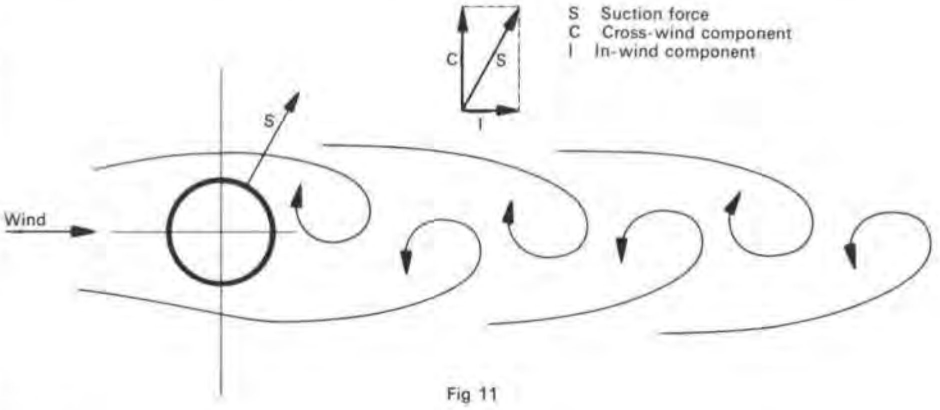


Fig. 11

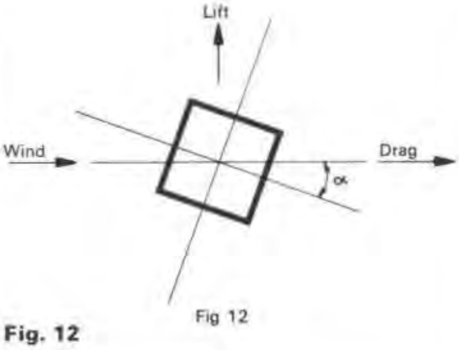


Fig. 12

forces can lead to instability conditions, resulting in large amplitude cross wind oscillations, i.e. galloping. In practice, however, this happens only to systems possessing very low stiffness. The classical examples are high tension cables and cable stays of various types, when their cross-sections are distorted by icing.

As far as the design of the grandstand is concerned, the galloping can be quickly disposed of since the high stiffness of the structure precludes such occurrence.

Our structure consists of a number of interconnected members of different properties and stiffnesses. Experience has shown that structures of this type are not susceptible to serious excitation caused by vortex shedding. This can be crudely explained by the reluctance of a collection of several interacting members, each having its own natural frequency to vibrate in unison.

Finally the buffeting by turbulence. The dynamic response of a building to this form and excitation depends principally on the natural frequency of its structure, and it is generally accepted that when this is higher than 10 Hz, the magnitude of this response can be considered negligible. Although, in our case, the natural frequency at 2.5 Hz is lower, the amplitude of velocity fluctuations for the turbulence component corresponding with this frequency, and the area over which it is highly correlated, were sufficiently small to be considered insignificant, and the design could proceed on a semi-static basis.

For this analysis, the following coefficients were used from *CP3: Chapter V: Part 2: 1972:*

Basic wind speed: 38 m/s
Topographical factor: 1.1
Surface factor: 0.83
Building life factor: 1.0

Hence a design wind speed of 35 m/s was determined with a corresponding dynamic pressure of 77 kgf/m^2.

The angle of the canopy is a little less than 5°, and the code recommended pressure coefficient of ± 1.1 was adopted, resulting in a design wind pressure of ± 85 kgf/m^2.

Due consideration was given to effects from frictional drag and wind on the fascias.

The design forces and moments are shown in Fig. 13.

Fatigue assessment

The structure was clearly going to be subjected to stress fluctuations, and some members to stress reversal. An assessment was made of the number of cycles which would reasonably occur in the structure life of 50 years. Levels of stress fluctuations were determined for two cases of mean stress, i.e. about a mean stress equal to dead load only, and about a mean stress equal to dead load plus live load. The results, when interpreted in conjunction with *BS 153*, clearly showed that fatigue would not be a serious problem, and it would be justifiable to design to normal stresses. However, as a precaution, stresses in welds were generally limited to 50% of the allowable given in *BS 449*.

Throughout the design, at welded connections, the best possible constructional details were used in order to remain in the higher class of details described in *BS 153*.

Lateral stability

The architect required that there should be no diagonal bracing in the plane of the roof. In order to provide stability against side loading of the cantilevered portion of the roof, the canopy was initially designed as a frame on plan, with HSFG bolts used to provide moment-resisting connections at the purlin/girder junctions. There was little in-plane stiffness inherent in this arrangement, and relatively large deformations could be expected. Up until then the stiffening effect of the roof sheeting had been ignored, though clearly if the deck to structure fixings were adequate, a substantial contribution would be made by the sheeting.

In recent years theoretical and experimental work has been carried out on the employment of light corrugated metal sheeting in stressed-skin construction. On this basis the design was modified to allow the metal deck to be effective as a shear diaphram, and the maximum horizontal displacement of the outermost purlin was reduced to about 7 mm.

One consequence of the method is that the connections between the sheeting and the supporting steel occur at the valleys of the troughing. To ensure watertight connections the fasteners were provided with special sealing grommets.

Precautions against brittle fracture

BS 153 was used as a guide in choosing ductile steel to *BS 4360* grade 43 C for all plated components and grade 50 C for the tubes, after considering applied stress levels, temperature range, material thickness and loading rates. Extensive non-destructive testing and the use of low-hydrogen electrodes throughout were specified.

The component which was felt to require special attention was the capping piece at the apex of each frame (Fig. 14). This unit is exposed to the elements high above the ground, is extensively welded and also subjected to high tensile stress concentration. An added problem was that after exhaustive enquiries, the fabricator was unable to obtain a solid 75 mm diameter rod, to grade 43 C. It was eventually agreed to substitute this item with a rod of material EN 14A. Weld procedure trials were specified, in particular for the butt welds to the central rod as this detail would be the most critical from the point of view of the possibility of hydrogen embrittlement. If these procedural tests showed cracking susceptibility, then preheating would be adopted.

However, due to the late and unexpected inclusion of a material outside the specified range (the EN 14 A rod), it was considered necessary to introduce preheat anyway for the welding of this particular item. EN 14 A is similar but not the same as grade 43 C steel, as it has further alloying elements including nickel and chromium which increase its hardness. In view of this, hardness tests were carried out, and also extensive ultrasonic examination of the completed unit. To ensure maximum notch ductility of the material, rods were firstly normalized by heating to a temperature of 900°C, and then preheat was applied to the rod at the time of fabrication, maintaining a minimum temperature of 100°C. Again low hydrogen electrodes were used for all weld runs. Stress relieving was considered but not adopted.

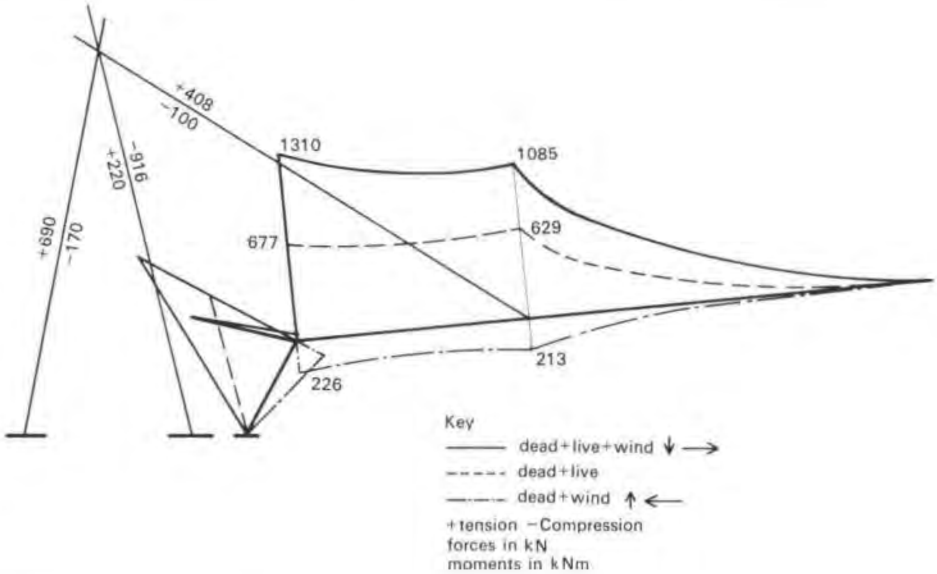


Fig. 13
Design forces and moments

Plate girders

Under the loading from own weight and imposed roof loads, the botttom flange of the plate girder is in compression. Lateral restraint is provided by 'T' shaped stiffeners designed to transmit $2\frac{1}{2}$% of the bottom flange force in bending to the purlin junction. The resulting moment is catered for in the girder/purlin friction grip connection. Considerable attention was paid to the detailing. The web plate was extended through a slot in the top flange at the connection with the front tie to obviate the possibility of lamellar tearing. Welds in the flanges of the girders are full strength butt welds, ground flush to avoid notch effects. These welds were deliberately positioned away from points of maximum bending moment. In no instances do web welds coincide with flange welds. As a general rule, welds across the main tension flanges of the girder were avoided. Hence at the haunches, the stiffeners were machined to fit exactly to the underside of the top flange.

Fig. 14
Support pyramid showing capping piece
(*Photo:* Harry Sowden)

The 'pyramidal' assemblies

The 'A' frames contain the only departure from shop welding. One box leg per frame was welded to the capping piece on site. This was however done at ground level to ease the task of the welders and also to facilitate inspection.

The detail at the base of the back ties required particular attention for both structural and visual reasons. Both considerations led to a double steel collar solution (Fig. 4). The bolt forces are transferred via the upper plate to the stiffeners. Two advantages seemed to be offered by this detail. It was possible to make the stiffener a suitable length in order to keep the stresses very low in the full strength butt weld of the stiffener to the tube. Secondly, the stiffener/upper plate joint is in compression and hence the possibility of lamellar tearing avoided.

At the joints at each end of the front ties and the top end of the back tie, HSFG bolts were considered advisable to eliminate racking, thus avoiding shear impact on ordinary bolts and the subsequent wearing of bolt holes under conditions of fluctuating stress. These joints (Fig. 6) consist of two tongue plates each butt-welded to the sides of a slot in the tube. The plates are joined at their innermost end by a circular rod to seal each tube. In the completed state, these plates pass either side of the capping unit wings or extended web plate of the girders. It was essential that care was taken at the fabrication stage to provide the stated minimum gap between the two tongue plates at the ends of the tubes. This was to ensure an effective gap closure between plies when the HSFG bolts were tightened.

As a precaution, two additional bolts were provided at each connection for the sole purpose of closing this gap, their contribution to the joint strength being disregarded in the group calculation.

The geometry of the capping pieces was dictated by the steepness of the back tie which caused only a small angle between it and the A-frame strut. The starting point for developing this detail was to make the centre lines of all the members coincident. The horizontal components of the forces are transferred through full strength butt welds connecting the wing plates and the central rod (Fig. 5). Because of the inherent rigidity provided by the perimeter walls of the box it was clear that the central rod would in fact take very little of the vertical force components of the system. Hence it was ensured that the butt welds connecting the wings to the side plates could safely transmit the vertical forces to the walls of the unit. Extensive testing was carried out on the rod, and rod to wing plate connections. The rod was subjected to Charpy impact tests in its prenormalized state, and a test piece consisting of 43 C plate, welded to the EN 14 A rod, was examined for hardness at the weld connection.

Each weld on every unit was ultrasonically tested stage by stage as the core and box was assembled.

Bolting

For all site HSFG bolting, general grade bolts were adequate, and these were specified to be used in conjunction with load indicating washers.

Tests carried out over recent years show that for general grade bolts, most metallic coatings are acceptable and no de-embrittlement treatment is required. Sheradizing was the finish actually adopted for the bolts, nuts and load indicating washers used on this project.

Testing and inspection

With the exception of the EN14 A rod, material testing was carried out at the instigation and under the direction of the fabricators. This was done ultrasonically, mainly at the works. In the terms of the specification, it was the contractor's responsibility to provide material free from harmful laminations. Any such laminations would have been detected after fabrication when NDT was carried out at the weld connections, and if found could have had serious repercussions on the programme. As most of the material specified was to grade 43 C (a grade which the British Steel Corporation will not guarantee to be free from laminations), the fabricators took the precaution of testing critical material at the prefabrication stage. This proved to be prudent as considerable defects were discovered in some of the plate material.

Testing arrangements were established at an early stage in conjunction with the fabricator in order that he could incorporate the requirements within his fabrication programme.

Extensive welder qualification and welding procedure tests were called for, and test pieces submitted to the appointed authority for examination.

Concrete superstructure

It was not considered feasible to place movement joints on the lines of the main frames since the reactions from the un-jointed canopy were transferred to the concrete superstructure on those lines. Two half-joints were placed at approximately quarter points of every fourth bay to form a suspended panel. The concrete structure was thus divided into roughly 30.5 m lengths. Low friction sliding bearings consisting of two thin pads of neoprene coated with polytetrafluoroethylene (PTFE) were used.

The terraced slab was first designed to consist of shaped precast elements spanning between the main frames. This was later rejected in favour of an in situ sloping slab with separate precast seating units above, because the latter did not impose as severe a limitation to the headroom available in areas below, and also provided a more satisfactory solution to the problem of waterproofing. (The areas beneath will include kitchens and restaurants.)

Because of the large forces in the back ties (687.5 kN) and also because of the 'tugging' effect due to stress fluctuations in the roof structure generally, it was decided at the start of the design to adopt details which would enable these forces to be transmitted from the tie down through the back concrete columns and into the pile caps in a continous fashion. A system of mechanical splices and couplers was introduced in the form of CCL *Alpha* tension splices.

The bar above the last splice (Fig. 4) was threaded to receive the steel base plate. Hence it was necessary to look for a bar with characteristics that would make it suitable for *Alpha* splicing, as well as enabling it to be threaded, and which would also meet the necessary design requirements for impact loading and stress fluctuation. Additionally, this bar would be expressed as part of the featured connection previously mentioned, and should not suffer any embrittlement effects from the protective treatments. The British Steel Corporation produce a threaded assembly which was considered to be suitable for this detail. It is a round ribbed high yield bar, with a precision cold rolled unified fine thread for maximum efficiency. Apart from the satisfactory mechanical properties of the bar, the chemical composition and known impact strength indicated sufficient notch ductility. To provide as much tolerance as possible for the contractor, the top two splices (Fig. 7), instead of being the single *Alpha* sleeve, were replaced by *Alpha* screwed couplers. These consist of two sleeves, each pressed on to reinforcing bars, but threaded at the remaining open end. The bars are then joined with an interconnecting stud, which enables the cranked bar detail which anchors the back tie to be rotated, and hence provides a fine adjustment to the inclination of the bolts until a late stage in the construction sequence. The couplers had a specified minimum strength of 471.5 N/mm², i.e. the characteristic strength of the reinforcement plus 15%. A limited series of fatigue tests had already been carried out by the manufacturers on the screwed coupler using a bar of diameter and type identical to that being incorporated in our detail. The mean stress and stress fluctuations set up for the tests were significantly more severe than those the canopy would experience, and a comparison of the results with the predicted fatigue criteria showed that the coupler would be satisfactory.

A specially fabricated temporary steel template, approximately 8 m × 6 m (Figs. 15a, b), was used to ensure the accurate positioning of each group of bolts, and of all the groups of bolts within each pyramidal assembly in relation to each other.

Fig. 15a
Steel template (*Photo: copyright* Greater London Council)

Fig. 15b
Corner of template showing backtie connection detail (*Photo: copyright* Greater London Council)

CONSTRUCTION

Foundations and reinforced concrete superstructure

The piling was installed as a separate contract by Lind Piling Ltd. The main contract commenced on site in February 1975 and was completed in October 1976.

One point of particular interest in the construction of the reinforced concrete superstructure was the casting of the sloping slab (25°). Concrete of normal workability (50 mm slump) was used. To control the downward flow of concrete a grid of timber battens was employed as a top frame. The dwarf wall shutters were fixed to this top frame, and the dwarf walls were cast with the slab.

The inclined holding down bolts were cast in with the aid of the large temporary steel template, which could be unbolted into two pieces for ease of transport and moving from one bay of the building to the next (Figs. 15a, b). The position of the reinforcement on plan at

Fig. 16
Internal view of structure at level 2 (*Photo:* Harry Sowden)

the top of the back column was critical, as it had to match the steel template. Alignment of the bars in this column was achieved by 20 mm plywood templates set by theodolite and this procedure was started at pile cap level (Fig. 7). The steel template was supported on concrete at level 3 and in order to do this, when the formwork and steel fixing for level 3 had been done, a strip over the columns was left unconcreted. The inclined plates to set the bolts were cantilevered over the uncast section. The bolts were placed, and concreted in by pouring of the infill strips. The plinths to the back tie and front strut were cast separately afterwards with the templates still present. This template was also used in the fabricator's yard for a trial assembly of the first back pyramid, to prove the geometry, and to ensure that the baseplate holes precisely matched the bolts cast-in on site.

The precast seat units were factory cast in individual purpose-made steel moulds. They rest on the dwarf walls, and are bedded on all four edges on mortar, gaps being left in the mortar on the long sides as weepholes for any water to flow down hill. A special sand was used for pointing to match the colour of the units.

A striated finish was specified by the architect for exposed in situ concrete to walls and columns. The appearance reveals very fine random ribs of 2 mm average width and average depth of 1 mm. It is an off-shutter finish, formed by glass fibre panels glued to the inside faces of ply shutters.

Erection of steelwork

Steel erection was started when the concrete was about 75% complete. The 22.1 m long plate girders were brought to site in one piece. The cranage used on site was a 60 tonne NCK Rapier Crawler, supplemented by a mobile for feeding the girders from the site storage area to the crawler. The only site welding required was to join the two legs of the A-frame, which could not be transported in one piece. This weld, a full strength butt, was made on the ground. The two legs of the A-frame were set up accurately on packings with their feet joined by a temporary cross piece. All four sides were welded without turning the assembly, thus an overhead butt was necessary. The weld was ground flush. Welder tests for vertical and overhead butts were carried out on site and a proportion of the A-frames were checked ultrasonically. No significant defects were noted.

The erection sequence of each element of the frame, consisting of two girders plus purlins, fascias spanning between them, one A-frame and two front and two back ties, was carried out as follows.

Temporary scaffold support towers were first erected, and one girder was lifted onto the holding down bolts at its base, and onto the jack at the top of the scaffold tower. The second girder was erected likewise, and the purlins and temporary bracing (if detailed for that bay) fixed.

The A-frame was lifted with its temporary crosspiece. Large holes had been provided in the baseplates in order for the complete A-frame to be lowered onto inclined holding down bolts. Special large washers cut from 25 mm plate were used. The A-frame was held temporarily from the capping piece by a *Tirfor* anchored to the wall at the bottom of the terrace. The back ties were then lifted singly and lowered over the threaded bars which formed the holding down bolts. The holes in both base plates were again oversize, 50 mm diameter for nominal 32 mm diameter threaded bars, and provided with special washers. Such oversize was enough for the top tongue plates of the tube to be inserted over the capping piece wing plates by rotating the tube about its base. The front ties were then fixed.

The frame was finally lined and levelled and the bases of the back tie, A-frame and plate girder leg were grouted. To transfer the large A-frame forces, until the grout had hardened, shims were placed under each of the eight stiffeners (away from the corners of the plinth).

After the grout had hardened, the jacks were released. The deflexions at each jack point were small and differences in deflexions at the tips of the individual girders proved to be negligible. It was important that this should be so in order to preserve the alignment of the front fascia beams.

INAUGURATION

The inauguration took place on Saturday 16 August 1977 in the presence of Her Royal Highness the Duchess of Gloucester.

Credits

Architects:
GLC Department of Architecture and Civic Design. Special Works Division
Main contractor:
F. G. Minter Ltd.
Steelwork fabricators:
A. E. Watson (Exeter) Ltd.
Precast concrete units:
Francis Concrete Ltd.
Roof sheeting:
Carter Horseley Ltd.
Piling contractor:
Lind Piling Ltd.

References

(1) BRYAN, E. R. The stressed skin design of steel buildings. Crosby Lockwood Staples, 1973.

Fig. 17 (above)
Rear view of stand (*Photo:* Harry Sowden)

Fig. 18 (below)
Front view of stand (*Photo:* Harry Sowden)

The GLADYS computer system

David Croft

INTRODUCTION

GLADYS is a computer system for the production of calculations for structural design in reinforced concrete and was developed in the London office of Ove Arup and Partners.

When, in 1974, the decision was taken to replace the firm's existing IBM 1130 computer with a new DEC System 10, the question arose as to what new software should be developed to take advantage of the time-sharing facilities of the new machine. Over the previous few years, in parallel with the batch programs that were run on the IBM 1130, the use of the Hewlett-Packard programmable desk top calculators had been steadily increasing. With the London office spread between a number of different buildings, co-ordination of program development was inevitably difficult and as a result there was considerable variation in standards and duplication of work. As there were roughly 250 engineers within the structural and building engineering divisions there was a sufficient internal market to justify the development from scratch of a completely new system which would combine the expertise gained from the writing and use of interactive Hewlett-Packard programs with the benefit of a centralized co-ordinated system.

Phase 1 of the development was started in 1975 and completed in February 1976. Since that time the use of the system has been closely monitored and as a result a decision has been made to proceed with a second phase of the development.

Fig. 1 shows an example of running the system.

DESCRIPTION OF THE SYSTEM

Code of practice and units

Where applicable, calculations are carried out in accordance with *BS CP110* and SI units are used throughout.

Hardware configuration

The computer used by Ove Arup and Partners is a DEC System 10, manufactured by Digital Equipment Corporation, which is designed for batch and time-sharing use. Access to the machine is primarily via terminals but input from punched cards, paper tape or magnetic tape is also available.

The standard terminal used by the firm is the Cableprint HCT 302 which is based on the Diablo printing mechanism. This is an impact typewriter of average speed (30 characters per second) and good printing quality. Connection to the computer is by either hard-wired or dial-up lines. Visual display units are also used when hardcopy output is not required. In addition there are two Tektronix storage tubes for graphic work.

With the GLADYS system, full use is made of the Diablo plotting facility. The typehead can be programmed to move across the paper in increments of 1/60 in. horizontally and 1/48 in. vertically upwards and backwards as well as to the right and down the page, which is adequate resolution for simple diagrams.

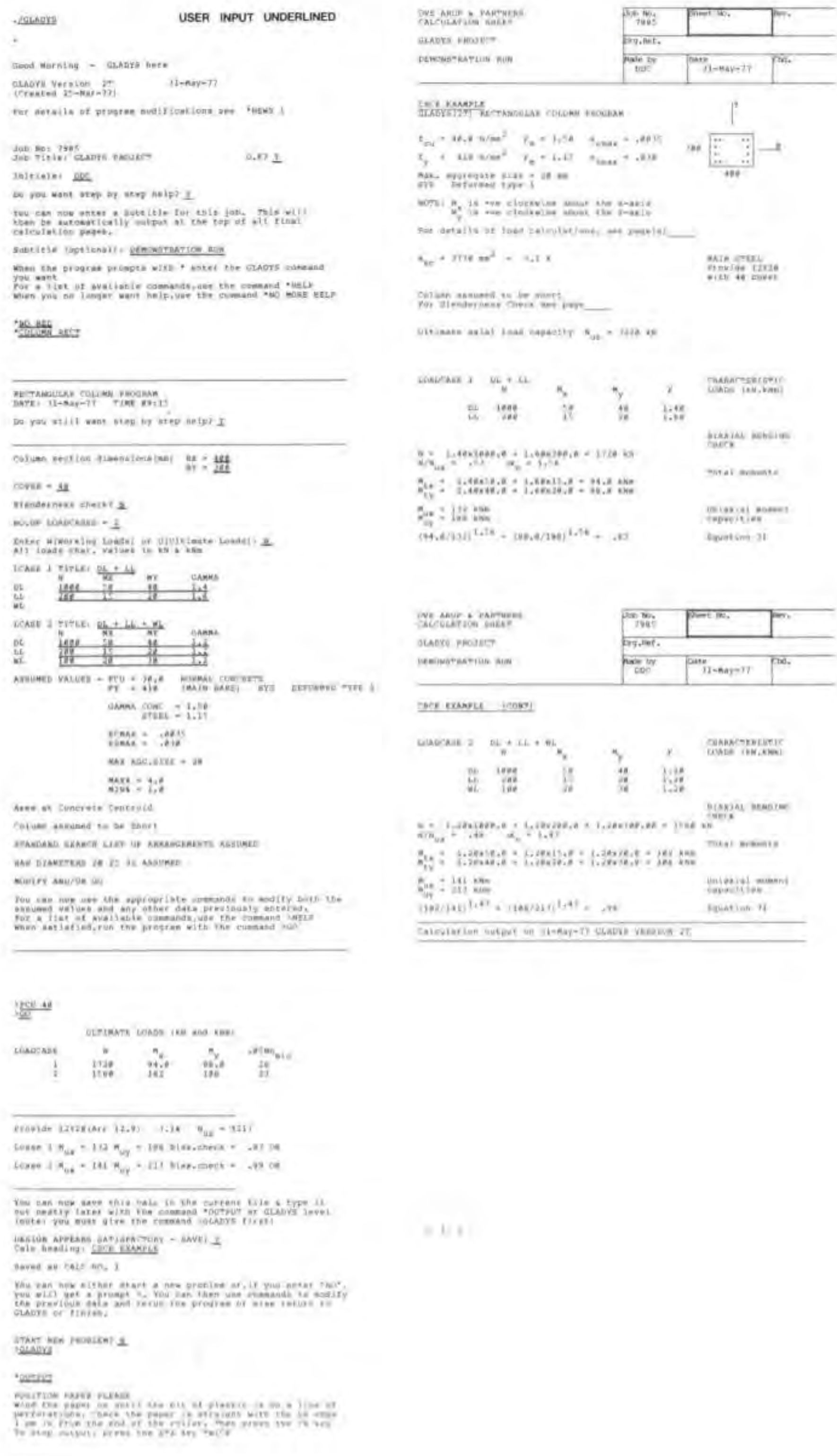


Fig. 1
Example of running the GLADYS computer system

Programs available

With completion of Phase 1 in February 1976 the following programs were available:

Beams: This program is in two parts which can be run either separately or together:

(i) Beam slab analysis
This program analyzes single and multi-span beams subjected to specified distributed and point loads. Column stiffness can be included. Support moments can be redistributed. Output options include maximum elastic and redistributed moments at supports and in the spans and bending moment and shear envelopes (either plotted or numeric values).

(ii) Beam section design
This program calculates steel areas required in beams for bending and shear. Bar arrangements can be either automatically selected or input by the user and checked.

Rectangular and circular columns: These programs analyze columns at ultimate load in accordance with *CP110* and may be used either to check a column with specified dimensions and reinforcement pattern or to select a suitable reinforcement arrangement for a column of specified dimensions. If requested, a check on the slenderness ratio is made and, if slender, the additional moments calculated.

Irregular columns: This program analyzes sections of irregular shape subjected to axial forces and biaxial bending at working or ultimate load.

Pad footings: This program calculates the reinforcement required in pad footings of specified dimensions in accordance with *CP110*. Checks are made against allowable bearing pressures. Bond and shear stresses are also checked. The footing may be square or rectangular, and support a single column of rectangular or circular section, which may be positioned eccentrically.

Pilecaps: This program can be used to design pilecaps for groups of 2–9 piles supporting a single column at the pile centroid.

Irregular section properties: This program calculates the elastic section properties of irregular sections.

User language

General: During the development of the system considerable thought was given to formulating the method of entering data and controlling the progress of the calculations. The following requirements were thought to be particularly important:

(i) The programs had to be simple to use and easy for new users to learn how to operate them.

(ii) At the same time, the programs should not be cumbersome to use once the user has become familar with them.

(iii) It should be possible to modify separate items of data individually without having to re-enter all the data for the problem. In this way alternative solutions can be produced and compared and the design optimized. In addition, where appropriate, parametric studies can be carried out to quantify the sensitivity of the answers to changes in the input.

(iv) Engineers, like all humans, make errors and, if anything, due to their method of working, are particularly error prone. Programs therefore should be written in such a way that not only is the probability of error minimized, but if an error is discovered it should be possible to rectify the situation as simply as possible.

(v) Frequently several different problems have large amounts of data in common, for example a set of columns of the same dimensions but with different loading conditions. It should be possible to enter one problem and solve it and then progressively modify the data and solve for each of the others in turn.

GLADYS and program levels

The GLADYS system was designed to meet these requirements and is a two-level system: GLADYS level and program level. The user calls up the system and enters at GLADYS level and subsequently calls up the program required; the user is then at program level. To run another program the user must first return to GLADYS level.

All information is input in one of three ways:

(i) Prompts
(ii) Questions
(iii) Commands.

Prompt input: A 'prompt' is a string of text output by the program asking for a specific piece of information and the user enters the data as specified. Where appropriate, the units in which the data should be input are defined in the prompt.

Questions: These are similar to prompts but require the answer 'YES' or 'NO' ('Y' or 'N' are also accepted). The form of the question and the fact that they end with a query (?) differentiates them from other prompts.

Commands: A 'command' is a string of characters entered by the user in order to specify a certain action to be taken by the program. As already described, the GLADYS command system is on two levels, i.e.:

(i) GLADYS level commands. When at GLADYS level and expecting a command an asterisk prompt (*) is output and the user must enter the required command.

(ii) Program level commands. When at program level and expecting a command the program prints a prompt (>).

In many cases a command is followed by one or more items of data. In such cases as with prompts, different items of data are separated by blanks.

Commands can be used to modify either data previously entered or any assumed values (see below). When the user is satisfied that the data is correct the solution is initiated with the command >GO.

The command HELP, given at either GLADYS or program level, causes a full list to be output of all the commands that are available at that time.

Assumed values: Data regarding material properties and safety factors is required for most of the programs. As these tend to be the same for different calculations, a set of default values is stored within the program. Any assumed values are output immediately after the prompt input and can be modified by giving the appropriate command.

Step-by-step help: All new or infrequent users of the system are recommended to opt for step-by-step help. This causes paragraphs of helpful information to be output which clarify the options available at each stage.

Standardization: Wherever possible, and appropriate, all prompts, commands, units and sign conventions are the same for all programs. In this way, once a user has become familiar with one program, he will find it relatively easy to use the others.

Report and fast modes: The normal level of output is generally produced in response to the >GO command. In some cases, the user might want additional information and the >REPORT command can be given instead. Alternatively the user may wish to suppress some of the output and the command >GO FAST can then be used. The latter is particularly useful when using the program to optimize a design by trying a number of alternatives.

Data checking

The main reasons for checking data are as follows:

(i) To check that the data entered by the user is sensible and consistent with itself

(ii) To ensure that the data does not extend the program beyond the limits of its validity, i.e. contrary to the assumptions made within the program

(iii) To warn the user when a value entered is unusually large or small and may be in error.

In the GLADYS system, data checking is carried out in the following ways:

Idiot value check: Generally each item of data input is checked against absolute minimum and maximum values. If it is outside this range, the value is rejected. In addition the value is checked against upper and lower query limits. If outside these, it is considered probable that an error has been made and the value is queried: 'Funny value – are you sure?' If the user replies 'YES' the value is accepted.

At data input time, no attempt is made to check data compatibility. The user is given the benefit of the doubt, in that he may have spotted an error himself and intends correcting it before giving the command >GO. The check is solely on the numerical value and the term 'idiot value' is used to indicate the level of the check rather than as a comment on the user. Typical errors that are trapped by this check are dimensions entered in metres instead of millimetres and vice versa.

Data compatibility checks: After the command >GO, the data is checked to ensure that it is compatible with itself. Absolute limit checks are also made where appropriate. If any of these checks fail, execution is halted, a message output and a prompt (>) is printed. The user must then modify the data and give the command > GO again.

Solution checks: Checks are made, as appropriate, as the solution proceeds. These checks depend on the nature of the program and cover such aspects as reinforcement detailing, strength, etc.

Debug systems: One of the keys to economic program development is the provision of efficient debug systems. In the GLADYS system these took the form of purpose-written sub routines which output the value of selected variables. These routines are now permanent parts of the system and can be activated at any time by means of the appropriate commands.

This facility is particularly important as it was always intended that the system should from time to time be modified as a result of demand from users, changes in the code, etc. It has to be expected that such changes will introduce new errors or activate existing ones. The provision of permanent debug routines greatly assists in tracing these errors.

An additional precaution is the provision of 'sleeping' statements which, provided the program is functioning correctly, are not normally executed. These and other solution time checks, such as limiting the number of iterations in iterative solutions, activate an error trap and the error is entered into the GLADYS log file. The user is instructed to report the error so that a post-mortem can be carried out but, should he omit to do so, the incident is diagnosed when the log file is next processed.

Development history

The Brief: In 1974 it was decided to order the DEC 10. The delivery date was scheduled for August 1975 and plans were made for an initial range of software to be available when the machine was commissioned.

As part of this preparation the Associate Analysis of each of the then four London Structural Divisions and the Building Engineering Division combined to prepare a brief for a set of interactive production programs. Thereafter they continued to fulfil the role of client and were subsequently responsible for the successful implementation of the system within their respective parts of the firm.

The brief specified a suite of programs that could be run from the terminal by engineers and would produce pages of calculations justifying the final design of typical structural elements such as beams, columns, foundations, etc.

Feasibility study: During the period from September to December 1974 the configuration of the proposed hardware was examined in relation to the brief and a feasibility report was produced containing:

(i) Estimates of the manpower and computer time required
(ii) Estimated costs
(iii) A proposed organizational structure
(iv) A programme for the development
(v) Draft program manuals for a number of the programs
(vi) Specifications for the FRUD and RECON libraries. These are libraries of standard subroutines that are used throughout the GLADYS system for input/output operations and for standard calculations in accordance with *CP110*.

Briefly, the proposals were for a team to be set up consisting of five engineers, one from each division, led by a project engineer for a total of 36 man months. When the system was complete, they would then return to their divisions and provide local centres of expertise regarding both the use of the machine in general and GLADYS in particular.

Arrangements were to be made for the development work to be carried out on another DEC System 10 so that they could be ready by the time the machine was installed.

Scheme design: With the acceptance of the Feasibility Report, work was carried out from January to March 1975 on the preliminary development of the system. This included the master GLADYS program and the partial completion of the subroutine libraries.

Program development: In April the project team was assembled and work began on the development of the individual programs. The new machine finally became operational in September and the system was transferred in-house. All the programs were substantially complete by the end of October and by then all the members of the project team had returned to their respective divisions.

Implementation: This was done between October and December 1975 and, in view of the wide use that was expected to be made of the system it was considered essential to set up a rigorous verification programme. At the same time an educational programme was required so that engineers throughout the firm could learn to use the machine efficiently. These two requirements were satisfied by dividing up the verification tests between all the divisions and during this period nearly all the structural engineers in the firm used the terminals. A range of problems for each program was selected that would test the different parts of the program. The results from GLADYS were checked by hand and other methods and the whole operation was co-ordinated by the Associates Analysis acting as the client.

During this period minor errors were reported and programs revised accordingly. Similarly minor improvements in the operating methods were incorporated.

Program modifications: Comments from users, both on the programs and the manuals, were collected and a list of the more major modifications required was agreed. As a result, the project team was partially re-assembled during January 1976 to carry out these agreed modifications.

Acceptance: When these modifications had been carried out and subsequently tested, the system was formally accepted in February 1976 by the Associates Analysis on behalf of the firm. At about the same time the system was approved by the London District Surveyors Association. They had been kept informed during the development of the system and had themselves contributed to certain aspects regarding the final calculation pages. Similar approval was received shortly afterwards from the Greater London Building Surveyors Association.

Comparisons with the Feasibility Report: The predicted and actual figures for the quantity of work involved and the manpower required are as follows:

	Predicted	*Actual*
Number of Fortran statements	15,000	38,000
Number of programs	10	8
Average statements/ program	1,500	4,800
Man-weeks for development	150	206
Statements /man-week	100	190

In addition approximately 30 man-weeks were used in the user trials and verification tests, roughly one day for each engineer in the firm.

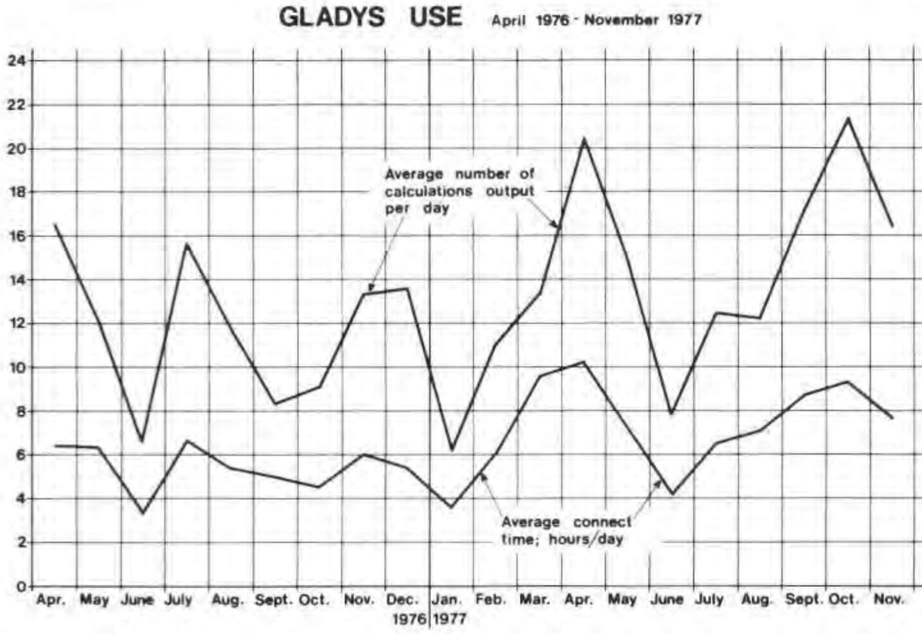


Fig. 2
Analysis of log file

ANALYSIS OF USE

GLADYS Log File

Details of all GLADYS runs are entered automatically into a central log file. This file is processed at monthly intervals and provides factual information on the use of the system and the individual programs.

The log file for the period April 1976 – November 1977 has been analyzed and Fig. 2 shows the average number of hours/day connect time plotted for each month. Also shown are the average number of final calculations output per day. The variation of use is seen to be somewhat erratic and this reflects to some extent the fluctuating workload of the firm during this period.

The averages for the 20 month period were:

Average connect time/day = 6.4 hours
Average number of calculations output/day = 12.9.

The relative use of the different programs based on the number of calculations output is as follows:

	%
Beam/slab analysis	50
Beam section design	6
Rectangular columns	12
Circular columns	0.5
Irregular columns	25
Pad footings	2.5
Pilecaps	1
Irregular section properties	3
	100

Discussion

Relative use: The order of relative popularity of the different programs is generally as had been expected. Predictably, the beam/slab analysis is by far the most popular. However, only a minority continue on to reinforce sections using the machine and this is not completely explained by the fact that the slab and flat slab section design programs are not yet available. It is apparent that the beam section design part of the program is not quick and flexible enough to attract engineers to use it. Surprisingly the logic of this particular program turned out to be the most complex of all the programs.

The irregular column program is frequently used because there is no alternative method that is practicable without making significantly conservative approximations. The rectangular column program is also relatively popular because hand methods, even with the use of design charts, are tedious and frequently unduly conservative. The other programs must in a sense be considered as loss-leaders as they cannot be justified except as part of a complete suite of programs.

Overall use: The justification of computer systems in the structural field on cost/benefit grounds is notoriously difficult. One of the problems is that the existence of a program tends to create a need that would otherwise not exist.

For example, it is now possible to analyze several different beams in less time than it takes by hand to do one. Previously, provided they were not too dissimilar, the worst case would be analyzed and all the beams reinforced the same. Although the quantity of reinforcement was greater, this would be more or less offset by the repetition on site. It is, of course, the engineer's job to optimize between these two approaches and at least it can be said that with the assistance of computer aids he now has a choice between two practicable alternatives.

On the other hand, there are many benefits which cannot rationally be quantified but are of significant value such as:

(i) Legible printing
(ii) Full statement of data and assumptions
(iii) Standardized solution methods
(iv) Standardized presentation of results.

CONCLUSIONS

The GLADYS system and the history of the development have been described. The use of the system has been closely monitored over a period of 18 months and, as a result, a decision has been made to proceed with a second phase of the development.

Kensington & Chelsea Town Hall

Tony Langford

Introduction

The new Town Hall for the Royal Borough of Kensington & Chelsea was officially opened on 31 May 1977. It has been described as the last work of Sir Basil Spence, but sketches for the proposed building were prepared by Sir Basil as long ago as 1965 and the design was carried out by the John S. Bonnington Partnership (formerly Sir Basil Spence, Bonnington & Collins).

Because of financial restrictions, a public enquiry, design modifications necessary to accommodate the Council's change in staffing needs and other delays, a start on site was not made until January 1972.

The site

The excellent 1.4 ha site is situated next to the Central Library adjacent to Kensington High Street. Other surrounding buildings are principally terraced domestic housing of brick construction. Many of the fine trees on the site have been preserved and among those in the central courtyard of the new building is a Giant Redwood, planted in 1967 in memory of Sir Winston Churchill. Although it is now only about 4 m high this will, in time, become a dominant feature.

Accommodation

The building is fully air-conditioned. The office accommodation provides for staff expansion to 1250 persons. A large assembly hall seats 750 and a secondary hall has a capacity of 200. A separate Council chamber provides seating for 70 members with space for guests and for 60 members of the public in a separate gallery. Additional accommodation is provided for the Mayor and Mayoress and there are seven committee rooms. A staff canteen and kitchen provide facilities for meals. In the basement there are spaces for about 450 cars.

The buildings

The main building is approximately 90 m × 70 m in area and covers about two-thirds of the site. Planned as a low, four-storey, horizontal structure with a roof plantroom, it surrounds an open courtyard and is raised on exposed columns to preserve a spatial aspect. The office façades are brick-clad, separated at each level by continuous horizontal bands of glazing. There is no vertical structure on the façades and the floors cantilever independently from the interior columns. A strong feature is the staggered profile produced by the overhang or set-back of each floor from the floor below. The Mayoral Suite and committee rooms are identified by the high, heavily moulded treatment of the window mullions. Situated in the south wing, these special areas protrude beyond the ends of their cantilever supports and overlook the Ceremonial Court. A three-storey office annexe, of similar construction to the main offices, is situated in the north-west corner of the site. This is used as separate lettable office accommodation, with separate entrances. Flanking the Ceremonial Court to the south, the Council Chamber and the Great Hall project from the main building. At the south-east, the strongly modelled Council Chamber stands on four cruciform columns over a reflecting pool of water. The Great Hall, at the south-west, rises from the brick paved courtyard, its plain elevations contrasting with the complex façades of the other buildings.

The architect has taken great care that all the materials used in the construction shall be in harmony with the buildings in the neighbourhood. The colour of the new brickwork and the white textured concrete of the Council Chamber match the soft red bricks and Portland stone of the existing Library. The sloping roof of the plantrooms over the Main Office and of the two halls are clad with zinc in the same manner as the Library roof. The moulded surfaces of the exposed office columns are in grit blasted concrete revealing gravel aggregates, their grey appearance providing a pleasing contrast to the brickwork of the new building.

The basement is at three levels and is approximately 140 m × 68 m on plan. With the exception of the unexcavated central courtyard, it covers the entire site. The depth varies from 7.5 m at the south, where it abuts the Library basement, to 12.5 m at the north-west corner.

Plantrooms and further offices, including a computer suite, are housed within, and there are public car parks on the two lower floors, with access from the two existing ramps situated on each side of the Library.

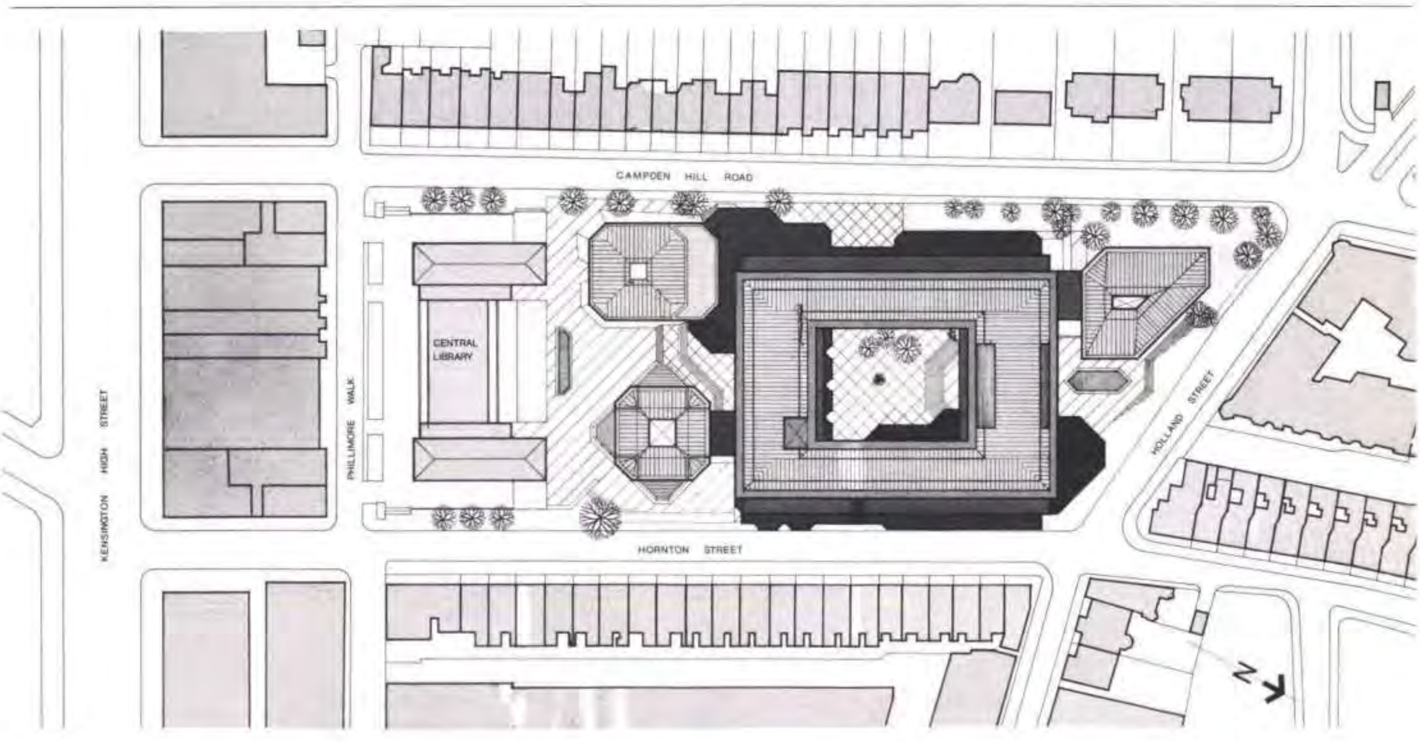


Fig. 1
Site plan

Fig. 2
Architects' east elevation

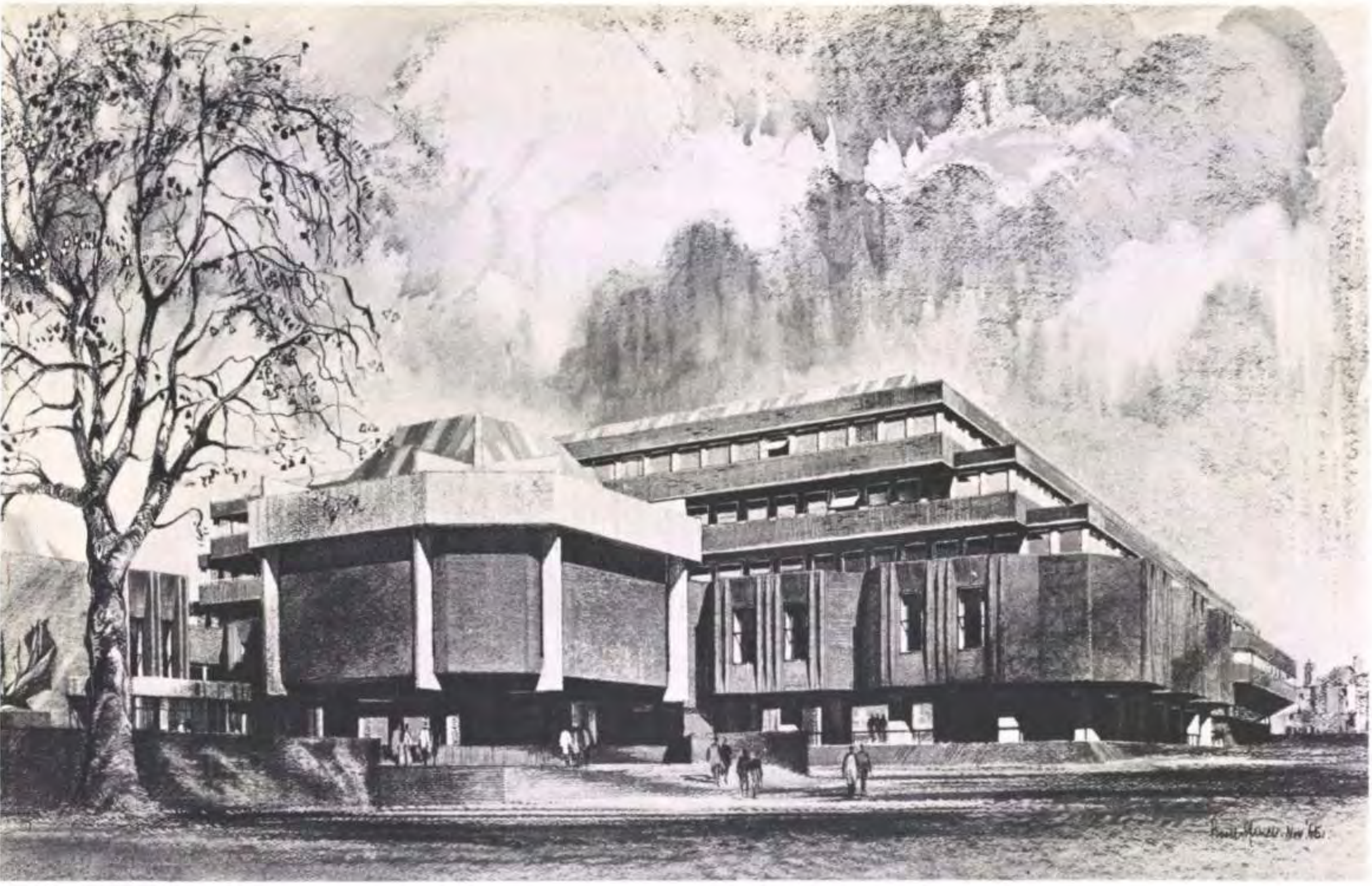


Fig. 3
Drawing by Sir Basil Spence
(*Photo:* Henk Snoek)

Fig. 4
Architects' model looking north
(*Photo:* Henk Snoek)

Fig. 5
Elevation from north side
(*Photo:* Henk Snoek)

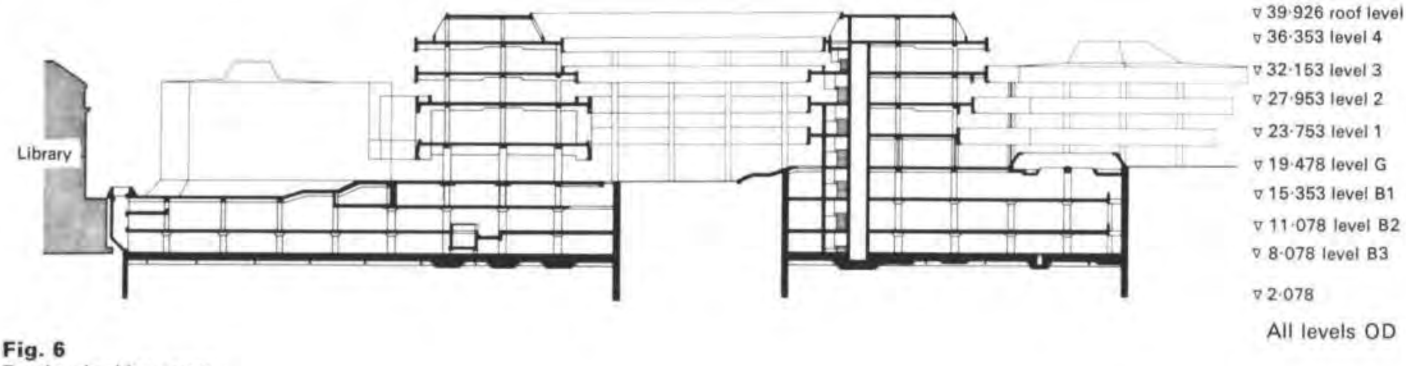


Fig. 6
Section looking west

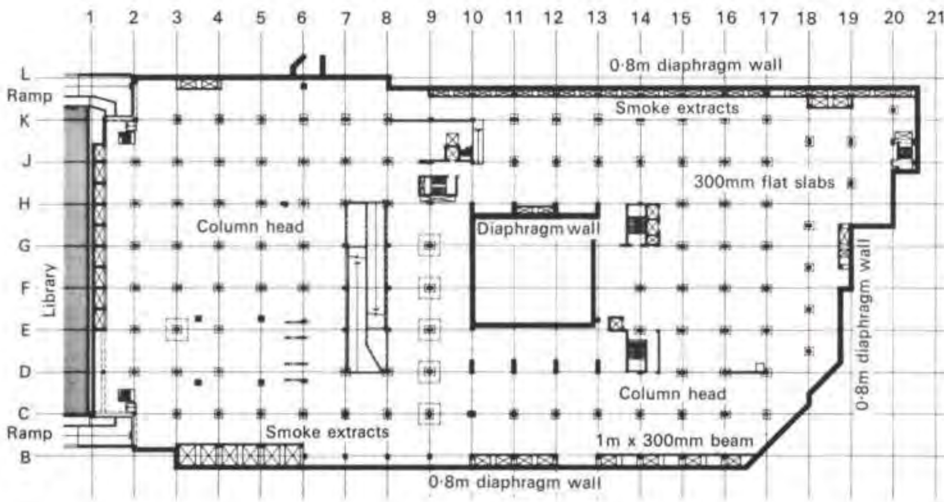


Fig. 7
Plan: basement level B2, car park

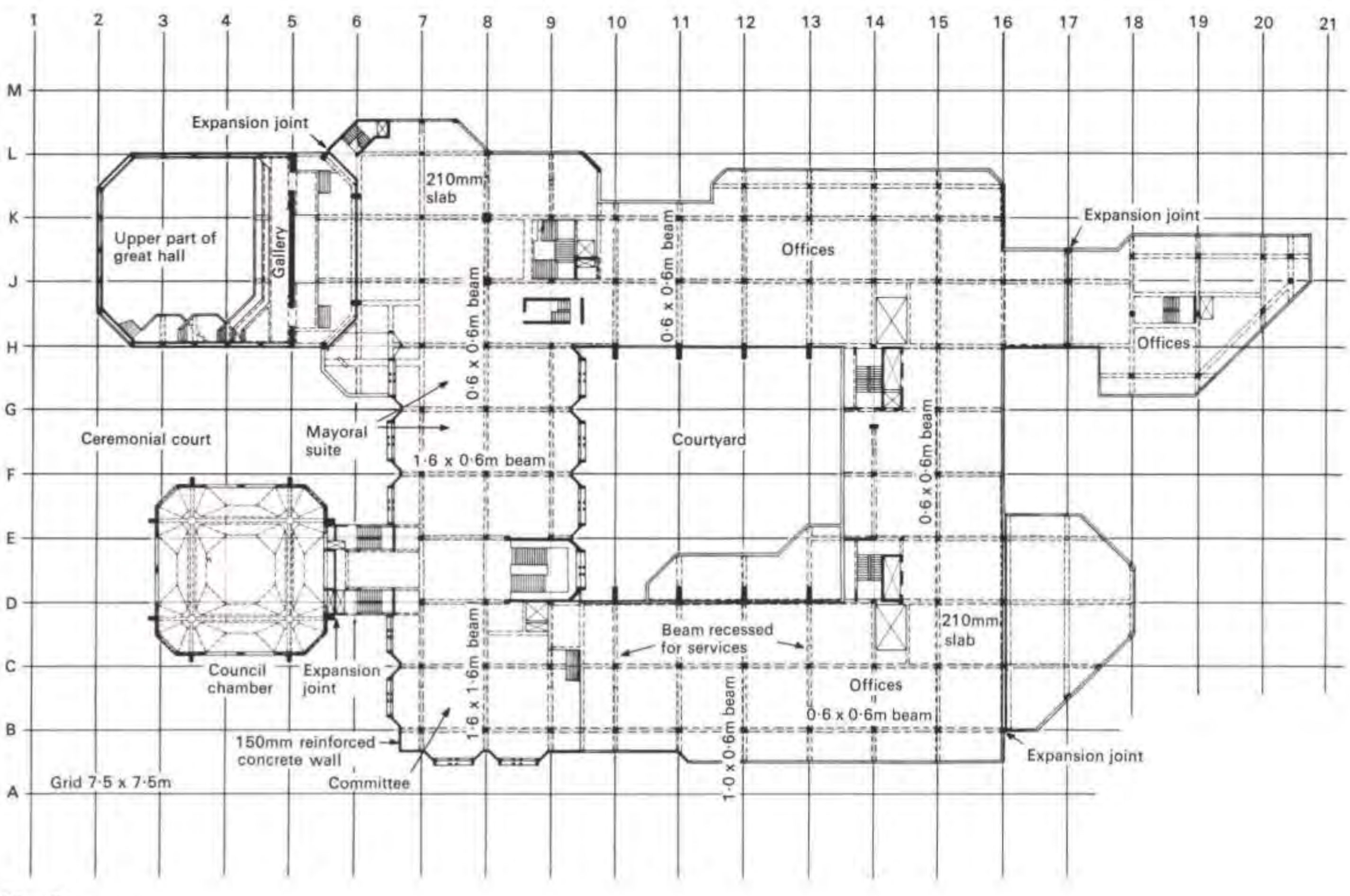


Fig. 8
Plan: level 7

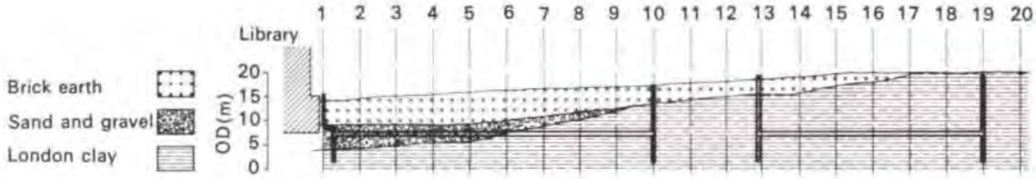


Fig. 8
North/south geological section

Site investigation

Nine boreholes were put down to a maximum depth of 45 m. A standpipe and three piezometers were installed and ground-water behaviour monitored. The investigation revealed that the site was underlain by London Clay, the surface of which had been eroded over three-quarters of the site area and replaced at the southern half by a dense gravel terrace.

Overlying the gravel and, in part, the London Clay, was a deposit of sandy clay (brick-earth). A geological section through the site shows a simple succession; however the deposits at the mid-section of the site were very irregular. Following days of heavy rainfall, large fluctuations in the ground water head were registered in the gravel. The piezometers installed in the London Clay showed that the hydrostatic profile was little affected by the under-drainage brought about by extraction of water from beneath the London Basin. A full hydrostatic head was therefore assumed in the calculations for the retaining walls. Triaxial tests on the London Clay indicated a bearing pressure of 350 kN/m² and, as the weight of excavated material was approximately equal to the average weight of the building, a spread foundation solution was adopted. In this case the most economical form was a raft.

Basement

An investigation was made into the cost and suitability of various methods of forming the basement retaining walls. Noise was a problem and, with surrounding buildings in close proximity, normal driven sheet piling was excluded. There were doubts concerning the ability of silent driven piles to penetrate the deep clay bed at the northern part of the site and the system was known to be unsuitable in gravels. A scheme for a concrete wall within a line of retaining bored piles proved to be expensive and space consuming. Substantial column loads were required to be carried around the periphery of the basement and a diaphragm wall, extended beneath the raft to act as a pile, offered the best solution and was therefore adopted. Advantage was taken of this extension by ensuring that the wall penetrated at least 2 m into the London Clay forming a cut-off to prevent water entering the gravel below the raft. There was an added advantage of extra stability during the temporary works. The construction technique satisfied the requirements of minimum noise and vibration during building operations.

Although the wall was to be propped during the excavation, movement was anticipated. On excavation, negative pore water pressures would be set up within the clay and, with time, these would dissipate as water was sucked through the fissures. Swelling of the clay and softening would follow, accompanied by movement of the wall. On the advice of Arups' Geotechnics staff the design of the wall was carried out on the basis that the shear strength $C' = 0$ and $\varnothing' = 25°$, from which $Ka' = 0.4$. Temporary propping of the wall would progress from the top downwards and free movement of the wall would be restricted, so there was a problem in judging if sufficient movement would occur to allow the pressures to drop to active values. In the final condition the walls were to be propped by reinforced concrete struts at the basement levels. Therefore, in order that the forces in the upper basement were not under-evaluated, the design was checked as a braced excavation to a rectangular pressure profile.

An initial design for a 600 mm wall proved too flexible and an 800 mm thickness was chosen. Maximum horizontal displacements in the order of 20 mm to 25 mm were estimated and the associated soil movements were expected to be such that no damage would occur to the buildings around the site.

The maximum vertical forces on the wall occurred on the western edge of the unexcavated central courtyard, where four massive columns each imposed a load of 800

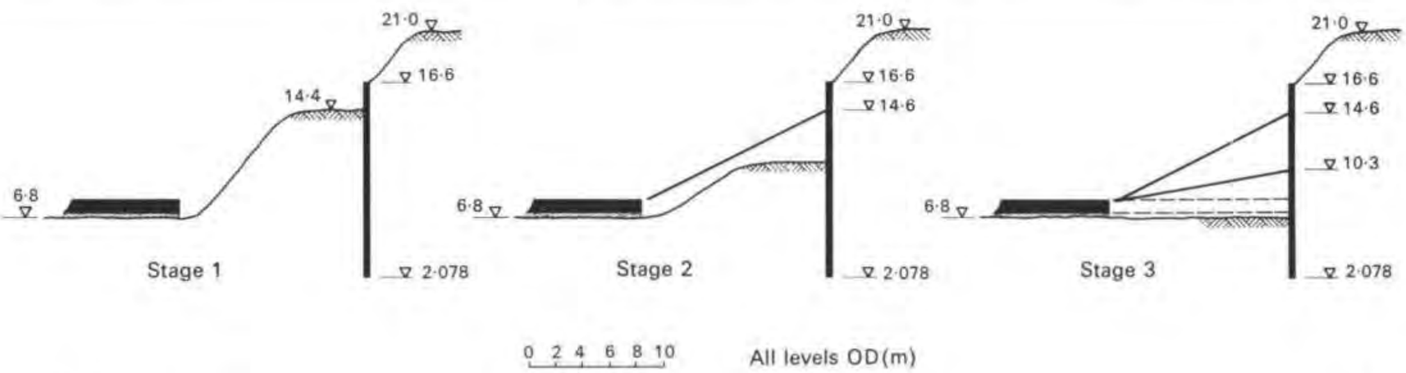


Fig. 10a
Excavation sequence for north wall

Fig. 10b
Diaphragm in wall showing *Rendhex* struts
(*Photo:* Tony Langford)

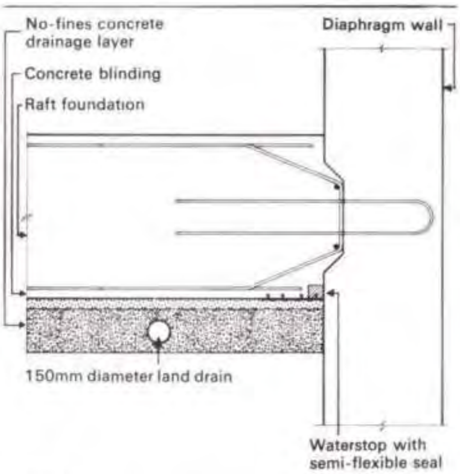


Fig. 11
Raft to diaphragm wall

Fig. 12 (right)
Elevation of Main office (Committee Rooms)
(*Photo:* Henk Snoek)

tonnes. The wall was formed as a series of 'T' sections in these positions but other high column loads occurred along the wall or immediately adjacent to it. The design of the wall under vertical load was based on normal considerations of side friction and end bearing, but the method of construction, under bentonite slurry, raised doubts about the bearing contact at the base of each panel. There was also an opinion that bentonite could reduce the shaft friction. Tests by Burland [2] and Farmer *et al*[1] found otherwise, but data was limited and after discussion with the District Surveyor it was decided to carry out a loading test on a diaphragm wall panel. The results of this test have been published elsewhere[3]; suffice to say that the ultimate load carried by the panel was 67% greater than that calculated and that the use of bentonite appeared to increase rather than decrease the shaft friction.

It was a condition of the diaphragm wall subcontract that the walls should not leak, reliance being placed on dense concrete and good workmanship at the joints. Along the top of the wall a deep in situ concrete capping beam tied the panels together and spread the loads from the peripheral columns. A land drain laid behind this capping beam surrounded the whole basement.

The columns within the basement were set out on a regular 7.5 m grid, whilst above ground level the configuration of the buildings was irregular, setting up a complex loading pattern on the raft foundation. A partial fixity solution at the junction of the raft and diaphragm wall was adopted, as the combination of continuity and the high lateral stiffness of the wall would have introduced forces into the upper floors which the structure could not take. After much discussion the analysis of the raft proceeded as follows:

(1) A column loading plan was drawn and from this the raft divided into areas of equal UD load.

(2) The immediate elastic settlements (dishing) within these areas were calculated by Steinbrenner's method assuming a flexible plate, and a contour drawing of soil settlement prepared for the whole site.

(3) The raft was then considered as a grid frame with nodes at the column positions, supported by springs, the stiffness values being calculated from an assessment of the coefficient of subgrade reactions derived from (2) above. The change in inertia due to the presence of the diaphragm wall at the periphery and the various spring stiffnesses, brought about by its possible different modes of behaviour, were included in the calculations.

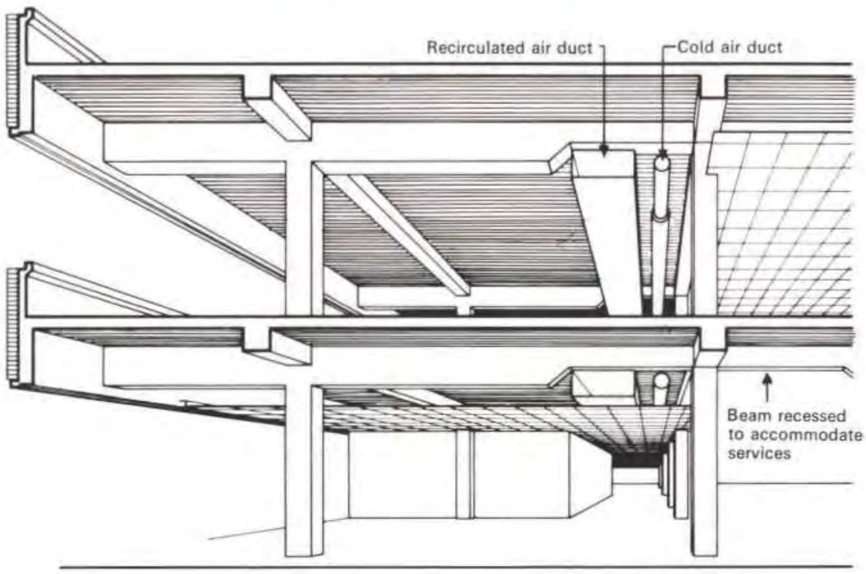


Fig. 13
Main office

(4) Bending moments and torsions at node positions were calculated on the computer, contact pressures being derived from the final reactions at the springs.

(5) Hand calculations were prepared to assess the effects of local bending moments and shears in the spans and the computer results adjusted or increased by these values.

(6) An inspection and a further series of hand calculations checked for anomalies in the results and ensured the equilibrium of each bay. There was also a check made to ensure that errors in the assumptions did not lead to insufficient consideration of local bending beneath a column.

(7) Final calculated settlements were compared with original assessments to ensure that the results made good sense.

The raft thickness was 0.9 m to 1.2 m, thickened locally to resist punching shear under the higher loaded columns. Additional thickening was provided at positions of high shear along the raft to diaphragm wall interface. The maximum elastic settlement plus long-term consolidation settlement was calculated at 80 mm, occurring just south of the central area. A 40 mm heave was expected at the north of the site where the excavation was deep and the loads light. Reductions in stiffness by creep were allowed in the performance of the raft and superstructure under this long-term settlement. The presence of the relatively thin gravel bed at the southern half of the site had little effect on differential settlement.

Fig. 14
Mayoral suite
(*Photo:* Harry Sowden)

Fig. 15
Internal courtyard
(*Photo:* Henk Snoek)

PVC water bars were used at every construction joint in the raft and a special detail was developed for the joint with the diaphragm wall. Within a layer of no-fines concrete, a system of land drains under the raft limited the water pressure that would otherwise develop, adding to the water resistance of the whole.

The two intermediate basement floors were designed as 300 mm flat slabs with column heads, drops being used only in those areas where loading was at its heaviest. The column heads, cast integrally with the columns, eased the fixing of beam-strip reinforcement around the heavy column steel. The forces from the basement walls were carried to the floors by struts spaced between the smoke extracts around the periphery of the basement.

The ground floor followed the slope of the site in a series of steps and irregular inclines and the presence of many openings into the basement added to the complex geometry. For these reasons a beam and slab solution was used at this level.

It was found that the existing Library ramps were not steep enough to connect to the new entry and exit points for the underground car parks. The lowering of these ramps would also expose the foundations of the existing ramp retaining walls. Therefore the intruding foundations were cut back and underpinned. The walls were then strengthened by inserting McAlloy bars and post-tensioning down onto the new enlarged foundation below.

Basement construction

The demolition of the few existing buildings on the site started in January 1972, followed by excavations to form working platforms for the construction of the diaphragm wall. A 1 m deep guide trench with concrete walls was constructed and this was temporarily strutted or backfilled. Excavations for the diaphragm wall followed, using the trench as a guide. Two tracked excavators were used; one with a grab mounted on a Kelly bar and the second, a rig with cable operated grab, for use under trees where headroom was limited. The excavations for each diaphragm wall panel, 7.5 m in length, were stabilized by the use of bentonite mud. Steel cages were lowered through the mud and suspended from the guide walls. Concrete was supplied by ready mix truck and placed by tremie pipe, the displaced bentonite being pumped away for reconditioning or disposal. Where the diaphragm wall crossed old basements, these were dug out and backfilled with a cement-stabilized granular backfill. Drain pipes crossing the route of the wall were sealed at the connections with the main sewer, a few of which were overlooked, the result being some sud-

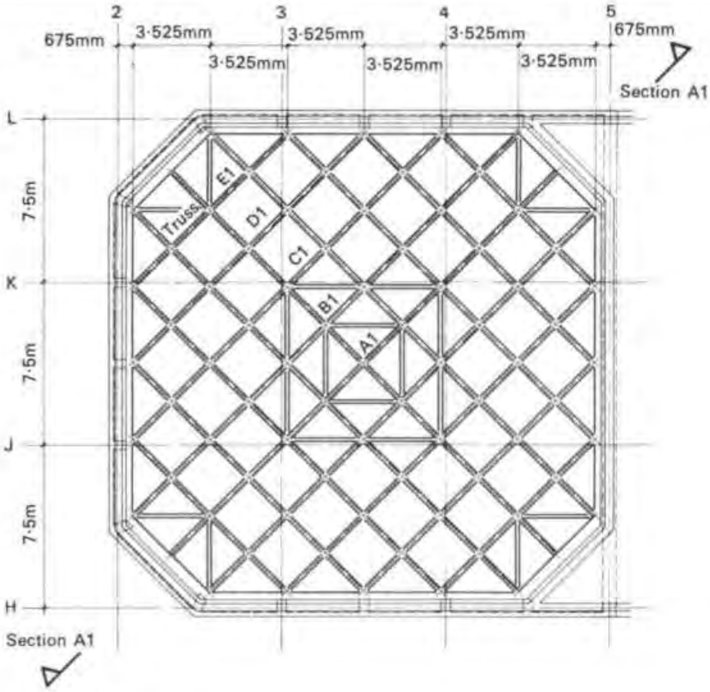


Fig. 16
Assembly Hall: steel roof layout

den losses of bentonite mud into the London sewerage system.

During excavations the diaphragm wall was supported by a strutting system. Consideration had been given to the use of ground anchors, but it was felt that the deformations associated with their use in London Clay would be excessive. There were additional problems in obtaining wayleaves. As the excavations would be open for a comparatively long period, a strutting system was preferred.

The main excavations started at both the north and south ends of the site to allow parts of the raft to be constructed. Berms of soil were left to support the diaphragm wall; these were then reduced to allow the installation of the top struts, each bearing onto the raft. Excavations of the berms in short lengths followed, the lower struts installed and the remainder of the raft cast to secure the wall. The struts used were 600 mm *Rendhex* box piles and these were removed as the basement was constructed and the thrusts from the wall transferred into the permanent structure. Wall reactions were in excess of 300 tonnes and nominal loads of 10 tonnes were jacked into the struts at the time of their installation. Waling beams were avoided by providing additional horizontal reinforcement in each wall panel and horizontal continuity was supplied from the capping beam, cast as the excavations progressed. During the excavation the central unexcavated area was retained by tensioned McAlloy bars running between the surrounding walls. The ties were placed inside tubes which had been thrust-bored across the retained dumpling of soil at two separate levels. Struts were therefore unnecessary.

Daily pours in excess of 200 m³ were quite normal in the construction of the raft foundation. The unnecessary restrictions of our specification with regard to the maximum pour length of 10 m soon became apparent. Theoretically adjacent pours should only be carried out after most of the shrinkage of the first pour has finished and in this context one could almost put up an argument for less curing, to accelerate the shrinkage process! A chequered pattern for pour sequences was adopted, but pouring up against adjacent bays was unavoidable. As the raft slab was continuously restrained by the gound and by the column thickenings, we reasoned that the pour length could be extended without any adverse effect, and so it proved. Pour lengths in excess of 17 m were therefore common, reliance being placed on good quality concrete with low slumps. Curing was by sprayed membranes applied to the concrete surface.

In the centre of the raft an arrangement of wet and dry drainage sumps surrounded by six columns each loaded to 10,000 kN necessitated a local increase in excavation of 5 m below general raft level. The contractor requested that the major part of this area be cast in one pour involving a total volume of concrete of 730 m³ with a maximum lift height of about 3.5 m. We were concerned by the possibility of cracking due to excessive thermal gradients set up by heat of hydration. After investigation it was concluded that the cracks could be avoided by keeping the surface of the concrete warm. It was proposed that this should be done by ponding the surface, by using insulating formwork on the sump walls and by boarding over the sump openings to retain the heat. It was feared that the depth of the pour could lead to relatively high plastic settlements and that rigidly fixed reinforcement beneath the upper surface could result in cracks forming over the restraining bars, leading to the inducement of further cracks from the effect of the thermal gradients.

The upper surface of the pour was about 1 m below the final top surface of the raft and it was therefore decided to take the risk of omitting any so-called crack distribution steel from this surface. Thermal gradients were checked with thermocouples cast into the concrete. The pour was completed in nine hours without interruption, after which the surface was ponded, but not as deep or continuously as we would have preferred. The maximum temperature in the concrete, recorded at three days, was 58.8°C. Taking into account the time lag and the fact that little stress would occur until after the outer concrete had started to set, the effective temperature differential was 19°C between the centre of the mass and the top surface. Shutters were removed slowly over a 24 hour period, seven days after casting. No significant cracking was observed in the walls or in the top.

Superstructure

The simple beam and slab solution of the office structure followed cost comparisons with alternative forms of construction. Set out on the 7.5 m grid, the 1 m deep beams cantilevered to the heavy façades and the subsequent reductions in moment and shear at the centre support provided us with the opportunity to reduce the beams in the middle of the building, assisting the passage of services in the ceiling space. The plantroom almost covers the roof and is a structure of reinforced

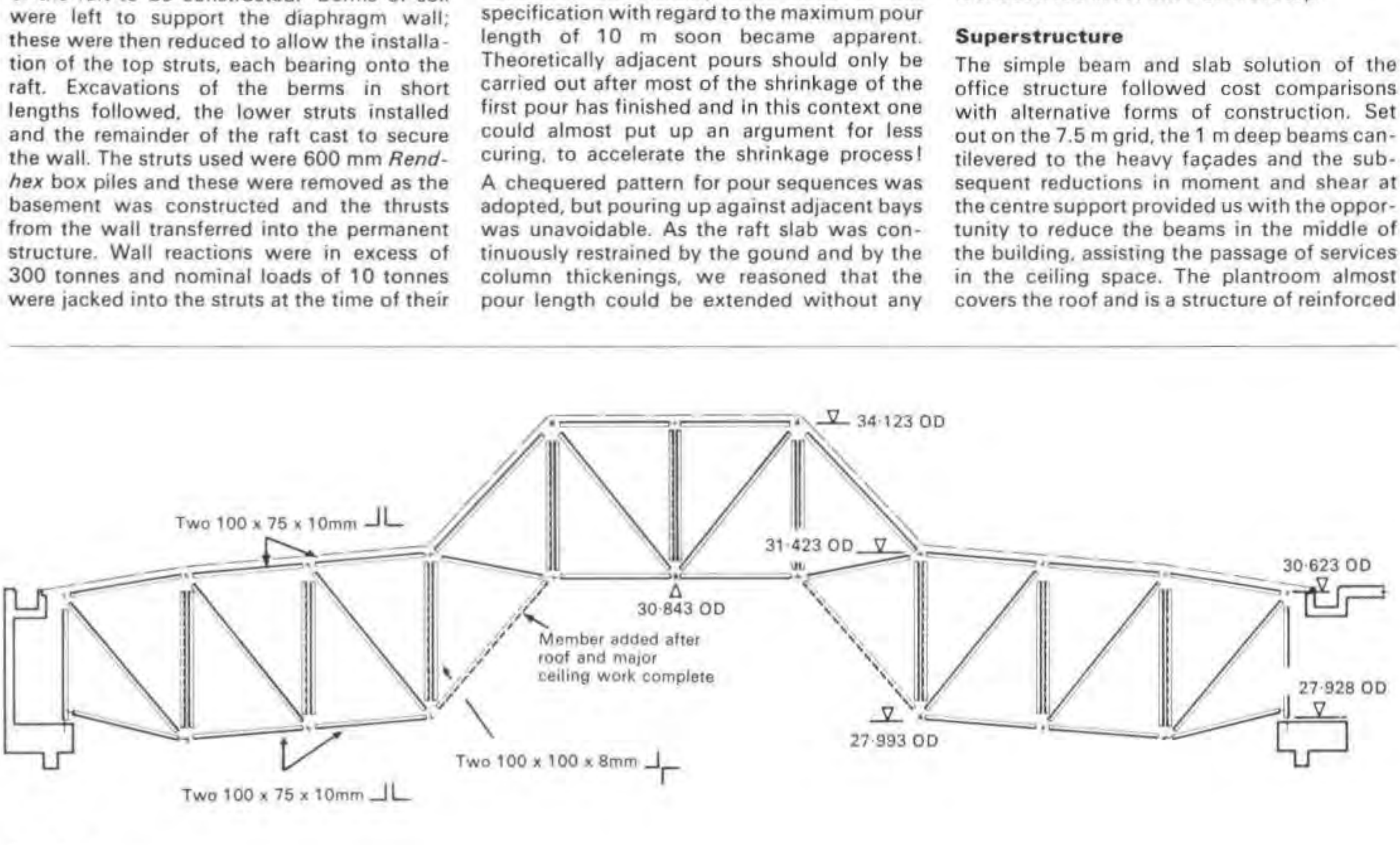


Fig. 17
Assembly Hall: elevation of truss

concrete portal frames set back from the edge of the building. Expansion joints in the building are 75 m apart, but the roof plantrooms have an intermediate joint as this part is more vulnerable to thermal movements.

The four lift shafts at the corners provided stability to the building, but their stiffness and position caused considerable restraint to shrinkage. The high stresses which would have developed at the junctions with the floors were relieved by leaving full-width bays in the floors unconcreted. These bays were cast as the final pours for each floor, by which time a significant amount of shrinkage in the completed concrete would have taken place.

The Great Hall

The Great Hall extends to a height of 17 m and is totally brick clad. The services for the air-conditioning run vertically up to the roof and are hidden within the space between the double skin walls. The roof has a double span of 22.5 m, sloping from a raised centre down to the eaves and the structure is a diagrid of steel trusses which is strongly expressed in the ceiling finish within the building. The raised centre of the roof was isolated from the remainder of the grid in order to simplify the design, and this was achieved by inserting the lower tie members around the centre after the roof finishes were applied. This procedure avoided the large forces which would otherwise pass into the centre and lighter members and connections resulted, with general economy throughout the roof. Erection, which took eight weeks, commenced from two temporary towers within the hall and the trusses were lifted in sections after fabrication on site. The final central ties were added after the roof had taken up its dead load deflection.

North wall

A gallery cantilevers from the west columns and from the north wall of the Hall. Pierced with large entrance openings, this two-storey high concrete wall is designed as a deep girder, spanning the full 22.5 m.

Fig. 18
Assembly Hall: brick mosaic
(*Photo:* Tony Langford)

Fig. 19
Assembly Hall: gallery
(*Photo:* Henk Snoek)

Fig. 20
Council Chamber: reflections from pool on moulded soffit of deck
(*Photo:* Harry Sowden)

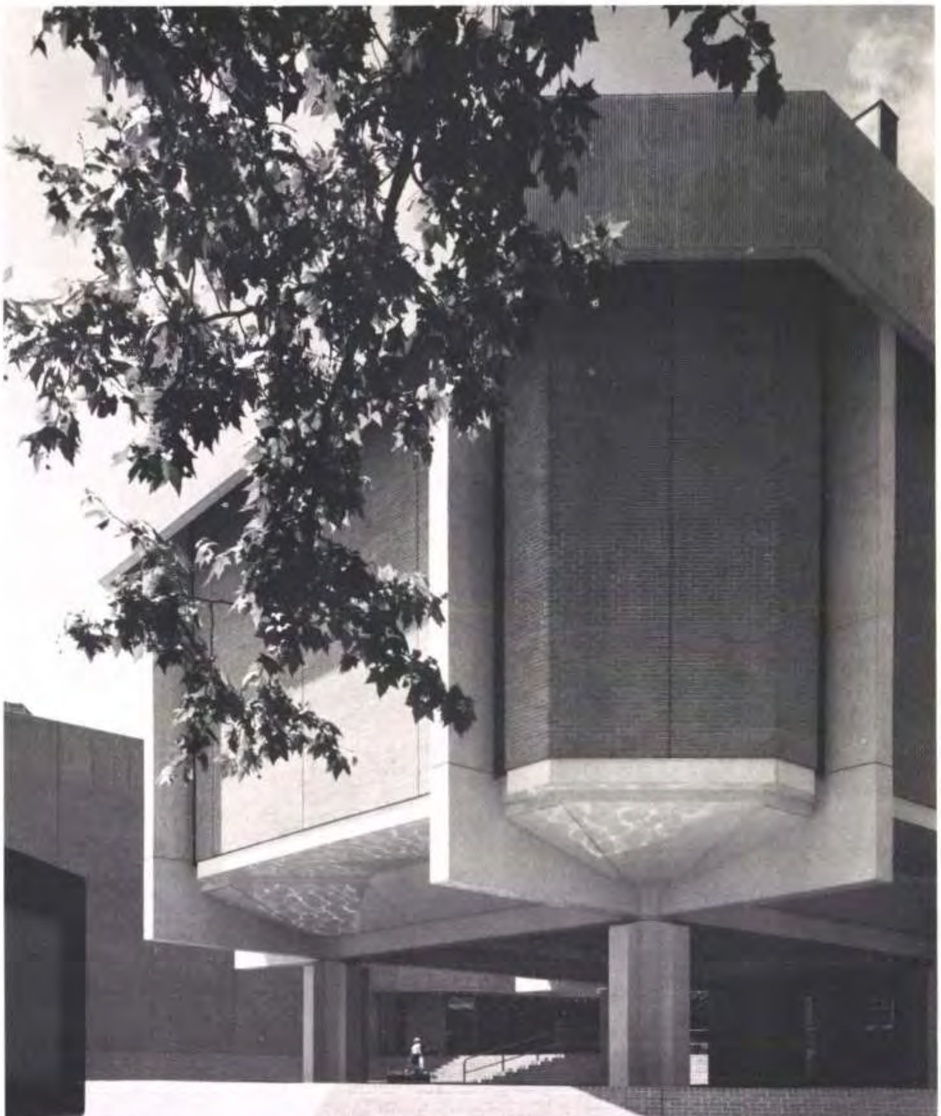


Fig. 21
Council Chamber
(*Photo:* Henk Snoek)

Fig. 22
Interior of Council Chamber
(*Photo:* Henk Snoek)

Council Chamber

The steel roof of the Council Chamber is carried on four trusses, set out on plan in a double cruciform arrangement. The trusses span 20 m onto eight circumferential concrete columns, tied at the eaves by a deep exposed edge beam. Each column bears onto the chamber deck and between them separate reinforced concrete sub-frames stabilize the double skin walls. The deck is modelled on the soffit by four mushroom heads, heavily faceted and broken by the downstanding beams, and the whole deck cantilevers out from the cruciform columns rising from the pool below.

The edge beam was cast in white concrete with a fine ribbed finish. The deck and columns were also in white concrete but all surfaces were grit-blasted. Hopton Wood limestone aggregates were used and the mix was gap-graded to highlight the grit-blasted texture, assisted in good lighting conditions by reflections from the pool. The cruciform columns were cast in one lift and the deck in three separate pours, each pour terminating at one of the many V joints in the concrete surface which emphasized the geometry of the soffit.

Conclusion

The controversy which has arisen around the new Town Hall has little to do with the architecture. From a tender figure of £6½ m in 1971, six years of inflation has pushed the building cost to a current £11.6 m, a fact which has not been overlooked by those who have tried to claim the Town Hall is expensive. However, the many diverse facilities now being provided and the improved working environment for both staff and Council will no doubt improve efficiency and be of lasting financial benefit.

Acknowledgements should be made to the principal contributors to the design. Brian Corbett and Richard Davies advised on geotechnical problems, and Bart Randerson guided us on the drainage. Jana Matyasova worked on the design from the start and finished the project as Resident Engineer. The Senior Resident Engineer was Malcolm Smith, and Valerie Williams did the major share of the layout drawings. Alan Frampton was involved from early days eventually taking over as Project Engineer. Les Dobinson and David Whitfield organized the detailing of 4,700 tonnes of reinforcement.

Credits

Architect:
John S. Bonnington Partnership
(Principal Architect, Anthony Page)
Quantity surveyor:
Reynolds & Young
Main contractor:
Taylor Woodrow Construction Ltd.
Diaphragm wall sub-contractor:
Tarmac Soletanche
Structural steel sub-contractor:
Dibben Structural Engineers

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