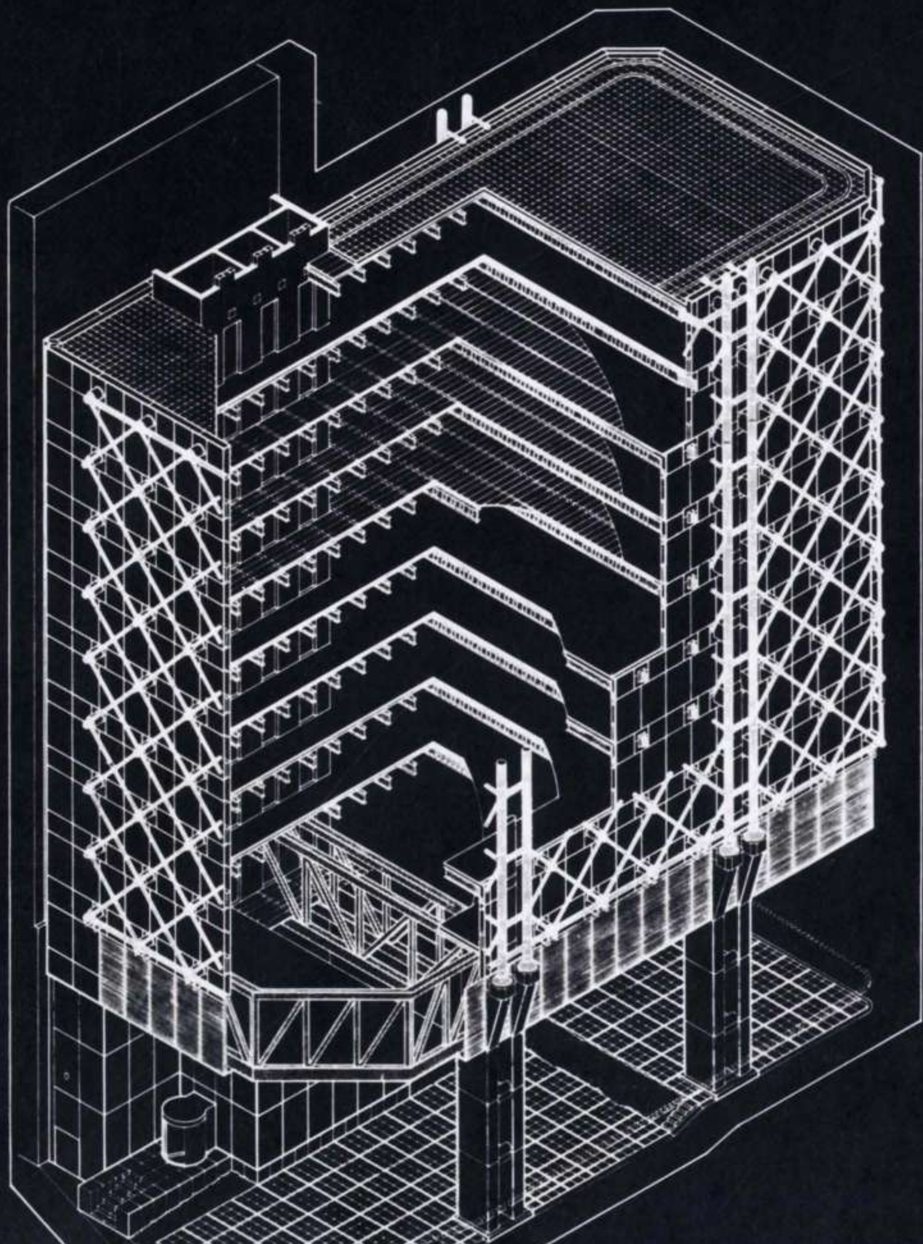
THE ARUP JOURNAL

DECEMBER 1976



THE ARUP JOURNAL

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Back Cover: Royal Exchange Theatre: Plan of roof at level of top chord

The role of the structural engineer in the building industry

Peter Dunican

This paper is based on a talk given at the Institution of Structural Engineers – Republic of Ireland Branch – 6 January, 1976

Before discussing the role of the structural engineer in the building industry perhaps we should try to establish what a structural engineer is, what he does and the nature of the building industry in which he plays such an important part. I think this is necessary because there is a lack of definition. For instance the Royal Charter of the Institution of Structural Engineers says that the main object and purpose of the Institution is, and I quote: 'To promote the general advancement of the science and art of structural engineering in any or all of its branches and to facilitate the exchange of information and ideas relating to structural engineering amongst the members of the Institution and otherwise'.

But the Charter does not define what a structural engineer is, at least not in functional terms, although the Institution does say: 'If you have an aptitude for mathematics and physics and are attracted by the creative challenge of the construction of the buildings of all kinds that are part of our environment, you have the makings of a successful structural engineer'. However, no mention is made of other finer feelings, like the ability to draw for instance.

And the Institution also goes on to say: 'When you see a bridge, an elevated motorway, a dam, a water tower or a large building or factory, it is the structural engineer who has been responsible for the design of the structure and supervised its construction. The strength and safety of foundations, of steel or reinforced concrete frames, floors and roofs, or arches and retaining walls are all his direct concern'.

This statement also seems to go some way in defining the work of the building industry.

My personal definition of structural engineering is that structures are assemblies of compatible elements arranged in statically stable, geometrically suitable and aesthetically acceptable systems which are needed to meet some external purposes such as the support of applied loading, the spanning or enclosure of space as in buildings and other similar artefacts which our society wants.*

The art and science of structural engineering is the economic design and construction of such systems.

So in brief, structural engineering is the applied science of design to ensure the feasibility, the construction, the stability, the economy and the aesthetic suitability of buildings and other structures.

To begin with, therefore, this is what a structural engineer does, or at least generally what he does at present.

It is not true, as some wit has suggested, that structural engineers mould materials they do not really understand into shapes they are unable to analyze so as to withstand forces they cannot assess and in such a way that the public does not really suspect.

When I started my engineering career in Victoria Street just before the War it seemed to me that structural engineering consisted mainly of the analysis of structural systems largely determined by others without too much reference to the potentiality of the available structural materials and their economic possibilities. To me the calculation of reinforcement seemed to dominate the design process.

*After much discussion in Council and in Committee, on 24 June 1976 the Council of the Institution adopted the following official definition: 'Structural engineering is the science and art of designing and making, with economy and elegance, buildings, bridges, frameworks and other similar structures so that they can safely resist the forces to which they may be subjected.'

The end of the War was the watershed not only for the practice of structural engineering but for many other activities.

New ideas

In 1945 the notion that Victoria Street was the beginning, middle and end of civil and structoral engineering and construction was being questioned. Consulting engineering practices began to spring up in odd places where before only architects appeared to flourish, for instance in Bloomsbury.

And the idea started to develop that perhaps the role of the structural engineer as an architectural handmaiden had more to it than fighting over the size of a column; and that perhaps it might be more profitable to consider first of all whether a column was necessary. The implication was obvious. Architects, structural engineers and building designers began to think about buildings in whole as well as in parts; to talk about integrated design where the parts *are actually designed* to fit together.

The art of structural engineering began to appear in a real form.

To me the art of structural engineering means designing a structure which not only fulfils all the specific functional requirements and other relevant physical conditions but which is also in harmony with its environment; where the structure contributes positively to its environment and forms an integral part of it. In other words we must look beyond the building.

All buildings must satisfy some conditions and the structure of a building must satisfy at least four: it must stand up, it must be capable of being built economically, it must be within the means available, and it must suit the building for which it is intended.

It is not difficult to find a number of viable structural systems which will suit a particular building but the final geometry of the system will depend on the form and function of the building and the available structural materials.

The structural materials will influence the constructional process, and the form of the building will be decisively affected by the geometry of the structure. The possibilities appear to be infinite and if they were, design would be virtually impossible. Fortunately, in practice, the choice is limited. Disciplines exist; some naturally, others we create for ourselves.

These self-created disciplines should be constantly and critically examined and reviewed. They usually present the most difficulty in our pursuit of the ideal solution which we all seek; the perfect structure.

Nevertheless, despite the limitations there is considerable choice and probably there is no ideal solution. The problem can only be solved pragmatically, by trial and error if you like. All design is a process of trial and error – the reconciliation of conflicting conditions; one makes assumptions; one tests these assumptions by detailed examination and modifies them in the light of what is found, and then re-examines them. This process continues until all the imposed conditions have been satisfied – until an acceptable solution has been found. This process can be short or long, depending on experience and on the stringency of the conditions which are imposed and on the time which is available to make the choice – the more time you have, the longer you appear to take to solve the problem. There is nothing like the spur of external pressure to help produce what is wanted quickly and economically.

But we must always ask ourselves – are we solving the right problem? Perhaps we are concerned too much with more tractable matters such as the detailed design of the structural members. Our first concern should be with the conception of the system itself and then with the way it is to be built. The method of building should be inherent within it. Some structures can be built in a variety of ways. But for every structure there must be a way of building. No engineer should design anything without knowing at least one way of making it.

Many engineers exaggerate the importance of structure in building and this is understandable. In their training they are usually concerned with relatively simple engineering constructions and they are not taught to appreciate the problems of the architect; that the only reason for the existence of the structure is the building, and that without the building, the structure has no meaning. On the other hand some architects do undervalue our contribution in achieving a viable architectural solution of the building problem.

But apart from any mutual lack of appreciation, perhaps a more significant impediment to achieving the required degree of collaboration between architect and engineer is the question of technical competence. It is unfortunate, but nevertheless true, that most engineers suspect the technical skill of the architects with whom they are working. This does not necessarily apply at the top level, but the suspicion certainly does exist within the drawing office. This often leads to a technical arrogance on the part of the engineer, which conflicts most unfortunately with the assumed intellectual arrogance of the architect.

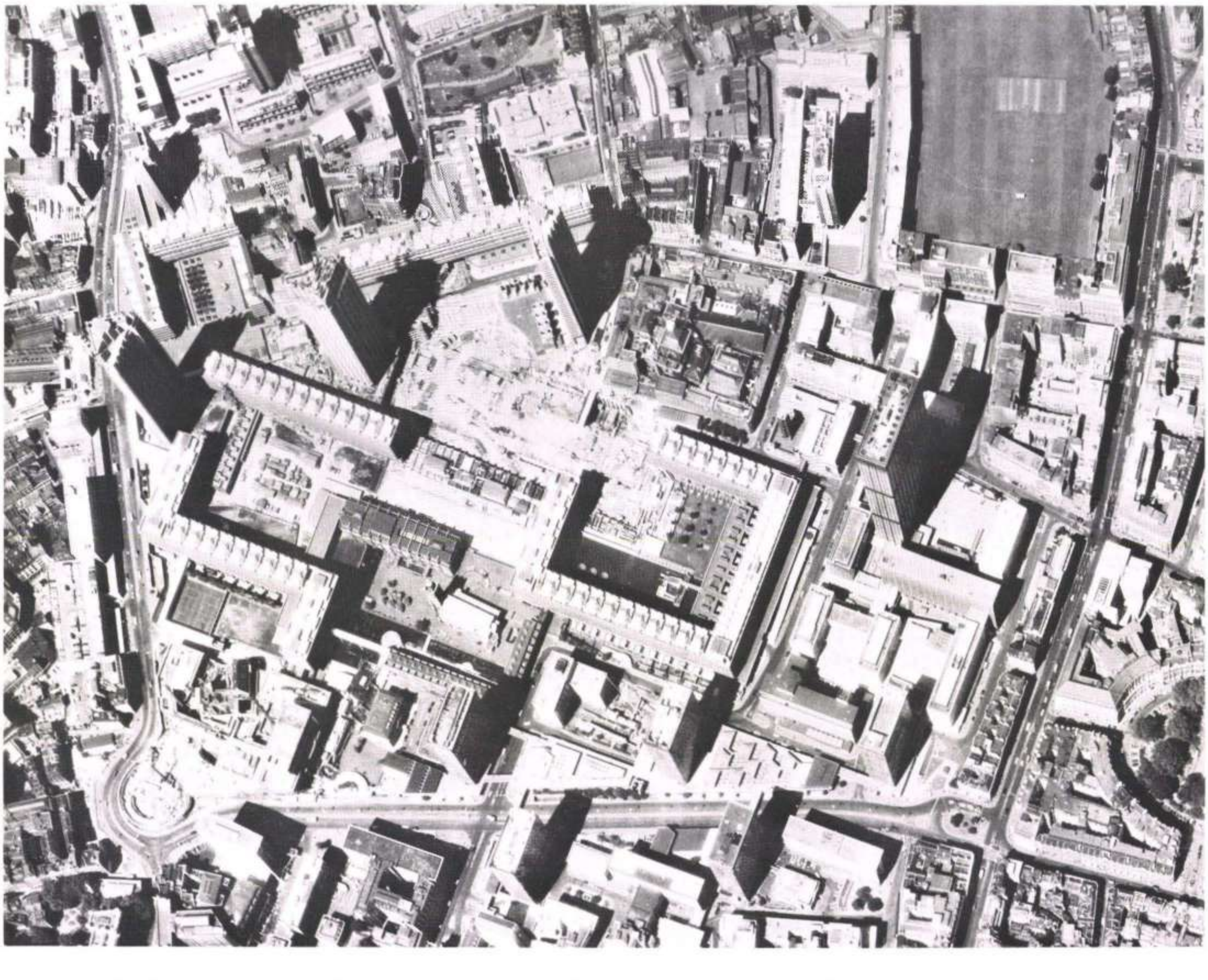
This may appear to be a sweeping generalization, which it is, but at least it has some germ of truth; mutual respect is imperative if we are to be successful.

The structural engineer's contribution to the design of a building has to be judged in its context. Perhaps it is unfortunate that in many cases the most successful structural solutions are those which are so integrated with the building that they cannot be seen, except perhaps by the cognoscenti. Structural acrobatics have a place, no doubt, but rather as the exception than as the rule; as Nervi says, 'structural acrobatics are a sign of false structural conception'.

But the biggest difficulty the engineer has is probably that of accommodating himself to the many seemingly conflicting philosophies of architectural design. For instance, there is the idea that the structure is the main architectural statement and the mechanics of the structural system should be directly expressed with the utmost economy of means – a complete, unalterable, finite unity. On the other hand, there is the theory of an additive or endless architecture in which the external structure consists of a series of repeating simple, well-proportioned, neutral elements creating some geometrical network without any emphasized boundaries or apparent limit. And there are many other ideas; long life, loose fit, low energy for instance.

There is no doubt that most architectural philosophies harbour some essential truth. These may be beyond the reasonable understanding of most ordinary folk but for engineers, who are supposed to think logically, it is not easy to reconcile often conflicting philosophies expressed sometimes in a language apparently intended more to confuse than to clarify.

However, it is imperative that if structural engineers are to work within architecturally conceived notions, they must trust the architect who is doing the conceiving. Mutual trust is essential. And so is faith. But it does not



Barbican Redevelopment Scheme: Architects: Chamberlin Powell & Bon (Photo: Aero Films Ltd.)

follow that the engineer need not understand what it is all about. Of course he must but first he ought to master his own technology. Most of us are not as competent as we should be.

The unfortunate truth is that we do not know enough about the materials which we are using or the processes of design, the analysis, synthesis and evaluation of the problem. Structural design is rather like trying to deal with a random input into a non-linear system.

So far I have concentrated on ourselves, on structural engineers and architects, but designing buildings is an activity which involves many different sorts of technologies and technologists. This is an omnibus term to embrace all of us, architects, structural, constructional, mechanical, electrical and other environmental services engineers, building economists, and quantity surveyors, etc., who are directly concerned. There are many others who are indirectly concerned.

Building is a multi-disciplinary activity which traditionally has been directed, rather than managed, by the architect in a simple, separate relationship with each of his technologist consultants. Building design has generally been approached on an individual basis in the same way as each building has been dealt with, as an individual project, seemingly in isolation.

Until recently it has not been accepted that the design of buildings is a process which is susceptible to the processes of formal management techniques, whatever these might be.

There were three main reasons for this situation, firstly we did not know exactly what design meant as an activity. Secondly we did not know how it was carried out, despite the fact that management is a function of design, and that design is a process of management. Thirdly, the design of buildings is an art as well as a science dominated by the idiosyncracies of the designers.

We know that structural design is particularly concerned with imposing order and method on construction and, like construction, it is a process which can be managed, although it is generally true that most designers do not recognize the managerial nature of the process which they are carrying out.

Design as a process

Clearly, to manage we must know more about design as a process and how it is carried out and perhaps, what is more important, how it should be carried out to make it more efficient.

If you do not know how you do whatever it is, and what might be wrong with the way you do it, and with its consequences, how can you possibly improve on what you do and, perhaps what is more important, on what you are doing it for. The main reasons for trying to improve the way we design is to reduce the effort and improve whatever it is we are designing to improve the product.

Design is the solution of a problem within a real resource envelope; and management must be concerned with making the most efficient use of the available resources; to produce what is required at the right time at the right price with the minimum effort. This requires all concerned to work closely together, that is, it requires team working.

Team working, to be successful, requires that the members of the team must not only be competent but they must also be compatible, be able to communicate with each other and above all to care for what they are doing. They must be totally committed to the task in hand. As design teams consist of people, so must management be concerned with how to get them to give of their best, and for each and every one of them to derive the maximum satisfaction – material, intellectual, professional and spiritual – from their work.

The individuality of the team members is most important – the strength of a team can be more than the sum of the strength of the individual members of the team – but for success it is necessary to recognize individual strengths and weaknesses because by doing so any imbalance within the composition of the team can be reduced, that is if it cannot be eliminated.

Broadly, design teams can be set up in a variety of different ways, but in the design of buildings there are two usual arrangements. The first, and the more usual, is through a series of separate individuals or groups, brought together under the leadership of one or another of the team, but usually the architect, to form an inter-disciplinary group to deal with a specific problem. The second, and not so usual, is through a multi-disciplinary group or entity which has within it all of the necessary disciplines or skills required for dealing with the design problem.

In each case the management problems are the same in principle, although they are different in detail, mainly because of the different communication problems.

Communication is an integral part of the management process. It is essentially a two-way system, which must ensure that the required information of the necessary quality and quantity is transmitted and received by all concerned at the right time. The right information at the wrong time can be even more frustrating than the wrong information at the right time, so therefore the timing of communication is most important.

Management of Design

The management of the design process must clearly recognize the inherent creative function of design, which, although susceptible to rationalization, cannot be pre-empted. As Chermayeff has said: inspiration is a special moment in a rational process.

Of course, to have any inspiration you must be capable of being inspired but unfortunately this is a state which cannot be preordained or predetermined. It is probably true that in the past, the need for inspiration has been overstated in the design of buildings, but nevertheless inspiration is the hallmark of good architecture. It distinguishes the good from the commonplace – and there are too many commonplace buildings being designed and built today.

The implication that the architect is the leader and organizer of the design team should be reviewed. This is a difficult position for the individual architect to sustain if at the same time he is personally responsible for the architectural design. This is because his emotional involvement in the design is likely to inhibit him in his role as team leader, where he would be required to make design decisions based on the advice given by other members of the team which may be in conflict with his preconceptions.

On the other hand the structural consultant may not recognize sufficiently that the design of buildings is essentially a compromise between what is ideal and what is possible and that structure is only a part, an important part no doubt, but still only a part of a whole.

Also the structural engineer does not concern himself as much as he should with the way in which the structure and the building should be made. He also is critical of the architect's apparent ignorance of the process of building; although he does get on quite well with him, probably because the architect has more understanding of structure than of the other engineering aspects of building.

There are also difficulties for the services engineer who naturally is preoccupied with his pipes and ducts and plant and the multiplicity of the possibilities. His defensive attitude towards himself arising essentially from criticism based on subjective criteria of measuring the performance of his work; his historical background, the conditions within which he operates and above all his apparent non-regard of the main objective of the exercise, to produce a building which satisfies the needs of the users, all add to the difficulties he has with the other members of the design team; perhaps the biggest single difficulty is with his fee agreement. At present he is not paid for producing what is required by the other members of the team when they need it, if they are to satisfactorily complete their design work.

Then we have the quantity surveyor, who must be considered as an important member of the design team contributing in a positive and definite way towards the design. He is not a creative designer but his role should not be limited, as many quantity surveyors appear to think, to forecasting prices and obstructing innovation and change because he is unable to estimate costs. Cost analysis and control can be a constructive contribution to the design.

Having established that the crucial problem in the design of buildings is that it is divided amongst a number of different disciplines, I would like to refer to another significant problem which has only been mentioned in passing, which is that design is divorced from production. In this sense building is rather particular and peculiar, if not unique.

Generally, the way in which design decisions are made in any multi-discipline, industrial situation is independent of what is being designed because the producers are integral participators in the design process. In building, however, production is usually independent of, and separate from, design, and designers with perhaps the exception of the engineers involved are not knowledgeable about the processes of production.

Therefore there is an inherent discontinuity – or credibility gap – in the building process if this is defined as all that goes on from when it is decided that a building is required until it is completed and ready for occupation.

So, if you believe that design concerns directly the process of production, the situation in the building industry imposes an additional management problem of some significance. This is not an insuperable problem, but it is inhibiting in that design decisions can more adversely affect the cost of the product. If the design is not entirely right for production the cost must be unnecessarily inflated. And cost is money and money is part of the resource envelope.

And it is here that I see the essential dichotomy confronting us today. Do we continue to concern ourselves with the very important narrow specialization of structural engineering or do we widen our horizons and apply the management and communication skills which we use within our design activity to the much wider problems of the industry?

To oversimplify this argument, already in our work we determine the way in which the structure should be built. This in turn, has a direct effect on the way in which the building is made so when we have finished our design why don't we get up off our stools and go out and build it? But I don't mean build it in the conventional sense but manage the building of it as a professional activity in the same way as we could manage the whole design process. We are numerate, we believe in order and method and usually we are what might be termed the precursors of the building process. We don't need to own plant, employ labour or be wealthy or have shareholders whose prime interests we have to serve. We don't have to be package dealers.

Conclusion

No doubt there are one or two arguments against this proposition which are discussable. However, all I would say now is that whilst I see the demand for our services as structural designers continuing strongly if not expanding, I do see that our central position in the building process would make it possible for us to take up a dominating role in the industry, not only to the benefit of the industry but for the benefit of the community as a whole, which, of course, is the main reason for our existence.

Structural Steel Design Awards 1976

Three buildings for which Ove Arup & Partners were consulting engineers have been named in this year's Awards.

The buildings

The Habitat Warehouse, Wallingford, Berkshire (job no. 4094), designed by Ahrends, Burton and Koralek, and the swimming pool at Richmond Recreation Centre, North Yorkshire (job no. 4243), architects Napper Errington, Collerton and Associates, each won an Award, whilst the Exhibition Halls at the National Exhibition Centre, Birmingham (job no. 4300), designed by Edward D. Mills and Partners, received a commendation.

The Habitat Warehouse and the NEC were both dealt with by the Building Engineering Division, Section B, and the Swimming Pool by the Newcastle Office.

Fig. 1 above
Habitat Warehouse, Wallingford, Berkshire (Photo: John Donat)

Fig. 2 right
Richmond Recreation Centre (Photo: Henk Snoek)

Fig. 3 below
The National Exhibition Centre, Birmingham (Photo: Handford)

Bush Lane House

John Brandenburger
Michael Eatherley
Dick Raines

INTRODUCTION

Architecture has essentially to meet the needs of people in the widest sense as well as at the most detailed level, and the resolution of these, together with the technical means of achieving solutions, is what provides our professional challenge.

When describing projects within the practice there is a tendency to concentrate attention on the technical aspects of our designs; partly because our expertise generally encourages a pride at this level and also, one suspects, because it is easier to provide more acceptable descriptions at this rational (non-subjective) level of design. So be it.

In this building the technical aspects of the design have involved by far the larger part of the time and effort of all those concerned. This article sets out the context of the building design and the main factors which generated the adoption of certain solutions, before going into more detail. It should, however, be constantly remembered that these solutions and the whole range of technology they have invoked are in order to meet the client's requirements.

The brief was succinctly stated: to provide the maximum permitted usable floor area for a high quality lettable City office building.

THE PROBLEM

With the Fleet Line Bill of 1970, London Transport sought powers from Parliament to acquire the freeholds it required along the route between the Strand and Fenchurch Street. The existing Bush Lane House adjacent to Cannon Street Station occupied one of these sites and in April 1970 the directors of Trafalgar House Developments Ltd. instructed us to test the feasibility of replacing the existing building with a new one, which, in addition to achieving the office area required, would allow the subsequent tunnel and station construction below the site.

We immediately started talking with the then New Works Engineer of London Transport (a Mr Turner, well known to those involved in the tunnel re-alignment at Barbican). Three main conditions emerged:

(1) Foundations had to be restricted in location and extent to those areas not to be occupied by tunnels, air shafts, escalators and machine rooms.

(2) A clear headroom of 10 m above ground level was required over a substantial part of the site area and supports to the building above were restricted.

(3) The ground area and site access had to be available to London Transport by June 1974. (This programme was subsequently scrapped.)

A feasibility study was prepared by August 1970 which demonstrated to the London Transport engineers that their sub-structure conditions could be met. In order to allow Trafalgar and London Transport to draw up their agreement for this development we were instructed in December 1971 to prepare designs and obtain planning permission.

Apart from imposing the normal town planning conditions of daylight, plot ratio and building lines, the City protects views of St Paul's from certain strategic positions, one of them being the Monument viewing gallery. The building is directly in line between these two and is restricted in height to 44 m.

Fig. 1
View from the Monument – photo montage

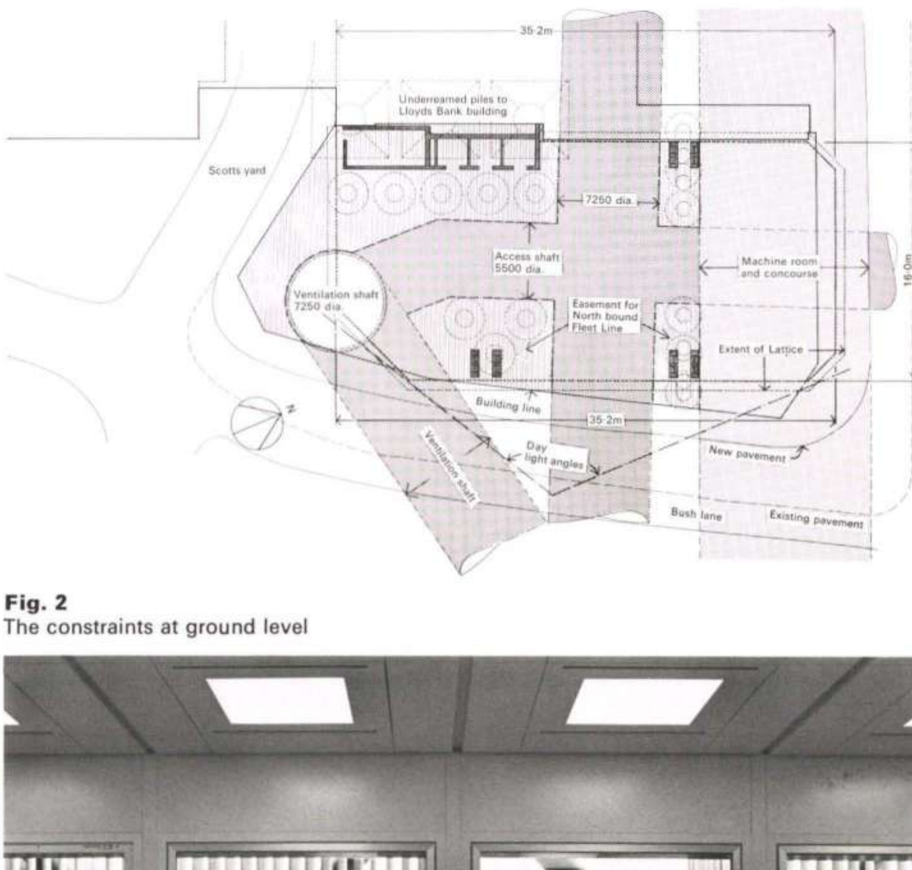


Fig. 2
The constraints at ground level

Fig. 3
Seventh floor view

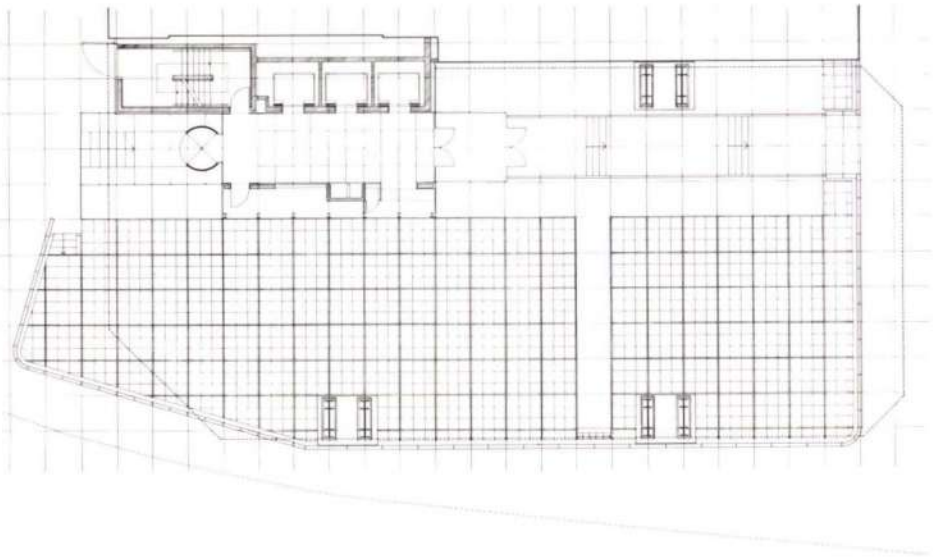


Fig. 4
Ground floor plan

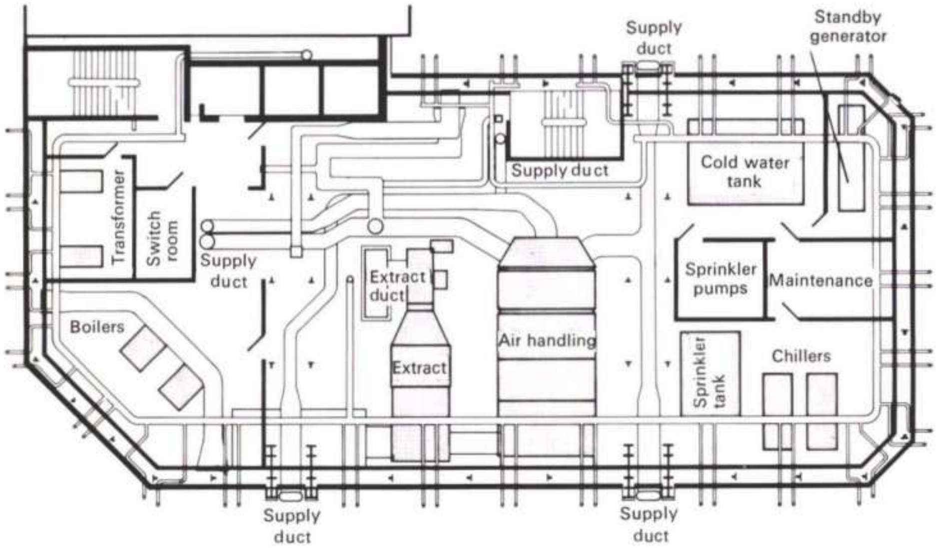


Fig. 5
Plantroom plan

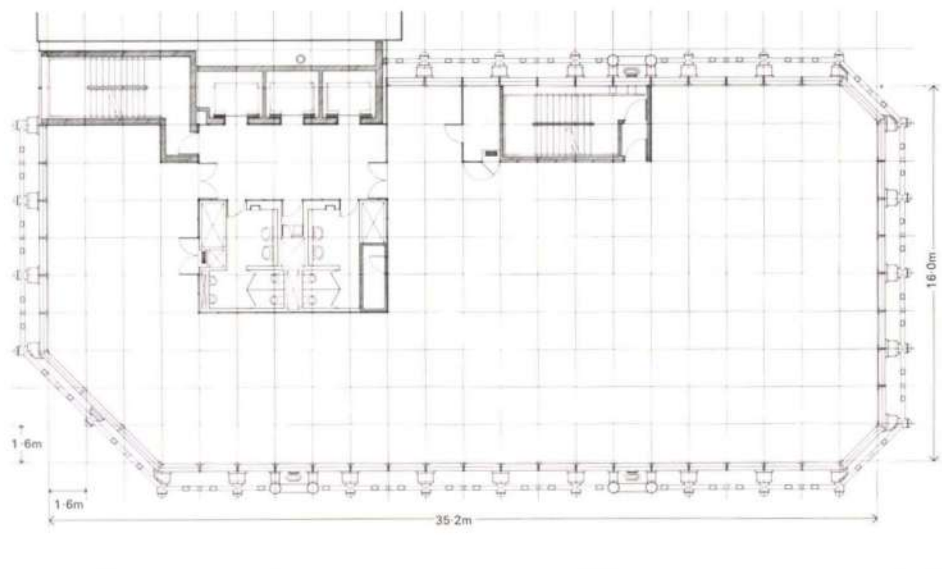


Fig. 6
Typical floor plan

THE DESIGN

The site is a small one, 1027.5 m² and the area per floor was further limited principally by daylighting restrictions. Also the City's height limitation, combined with London Transport's headroom requirements, meant that the permitted plot ratio area had to be planned into a somewhat compressed building envelope.

The tunnel layouts imposed a stringent limit on the area available for foundations, and this determined that it should be a lightweight building. Further, the pre-determined column positions, combined with the clear headroom requirement, dictated that a substantial area of the building would be cantilevered from the inset columns.

The eight office floors are planned over the first floor plant room, which contains all primary services and the air handling plant for the air conditioning. Each typical floor has a lettable area of 435 m² and is approximately 35 m long and 16 m wide. It is supported by the lift core and three columns set 11 m in from the extremities of the building. The main lift circulation core is planned against the party wall of the adjoining Lloyds building, so that the office areas gain maximum benefit from daylight and the impressive views over the City towards St Paul's and of the River and Tower Bridge.

a

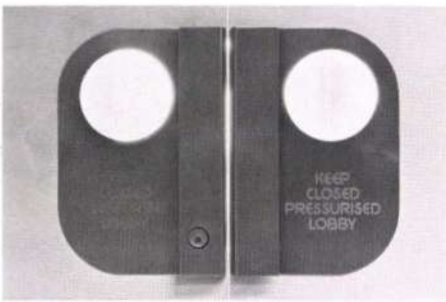
b

c

Purpose made graphics for Bush Lane House were designed by Pentagram Ltd. Photos **a** to **f** show some applications

Photos **g** to **o** show various aspects and details of the building

d



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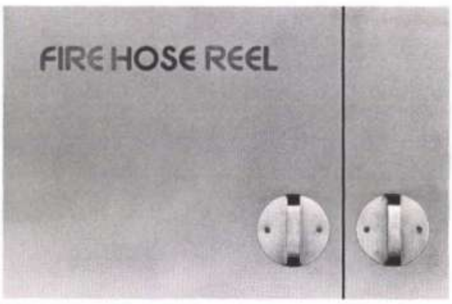

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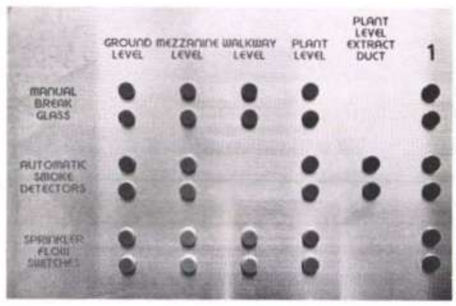

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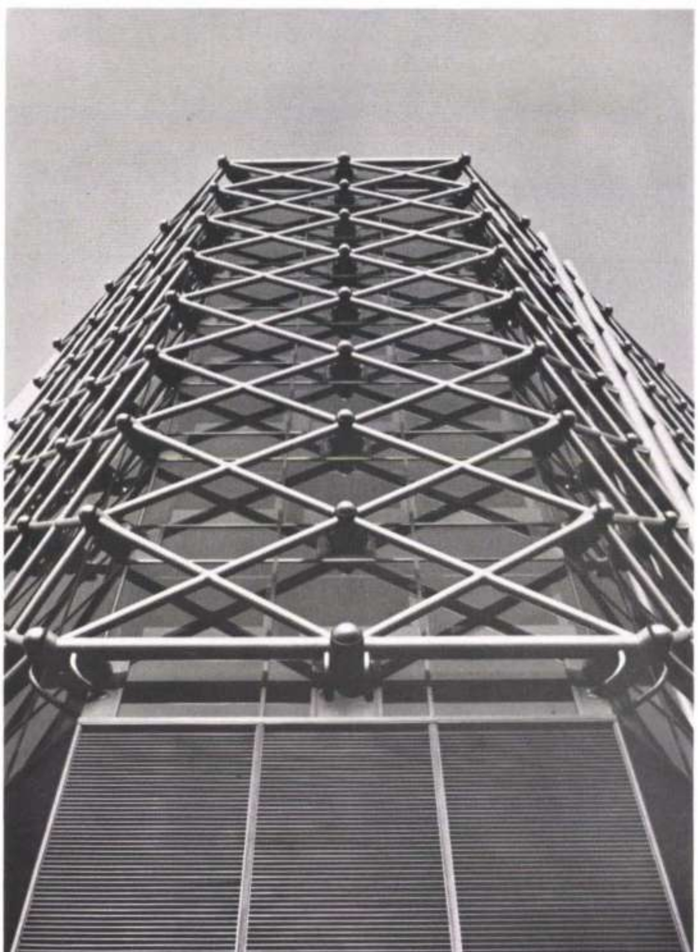

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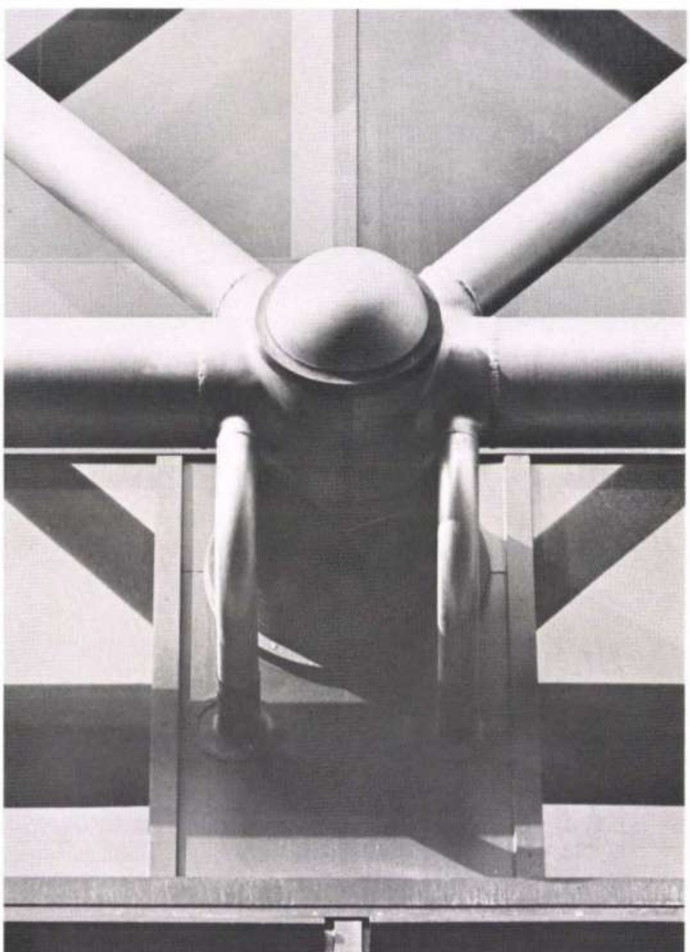

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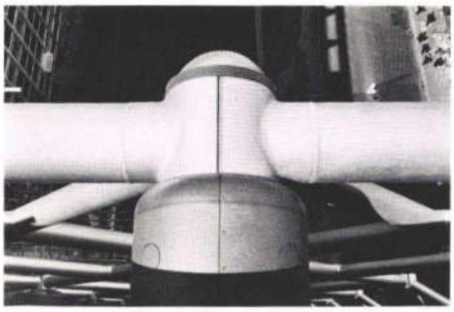
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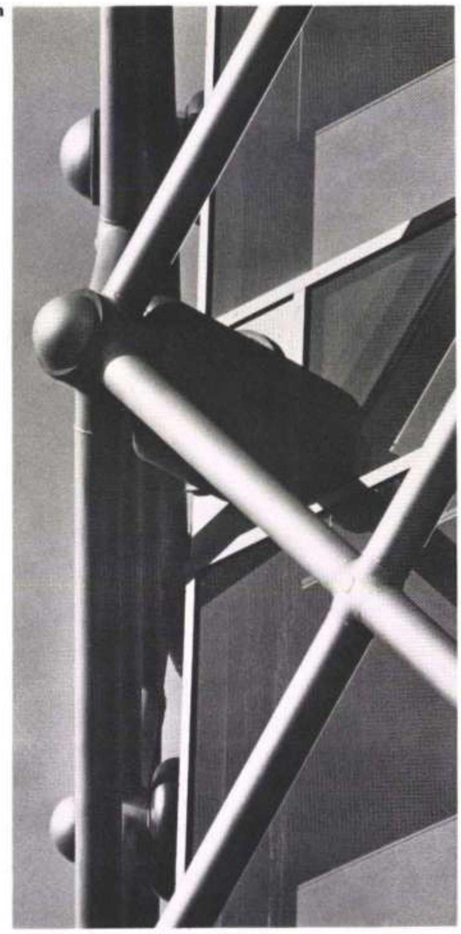
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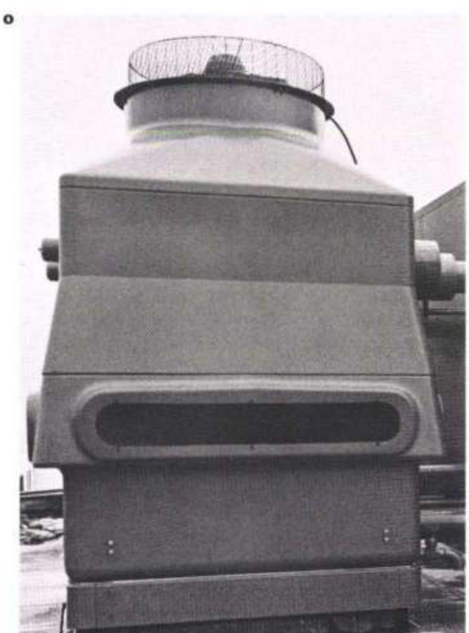
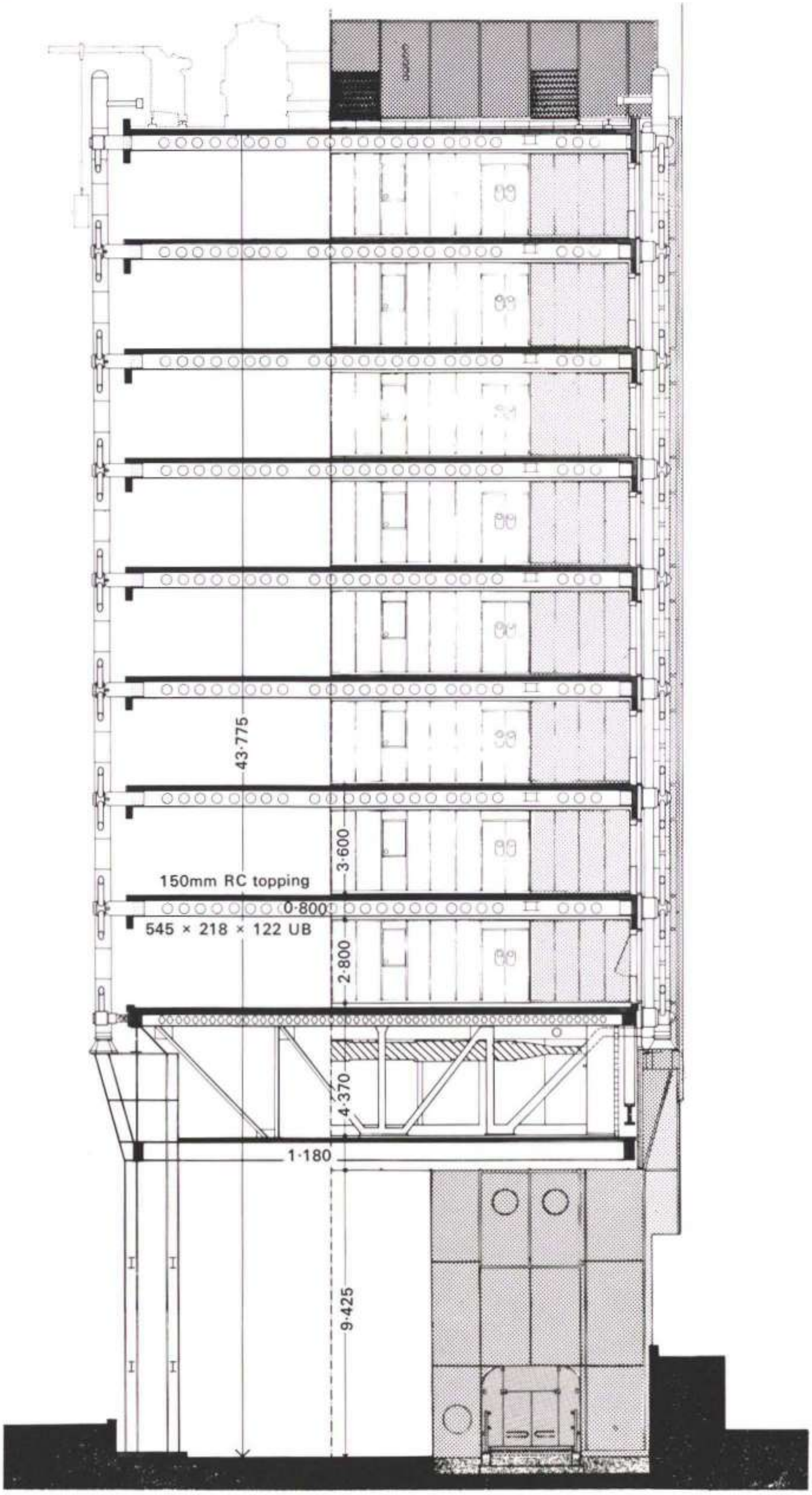




The ceiling is designed to encourage tenants to plan partitioned layouts within the modular discipline. 1.6 m square modules are separated by a recessed channel able to receive standard partitioning, and each module contains specially designed lighting and air supply or extract which, with an adaptable system of air volume control, provides conditions which allow a wide range of office layouts. Similarly an underfloor trunking system allows distribution of power and telephone wiring within the modular discipline.

Apart from the stainless steel lattice and grey glass all external surfaces are clad in aluminium which, except for the red external air ducts, are anodized dark grey. The panels cladding the lift core, first floor soffits and the roof plant room are honeycomb cored to ensure flatness. Normal external maintenance is restricted to washing.

Internally the suspended ceiling with its integral lighting and wall cladding are stove-enamelled metal panels. Floors are carpeted.

STRUCTURE

Two structural systems of resolving the floor loads into the isolated and inset columns were investigated; first, a frame sustaining the loads over the full height, and second, columns carrying the loads down to a heavy girder at the plantroom level. The most important objection to the latter was the limitation it imposed on the plantroom, where heavy girders of any form would severely have limited the size of openings for the intake and exhaust, and for access to the plant. It was not possible to take in air through the floor of the plantroom owing to the risk of ingesting stale air from the LTE vent shaft. Fig. 10 illustrates the plant layout and the extent of the equipment and ductwork.

A full height lattice frame largely overcomes these difficulties. The floor loads are transmitted to the columns at each level so that the individual members remain quite small and the plantroom structure allows space for the air openings and for access.

During design the structure developed into two principal elements: the plantroom at the 9 m level supported by structural steel trusses, and the steel lattice above carrying the eight office floors. These are connected at first floor level by links which transmit lateral forces but allow free relative vertical movements. Such a division of the structure avoids the geometrical problem in a diagonal system of coping with the differing headroom requirements: 4 m in the plantroom and 2.8 m in the office floors.

It also made possible an early completion of the lower structure, which enabled the installation of plant to proceed, and in itself formed a high level platform for the erection of the upper floors. The division in the structure is reflected in the curtain wall where a movement joint is incorporated at first floor level.

In order to exclude traffic noise, measured at up to 90dBA around the perimeter of the original building, the external wall consists of a double skin curtain wall with fixed glazing.

The external skin is 10 mm thick grey glass and the internal skin 6 mm clear glass and there are aluminium vertical blinds between the skins to reduce glare and solar heat gain.

Office areas are air-conditioned, the air being supplied through ceiling diffusers which are designed integrally with the special light fittings. This method was adopted in preference to the more usual perimeter induction units as these would have significantly reduced the usable floor area. Glazing of the internal skin is limited to 50% of its area to reduce the cooling load and to ensure satisfactory comfort conditions close to the perimeter.

Fig. 7 above
Cross-section

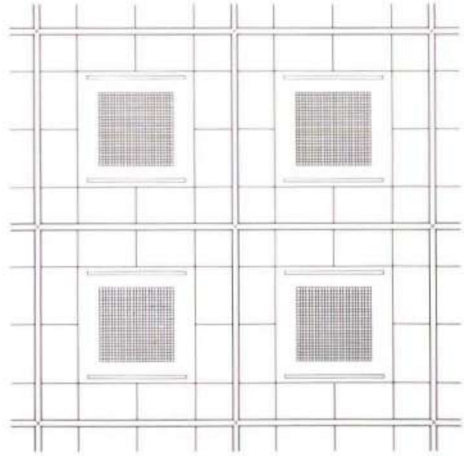


Fig. 8 right
The ceiling modules

Fig. 9
An office – from the prototype

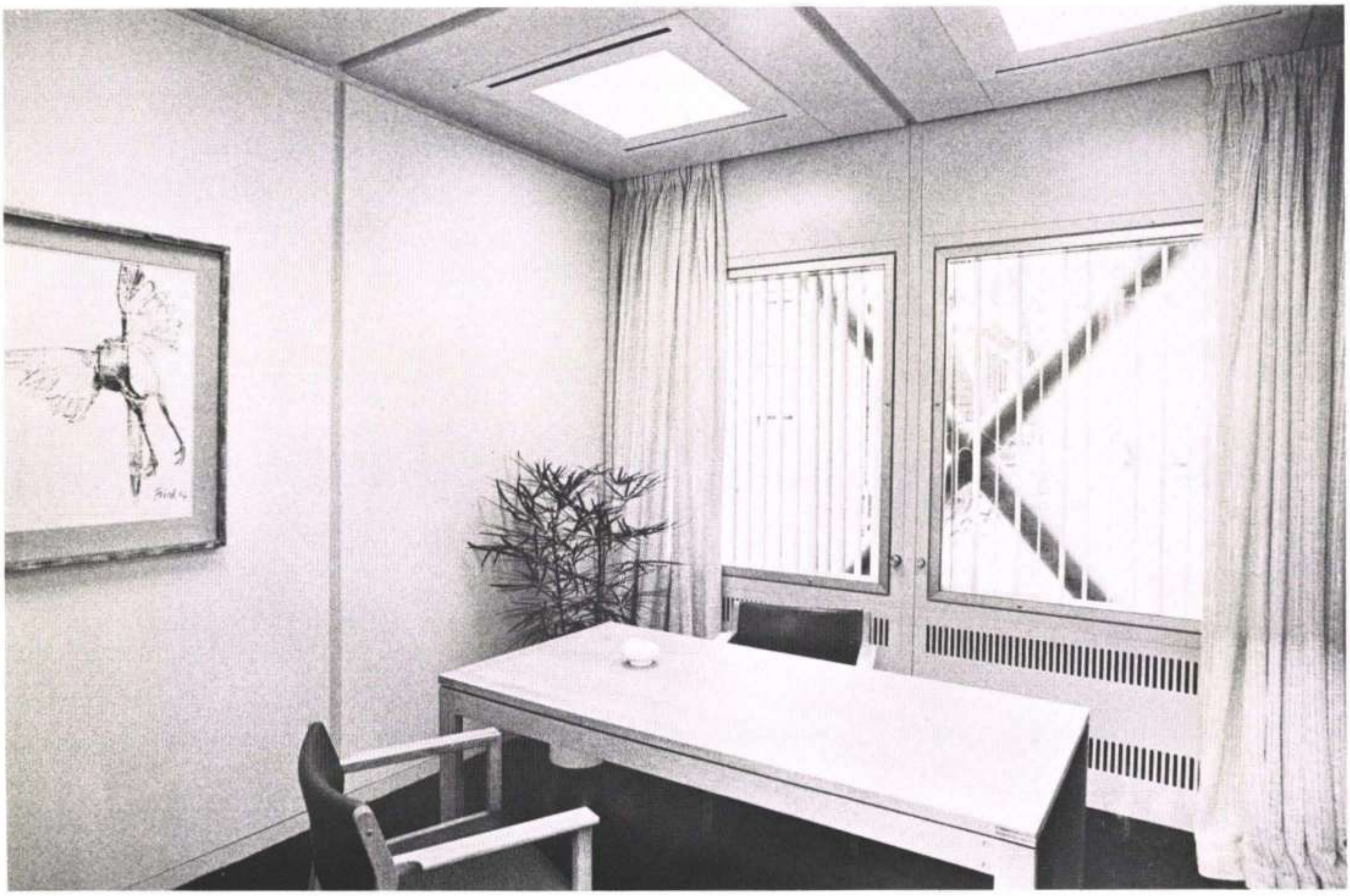
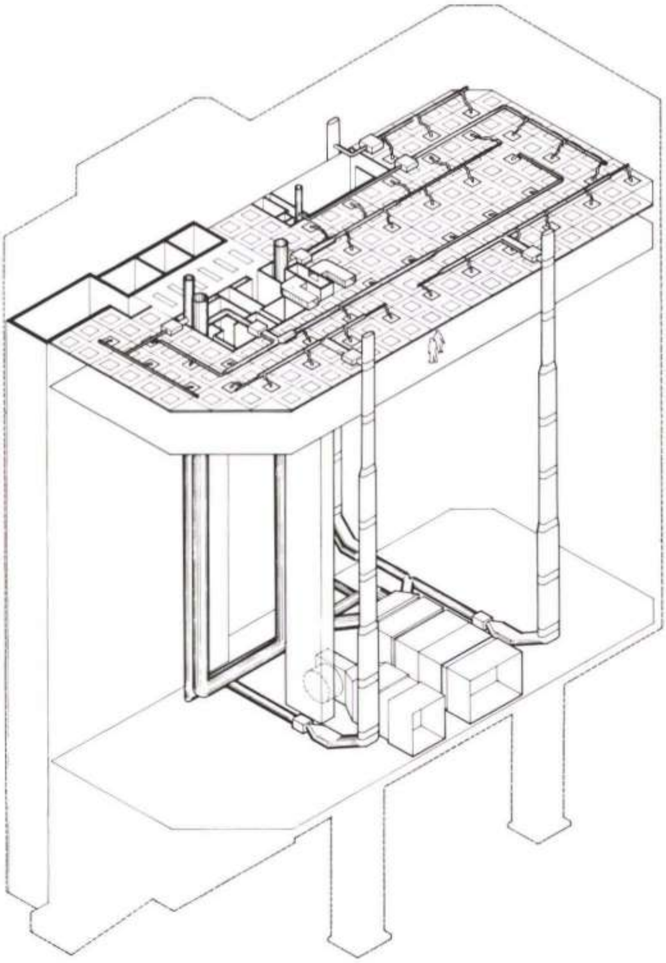


Fig. 10
The air conditioning system



The lattice is located outside the building envelope. This leaves the office space uninterrupted by any part of the structure and avoids a conflict between the diagonal members and the curtain wall.

Fire protection of the lattice depends on water-cooling the tubular frame. This method is particularly suited to a structure with few horizontal members and provides the required fire resistance without having to increase the apparent dimensions of members with externally applied protection.

The first proposal was to make the lattice in carbon steel protected by a durable paint system. The building would then have been classed as a special structure with the steel frame subject to periodic inspection by the District Surveyor. The client considered that this condition would adversely affect his investment, and also specified that all external maintenance should be limited to washing only. Cor-ten and also various ways of cladding the frame were discarded, mainly because of the weathering difficulties and, following instructions to investigate the use of stainless steel, the latter was ultimately adopted.

An important stage in the design was the erection of full-scale prototypes. One was of a typical lift lobby and the other comprised a single, fully-fitted office unit and two storeys of curtain wall and lattice. This took place in October 1972 and provided useful information about some of the problems of fabricating the lattice and about the behaviour of various stainless steel finishes in a London atmosphere.

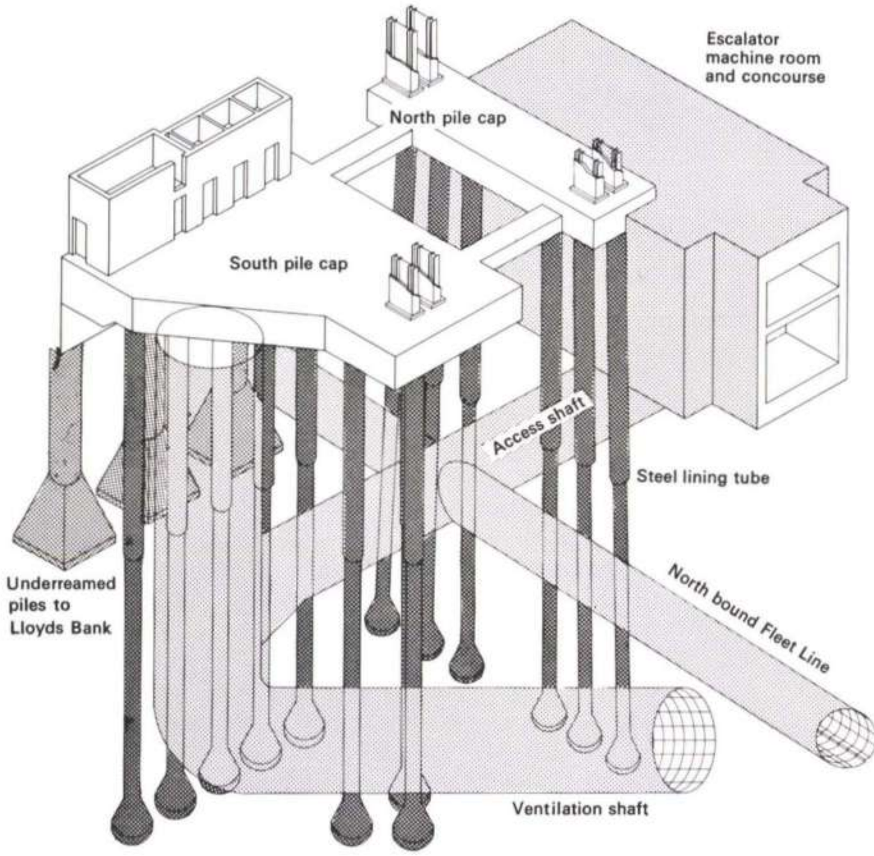


Fig. 11 left
The foundations

The foundations[1]

The design essentially was one of obtaining the greatest load capacity from a limited area of ground without surcharging any part of the future workings, without endangering the large underreamed piles below the Lloyds building and involved assessing and allowing for the ground movements associated with tunnelling. An impression of the restrictions imposed by London Transport and by the Lloyds building is given in Fig. 11.

Two sizes of deep bored piles were adopted: Eight piles of 1.25 m shaft diameter, with 3 m underreams for working loads of up to 940 tonnes under the south pile cap, and six piles of 1.07 m shafts and 2.5 m underreams for 700 tonnes under the north pile cap. The piles, 37 m long, terminate in London clay at an elevation of 27 m OD, having the upper 18 m encased in bitumen-coated steel lining tubes to the level of the workings. The linings serve to avoid down drag forces on the piles as the ground settles with the tunnelling work.

Clearance was a major consideration in both the design and installation, and to obtain a satisfactory space between underreams, the piles under the north pile cap are raked at 1 in 18. These piles are also specially reinforced to accommodate the lateral earth pressures to be expected with the very close excavations to the concourse and machine room. Three of the piles contain inclinometer tubes which will be used to monitor lateral displacements resulting from the tunnelling work.

Installation of the piles relied on laser plumbing techniques to maintain the required order of accuracy. The borings were inspected and filled within 12 hours of forming the underreams, but in the case of the piles close to the Lloyds building the period was reduced to eight hours after boring below the existing underreams.

The floors

The floors are of composite steel and concrete construction and span 18 m with an overall depth of 700 mm. Floor beams supporting concrete planks coincide with the planning grid at 1.6 m centres, with alternate members projecting through the curtain wall, loading the lattice at its primary node points. The steel is fire protected by a ceramic fibre spray.

The lattice frames

The lattice is based on a series of prefabricated frames made up of stainless steel tubes welded to cast nodes. The frames are of two kinds; the columns which weigh up to 13 tonnes and measure 9.8 m high (one third overall) by 4.8 m wide (three grids), and the lattices of up to 4 tonnes and 14.4 m (one half the overall) by 3.2 m (two grids). The cooling water is contained vertically within each frame width, and except at the top chord level, the side connections are sealed. This allows the majority of site joints to be bolted with high strength friction grip bolts at the primary node connections. Site welds occur only at the levels where the frames fit on top of each other, and at the side-to-side connections along the top chord (Fig. 12).

Fig. 12 below
A column being erected

1. Mitchell, J. M., and Treharne, G., 'Pile Foundations Designed for Nearby Tunnelling'. Paper presented at the European Conference in Vienna in April 1976 on Soil Mechanics and Foundation Engineering.

Lattice loading

Floor loads are transmitted to the main columns primarily through direct axial forces in the east and west lattice members with a small reliance on the development of shear and bending moments in certain areas. At the top and bottom of the lattice, the corner geometry results in the chords shedding a proportion of their axial loads into the roof and first floor.

The floor beams tend to generate significant out-of-plane bending in the lattice members.

This effect is limited by applying controlled upward jacking force to the centre of each floor beam prior to casting the concrete topping. The force is released after the maturing of the concrete leaving a residual hogging moment at the connection.

Lateral stability depends on the floors distributing loads to the lift core and lattice. In the long direction, the core is comparatively stiff and assumes most of the load. But across the building it is more evenly shared with the end frames. These transfer the horizontal load to the bottom level and thence into the plant room structure through the corner links. The plantroom structure incorporates internal cross frames which act with the main columns to form portals leading the forces to the foundations.

The exposure of the lattice results in cyclic temperature differences developing with respect to the rest of the building. To a large extent the design allows free thermal movement within the structure. Vertically, there is restraint only near the lift core and an expansion in the lattice simply raises the floors very slightly; laterally the floor-to-node connections are flexible but at the top and bottom corner, the restraints needed for stability generate local stresses in the members.

In the event of fire, the circulation of hot water inside the tubes leads eventually to a similar but more severe problem of differential temperatures. In these circumstances the stresses in certain of the corner members approach yield; and after a serious fire these members would be examined.

The effects of ground movements are mitigated by the comparatively high overall shear flexibility of the lattice. Stresses are, therefore, low.

The construction

The first phase of this construction: the piling, pile caps, core and plantrooms, began in 1972, was complete by 1973. It was only with the erection of the lattice and steelwork to the upper floors that there was a departure from conventional practice. The assembly of the large lattice frames and their close relationship with the curtan and planning grid made it imperative to observe very close tolerances.

The contractor in effect decided to erect the floor steelwork first, supported by temporary posts from the plantroom, and then to offer up and hang the lattice frames from the floors until their assembly was complete. By employing packs in the temporary posts it was possible to correct for the accumulating deflection in the plantroom and to make firm adjustments to the level. Similarly, the alignment was controlled by adjusting the diagonal bracing to the posts. The load was transferred to the lattice only when all the lattice joints and welds had been made.

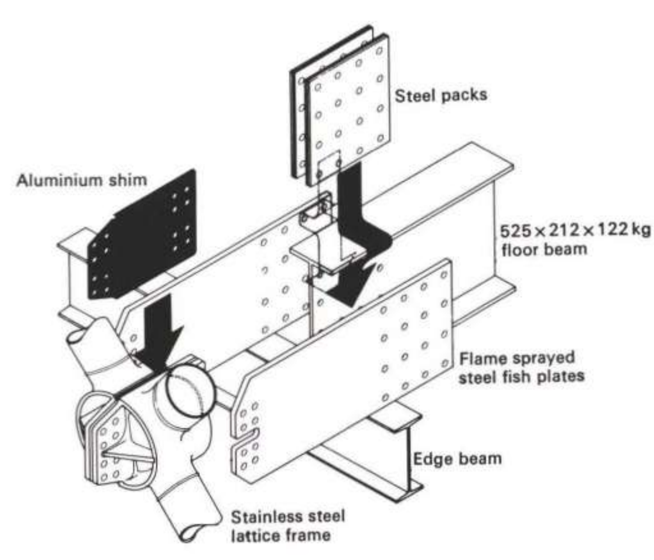


Fig. 13
Primary node assembly

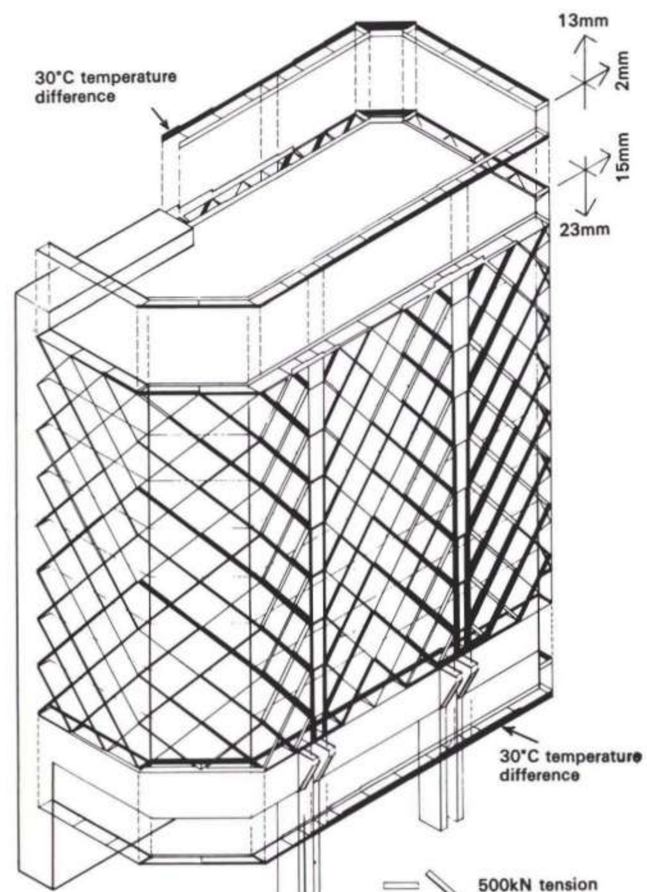


Fig. 14
Lattice forces for maximum vertical loads and for 30°C temperature difference

Analysis of the lattice frame

A standard three-dimensional frame computer program was used for analysis of the basic dead and live load condition. The model contained members to represent the lattice, the floors, floor links, core and main columns. The analysis included load cases to cover staggered patterns of live load and a temperature difference between the lattice and floors.

A series of plane frame analyses was adopted for separate considerations of wind loading, tilting of the core and differential settlement between the core and north pile cap. The effects were then combined with the previous results.

The stability of the lattice columns was checked by proving that any mode of buckling involving horizontal translations of the floor corresponds to a critical Euler load greater than the Euler load of the column, pin-ended between floors that do not move. The effective length used in the design was therefore taken as equal to the storey height.

For the condition of out-of-plane bending in the lattice an iterative programme was prepared for a desk top computer. It accounted for the influence of one floor on the others during the jacking procedure and gave bending moments in the lattice and floor beams. Fig. 14 illustrates the pattern of direct forces in the lattice for typical load cases and indicates some of the critical deformations.

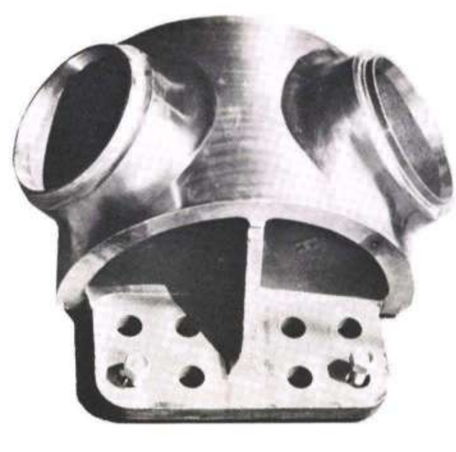


Fig. 15
A typical node

The nodes

Design of the nodes was determined by the structural and water cooling requirements. This was modified at the foundry to ensure directional solidification after casting with maximum soundness and freedom from large defects.

The behaviour of the nodes was examined by computer shell finite element analysis. Loads were applied at the tube ends to represent combinations of axial forces and bending moments with the nodes fixed at the planes of symmetry. The resultant principal stresses in each element were checked against a limit of 200 N/mm^2 at working loads and, where necessary, minor adjustments made to the thickness of steel. A typical top node is illustrated in Fig. 16 showing the pattern of elements and contour lines giving the thickness of material.

A full-scale load test on a pair of intermediate level nodes was carried out by the CEGB testing authority at Cheddar. The test was designed to simulate combinations of axial forces and bending moments factored by up to twice working load. The rig incorporated micrometer gauges to monitor deformations. It was established that at a load factor of 2.1, which represented the capacity of the jacking system, there was no evidence of yielding in any part of the castings or in the bolted connection between them.

Column 'K' joints

These joints were divided into a series of rings as shown in Fig. 14. Each ring was subjected to forces at its point of contact with the intersecting tubes. The resulting stresses adjusted on the basis of compatible deformations between the rings were again limited to 200 N/mm^2.

Tubes and welds

Axial and bending stresses at the weld positions were calculated for the various load combinations. A plot of the stress unit factors was used as a guide in the fabrication test procedure; wherever the factor exceeded 0.85, the weld was fully radiographed.

Bolted joints and fish plates

An exploded view of a typical bolted joint is shown in Fig. 13. The aluminium shim had two functions. First, by its supply in a variety of thicknesses it served on site to regulate the tolerance. Second, it gave the centre interface of the joint an acceptable slip factor for high strength friction grip bolts. Slip factor tests showed that even by coarsely maching and grit blasting the faying surfaces it was not possible to achieve factors of above about 0.3 for cast stainless steel to stainless steel connections. The insertion of an aluminium shim between the surfaces brought about a notable improvement.

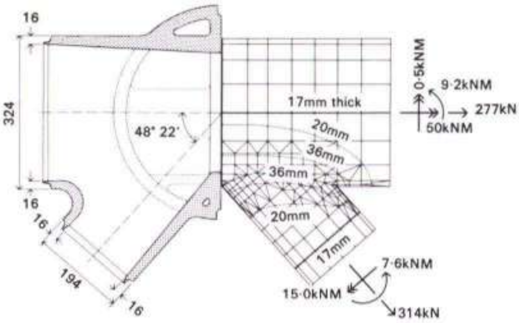


Fig. 16
Top node section and finite elements with typical applied forces

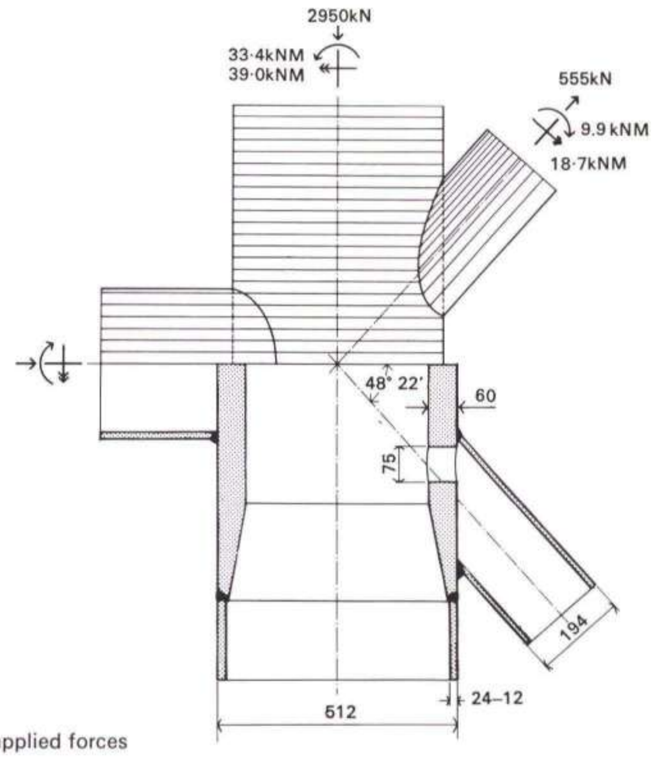


Fig. 17
Column joint with typical applied forces

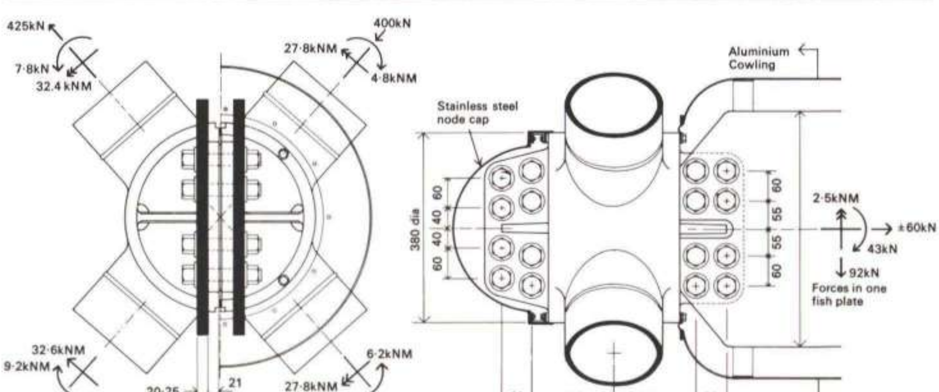


Fig. 18
Node bolt details with typical applied forces

The possibility of plastic flow or creep in thicker 6 mm aluminium shims was investigated theoretically and also experimentally on the basis of a short-term, high load, creep test and found not to be significant.

General grade carbon steel high strength friction grip bolts were used and a double line of defence against corrosion adopted. The inner and outer groups in each node are contained within a sealed aluminium cowling and a stainless steel node cap respectively; each bolt is protected by a cap of impervious polyester resin. To protect the thin top surface of the aluminium shim, the recess between the two castings is sealed with polysulphide. Sheets of vermiculite are used to fire protect the fish plates and ceramic fibre blankets used for the inner bolts. The outer bolts are protected by ceramic sprayed into the node cap.

The loads and moments acting on a typical node are shown in Fig. 18. These are resolved into shear forces at the fish plate to node interfaces and into combined shear and axial forces at the central interface between the two castings. In practice it is only at the top joints that a net tension develops across the interface and here the site weld is designed to carry the full effort. The stability of the fish plates was checked under the most severe combinations of load and lateral movement.

STAINLESS STEEL[2]

The material

The requirement was for a material of strength comparable to structural steel, good corrosion characteristics and suitable for the manufacture of nodes of 21 different types and of three different diameters of tube of variable thicknesses.

The choice effectively was between a high grade casting alloy or a high grade wrought alloy, type 316. Both materials meet the strength and corrosive requirements; both are readily weldable, and the costs were comparable. The advantage of the cast material lay in the facility of spinning tubes of varying wall thickness for a given outside diameter, and of casting nodes which met the strength and water-cooling reauirements and formed the basis of a simple fabrication. A disadvantage of the 316 alloy is its high coefficient of thermal expansion which would have aggravated the problem of differential thermal movement.

The casting alloy adopted known as Paralloy 3FL is a proprietary to APV Paramount Ltd. It has a duplex austenitic ferritic crystalline structure with the analysis of 22% chromium, 8% nickel, 3% molybdenum, and the physical properties shown in Fig. 19. It is suitable both for casting and welding, and although it was in fact developed specifically for Bush Lane House, it is very close in composition to alloys used in the process industry. Prior to its use on Bush Lane House an extensive test programme was conducted on cast and welded samples to verify its suitability.

Tube production

The centrifugally cast steel tubes are of three sizes: 194, 324 and 512 mm outside diameter, with the wall thickness varying from a minimum 9.5 mm in the small tubes to 64 mm in the columns at the joint positions. The casting process is simple: a measured weight of liquid steel at about 1600°C is poured through spouts into the ends of a rotating steel mould lined with refractory material. The tube is withdrawn while hot and allowed to cool in still air.

Node production

The castings made by the sand casting process vary in weight from 36 kg to 295 kg. Timber patterns are made for the inside and outside shapes with allowance for the effects of shrinkage and from these the single-use moulds and cores are formed by packing sand with a bonding agent against them. The sand moulds are oven dried and then placed into steel mould boxes ready for the pouring of the metal.

After casting, dross and casting heads were removed. They were heat treated, pickled, scurfed to obtain the required surface finish and then machined in preparation for fabrication.

Testing included an analysis of each pour, extensive dye penetration checks and radiography of all weld preparations. All repair welds and selected nodes were fully radiographed.

Lattice fabrication

The structure consists of 53 prefabricated frames: nine columns of three types and 44 lattice frames of 24 types. With the exceptions of the corner and end units the geometry is constant and the differences are largely due to variations in wall thickness.

Typical weld preparations are illustrated in Fig. 20. The spigot and socket detail eliminated the need for a backing run or gas purging and aided location of the pieces during assembly.

Fig. 19
Comparison of mechanical properties of cast and wrought corrosion-resisting steels (Typical values)

Property	*BS4360:1972* Grade 50C	*Paralloy 3FL* (cast) 25/5/2 Cr/Ni/Mo		*AISI 316* (wrought) 18/10/3 Cr/Ni/Mo	
		(Static)	(Tube)	(Standard)	(High proof)
Yield stress (N/mm²)	355 (minimum)	–	–	–	–
0.2% proof (N/mm²)	–	383	400	170	316
0.5% proof (N/mm²)	–	460	475	–	–
Ultimate tensile strength (N/mm²)	490–620	676	725	600	618
Elongation 5.65 / So (%)	20 (minimum)	30	35	40	35
Charpy V-notch (J) – 20°C	41* (minimum)	30	45	–	40
Modulus of elasticity (kN/mm²)	207	196	196	193	195
Coefficient of linear expansion 10 –6/°C	11	11.3	11.3	18	18

* For wrought plates.

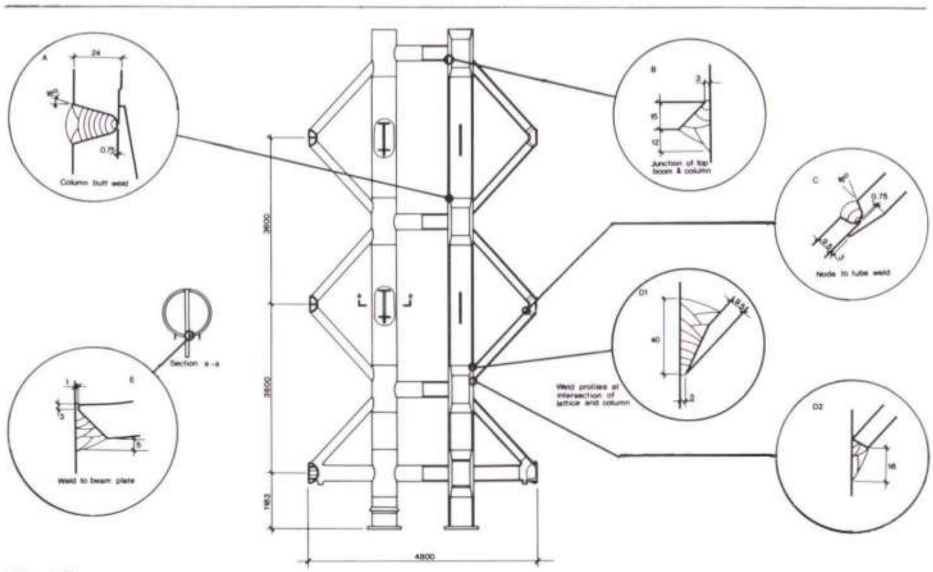
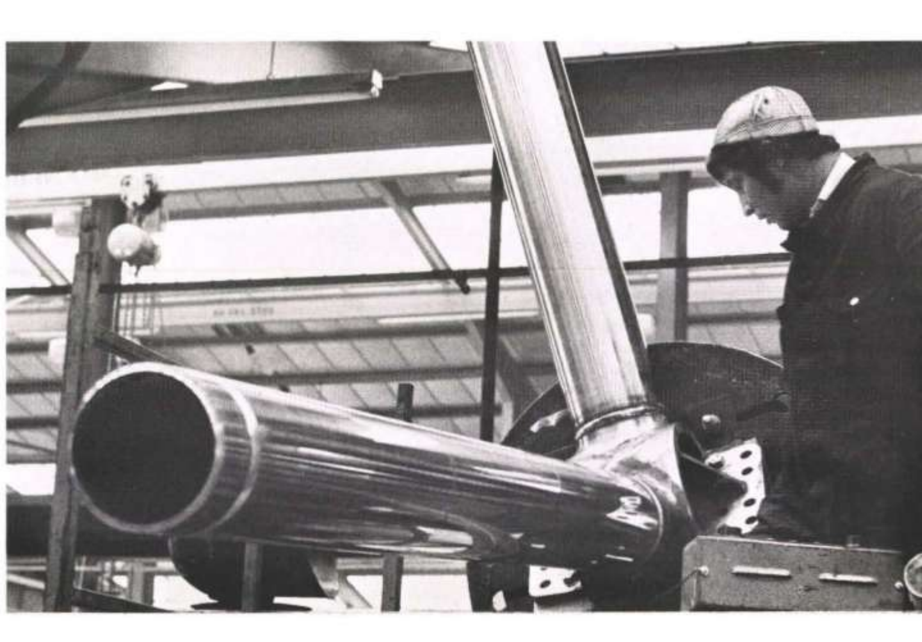


Fig. 20
Column frame and weld details

Tests established that the penetration of the welding arc was such as to fuse the 0.7 mm step and provide a full penetration butt weld equal to the inner tube. Manual metal arc welding was used with the majority of the welds executed downhand by rotating the pieces and by jigs designed to rotate through 180°.

Fig. 21
Fabrication of sub assembly



[2]. Denney, A. K., and Shuttlewood, J. R., 'Welded Stainless Steel Castings – A unique Building Application'. Paper presented at International Conference on Welding of Castings, Bradford, September 1976

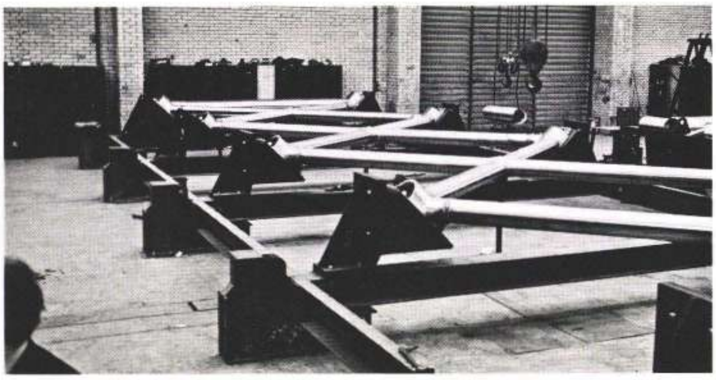
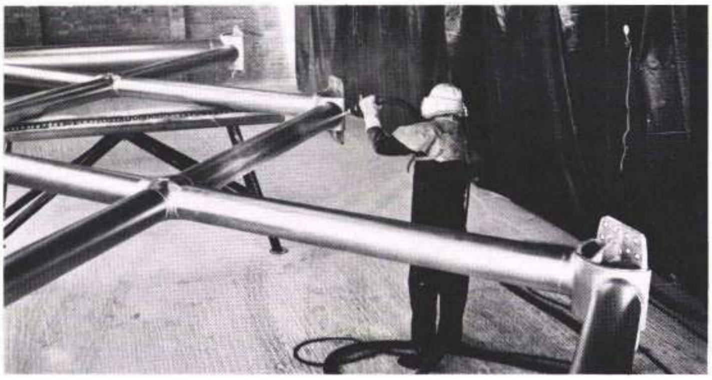


Fig. 22
Lattice frame in fabrication jig

Fig. 23
Glass bead blasting



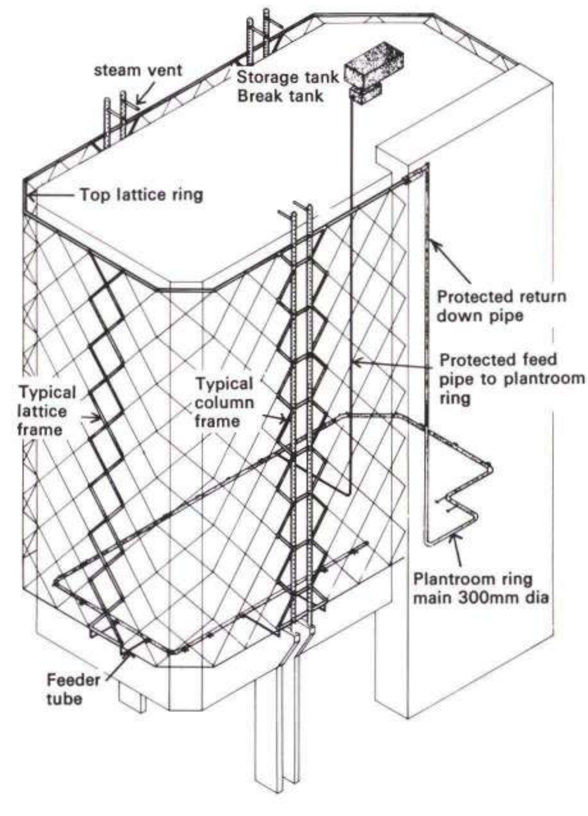


Fig. 24
The water cooling system

Fabrication took place in two stages with sub-assemblies welded into the complete frames. Tolerances of about ± 2 mm in a complete frame were achieved by studying and allowing for weld shrinkage in each joint and by holding the pieces in heavy steel jigs during assembly. The specification called for a 10% radiographic check on the welds generally, together with a 100% check on those designed as highly stressed. No unacceptable defects were found. Each frame was subjected to a water pressure test of 0.6 N/mm^2 before dispatch from the works.

The final stage was to blast the panels using glass beads. Tests showed this provided the required appearance and was the most satisfactory in terms of surface corrosion. The panels were then protected with a temporary plastic coating against surface damage during the contract.

WATER COOLING[3]

The principles

Structural steelwork is normally prevented from losing its strength in fires by applied insulation, so that its temperature does not exceed a critical value of 550°C. In water-cooled systems, the aim is to remove the heat reaching the steel sufficiently rapidly that its temperature does not rise too high. Removal of the heat is achieved by raising the water temperature (specific heat) but more effectively by evaporating it (latent heat). Whilst 1 kg of water absorbs 314 kJ for a 75°C temperature rise, it absorbs 2150 kJ in evaporating.

Heat is readily conducted from the steel to the cooling water and it has now been shown by calculation and by tests that as long as water remains in contact with the steel throughout the fire, its temperature will not exceed critical values, except under very unusual circumstances (e.g. exceptionally thick steel). The problem in designing a water-cooled system thus becomes one of making use of the latent heat of evaporation of water whilst ensuring that water is always in contact with the steel that is exposed to fire.

The essential elements in a water cooling system are:

(1) A means of separating steam from water and venting it safely to atmosphere

(2) A means of maintaining the head of water in the system, making good evaporation losses

(3) A means of ensuring adequate flow of water to all parts of the system, avoiding unintentional steam traps.

Subsidiary considerations include the provision of suitable facilities for filling and draining down, treatment of the water to prevent freezing and corrosion and provisions for maintenance.

In the majority of completed buildings of this type the technique has been to interconnect the structural members with pipework so that in the event of fire the water circulates and steam is separated in a tank on the roof. This also serves as a reservoir to replenish and keep the system full of water, thereby maintaining a large volume of water for cooling. Bush Lane House is of this general type.

Assessment of the fire resistance of structural elements normally follows established furnace test procedures. For water-cooled structures, this is more difficult as the system involves a large part of the building. There has consequently been an effort in the last 10 years to develop a theoretical, as opposed to a test, basis for determining the performance of a system.

The design

The general procedure adopted is summarized as follows:

(1) Establish the period of fire resistance
This was governed by the GLC By-Law and is taken as one hour in the lattice.

(2) Determine the maximum fire temperature
This was obtained from the standard time temperature curve and the temperature of 930°C corresponding to the end of the one hour period was adopted.

(3) Establish the extent of the fire
The performance is generally related to this since in a circulating system all the water may be involved and the heat from a fire on several floors must be absorbed by the same water as that from a fire on a single floor. In Bush Lane House each floor has a one hour compartment rating, nonetheless a fire extending over two floors was considered.

(4) Check the maximum steel temperature
This is normally for a fire low down in the building where the thickness of steel is greatest and the boiling point of water highest. The calculation depends on heat transfer theory with the fire temperature, flame temperature and duration of the fire as the main variables. The maximum steel temperature near the foot of the columns was found to be 350°C.

(5) Calculate the water storage for replenishment

Determine the time taken to heat the water to boiling and if this is longer than the fire resistance period no additional storage is required. For a one floor fire the time to boil the water is 1$\frac{1}{4}$ hours and for a two floor fire, $\frac{1}{2}$ hour. A fire on the top floor is critical because as soon as evaporation from boiling the water exhausts the reservoir, the water drops below the level of the top chord, leaving it unprotected. A reservoir of 6 m^3 provides the water for a two floor fire for a period of 1$\frac{1}{4}$ hours.

(6) Check that conditions exist for full water flow

Containing the water in individual frames of the lattice ensures an ordered circulation and avoids freak patterns where some

[3]. Based on information supplied by T. O'Brien.

members might fill with steam. To avoid the by-passing of any part of the top member it is made as an incomplete ring. It connects all the tops of lattice and column frames and at one end is jointed to a down pipe located in the protected space behind the lift core. The down pipe feeds into a ring main at the plantroom level which in turn feeds the lattice frame. Loss of water is made good from the header tank which feeds cold water down in a protected shaft and into the plantroom ring main (Fig. 24).

In the first stages of a fire, local heating occurs and the water begins to circulate on a local basis between heated and unheated parts of the lattice while steam forms and condenses within the system. Ultimately, as the fire spreads across an entire floor the circulation changes to an upward flow in the lattice, horizontal along the top chord and downwards only in the protected downpipe. Steam separates as the steam/water mixture flows through the top of the main columns from which vents disperse the steam over the roof.

The chemical solution

Although the system has been referred to as a water cooling system, the water used is treated to prevent freezing and to inhibit corrosion in the pipework. The chemicals used to deal with these problems are potassium carbonate to lower the freezing point and potassium nitrite to inhibit corrosion. 25% potassium carbonate depresses the freezing point to about $-12°C$. 0.85% potassium nitrite in conjunction with the carbonate at a pH of about 12 is an effective inhibitor. Joints between stainless and mild steel parts are protected by rubber gaskets. To limit the concentration of trace elements in the water, particularly chlorides in stainless steel tubes, demineralized water was used rather than mains water.

Testing and filling

Prior to filling with the solution, the system was pressure tested to a maximum of 0.45 N/mm^2 at the top, using water containing a fluorescent dye. Each joint was inspected and no leaks were found in the lattice.

The procedure should be one which leaves a uniform concentration of chemicals. Approximately 100,000 litres of solution were required and this was delivered premixed to the site in six tanker loads. The solution was pumped into the system through the header roof tanks and artificially circulated by pumps connected into the plantroom ring main.

Thermal expansion

The water level fluctuates under normal cyclic temperature variations and this variation is taken up in the space at the top of the columns and in the break tank on the roof. It was not considered practical to accommodate the volumetric expansion of solution that occurs under fire conditions. The system overflows on to the roof and this has been allowed for in the estimates of water volumes required. Thermal movement of non-structural pipework has been catered for by the provision of movement joints.

Maintenance

Water cooling systems are designed to be low maintenance installations. However, at this stage in their development they cannot be considered to require no maintenance at all. Apart from the obvious monitoring for leaks, the main item is to check the concentration of chemicals in the solution. Whilst potassium carbonate at the concentrations used is nowhere near the solubility limit, inhibitor concentrations can change particularly under bacterial action.

Maintenance will be carried out annually. The solution will be circulated by means of the plant room pumps and samples taken from the tops of the columns.

LOBBY PRESSURIZATION

Section 20 of the London Building Acts applies to buildings over a particular height and/or cube. It provides for more than usually stringent control over the building design related to fire and, in particular, fire protection, means of escape and the needs of the London Fire Brigade. These requirements are set out in a Code of Practice, clauses of which are then made a condition of by-law consent in each case.

In office buildings over 30 m high, to give firemen a chance of recovering from smoke or lack of oxygen, one such requirement is for the lift lobby to have access directly to fresh air, either by being planned externally or alternatively by introducing an air shaft (25% of the lobby floor area) within the building. Also external walls are required to have generally distributed, manually controlled openings to allow the dispersal of smoke with the overall area of opening representing at lease $2\frac{1}{2}\%$ of the floor area.

To overcome the reduction in lettable floor area which would have resulted from complying with these requirements we proposed to the GLC a system for mechanically ventilating the lift lobbies. Essentially the design had to ensure that in the event of a fire all lift lobbies would remain free of smoke by supplying air to achieve sufficient pressure to prevent leakage of smoke from the office areas through the fire check doors. The main features of the system are:

(1) A supply and distribution system able to provide the quantity of air required to maintain the required pressure. This was defined by the GLC as a 5 mm water gauge difference across the lobby doors.

(2) A triggering system wired to the manual and automatic smoke detector fire alarm system (which also stops the conditioned air supply fan) with manual override controls on the fire control panel in the ground floor entrance lobby

(3) Automatically controlled openings in the office external wall to allow air pressure release in order to maintain the pressure differential at the lobby doors

(4) Stand-by electrical generation with sufficient output to ensure the system against mains failure

(5) Mechanically operated smoke extract ventilation, manually switched from the fire control panel.

An extensive report setting out the theoretical design, our detailed proposals and backed up by precedent (mainly in Australia and San Francisco) was submitted to the GLC. In this we were greatly helped by Gordon Butcher who, at the Fire Research Station, had already done considerable work on lobby pressurization. This was approved, subject to a large number of qualifications, one of the most significant being the demand to add a sprinkler system. The London Fire Brigade, throughout our discussions on lobby pressurization, had shown real concern over the unpredictable nature of smoke under the intensively hot conditions which could occur from a fire fed by office equipment and furniture and this demand was made to help reduce the likely temperature.

STAND-BY GENERATION

With the need for a set to provide emergency power to the lobby pressurization installation the client decided that this should be increased to 250 kVA to provide sufficient output so that the building could function under 'power-cut' conditions. (This decision was made after two bad winters and following the effects of the three-day week.)

CONCLUSION

The acceptance by London Transport of our outline design in August 1970 allowed them and Trafalgar to begin negotiating the conditions of their agreement for the redevelopment of the site. In December 1971 we were instructed to prepare scheme design and obtained full planning consent in September 1972 (after refusal of our first application).

Demolition of the existing building began in July 1972 and was held up by request of the Director of the Guildhall Museum, who wished to carry out an archaeological survey of the remains of what had once been the Palace of the Roman Governor of Britain which extended beneath our site.

Piling by Cementation Ltd. began in October 1972.

The main building contract by Trollope and Colls, which included the steelwork erection by Boulton and Paul Ltd., began in April 1973 and was completed in September 1976.

Acknowledgements

Our client played an active role throughout all stages of the building design, both through regular and formal design meetings and, as effectively, through frequent informal contact.

Trafalgar are professional clients, responsible for a number of City office buildings designed by several architects and have, therefore, acquired the expertise and experience to take a real part in the design of their buildings.

Although demanding, we have welcomed this close attention and believe that the building has gained from it.

Once it was suggested that we should consider stainless steel for the lattice a practical difficulty emerged. There was no experience in the structural steelwork industry of the use of the material and its responses to our enquiries were quite negative. We approached the process plant industry where knowledge of the material exists and met an enthusiastic response from APV Paramount Ltd. who manufacture stainless steel equipment and vessels for the dairy and brewing industries. It was left to us to develop the structural expertise but the quality and precision which APV and their subsidiaries achieved, of castings and at the fabrication stages, contributed significantly to the outcome.

For the development of a structural vocabulary for stainless steel, taking into account its different metallurgical characteristics from carbon steel, we were dependent on our own R & D and it was they who wrote the report on which the GLC accepted the use of this material as well as assisting with the analysis of the lattice design.

Similarly, fire protection by water filling the structure, although several examples existed overseas, was new in this country and a tribute to the work done by R & D must be not only that the GLC approved our design, but that they were prepared to waive the need for any tests.

Credits

Client:

City & West End Properties Ltd.

Architects, Engineers & Quantity Surveyors:
Arup Associates

Main contractor:
Trollope & Colls

Steelwork subcontractors
Fabrication and erection:
Boulton & Paul

Stainless steel casting:
APV Paramount

Stainless steel fabrication:
Burnet & Rolfe

Royal Exchange Theatre

Patrick Morreau

Introduction

The Manchester Royal Exchange, within which the Royal Exchange Theatre is now housed, was built during the First World War and is itself an extension of the Cotton Exchange built between 1866 and 1874, which in turn replaced two earlier exchanges. An air raid in 1941 destroyed the original part of the building but left the present hall largely undamaged. Although the Manchester Cotton Exchange at one time numbered 12,000 members, trading ceased on 31 December 1968 and the Great Hall stood empty.

In 1972 the '69 Theatre Company, at that time based at Manchester University Theatre, obtained a 25-year lease from the Prudential Assurance Company, owners of the Royal Exchange, and set about the formidable task of creating, within the first and mezzanine floors of the Exchange, a theatre to seat 700, complete with workshops, rehearsal rooms, offices, bars and a restaurant, along with cloakrooms, toilets and ticket offices.

Richard Negri, an artistic director of the company, was appointed to be responsible for the conceptual design of the theatre itself, and Levitt Bernstein Associates as architects for the entire project. Ove Arup and Partners were appointed structural engineers, and subsequently their brief was extended to include fire safety engineering.

The initial conception of the theatre, a development of the ideas used by the '69 Theatre Company in their earlier productions, was of a free-standing concentric auditorium within the Great Hall, creating a close and vibrant relaship between performers and audience. This intimacy was to be achieved by seating the audience around the performing area, at the level of the stage and on galleries above, so that none should be more than 10 m from the centre of the stage. Actors were to be able to enter from several points around the perimeter, scenery to be flown from above the performing area, which itself was to be adaptable to different sizes and configurations, and the traditional 'upstage', 'downstage' and proscenium dispensed with.

This concept, which has remained little altered by the vicissitudes of design development and budgetary rigour, evolved into an open stage theatre, seven-sided in plan, with stage and seating for 450 at the level of the Exchange floor, and two galleries above, each of which seats a further 150 people. Although the Exchange building itself provides protection from the wind and rain, a further enclosure is needed to provide, within the auditorium, a controlled thermal and acoustical environment; thus the theatre is clad with toughened glass and roofed with metal decking.

Structure

The structural design started, as it commonly does, with a site investigation, with the difference that in this case the site to be investigated was the structure of the existing building, and the data sought was not the bearing capacity of the soils but the strength of the existing floors, columns and foundations. In this investigation we were immeasurably helped by Mr T. Pevitt of Bradshaw Gass and Hope, the Bolton firm who had been the architects and engineers of the original building and who have, for one reason or another, been involved with the Exchange almost continuously for 60 years. They were able to provide us with copies of all the original drawings and to inform us where and how the actual construction was changed from that shown on the drawings because of materials shortages during the closing years of World War One.

This investigation revealed that the existing floor construction was not capable of supporting the two galleries or the roof of the auditorium but that limited additional load could be carried by the brick piers of the Exchange. The architectural and engineering problem, then, was to arrive at a structure which would transfer the loads of the upper levels of the auditorium to these piers without obstructing free circulation on the Exchange floor, without sacrificing the sense of the Great Hall as a vast, single volume, and without generating more load that the piers could support. The limitations set by this problem of support became, in the words of the architects, the most influential factors in determining the final form of the auditorium and assisted rather than hindered the development of the design.

The auditorium is placed directly under the central and largest of the three glazed domes which light the Great Hall, and it is the piers that carry this dome which are used to support the roof and galleries of the auditorium. These piers, of solid brick construction faced with scagliola, are approximately 3 m square and define a rectangle 30 m by 21 m, within which the auditorium is symmetrically located.

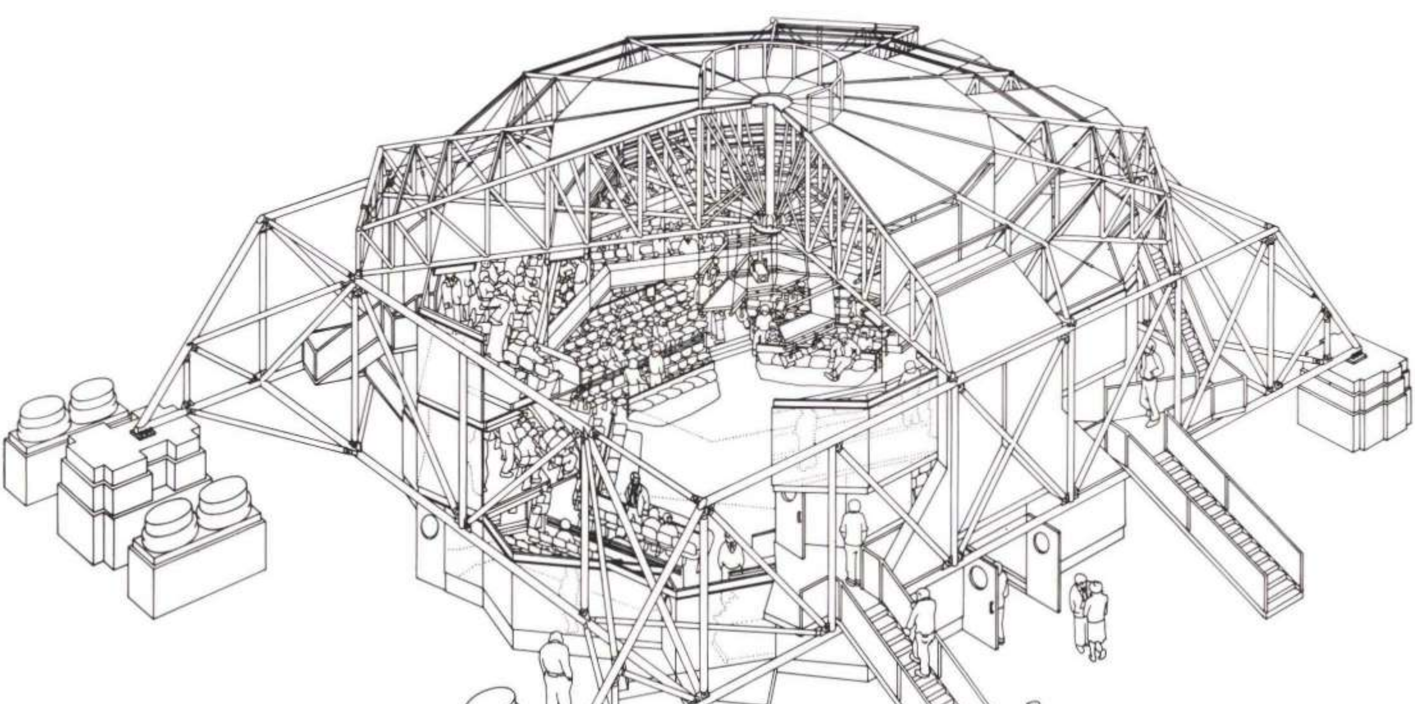
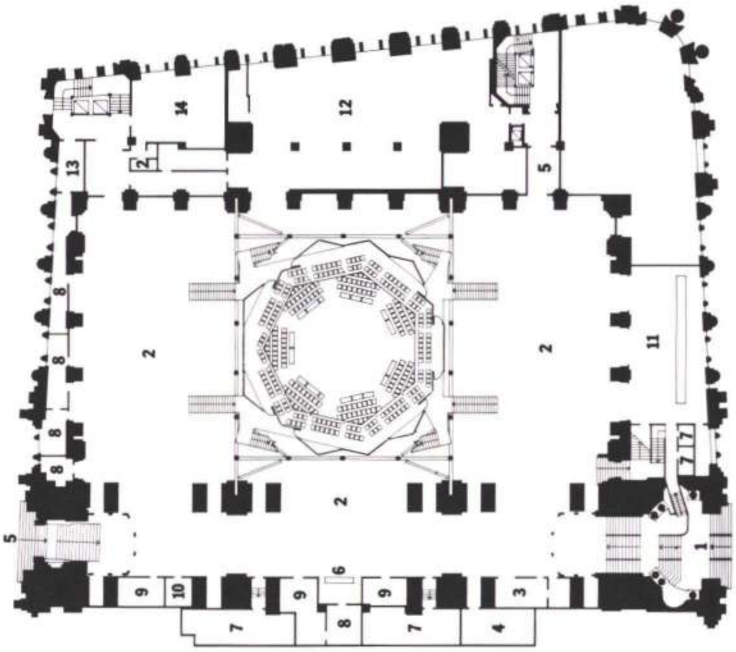


Fig. 1
Axonometric of the auditorium (Photo: Courtesy of the architects)



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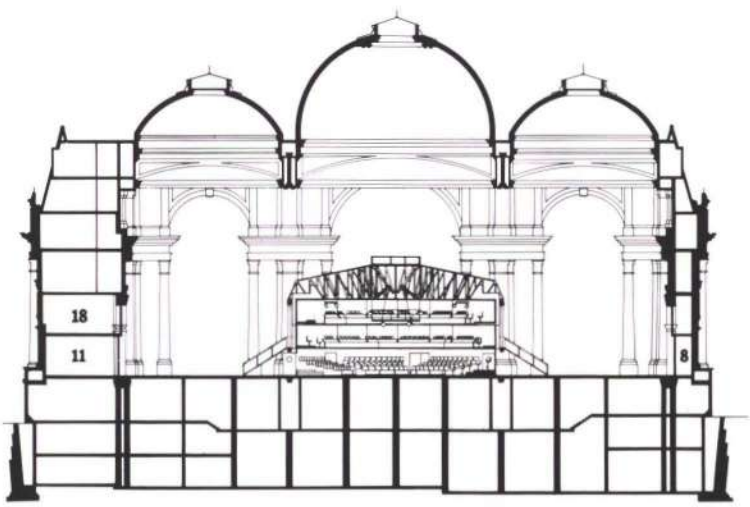


b

Figs. 2 a and b
Plan and section through the Great Hall of the Cotton Exchange showing the position of the theatre (Reproduced by courtesy of the architects)

Fig. 3
One half of the interior of the Great Hall of the Cotton Exchange.
The centre dome, beneath which the auditorium is located, is in the foreground
(Photo: Elsam, Mann & Cooper (Manchester) Ltd.)

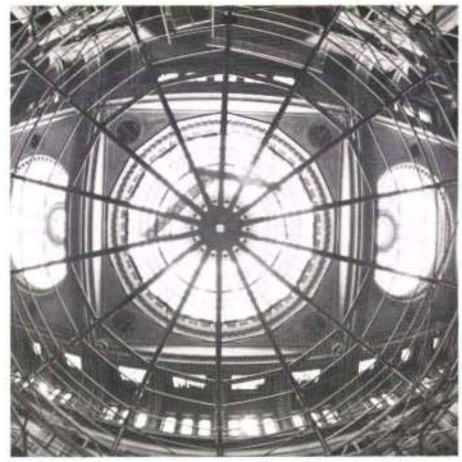


Fig. 4
The roof of the Exchange seen through the auditorium structure
(Photo: Picture Coverage Ltd.)

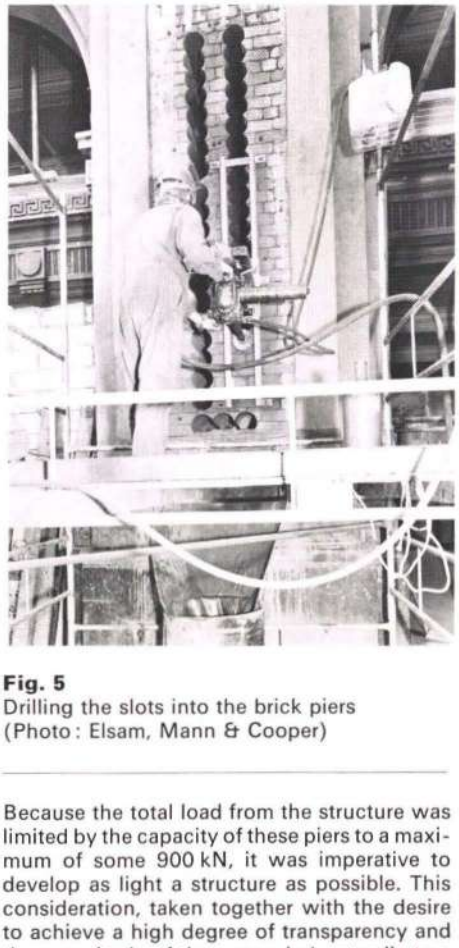


Fig. 5
Drilling the slots into the brick piers
(Photo: Elsam, Mann & Cooper)

Because the total load from the structure was limited by the capacity of these piers to a maximum of some 900 kN, it was imperative to develop as light a structure as possible. This consideration, taken together with the desire to achieve a high degree of transparency and the magnitude of the spans, led naturally to a steel structure of some form. Early, and fruitful, discussions with the City of Manchester building officials regarding fire safety led to an agreement to permit the steelwork to remain unprotected and thus avoid the cost and additional weight and bulk of fireproofing. This investigation of fire safety carried out by Ove Arup and Partners was of fundamental importance to the feasibility of the theatre and is described separately in more detail.

Following the investigation of a number of possible schemes, it was decided to adopt a system of tubular steel trusses from which the galleries would be suspended. Primary trusses 4.7 m deep span the 30 m between the piers and support secondary trusses of the same depth thereby forming, in plan, a square some 21 m × 21 m enclosing the auditorium.

The roof of the auditorium is formed by seven radial trusses 3.3 m deep at the centre and spanning up to 25.5 m, supported directly upon the top chords of the primary and secondary trusses. The roof trusses radiate symmetrically in relation to the seven-sided

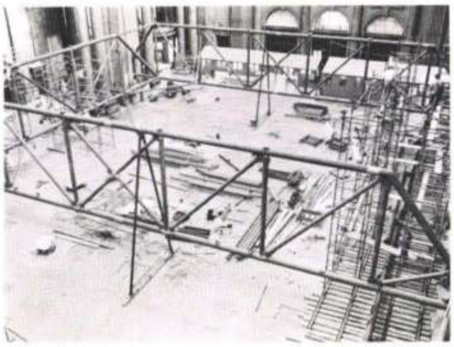


Fig. 6
Erection of the primary and secondary trusses (Photo: Elsam, Mann & Cooper)

auditorium but not, of course, in relation to the supporting square, and this geometrical anomaly gives rise to differing lengths of roof trusses. In order to maintain the symmetry of the primary and secondary trusses and to avoid large variations in the spans of the roof trusses, additional ones (which became known as the back of gallery trusses) were used at the corners of the square.

All of the trusses were fabricated from circular hollow sections, using grades 50C and 43C steel, and in diameters ranging from 270 mm for the chords of the primary and secondary trusses to 90 mm for the diagonals of the roof trusses. In order to ensure that difficulties in obtaining material did not interfere with a very tight design and construction programme, the client purchased the bulk of this material directly from British Steel Corporation as soon as the sizes and wall thicknesses had been determined from our preliminary calculations.

The roof trusses are fully welded except for a bolted connection at the centre where the seven trusses intersect. The primary and secondary trusses were fabricated with the verticals generally shop-welded to the top and bottom chords and the diagonals attached on site using single 50 mm diameter pins. They were shipped to the site in two sections which were welded using full penetration groove welds. The roof cladding is *Holorib* metal decking supported on 75 mm square tube sections.

The two seating galleries, which are suspended from the roof trusses by 25 mm diameter rods, are of concrete fill on *Holorib* deck supported on rectangular hollow sections. These were shop fabricated in segments and the junctions welded at the site. In addition to the galleries for the audience, there are two lighting galleries: one around the inside of the upper seating gallery just below the bottom chord of the roof trusses, and the other suspended above the centre of the performing area and reached by catwalks from the outer gallery.

Access to the upper levels of the auditorium is by four staircases which are supported either directly on the lower chord of the primary trusses or suspended, in rather an *ad hoc* fashion, from the roof trusses.

The support of the primary trusses on the brick piers presented two problems: to ensure that the load was imposed concentrically on the piers, and to allow sufficient freedom for the horizontal movements caused by elastic and thermal effects, while at the same time providing adequate restraint against horizontal and dynamic forces. Because of the great depth of the trusses and the size of the piers it was necessary to cut slots in the brickwork 500 mm wide, 2.75m high and 1.83 m deep, into which the ends of the trusses were inserted. In order to counteract the outward thrust resulting from local arching action above and below these slots, high strength steel rods and large round bearing plates were installed and the rods post-tensioned. The round 'washers' have since

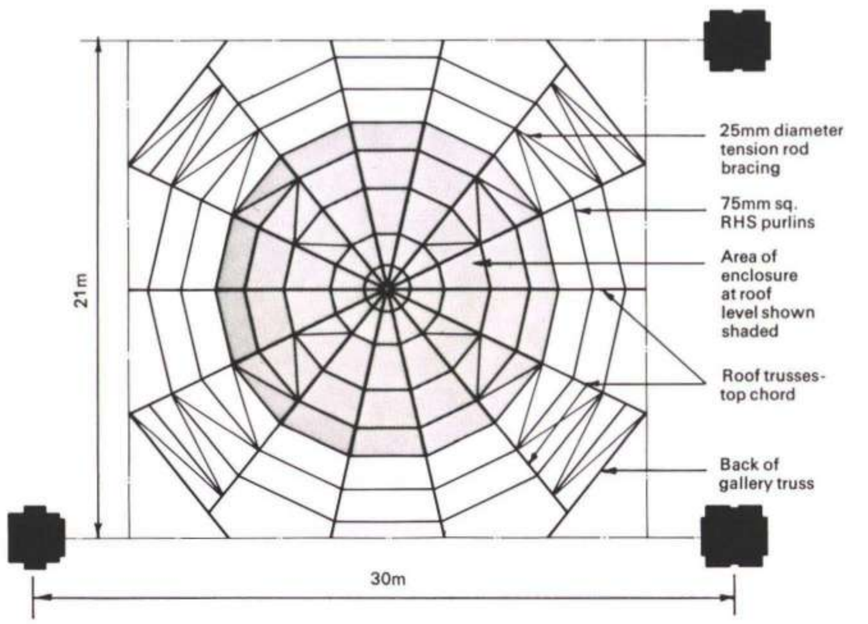


Fig. 7
Plan: roof at level of top chord

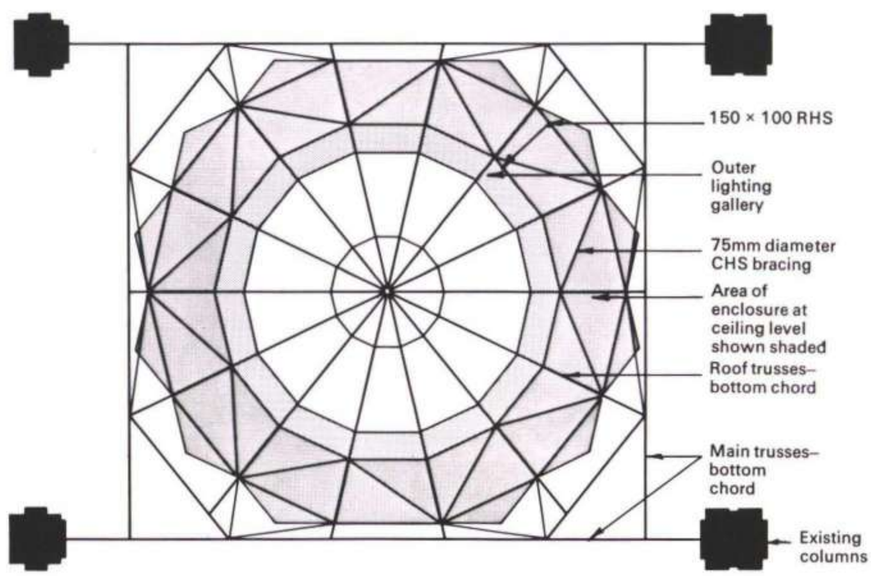


Fig. 8
Plan: ceiling at level of bottom chord

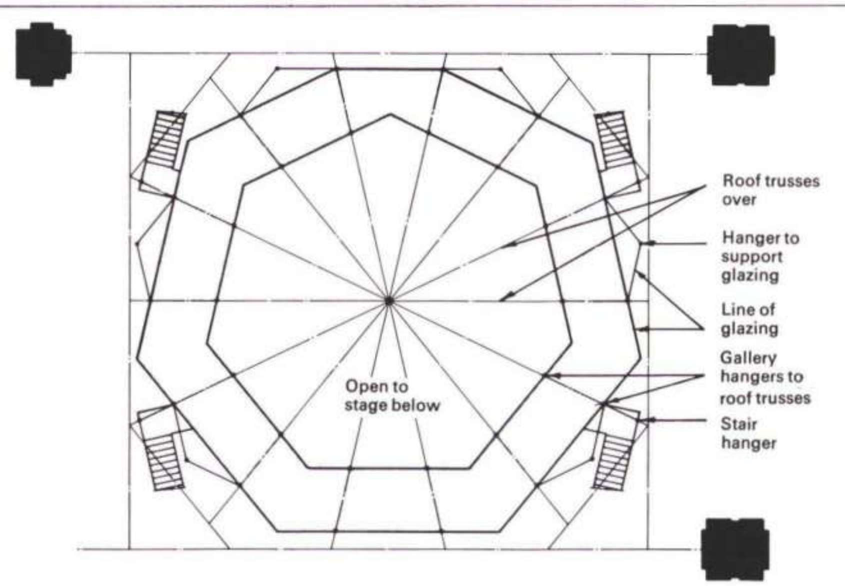


Fig. 9
Plan: upper gallery

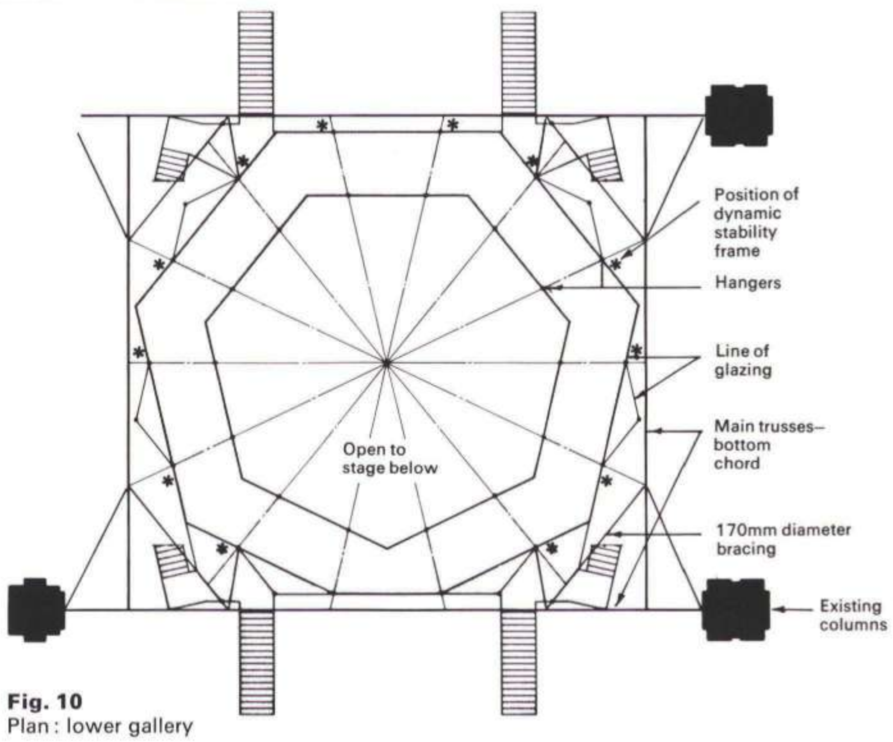


Fig. 10
Plan : lower gallery

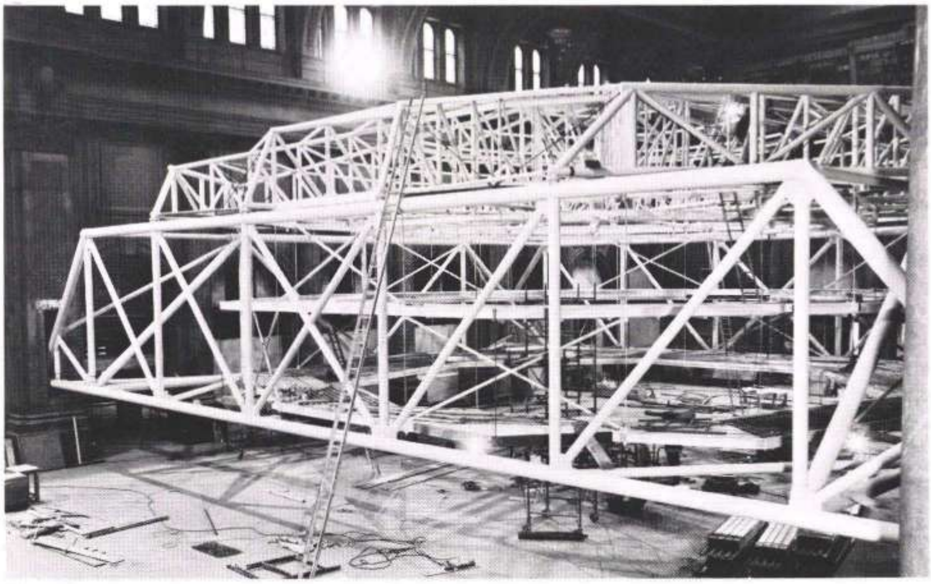
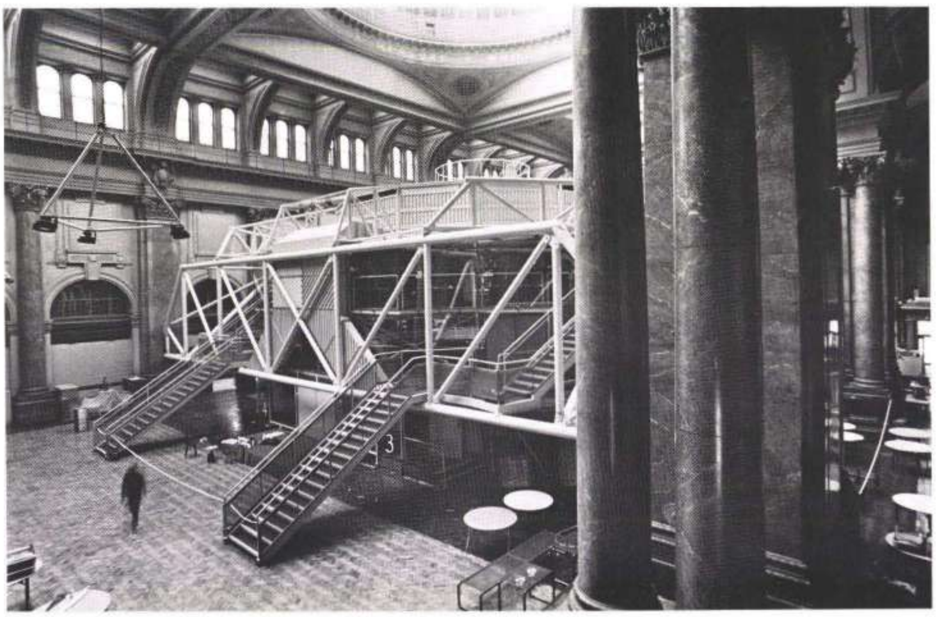


Fig. 11
The completed framework of the auditorium (Photo : Elsam, Mann & Cooper)

Fig. 12
The completed auditorium



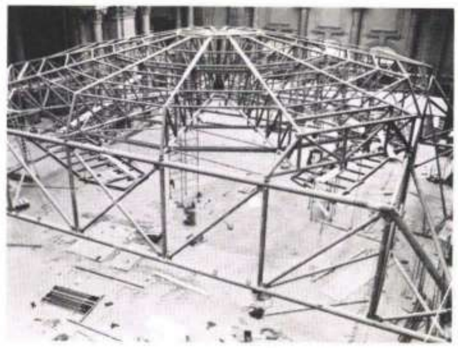


Fig. 13
The roof structure completed and the installation of the upper gallery (Photo : Elsam, Mann & Cooper)

been painted to blend with the surrounding scagliola – a whimsical touch considering the consistently bold expression of the structure elsewhere. The trusses themselves rest on CCL bearings.

The large spans, together with the type of structure adopted, give rise to quite large static deflections. Because of the comparatively low self-weight of the structure, a large proportion of these deflections is caused by superimposed loads and therefore, although the theoretical deflections can be calculated with tolerable accuracy, the actual deflections result from loading patterns which can vary widely. The situation is complicated further in the suspended structure by the way in which loadings at one level will induce deflections in other levels supported by the same trusses. The detailing of many elements has therefore taken account of a large range of movements and this has led to connections for the ventilation ducting and electrical services, main stairs and most particularly the glazing, which permit sliding and rotation to accommodate vertical displacements of up to 65 mm.

Dynamics

The dynamic performance was considered under two headings : the comfort of the audience, and the susceptibility of the total structure to damage resulting from excitation arising either from rhythmical movement of the audience – an exceptionally brisk rendering of 'Auld Lang 'Syne' – or from would-be vandals tuning themselves to the natural frequency of the structure. In assessing the effect of vibrations on comfort we took account of the unconventional nature of the structure and the fact that any vibrations of noticeable amplitude would most likely be caused by mass movement of the audience and occur for only a short time. In examining the possibility of damage to the structure, we investigated the amount of power that would be required to produce yielding at critical points, and then judged whether such power inputs were within the realm of possibility.

For vertical excitation the structure was considered as a lumped mass/lumped spring system, and for horizontal and torsional excitations as a system with seven degrees of freedom for which the individual modes of vibration were examined separately and in combination. The calculated lowest natural frequencies were : vertically 3.35 Hz ; horizontally 5.1 Hz parallel to the main trusses and 4.4 Hz perpendicular to them ; and torsionally 18 Hz. In the absence of any significant inherent damping, the vertical and horizontal natural frequencies were considered to be too low in terms both of comfort and safety, and it was decided that some form of damping should be introduced.

The main difficulty in developing a damping system proved to be devising a method of linking the auditorium structure to the Exchange floor which permitted the large elastic deflections mentioned earlier while at the same time keeping the dynamic movements to less than 0.1 mm. The system adopted employs

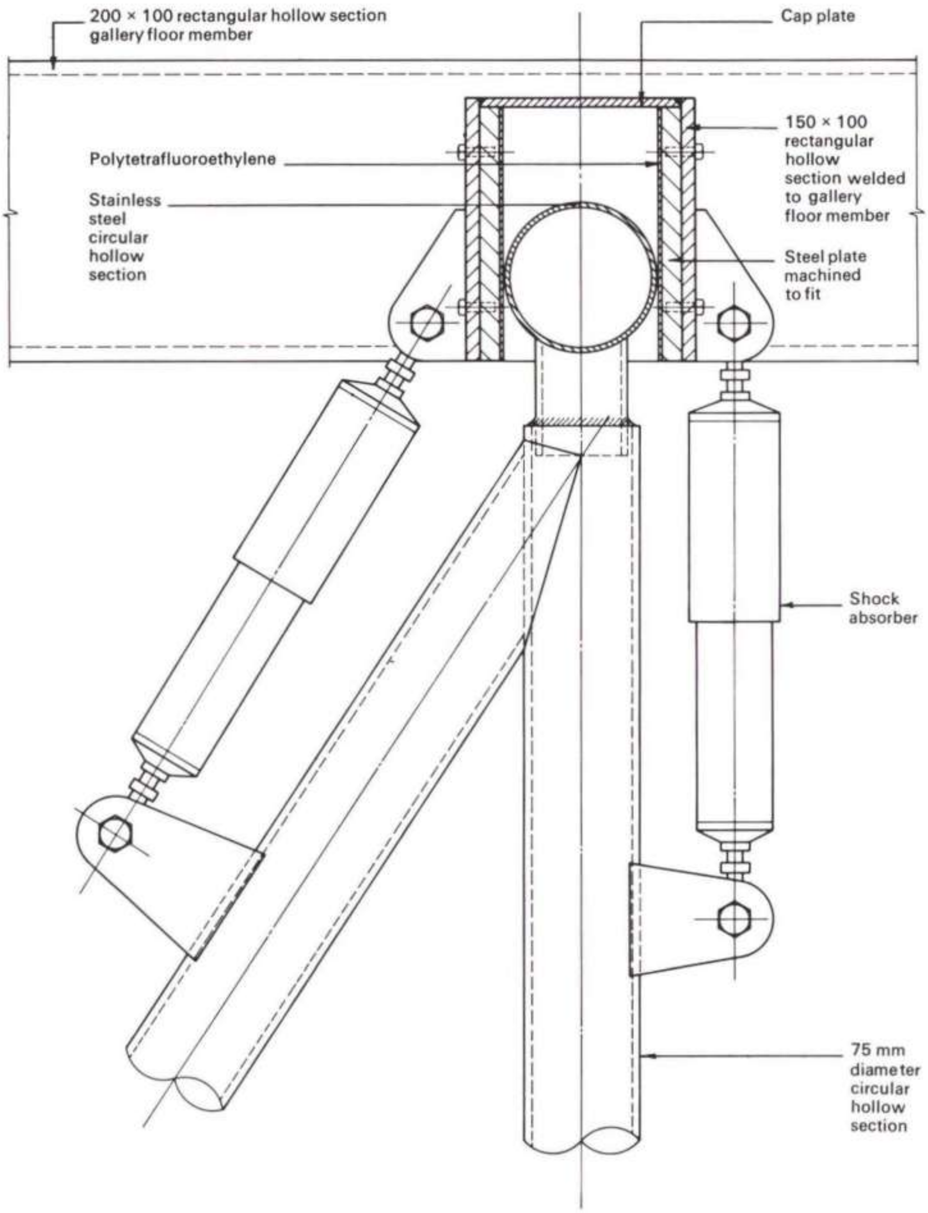


Fig. 14
Shock absorber

telescopic shock absorbers – modified slightly from those used for heavy Ford trucks – mounted on frames which are connected to the back of the lower gallery through an assembly incorporating PTFE surfaces sliding against stainless steel tube.

We have been exceptionally fortunate in having had the opportunity to check our theoretical calculations against the actual performance of the structure under dynamic tests carried out by the Building Research Establishment. The tests were conducted after the damping frames had been installed; difficulties with the equipment prevented testing both before and after the damping was introduced. A suggestion of vertical response was found between 4 and 5 Hz but the excitation system did not have sufficient power to establish a definite resonance; the damping associated with the response was thought to be about 6% of critical. In what we had regarded as the most sensitive direction, perpendicular to the main trusses, a first mode resonance was found at 3.92 Hz (compared to our calculated value of 4.4 Hz) with associated damping of between 1.4 and 1.8%. An attempt was made to excite the structure by three people trying to tune themselves to the resonant frequency, using the oscilloscope trace as a guide, but they were quite unable to develop any response. All in all, the results of the tests provided a gratifyingly close correlation with the calculations and were altogether reassuring with regard to the behaviour of the structure.

Fire safety

Fire protection received very careful consideration and because it was an unusual situation we, in co-operation with the City authorities, carried out a full fire engineering design appraisal.

Means of escape were first examined. The open structure of the theatre was considered by the fire authority to be an important safety feature because it assisted in the early detection of a fire, but it did mean that there was no effective fire barrier between the auditorium and the Great Hall. It had been suggested that the hall, on account of its great size, would constitute a safe refuge for an escaping audience, but after we had estimated the likely size of fire and the patterns of smoke movement, we recommended that the audience should be able to leave both the theatre and the hall within the escape time normally accepted for theatres. Accordingly crowd movement was carefully analyzed and a generous number of exits provided.

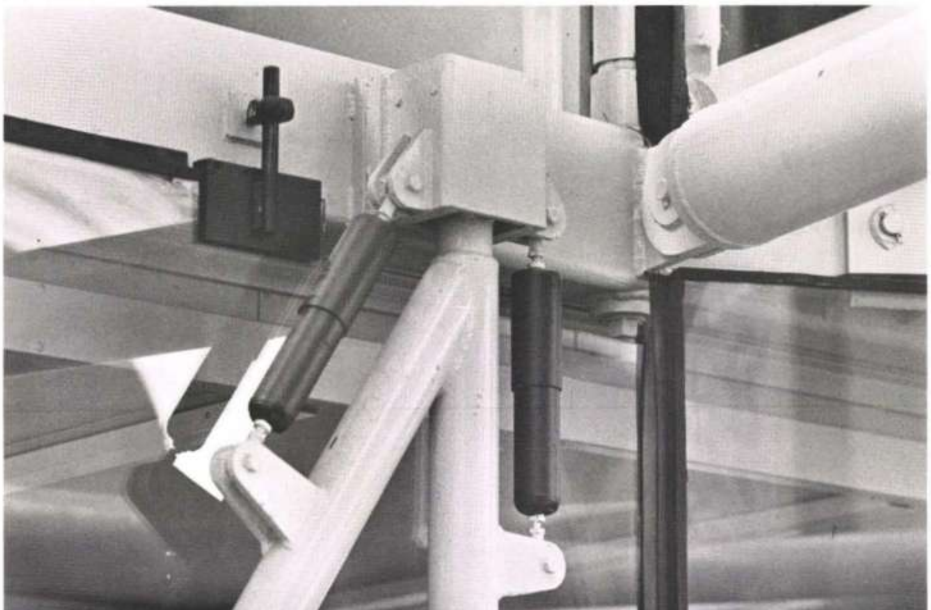


Fig. 15
Dynamic stability frame head detail
and sliding/rotating connection of glazing

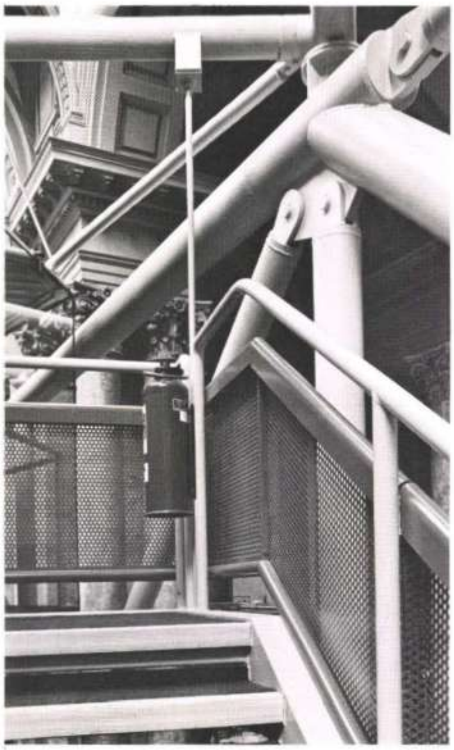


Fig. 16 right
Stair landing at upper gallery

17

18

19

Fig. 17
Connection detail
Fig. 18
Louvres around the top can be adjusted to control the reverberation time within the auditorium
Fig. 19
Ventilation ducts at the auditorium roof. Air is distributed around the perimeter of the roof and through the 'trouser leg' ducts (see Fig. 12) to the interior of the auditorium (Photo: Elsam, Mann & Cooper)
Fig. 20
Pulleys for flying scenery
Fig. 21
Interior of auditorium (Photo: Picture Coverage Ltd.)

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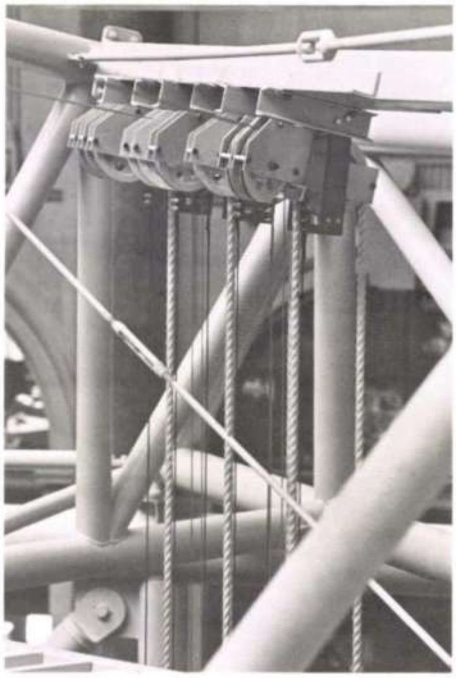

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The structural aspects of fire safety were then examined. Conventional fire protection of the structure would have been both too costly and too bulky, as well as bringing the total weight to a level beyond the capacity of the existing structure. The inescapable conclusion was that the steel had to remain unprotected and we estimated that, if a fire were to continue unchecked after evacuation, the floor of the Exchange could survive the collapse of the structure and there would be no extra hazard to the fire fighters.

The third part of our fire safety appraisal dealt with the actual risk of a fire taking place, and in order to reduce this risk to a minimum, non-combustible or low flammability materials have been used throughout. In addition, arrangements have been made to ensure early detection of a fire and for continuous surveillance by the staff whenever the public is present.

These were the grounds on which a waiver was granted of the building regulations requirements for the fire resistance of the structure. A subsequent full scale trial, with an invited audience, of the means of escape confirmed our calculations of the time required to empty the building and, it is hoped, will assist in the application of similar techniques of rational fire safety analysis to future projects.

GRP for cladding: criteria for selection

Turlogh O'Brien
Tony Read

Introduction

Developments in technology have progressively led towards a wider range of materials being available with which to design any building component. It is only necessary to look at the external walls of modern buildings to see the range of materials that can now be used for cladding. Apart from satisfying architect's desires for visual diversity, these different materials do not usually indicate that any different performance is required from the fabric of the buildings. If the requirements are similar, then the various materials may be considered to be alternatives to each other. What criteria can therefore be used to make a selection?

The following notes have been written largely from the viewpoint of selecting materials for cladding. It is in this market that glassfibre reinforced plastics (GRP) receives the greatest amount of competition and for which architects require evidence of cost effectiveness in relation to performance. GRP requires a strong *raison d'être* in such a situation where low cost/good appearance solutions are required. It does not take much study to find that the foremost reason why architects choose GRP for building components is a visual one. It is either the ability to shape the material to some special form or the surface colouring and texturing possibilities. The characteristics of lightweight and better durability (compared with painted steel or timber) are utilized by component manufacturers in the marketing of standard items. However, mouldability and aesthetic versatility stand out as the principal advantages of custom-moulded GRP when compared with alternative materials for the same situation.

There may be situations where GRP is chosen for its mouldability and lightweight characteristics as the *only* material suitable for achieving a particular architectural concept (perhaps a lightweight dome or complex roof panel) but in so doing a very 'uneconomic' design may have to be chosen to cope with the long term mechanical performance in conditions of creep or fatigue.

No material is perfect for every use, and the advantages must be balanced against the disadvantages.

The most obvious disadvantages of GRP are its lack of stiffness, its combustibility and the fact that it needs a high degree of manual skill in the production of components. Whilst the first of these is a limitation that can usually be 'designed out', and the last is within the capability of the fabricators to deal with, the second limitation is more intractable. There is no way a combustible material can be made incombustible. Although the technological inventiveness of the resin and GRP industries has produced materials with better fire characteristics, the basic problem remains. Clearly it is so fundamental that it must be taken as a matter to be considered very early in the materials selection process.

SELECTION CRITERIA: GENERAL

In order to try and put some logic into the criteria for selecting GRP for a building component, the attached flow chart has been prepared. Obviously there is no unique way in which such a chart could be compiled and different people will have their own hierarchy of critical decisions. However, this chart enables one to review the essential questions that must be answered before work should proceed.

Behaviour in fire

The all important question of behaviour in fire is put first as there is little point in continuing design studies if a satisfactory answer cannot be given. In situations where the regulations require a non-combustible material, GRP cannot be used. In other cases a limitation on the flame spread across surfaces to inhibit fire development, fire spread up the outside of buildings and hazard to adjacent buildings by radiation from large flames, will be required.

It is now possible to design GRP components using carefully formulated resins to achieve both Class 0 as defined in the Building Regulations for fire propagation, and useful periods of fire resistance with respect to panel integrity and insulation. These levels of performance cannot always be achieved without sacrificing some other property, for example colour stability or surface durability. Similar sacrifices also have to be made when the requirement is for a Class 1 surface spread of flame.

However, there can be situations in which compliance with the letter of the current regulations in these respects may still not achieve a safe building. Aspects of smoke evolution and contribution to the total fire load in the building could be important. Flame retardant materials with low heat propagation in the early stages (development) of a fire, will still burn, if combustible, during the main (fully developed) part of a fire, and combustible linings have a very dramatic effect on flaming.

As GRP is not the only lightweight, mouldable material, it may be unwise to persist too far with design in GRP where the fire behaviour will be less than satisfactory. The increasing development of GRC (glassfibre reinforced cement) provides designers with an incombustible material which is capable of giving many of the desirable features of GRP. However, GRC cannot yet match GRP in its aesthetic versatility, and it is this feature which must be considered next.

Appearance

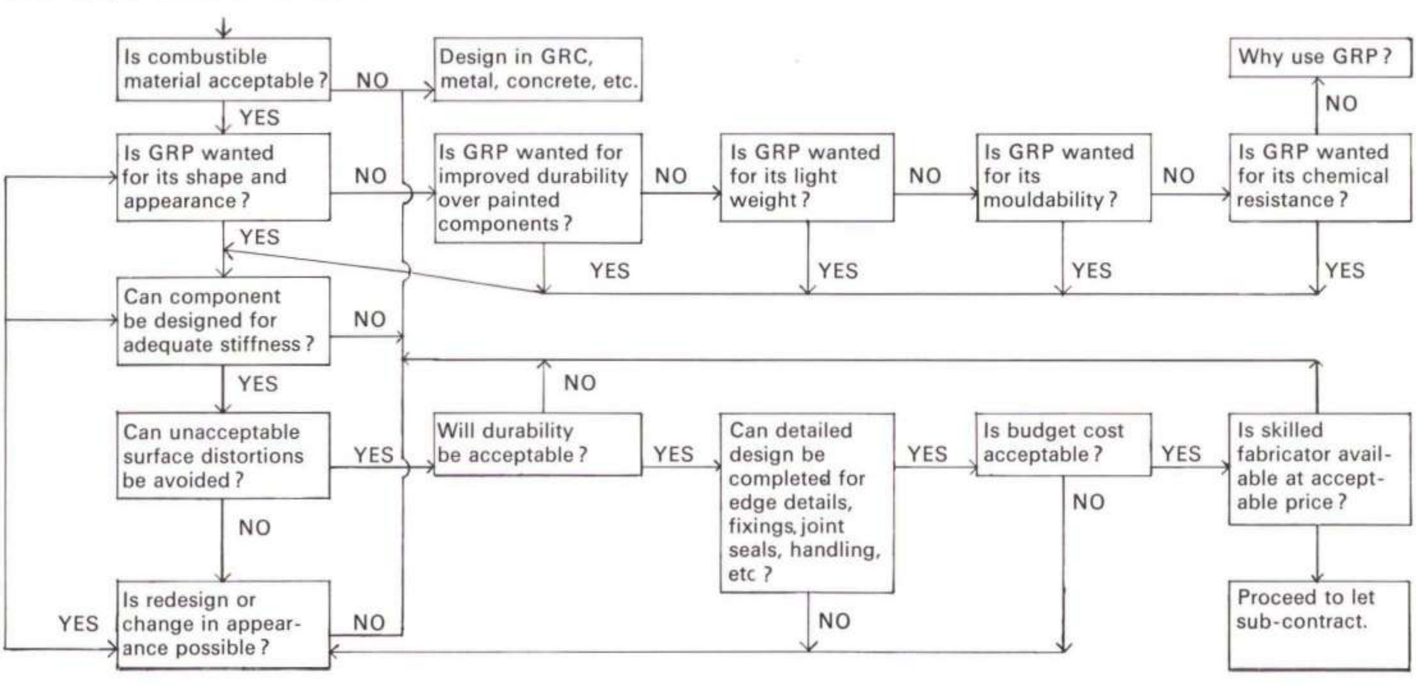
Many will dispute that GRP has a 'natural' appearance, the material so frequently being used to imitate others. The most characteristic GRP appearance is the smooth or very slightly textured semi-gloss finish achieved by forming it in a GRP mould which has not been highly polished. Full gloss is achieved either by very thorough mould polishing, or by the use of post fabrication surface finishes, such as polyurethanes.

The ability to simulate economically the appearance of other materials, particularly stones such as riven slate, is clearly an important and valid reason for using the material. It is not relevant to this paper to consider the architectural validity of Georgian porticos on new houses, but it is relevant to note that this usage of GRP stems directly from considerations of appearance.

The importance of colour in buildings is another hotly debated topic. GRP provides a means of achieving colour that adds greatly to the range of techniques available to the designer.

Although the most frequent 'natural' GRP finishes use pigmented resins, it could be argued that the translucent unpigmented material is the more 'natural'. The ability to incorporate translucent sections into otherwise

GRP component selection flow chart



opaque components without having to glaze the material in, is a characteristic of GRP which is unique in materials available for use in purpose designed components.

Mechanical properties

The mechanical properties of GRP will not generally play an overriding part in determining its suitability for cladding applications. It is usually possible to design a panel to meet the requirements of wind loading. However, it may be possible to impose severe performance requirements on other components, such as roofing, by way of deflection limitations which will make use of the material most uneconomic. It should also be borne in mind that where GRP is used in a truly load-bearing situation, the material specification chosen and control required during manufacture will place it on a par with 'engineering materials' and not 'cladding materials' in terms of cost. Because so little has been known about the long-term mechanical performance of GRP in creep and fatigue, it has been customary to design on the short-term properties but using high factors of safety (up to 10) to allow for this. As more information becomes available, the precision of structural design will increase and it will be possible to achieve more economic GRP structures with confidence.

Durability

The question of durability has already been touched upon. GRP has yet to establish itself as a material of proven durability even over the periods of time currently being quoted (30 years plus). The sensitivity of the gel coat to defects during fabrication or to damage before building completion is well known. In few other long life materials is a thin skin of material asked to do so much. Plastics coatings for sheet steel are obviously another in this category, but do we equate GRP with sheet steels?

If the requirement is to achieve greater durability than 'traditional' painted materials in a particular use, then GRP can offer very clear advantages. Replacement of timber weatherboarding by GRP panels of the same profile is a clear example.

Thus the durability of GRP must be considered in comparison with competing materials, some of which have proven long, useful lives and some of which required maintenance every few years. The effect of small changes in the materials making up GRP on their durability must constantly be borne in mind. It is a source of uncertainty to users who feel that they have little or no control over this aspect, it being part of the 'black magic' surrounding the material.

Weight

GRP has unquestioned advantages where a lightweight, strong material is needed. Unfortunately, in building, the potential cost savings on structure and foundations can often not be fully realized, and in other cases weight is needed to give airborne sound insulation. However, there are excellent examples of GRP clad roofs showing the lightweight advantage, often combined with partial translucence.

As joints in buildings are a frequent source of trouble, the ability of GRP to be made and handled in large units, because of its light weight, is often claimed to be an advantage. However, fewer joints also means fewer places to accommodate movement and tolerances. A careful balance must be struck.

Mouldability

Mouldability is apparent in applications where the appearance of other materials is being reproduced. However, this feature is frequently used to produce large panels of complex form. Such shapes may also be formed in precast concrete or GRC, but the lighter weight of GRP, combined with its mouldability, enables these complex shapes to be formed in larger panels.

The limits on mouldability are similar to other materials with respect to mould shape and construction to permit easy demoulding. GRP produces different constraints on subtleties of mould detail as its method of forming from resin and glass is different.

Chemical resistance

Careful selection of resins can confer a useful degree of chemical resistance on GRP, and this is exploited in a number of applications.

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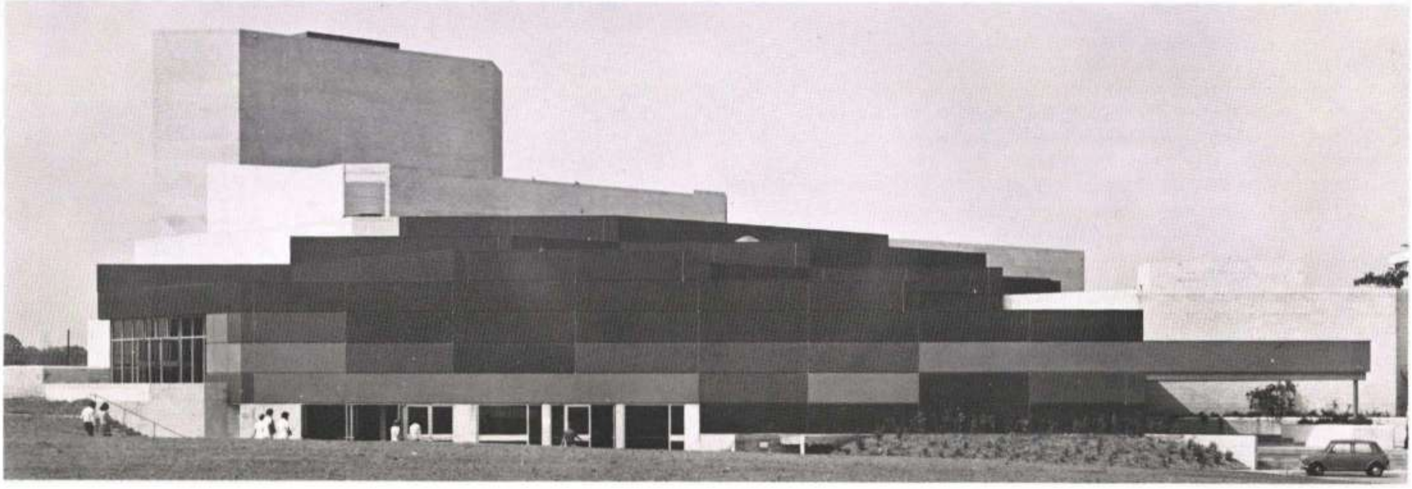

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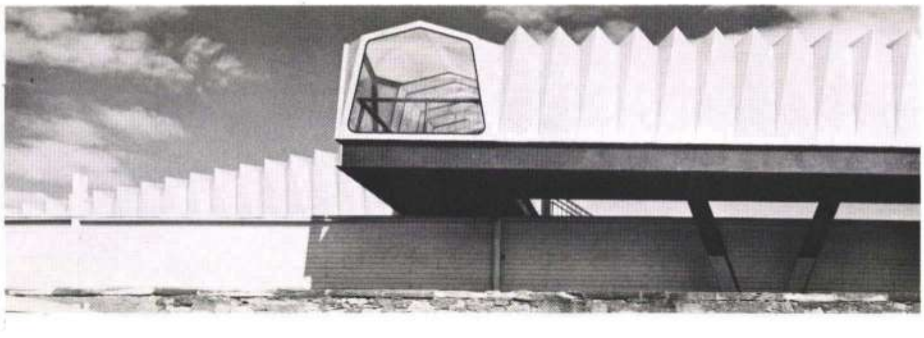
Fig. 1
Warwick University Arts Centre. Job no. 3855
Architects: Renton Howard Wood Levin Partnership.
Flat cladding panels in orange, brown and buff

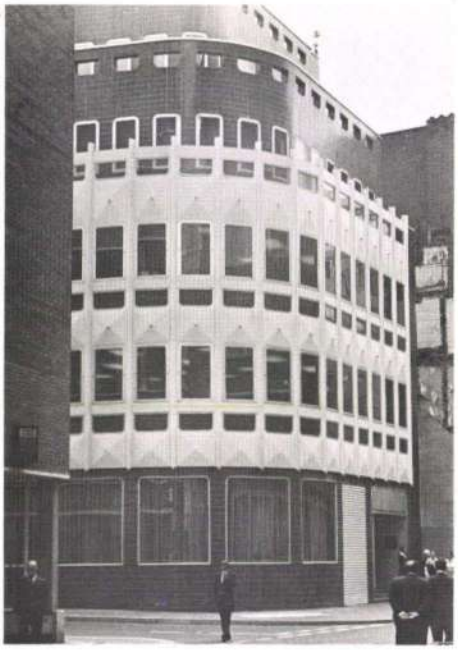
Figs. 2 & 3
Ardrossan Ferry Terminal, Scotland.
Job no. 2614.
Architects: Law & Dunbar-Naismith.
Translucent folded plate walkway units

3



Note
The photographs in this article show examples of GRP in which the Ove Arup Partnership has been involved. They are not intended to illustrate any particular points in the text.

4



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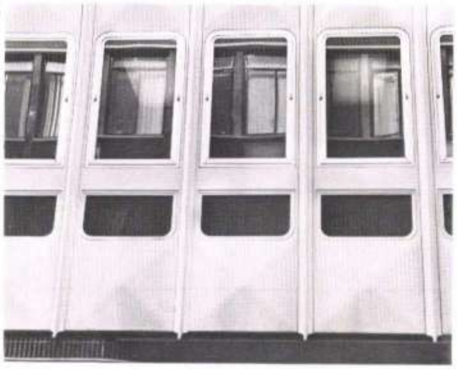

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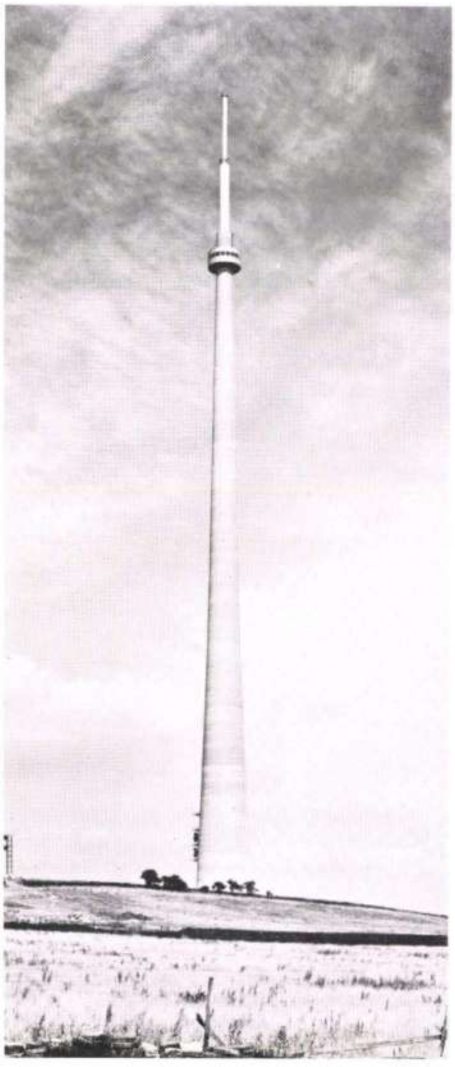


SELECTION CRITERIA: DESIGN CONSIDERATIONS

Assuming that GRP has tentatively been selected for a particular use based on consideration of the general attributes given above, thought must be given to a number of design considerations before final selection is made. The flow chart indicates the questions to be evaluated.

Design for stiffness

In most GRP applications (particularly cladding) it is found that the limiting property is not stiffness. The low modulus of elasticity can lead to unacceptably large deflections under normal working loads, even if these are only wind loads. Design must therefore be based on shaping to achieve adequate stiffness and on the incorporation of various ribs and composite sections. (Reference 2 gives examples of techniques that can be used.)

Design for adequate stiffness may, in some instances, only be possible if other design parameters are changed. It may be panel profile in the case of a cladding application, or section depth in the case of a window frame. The designer will have to decide if these changes can be accommodated or whether it would be better to switch to another material.

Surface distortion

The smoother and more glossy the surface finish, the more the appearance will be marred by surface distortions. These can arise both from inaccuracies in the mould (e.g. obtrusive 'flash lines' at points where the mould must be split for demoulding) and from shrinkage effects associated with stiffeners.

On high quality architectural GRP work, of relatively simple profile, this factor is one of the major ones leading to disappointment with the results achieved. No other material has this problem as acutely as GRP; though thin sheet metal cladding and fascias are a similar example. As with metals, the surface distortions can be exaggerated by thermal movements occurring after fixing to the building.

If surface distortions are likely to occur on a particular component, obviously re-design is required. In some instances it will be found better to re-design in an alternative material.

Design for durability and maintenance

Once design has proceeded to a stage at which the stiffness and surface of the component have been resolved, it is important to re-assess whether adequate durability can be achieved with the resins that will have to be chosen to meet the fire performance required. In addition, the mode of behaviour of the component under load must be carefully considered to ensure that local stressing does not occur in such a way that cracking of the gel coat would be expected.

At this stage the procedures that will eventually have to be followed to maintain the GRP should be considered and their feasibility assessed. It is worth remembering the basic tenet of good building that if a material is used whose designed life is less than that of the building, facility for its maintenance should form part of the design.

Detailed design

As with any other material, the detailed design is as important as overall design in achieving the result required from a component. Once the principles of detailed design with GRP have been mastered, there need be little difficulty in applying them.[2] Yet, despite this, the design of edge details, fixings and jointing is frequently inadequate. How often has one seen a bolted flange edge detail where the flanges are of a completely inadequate stiffness and so distort grossly between bolts? Similarly, cladding applications can be seen where the jointing material is falling out.

It is unusual to find that detailed design cannot be carried out within the context of a fixed outline design, but in some cases it may be

Figs. 4 & 5
Mumford Court Offices, London. Job no. 4148
Architects: City of London Architects Dept.
Double storey window units in white with mullion cover units

Fig. 6
Emley Moor Television Mast. Job no. 3429
Cylindrical cladding to the top steelwork mast

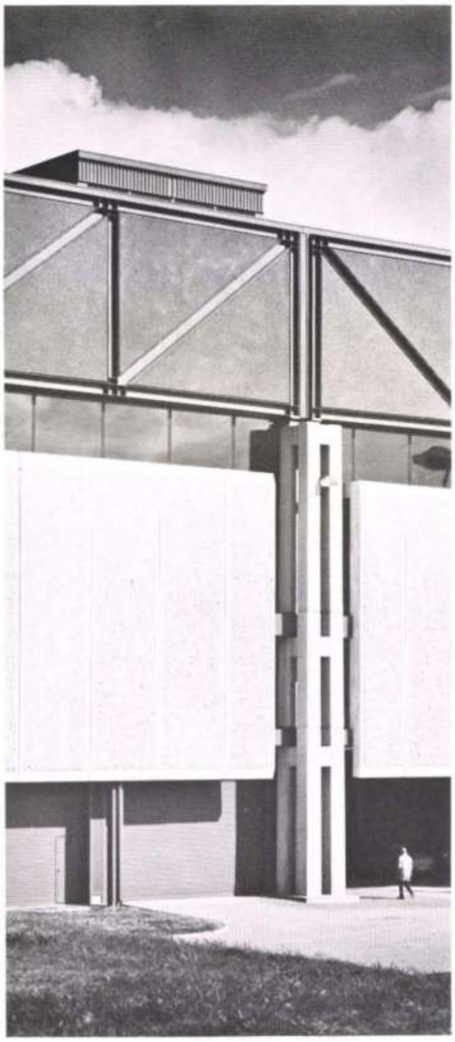
Fig. 8
Metal Box Head Office, Reading. Job no. 4304
Architects: Llewelyn-Davies, Weeks, Forestier-Walker & Bor
White internal lining units to walls

Figs. 7 & 9
Horizon factory, Nottingham. Job no. AA175
Designers: Arup Associates
Coffered ceiling units used as permanent formwork and housing lighting and air extract/intake points

Fig. 10
Horizon factory, Nottingham. Job no. AA175
Designers: Arup Associates
Triangular infill cladding to steelwork truss in brown aggregate finished GRP sandwich construction

Figs. 11 & 12
Bletchley Leisure Centre Walkway.
Job no. 4196
Architects: Faulkner-Brown, Hendy, Watkinson, Stonor.
White, self-supporting arch units over link between the car park and leisure centre

10



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necessary to re-design parts of the overall component to achieve more satisfactory details.

Budget cost

The great problem with budget costs is the stage at which they are established with any meaningfulness. Too often a budget is arrived at before any consideration has been given to the design considerations. As a result the design is straight jacketed into a form that may not achieve a satisfactory result.

In many cases it is necessary to have a budget cost early in a project. The costs then established for a component are obviously going to be based on updated historical data both for costs in GRP and in other materials which could achieve the same result. Assuming that it is possible to establish this reasonable target, then the detailed design in GRP must be geared to meeting it. If it cannot, as could often be the case, then re-design in one of the alternatives may be necessary.

The essential point here is that GRP is generally not a low cost material, and that for very many components if lowest cost is the overriding objective, GRP is unlikely to be the most appropriate choice of material.

Design to use the skills of fabricators

We have already touched on the fact that the skill needed in production is a disadvantage of GRP. The low capital cost of setting up a GRP fabrication shop should not disguise the fact that laminating GRP is a skilled job. Despite the length of time that it has been a fully commercially available material, the recognition of laminating as a skilled task seems very slow in taking hold. In times of high demand in the industry, beware the fabricator who says that he can rapidly increase his production capacity by recruiting new operatives.

Because it is a material whose final qualities are so dependent on workmanship, the way a component will be formed is a matter which must be considered by the designer. The more the design takes account of the basic techniques, the more likely it is that a satisfactory job will be achieved. To take an example, if for reasons of local stiffening associated with a fixing point, a greater bulk of material is required at some point, this can easily be done in GRC by spraying until sufficient material has been built up. With GRP, thicknesses can be built up by spraying, but great care is needed. More usually the area would be detailed using box or rib construction to give the same effect but with economy in use of material.

Prototypes

It is a cliché to say that the building industry makes inadequate use of prototypes of components or elements proposed for particular buildings. The 'prototype' is tested out on the actual building. In the GRP field there is much to be said for commissioning prototypes to be made in advance of letting a sub-contract. Not only can panel detailing be tested out, but they can be used as reference standards to show clearly the quality required. The main difficulty, of course, is in the high cost of tools from which to make a prototype for anything but the simplest panel.

CONCLUDING REMARKS

The discussion in this paper has concentrated on examining the logical reasons which may lead to the choice of GRP for a building component. A number of questions have been posed which must be considered before the feasibility of a particular use can be established. It has been pointed out that for various reasons it may turn out better to design in an alternative material. This conclusion could arise not because GRP is an inherently poor material, but because it may not be the most suitable for the particular use. However, many valid uses have been identified.

Although the procedures to be followed in design do not form a direct part of the materials selection, it is worth commenting on this aspect as far as GRP is concerned. Design work for purpose-made GRP components could be done by the design team for the building (e.g. the architect), by a specialist consultant or by a fabricator. If the architect is not to do it himself he must retain a consultant, or get a fabricator to do it. But in the latter cases, he may find that a commitment is required to the particular material before the feasibility of its use has been established. This may cause problems.

If an architect is unfamiliar with GRP, it is essential that he studies the material, or retains advice and assistance before final commitment becomes necessary so that appropriate materials selection may be made. It has been a thesis of this paper that the criteria to be considered at this stage extend into matters of detail. Success with GRP, as with many other materials, requires that design be carried through in detail. The concept may be excellent but the execution appalling due to poor design in detail.

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