THE ARUP JOURNAL

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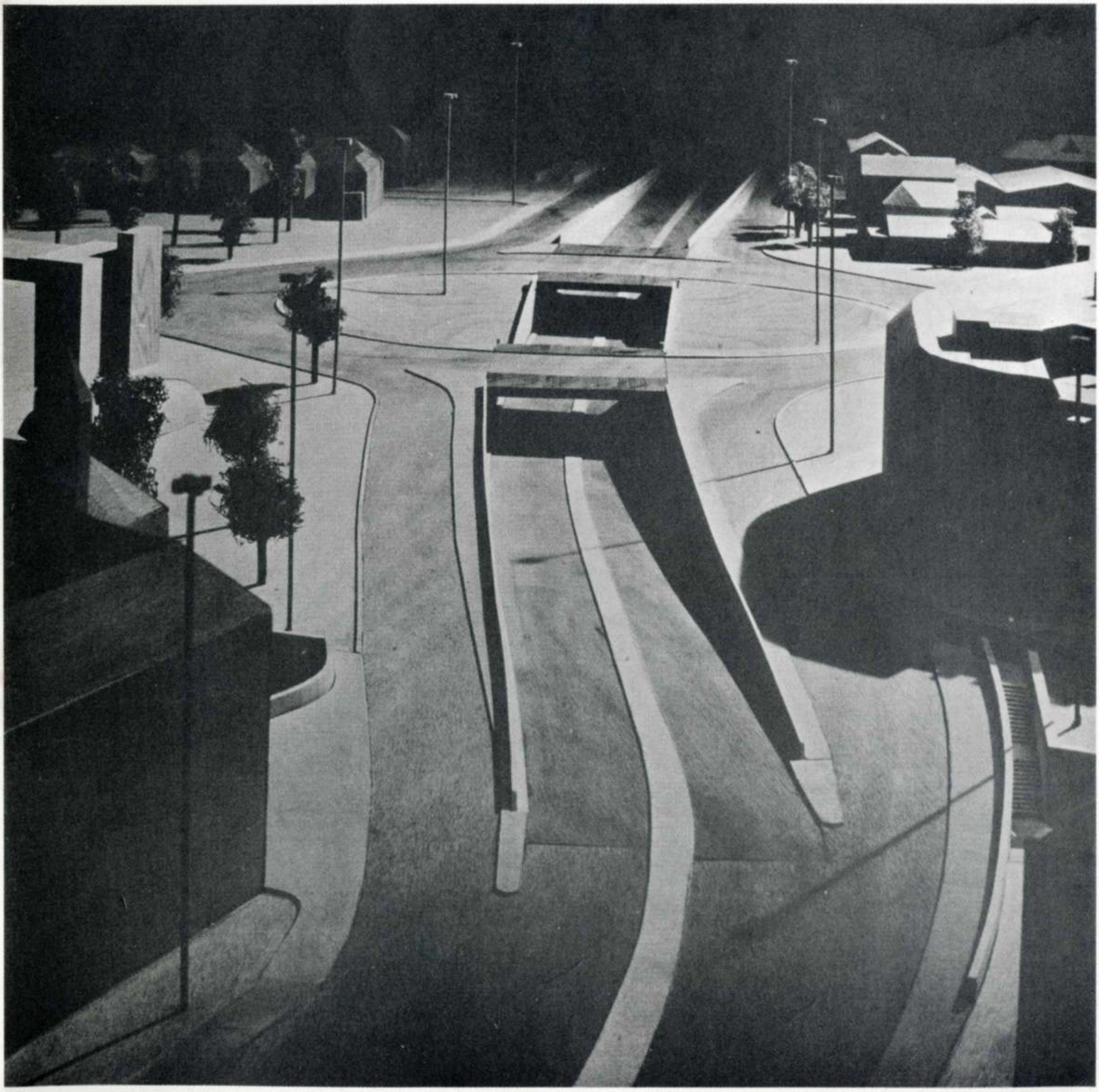

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The front cover shows a model of the St. Albans Road Underpass: view from the south. (Photo: Henk Snoek)
The back cover shows subway no. 6: view from inside model with low ramp wall in background. (Photo: Georg Rotne)

Watford Central Area Redevelopment: Phase 3

Svend Jensen

The following can only be considered as an introduction or as the first instalment of a serial which will be continued in a couple of years. The job is still in the design stage, and it may seem rather forward to write about it now as there will undoubtedly be some modifications to the scheme.

It is not the intention here to go into the design and analysis in very great detail but merely to give an outline of the scheme generally and to describe the various structures.

Highway proposals

In 1964 the Watford Borough Council and the Hertfordshire County Council adopted The Central Area Plan for the Redevelopment of Watford. The traffic aspects of this plan were investigated by the Borough Engineer, who carried out a traffic survey and based his proposals on the forecast traffic flow in the peak hours of 1980. These proposals are very extensive, but the system broadly consists of a one-way ring road combined with a north-south traffic route and surrounded by a network of outer routes which serve as accesses to the ring road.

Extensive work in the centre of a built up area naturally calls for careful programming and Mr. Brand, the Borough Engineer, planned the scheme to be carried out in eight independent phases. In phase 1, part of the High Street was turned into a service precinct. To take the diverted traffic a new road was constructed and some existing roads were improved. Phase 2, which is being carried out at the moment, is the construction of a small gyratory system in the south of the Central Area.

Phase 3 is one of the main stages and our involvement started in 1966 when we were appointed consulting engineers for the major structures. The road layout has been done by the Borough Engineer and throughout the scheme we have worked closely with his Special Project Section headed by Brian L. Williams. This collaboration has been very successful and the constant exchange of ideas has led to improvements in the design.

The extent of Phase 3 is shown in Fig. 1. Rickmansworth Road and St. Albans Road (A412) will be widened considerably and the A411 will be diverted from the High Street to a new road, Eastern Relief Road. Where the A412 intersects the A411 a grade separated roundabout, St. Albans Road underpass, will be constructed. At the junction of Cassio Road and Rickmansworth Road the right turn from the former will be taken by means of a slip road passing under the south-east carriageway of Rickmansworth Road. Part of the High Street will become a pedestrian precinct, and a bridge will be constructed to carry the A411 traffic over it at Exchange Road. A major pedestrian subway will link the High Street shopping area with the Civic Centre, under the A412.

General layout

But who would spend an afternoon exploring Slough, or a morning doing Watford? (Elizabeth Beazley: 'Design and Detail of the Space between Buildings'.)

We actually spent several mornings doing Watford and we worried a lot trying to find an acceptable way to integrate the structures into the existing environment. It does not require much effort to spoil the character of a place with a major road improvement scheme, in fact it is difficult not to. Engineering requirements and economic considerations determine the type of structure but irrespective of type, we felt that the right solution for this scheme was to avoid a spectacular *tour de force* and instead go for clarity and simplicity. We tried to express the function of the structures and to get the right proportions. We have also made an attempt to keep visual intrusion to a minimum and to make the structures sit in their surroundings instead of cutting through them.

The problems regarding layout are to a large degree three-dimensional and throughout the design we have made extensive use of models. Some of these were made in the drawing office using the Abbott Model Kit where steel pins are pressed into a baseboard. The roads and structures were made in cardboard or plywood and we used sawdust for landscaping. These rough models of course bore no comparison to the solid models produced in the model room but they have been very useful in the design.

St. Albans Road Underpass

The principal structure in the scheme is the underpass at the A411-A412 intersection.

It was originally envisaged that through traffic on the A412 should be taken over a surface level roundabout by means of a flyover and several alternative layouts were considered. A rough design was made for a 63 ft. (19.2 m.) wide viaduct with the approach ramps 200 ft. (61 m.) and 300 ft. (91.5 m.) long respectively and an elevated structure of 580 ft. (176.8 m.). As the major part of this structure would be more than 20 ft. (6 m.) above ground level it would overshadow the surroundings and be out of scale with the existing buildings. We worried considerably about the impact of this flyover on the town centre, and although we tried to console ourselves with the thought that some of the disadvantages might be overcome when the surrounding buildings were redeveloped, we could not get rid of an uneasy feeling that it was a wrong solution.

A re-examination showed that by modifying

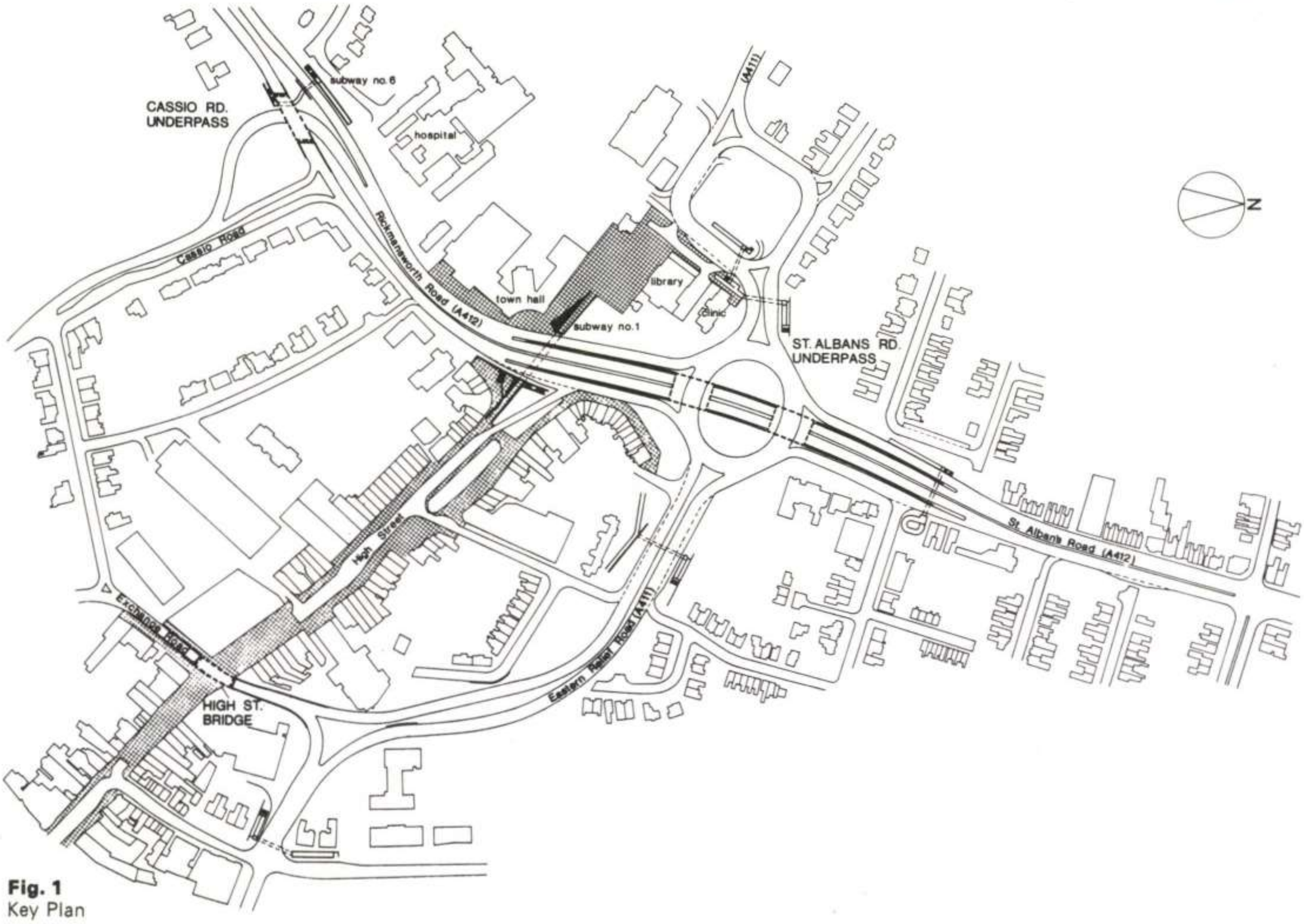


Fig. 1
Key Plan

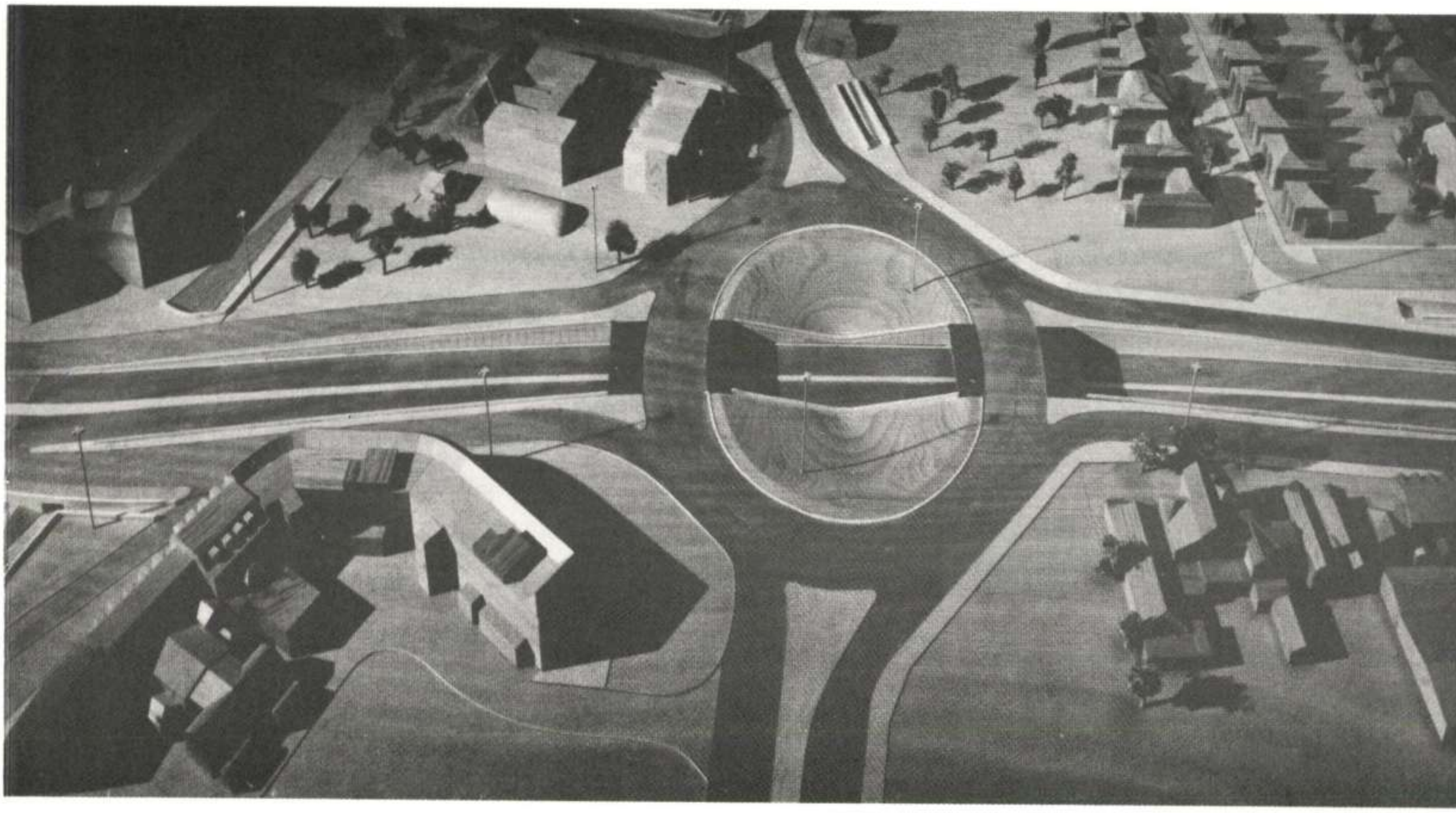
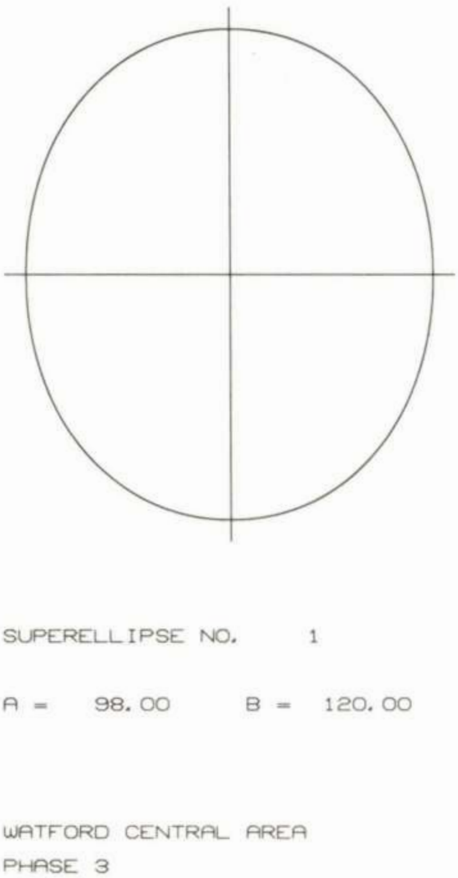


Fig. 2
St. Albans Road Underpass
Aerial view of model
(Photo: Henk Snoek)

Fig. 3
Superellipse



the road layout it would be possible to take the A412 under the roundabout. This involved moving the intersection approximately 30 ft. (9 m.) further north and it increased the land requirement. However, as the underpass structure was approximately 30% cheaper than the viaduct a considerable overall saving was achieved. The underpass solution reduces the noise level but, most important, it overcomes the problem of extreme visual intrusion.

Fig. 2 and the front cover show the underpass which has an overall length of 1,050 ft. (320 m.) and a width between retaining walls of 58 ft. 8 in. (17.9 m.). In plan it is gently curved and the maximum gradient of the carriageway is approximately 1:20.

We considered covering the underpass throughout the length of the roundabout, but apart from being uneconomical it was also technically unsatisfactory. The other much more attractive alternative was to leave the underpass open and just have two decks to take the roundabout carriageways over it. The difficulty here was that the rectangular opening in the roundabout island between the two bridge decks was out of harmony with the original irregularly shaped roundabout. By altering the shape of the island to a superellipse with the axis related to the centreline of the road we got a symmetrical arrangement and better relationship to the underpass.

The superellipse was originally proposed in the early 1960's by Piet Hein to solve a roundabout problem in Stockholm. The formula for the superellipse is:

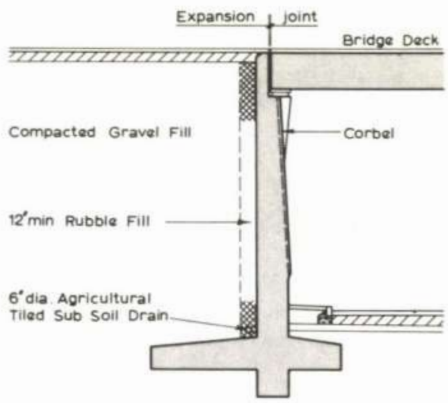
$$\left(\frac{x}{a}\right)^n + \left(\frac{y}{b}\right)^n = 1, \text{ where } n \geq 2$$

With $n = 2$ this is of course the ordinary ellipse, but when n increases, the oval develops flatter and flatter sides, becoming more and more like a rectangle which is the limit as n approaches infinity. In Stockholm Piet Hein used $n = 2.5$ whereas we in Watford used $n = 2.1$. There is now a programme available in the Computer Room which will draw superellipses for any given values of a, b and n (see Fig. 3).

To open up the central part of the underpass and provide a more pleasant environment for the motorist we have dished the roundabout. This also results in an appreciable saving in the cost of retaining walls and we believe it improves the view from the surface roads.

The underpass width is very restricted and near-vertical walls were the only solution. Icos walls with or without ground anchors were considered but ordinary cantilevered reinforced concrete retaining walls proved the cheapest solution. The solid parapets will be bush hammered and underneath them will be

Fig. 4
St. Albans Road Underpass
Section through abutment



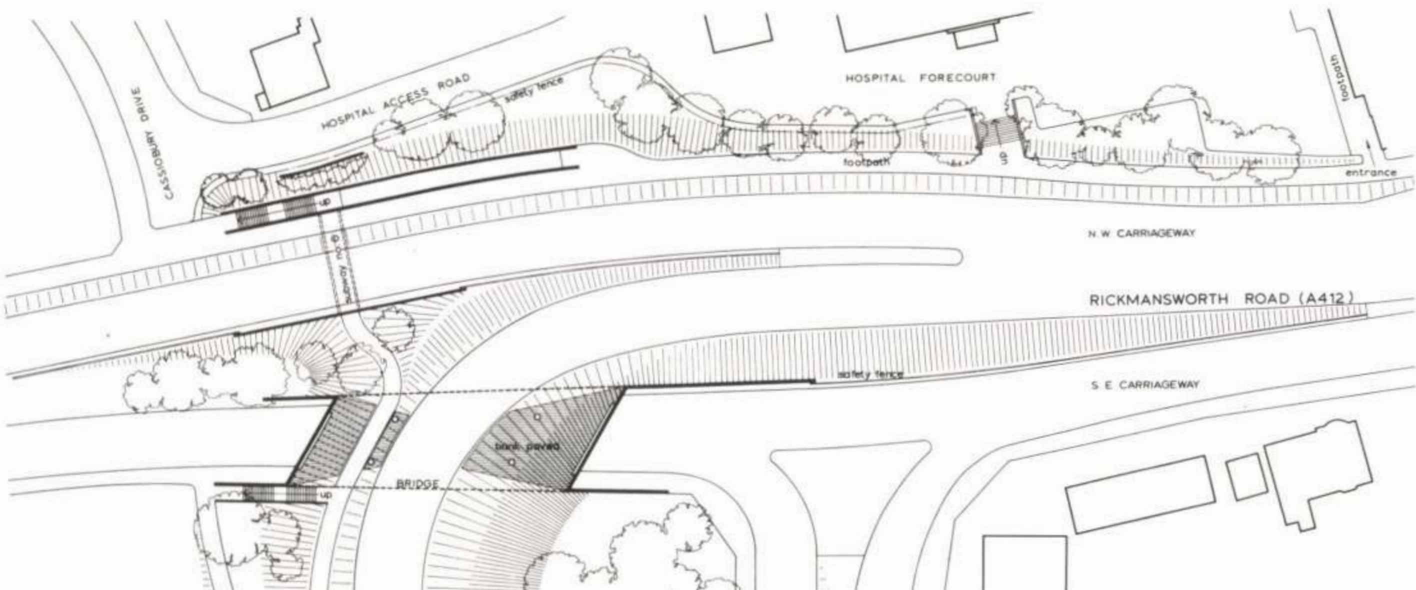


Fig. 5
Cassio Road Underpass
General arrangement

Fig. 6
Cassio Road Underpass
View of model from north-east
(Photo: Henk Snoek)

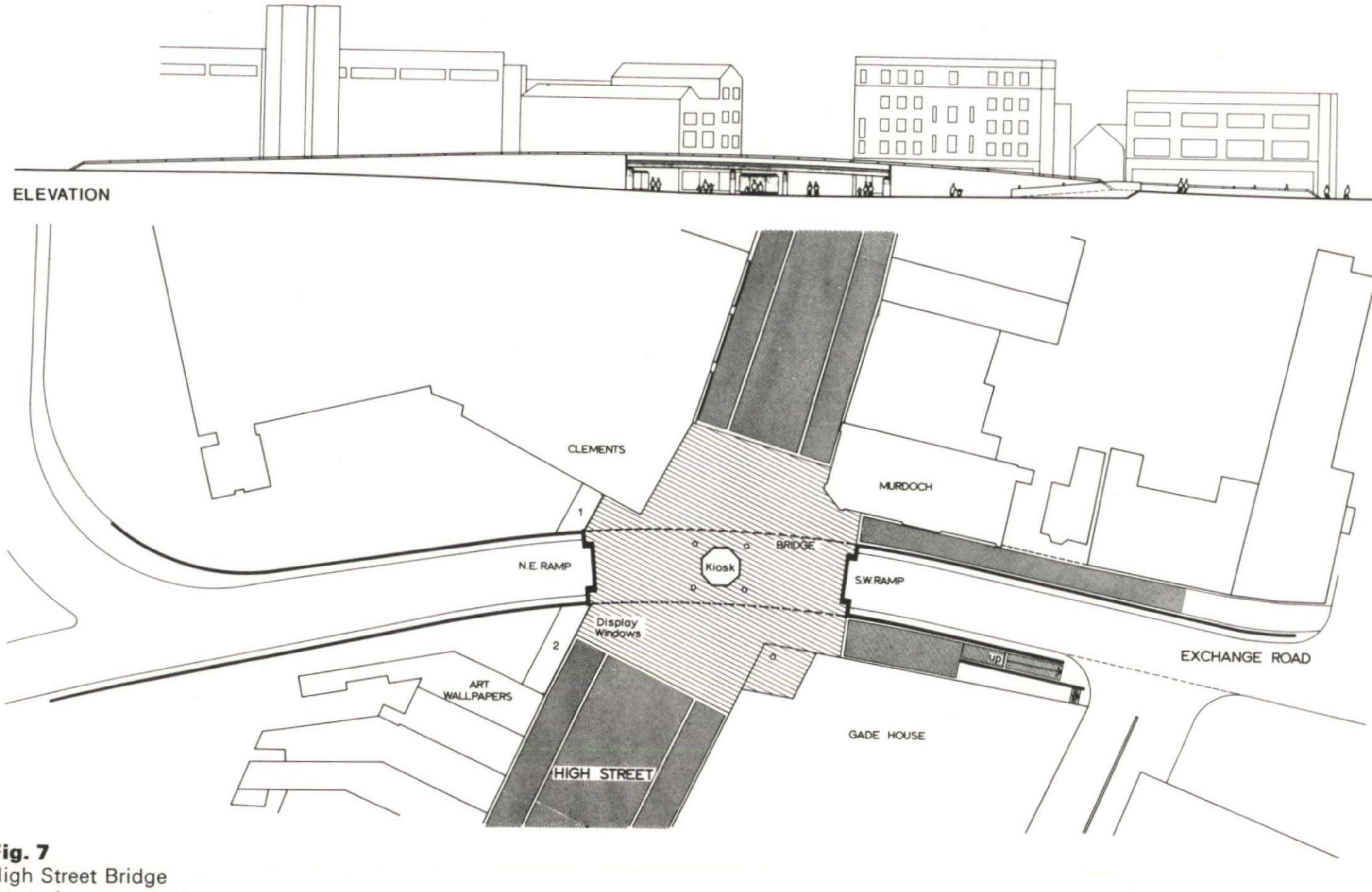


Fig. 7
High Street Bridge
General arrangement

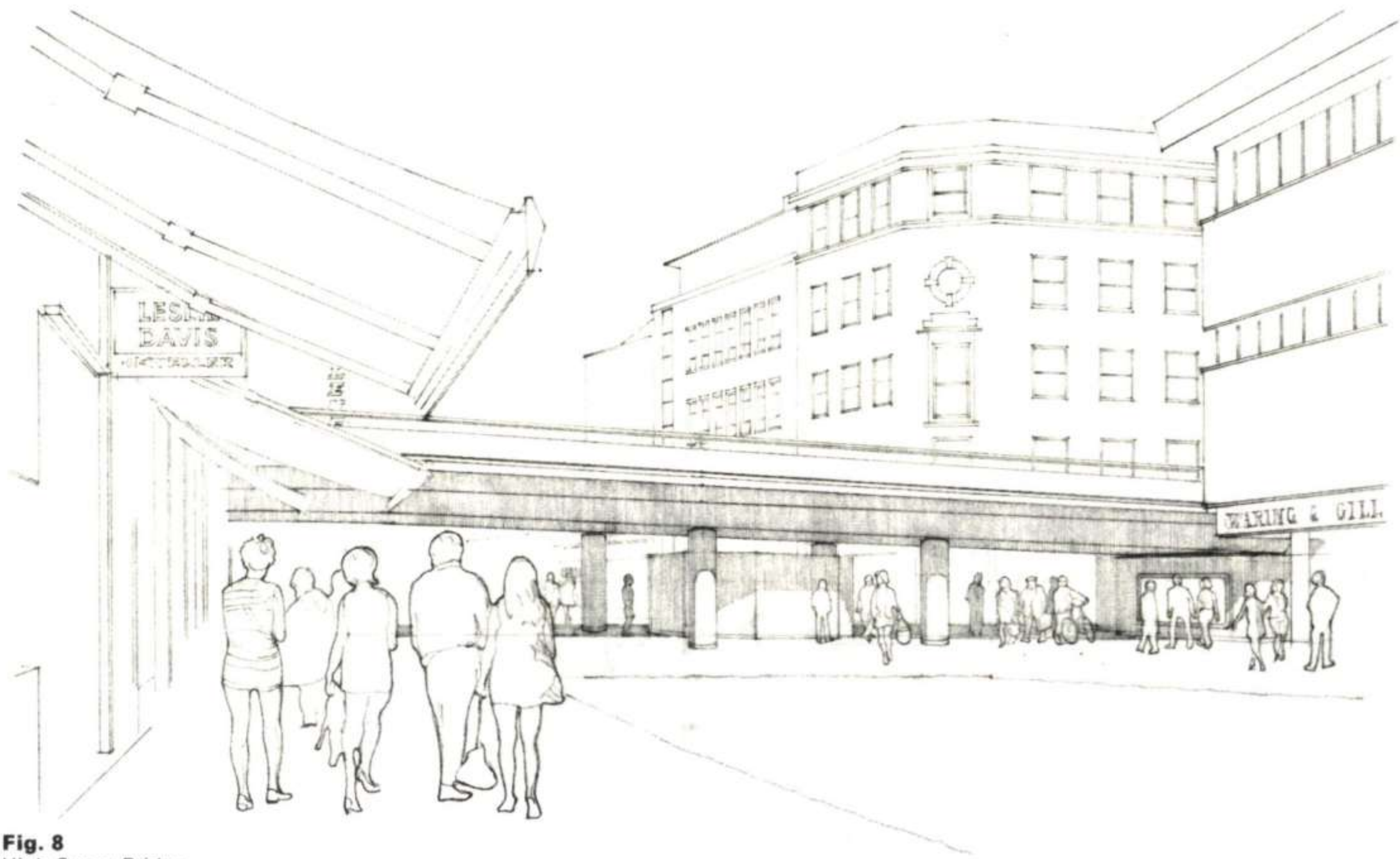


Fig. 8
High Street Bridge
View from north
(Drawn by Georg Rotne)

a deep recess which will give emphasis to the horizontal direction. The main body of the wall will have a vertical boarded finish with a bush hammered recess at the bottom to allow for splash from vehicles.

The two single span decks each have a span of 60 ft. 8 in. (18.5 m.) and the width is approximately 92 ft. (28 m.) of which the carriageway takes up 44 ft. (13.4 m.). The additional width is provided to give adequate visibility for traffic on or joining the roundabout. A large number of cost comparisons were made between precast concrete, in situ concrete and composite steel and concrete decks. Allowing for the effect of the structural depth of the bridge deck on the cost of the excavation and retaining walls the most economic solution was a 3 ft. (0.9 m.) deep, solid, in situ, post-tensioned, concrete slab with tapering edges. The deck will rest on laminated rubber bearings at 8 ft. (2.4 m.) intervals. As shown in Fig. 4 these sit on corbels cast integrally with the abutments which form part of the underpass walls.

Cassio Road Underpass

To avoid spoiling a hockey pitch at the West Herts Sports Ground the slip road joining Cassio Road with Rickmansworth Road is on a very tight curve. It starts off at ground level at Cassio Road and cuts approximately 20 ft. (6 m.) into the ground at the bridge, after which it starts rising again. The dual carriageways of the Rickmansworth Road are separated in this area and to meet the slip road the north west carriageway is taken down to 13 ft. (4 m.) below ground. A footpath had to be provided along the slip road, and pedestrian movement between the slip road and both sides of Rickmansworth Road had to be allowed for.

For the bridge over the slip road and the pedestrian way system two main alternatives were considered. One scheme was for a single span skew bridge between high abutments where sight line requirements on the lower road necessitated a large deck span with the high abutments giving a tunnel-like effect. Pedestrians would be taken under each carriageway of the A412 by separate subways but a footbridge over the north-west carriageway was also considered. The other scheme was for a three span bridge with partly buried abutments. This solution opens up the area for the motorists and especially for the pedestrians who, instead of crossing the south-east carriageway in a subway, can pass under one of the side spans of the bridge. Fig. 5 shows the general arrangements of the second scheme which is both cheaper and more practical.

The side spans of the continuous bridge are each 30 ft. (9 m.) and the central span is 62 ft. (18.9 m.). Because of the acceleration lane from Cassio Road, the width of the carriageway varies so that the bridge is tapered in plan with an average width of 42 ft. 6 in. (13 m.). The cross section of the deck is a solid reinforced slab, 3 ft. (0.9 m.) deep over the central portion with the edges reduced in depth and shaped to lighten the appearance. The deck rests on rubber bearings and is fixed at one abutment. The intermediate supports are circular reinforced concrete columns which will be founded on individual bases. Abutments and wing walls will also be of reinforced concrete and generally have a vertical boarded finish.

Where the footpath runs along the slip road and under the bridge it is raised above the level of the carriageway. In the central reserve on the north side of the bridge it cuts into the bank and continues in an 8 ft. (2.4 m.) wide subway under the north-west carriageway of Rickmansworth Road. To open up the subway ramp and steps, and to give an attractive view from inside the tunnel, the north-west wall is kept at parapet height. The slope from the parapet up to existing ground level is so steep that it has been necessary to introduce a short length of wall at high level.

To minimize loss of land to the hospital it was originally envisaged that a 500 ft. (152 m.) long retaining wall with footpath at the top should be provided along the length of Rickmansworth Road which is in a cutting. It was highly desirable to do away with this wall and we prepared an alternative scheme with banks where the footpath is set into the slope. This results in a considerable saving in construction cost, opens up the area and separates the public footpath from the hospital forecourt. Trees and bushes planted in these banks will act as sound baffles, shielding the hospital from some of the traffic noise.

High Street Bridge

Several schemes were considered for linking Exchange Road and the new Eastern Relief Road where the A411 crosses the High Street pedestrian precinct. Ideally vehicles should be taken under the precinct, but it would be impossible to provide suitable approach ramps and underground services would be completely disrupted. A surface level road with a pedestrian subway underneath would again cause major disruption to the services and be wrong on planning grounds as the approach ramps would draw people away from the existing shop fronts. The original proposal for a bridge taking vehicles over the pedestrian precinct also imposes problems but it is considered the only feasible solution.

Fig. 7 shows the general arrangement. The horizontal alignment was largely fixed by surrounding buildings which results in a skew crossing and a bridge curved in plan with a 540 ft. (165 m.) radius. The length of approach ramp that can be accommodated in Exchange Road is limited and to obtain maximum headroom under the bridge the gradient has been made 6%.

It would clearly be wrong for the abutments to project beyond the building lines as this, apart from increasing the interference to underground services, would disrupt the continuity of the High Street. In fact to minimize loss of amenity to shop owners the south-west abutment in Exchange Road has been set even further back.

This leads to an elevated structure approximately 110 ft. (33.5 m.) long. A single span deck was discarded as it would be excessively expensive and reduce the headroom to an unacceptable 7 ft. (2.1 m.). Of the alternative layouts considered, a three span bridge, with a group of four circular, reinforced concrete columns forming the intermediate supports, was chosen. The central span is 22 ft. (6.7 m.), the side spans are 44 ft. (13.4 m.), and the overall width is 32 ft. (9.8 m.). The reinforced concrete deck has a cross section similar to the bridge at Cassio Road but with a maximum depth of 2 ft. 9 in. (0.84 m.).

There is at this point no definite axis to the High Street and we found that abutments and columns placed at right angles to the centre line of bridge gave not only the simplest and cheapest solution but also the best appearance. The abutments and ramps will be of in situ concrete construction and, in order to be contained within the width of the road, the ramp walls will be L-shaped in section. To make pedestrians less aware of overhead traffic, and to protect them from noise and splash, the parapets will be made solid for a height of 2 ft. 3 in. (0.69 m.). They will be surmounted by a single horizontal aluminium rail which brings the total height up to the required minimum of 3 ft. 3 in. (0.99 m.).

The High Street is a busy shopping area with a pedestrian flow which at present is over 8,000 per hour in the peak periods. As we saw it, the problem was to reveal the structure of the bridge in an honest manner and at the same time treat it as an element in the street, as part of the shopping environment. In the design we have, therefore, underplayed the bridge and instead tried to make a 'place'. The area underneath the bridge should not be a dark, waste space but an amenity, a place where people can congregate. To help achieve this we have, as shown in Fig. 8, placed a kiosk centrally between the four columns. The kiosk will be set well clear of the columns and be of light construction showing clearly that it takes no part in supporting the bridge.

On the north side of the street it will be necessary to demolish some of the existing buildings to take the bridge through. An opening here would destroy the pattern of the street and it is, therefore, proposed that shop windows or showcases should be provided between the abutments and the remaining buildings until new development takes place. It is also suggested that display cabinets with front access should be fixed to the faces of the abutments and to the ramp walls in Exchange Road—see Fig. 9.

Subway No. 1

The subway which will link the High Street Shopping Area and the Civic Centre under the A412 can be seen at the extreme left in Fig. 2. Due to its position, the importance of the areas it links and the number of pedestrians using it, this subway will not be a standard one. The finishes will be different, it has been given a generous width and the approach ramps have been opened up by wide steps at the sides. On the north-west side these will extend over the whole length of the ramp which will give extra light, easy access at any point, and provide a setting in the form of a plinth for the Town Hall.

The Borough Council has decided that toilets shall be provided in this subway and we are at present modifying the layout to allow for this.

Lighting

Alternative methods of lighting have been examined and a scheme using mainly high masts has been recommended as the most suitable.

Lighting by conventional columns was rejected because of the visual effects, both by day and night, of the many columns required. With the grade separated carriageways the height of the lanterns above ground level would cause confusion and possibly glare. A combination of conventional columns and low level parapet lighting in St. Albans Road Underpass would overcome the problem of variable lantern height, but parapet light would not be entirely satisfactory with the wide carriageways and the installation, running and maintenance cost would be very high.

Alternative mounting heights, using 20, 25 and 30 metre masts were considered and 25 metre high masts were found to be most economical. Each mast has a unit consisting of four 1,000 watt colour corrected mercury lamps with special cut-off lanterns and each replaces approximately six conventional light columns.

Under the St. Albans Road Underpass bridges, the lighting will be reinforced by cornice mounted subway lanterns placed between the corbels.

At the High Street Bridge a scheme has been proposed for lighting by means of lanterns fixed to the surrounding buildings, and in principle the owners have agreed to this.

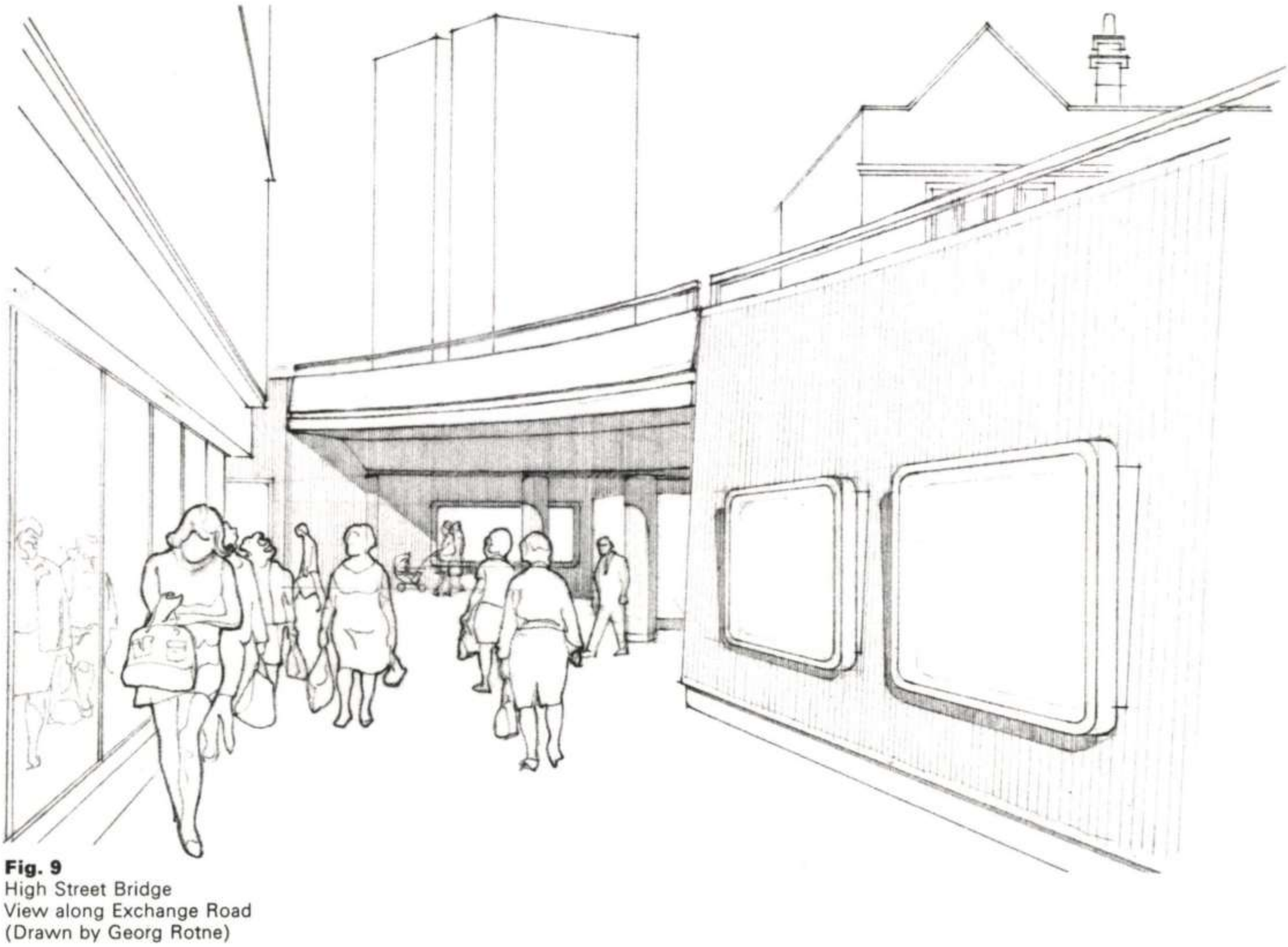


Fig. 9
High Street Bridge
View along Exchange Road
(Drawn by Georg Rotne)

The Avoncroft Museum of Buildings

John Martin

I am going to begin by quoting Sir Hugh Casson, President of the Avoncroft Trust.

'Near the village of Stoke Prior in Worcestershire work is well under way on the first museum of buildings in England. One house is already up and plans for others are far advanced.

The purpose of the museum is to preserve, by removement and re-erection, representative examples of timber framed buildings which would otherwise be demolished and lost for ever. At the same time the museum will, wherever possible, restore or advise upon the restoration of buildings on their present sites.

On the Continent the need for museums of this kind has been accepted for many years and many of them, having been set up with state support, are now run at a profit. With the exception of St. Fagan's Museum near Cardiff, we in this country have been desperately slow in following their example.

Yet Britain, and especially England, is by far the richest of all European countries in its surviving examples of mediaeval buildings. From the great open roofed halls to the simple cottages and farm houses, English timber framed buildings have shown an extraordinary ability to survive. Today, however, the potential destructive power of new roads and urban redevelopment far exceeds anything experienced in the past.

It is to save at least a few of these irreplaceable buildings that the museum has been established. With your support it will shape and grow into a most valuable exhibition, where people with a love of history and our heritage from this country and from abroad will be able to recapture the way our ancestors lived and the beauty and craftmanship of their buildings.

But it will not be just an exhibition. Carpenters will be trained at Avoncroft in the skills of restoration, architects will learn, and have already done so, techniques from the buildings being re-erected there and art historians will be able to add to their knowledge of our mediaeval past. Just as important, school children will continue to use the museum and do projects on the material there as they have been doing in the past year.'

That message, as you probably guessed, is part of an appeal by Sir Hugh Casson to raise funds for the Avoncroft Trust.

Perhaps I had better explain how we come into it. One day, a year or so ago, Mr. Charles, who is the architect behind the whole project, called at our office and met Peter Dunican. He asked him if we might be able to help with one of their projects at Avoncroft. Now it just so happened that it was Mr. Charles' birthday and it was also Peter Dunican's. Whether that had anything to do with it I do not know, but in fact we did agree to do what we could to help.

Before long I found myself at the wonderful Old Mill at Churchill in Worcestershire, which is Freddy and Mary Charles' home. It was a visit I shall never forget. They introduced me to a new world. The Mill itself is a wonderful example of mediaeval building and they have really given it new life. I was taken to the Avoncroft site and there I met Mr. Greiner who is the Master Carpenter in charge of the work. If the idea of Avoncroft depends on Freddy Charles, then the execution of the work certainly depends on Gunold Greiner. I had never met a man who more fully deserves the title of Master Carpenter. He had just about finished reconstructing the Bromsgrove House, and this he had carried out almost single-handed. The Bromsgrove House was first built about 1450, and this date was established during demolition by the discovery of a coin lodged in one of the joints of the hall tie-beam. It was a groat of Henry VI. This house was the only substantial building of the Middle Ages which still stood in the town, although traces of other buildings, notably a great timber cruck, also at the museum, have been found, from time to time, during demolition.

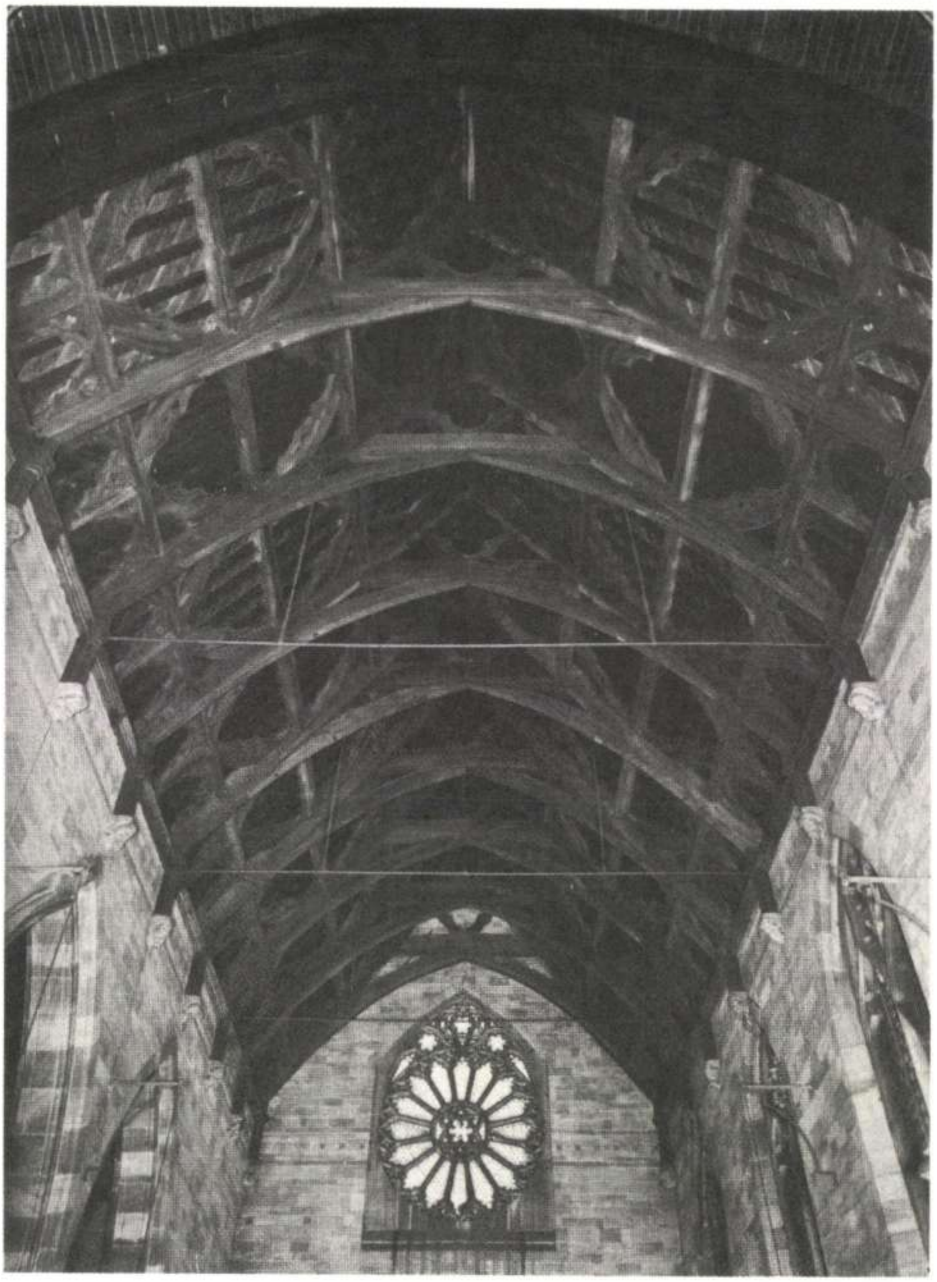


Fig. 1
Guesten Hall roof on Church of the Holy Trinity at Worcester
(Photo: National Monuments Record. Crown copyright)

To begin with, the demolition of the Bromsgrove House was not properly prepared, and many of the main timbers were broken. However, eventually all the timbers were taken down and taken to Avoncroft, and for the next three years there they were stored.

In the summer of 1965 Mr. Greiner, in pre-war days a refugee from Germany whose father had been minister, artist, and wood sculptor, offered to erect the house provided that there was no intention to re-create 'Ye Olde England'.

The timbers were laid out in the field for initial sorting. The only clue to their correct position within the frames was the exact fitting of mortices and tenons and the position of peg holes. Broken sections of timber could often be matched only by finding splinters that united their natural grain. The shape, size and position of the missing timbers were ascertained by the mortices and peg holes that survived. Hardly a single timber did not need repair.

By the early spring of 1966 all of the timbers of the Solar Wing were repaired, and the erection of the main frame work was nearly completed by the end of May. The wall frames were enclosed by wattle, and the roof covered in polythene by October. Then the same sequence of operations was started for reconstructing the Hall. The plastering of the wattle panels went on at the same time. Shortly before Christmas the frames of the Hall were ready for erection. The re-instatement of the chimney stack took place on 27 January 1967, and the main structure was complete.

Then followed the construction of the Hall

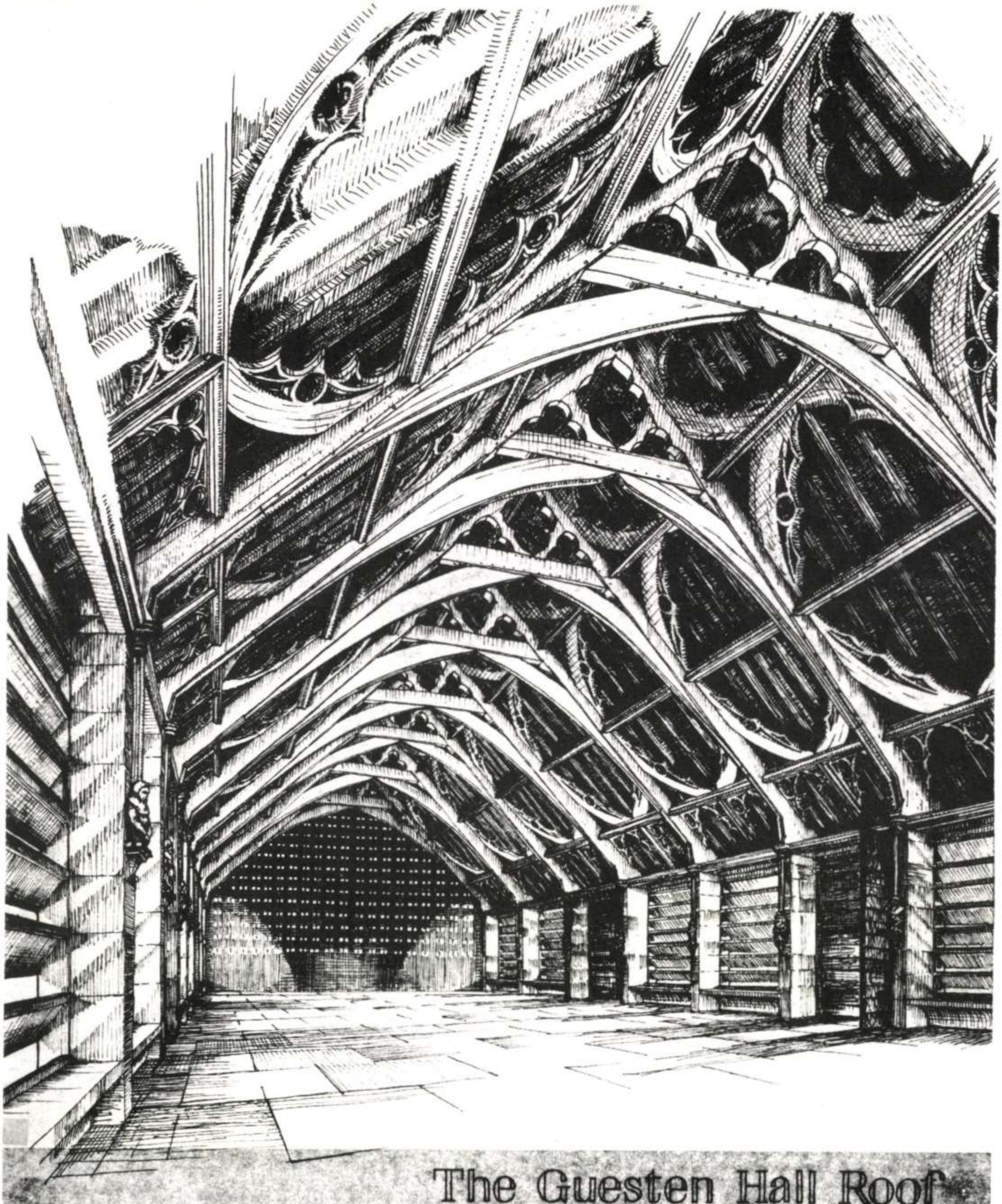


Fig. 2
The Guesten Hall roof
as it may appear on the
proposed new sub-structure
(Drawn by Freddy Charles)

roof and a series of detailed decisions concerning doors, window shutters, staircase, smoke outlet, and so on for which evidence was minimal if not absent, and finally the major operation of roof tiling.

The completed house shows not only the technical and architectural quality of the oak frame tradition, but also the true character of this type of building. There is no room to describe the construction of the building in detail, but some idea of the plan can be gathered from the photographs. I found the whole thing very exciting. The timber frame was a beautiful piece of prefabricated building construction. In those days people really understood how to use their tools. It was only possible to feel traces of adze marks by running your hand over the surface of the wood; there was nothing to be seen. The joints fitted perfectly, and the oak pegs drew the timbers tightly together as they were driven home.

There was only one possible sequence of erection for the building. The nature of the jointing determined that. So as Mr. Greiner steadily carried on with the process of reconstruction he really re-lived the original building contract. He learned to distinguish between the handicraft of the two different building teams who must have worked on the job, and he marvelled at the way the characteristics of each piece of oak timber had been studied and used to the best advantage.

Our job was to help with the re-erection of the Guesten Hall roof. This was built in the 1320's for the reception of guests at Worcester Cathedral, and it was partly demolished over 100 years ago. The roof was re-erected at Holy Trinity Church which was then being built nearby. This church is now awaiting demolition and the roof is thus threatened a second time.

The Guesten Hall roof is said to be the finest of its kind in existence. You can get some idea of it from the photograph, but don't be put off by the tie rods. The span of this old roof used to be about 36 ft. (11 m.) and the span of the church is only 29 ft. (9 m.). I am not sure whether this was because the church walls were built first and the roof was found later, or whether, as I have heard suggested, it was just that the Victorians had strong ideas about the correct pitch for the roof. Anyway the net result is that in transferring this roof to the church, every single joint has been cut and weakened and hence the tie rods.

The challenge is to design a suitable new building for the roof and to reconstruct the roof to its correct pitch. This of course means remaking all the old joints, a task which would make a lesser man than Mr. Greiner blench. So that we could make some sensible suggestions for a sub-structure which might be appropriate for this roof, we first studied the behaviour of the roof itself. You can see from the photographs that the main frames consist of principal rafters, knee braces, a collar and a pair of struts at the top. The whole thing of course is jointed with wooden pegs.

I was not at all sure to what extent this roof would depend upon lateral restraint at wall plate level. I had first thought quite rigid lateral buttressing would have been necessary, but a visit to the ruins of the original building seemed to indicate that the walls were very tall and really quite slender. We analysed this roof, and I am sure the original builders would have been amused to see us use the computer. Sure enough, it turned out that very little lateral restraint must have been provided. Only a very small horizontal movement at wall plate level is necessary before sufficient strength is mobilised within the truss.

It was very interesting to see how the number of oak pegs provided to joint the pieces compared with the theoretical requirements. I am sure the master builders who put up that roof understood their building every bit as well as we understand ours.

Design work is not complete and more money still has to be found for this project, but I look forward to the next stage enormously. There is something of great value to be learned from studying these old buildings. It is refreshing to see the loving care that went into this work and it is exciting to see how the principles which we try hard to follow in our modern buildings were applied so successfully by those craftsmen. They understood their materials and they knew just how they should build with them. The final design makes this most evident.

I recommend to all of you a visit to the Avoncroft Museum, and I hope that if you do go there Mary or Freddy Charles and Gunold Greiner are there to tell you about their work. In this short article I have used photographs given me by Freddy Charles and quoted freely from descriptions of the building which he has written. If you are interested in learning more about timber frames and, particularly, cruck construction and its derivatives there is a very interesting paper by him in our library. Finally, if you think you can persuade anyone to contribute to the Avoncroft Trust, I have in my office a large stack of descriptive pamphlets with an application for membership and blank bankers' order forms attached.

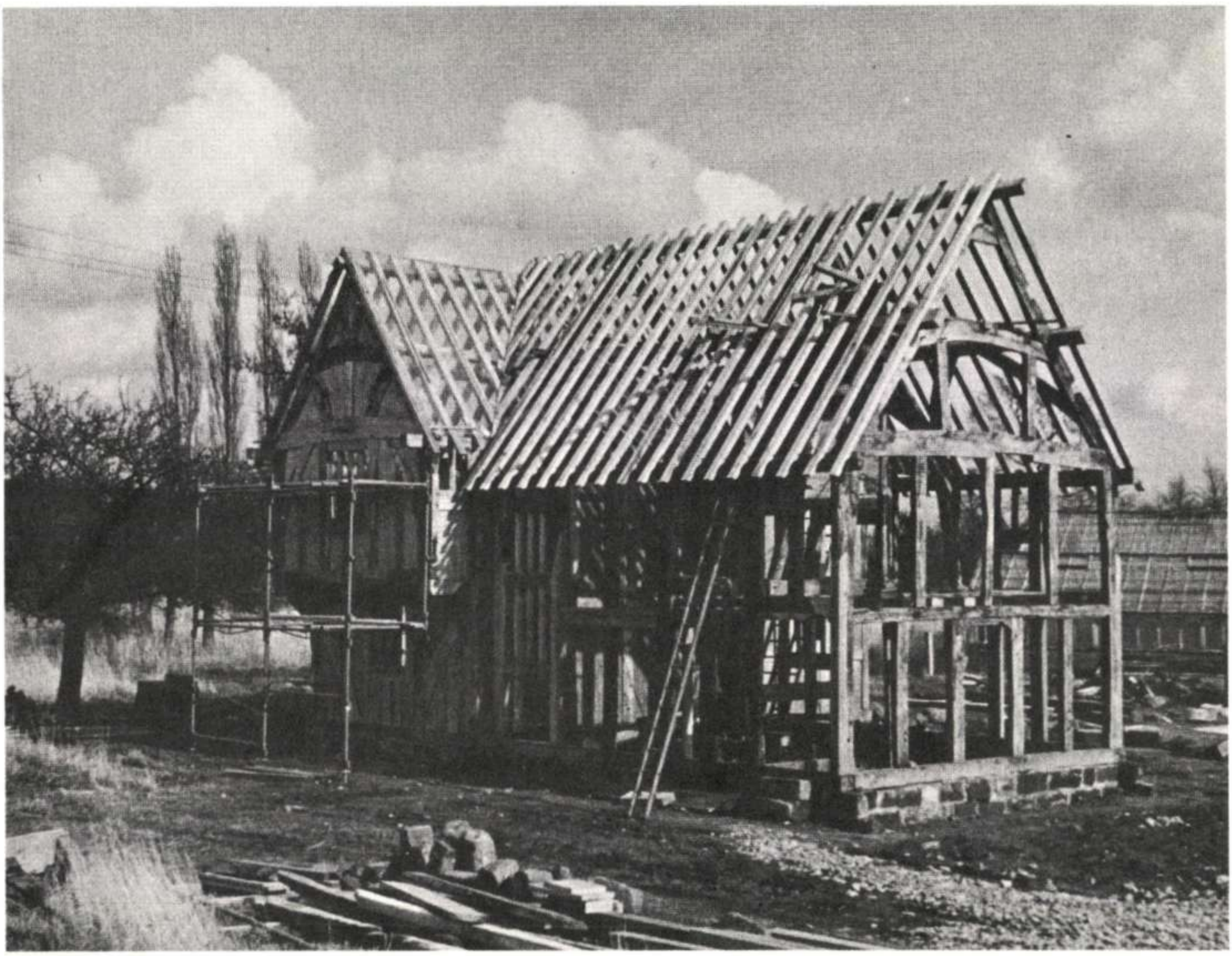


Fig. 3
The Bromsgrove House—
frame of hall nearly completed
(Photo: John Aulton)

The rebuilding of the London Stock Exchange: The tower

P. J. Thompson and
D. M. Griffiths

This article follows on from the one which appeared in the July 1968 *Arup Journal* which dealt with the foundation problems associated with the tower.

The Stock Exchange is composed of completely separate elements for design and construction purposes and, although these are interconnected at low level, the tower block is separated structurally and is considered separate. It comprises 28 floors from ground level, supported externally by a precast facade and internally by an in situ core containing all services. Owing to the general integration of the lower floors with the other buildings all floors are cast in situ up to and including the fourth floor and are generally 12 in. (305 mm.) solid reinforced concrete. From the fourth floor to the roof the external facade consists of load-bearing precast spandrel units and precast columns; below this level there are no spandrels. A section through the tower is shown in Fig. 1 which shows also the market area on the right and the post office on the left.

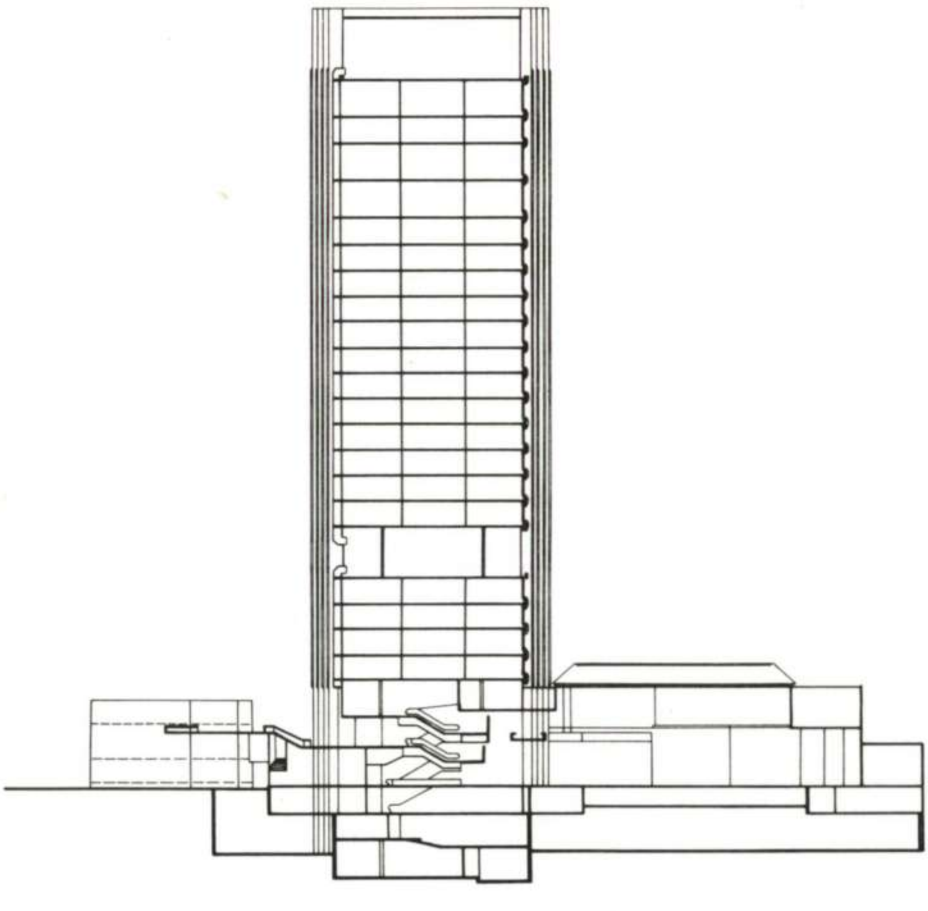


Fig. 1
Section through tower
(Drawn by Desiré Wishart)

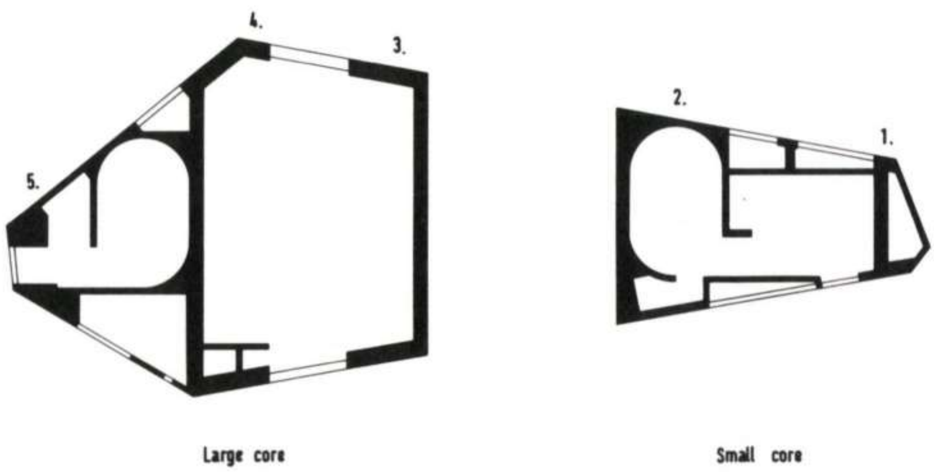


Fig. 2
Plan of cores for sliding
(Drawn by Susan Pickard)

The core

The core was to be slid. After much discussion between architects, engineers and contractors it was agreed that the core should be split. The contractor had a strong taste for an overall time-saving which would be reflected economically in the long term, although the actual cost of the construction of the core was greater. Sliding also meant that our single core now became two separate cores of different plan shapes (Fig. 2). This separation was to cause stability problems for the smaller of the two cores in its temporary condition. A high degree of discipline is required for the whole design team if a slide is to be successful and everyone's requirements must be carefully coordinated as there can be little or no alteration once the slide has commenced. As we were responsible for the stability of the towers we in fact produced drawings showing every hole, chase and inset throughout the tower height. We usually baulk at this, it being the architect's responsibility, but in this case we felt that it was reasonable for us to do this and it also helped the detailing group greatly as they could be quite sure that all holes were shown from an early date. From our calculations we found that a 4,500 lb./sq. in. (31 N/mm²) concrete would be adequate and asked the contractor to design a suitable mix. He had already sub-contracted the manufacture and operation of the shutter to a specialist firm who offered their services as concrete consultants.

For a fee, they would design a mix suitable for sliding and provide concrete control engineers on a 24-hour basis to constantly check the mix and to vary it, if it were found necessary to do so, through climatic or other reasons. We agreed to this and in due course received a report plus design curves based upon the aggregates and cements supplied by the main contractor. We found this mix to be far removed from 1 : 2 : 4 and in our opinion it is most important that concrete for this type of work should be designed by people who know and have experience of sliding, and know just what sort of factors to take into consideration. The jack-rods were placed at about 6 ft. (1.8 m.) or 8 ft. (2.4 m.) centres and the vertical reinforcement was organized to deal with the situation which arose around the jacks, namely that for about 18 in. (460 mm.) no vertical steel was possible. This created some awkward situations where we had critical points on the surface with the jack-rod running right through them. We prepared a typical layout of horizontal reinforcement which we passed to the main contractor for comment. The working space both externally and internally on a slide is very restricted and it is important that the bars are of such length that they can be fed in and adjusted with sufficient tolerance. While we were detailing the core the contractor also prepared a set of key-control drawings which told him exactly what had to happen at any one particular level, i.e., what chases had to be put in, what quantity of concrete would be required, how much steel would be required, what fixings had to be left in and also an estimate of how many men he would require at this particular time. We found that the contractor went to great lengths in preparing these drawings and when we came to the constructional side they certainly justified all the time and care that had been spent on them.

On completion of the foundation raft (see the July 1968 *Arup Journal*) the slip-form was erected. This consisted of three main levels, upper level for placing vertical reinforcement, middle level for placing the concrete into the

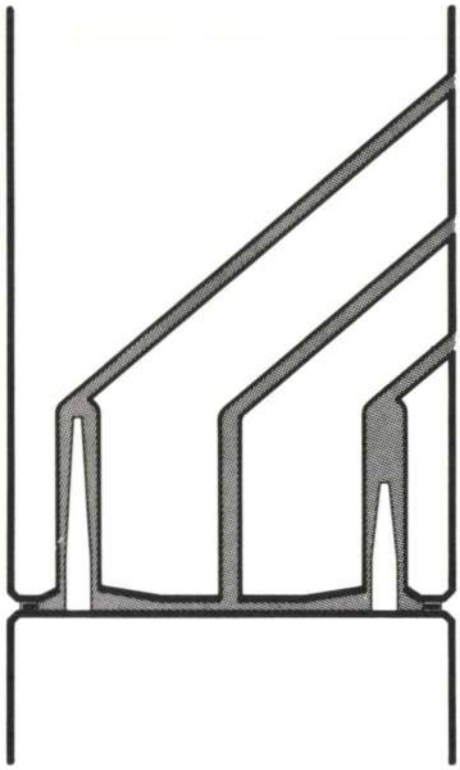


Fig. 3 (a)
Column joint,
fourth floor and above
(Drawn by Susan Pickard)

4 ft. (1.2 m.) deep shutter proper which occurs at this level, and the third and bottom level for concrete finishing. The shutter fabrication took about two to three weeks to complete. When at last we did start, our progress was very slow as the machinery installed to get the concrete from the mixer into the tower crane skip, from the skip on to the central platform, had not been designed to cope with the vast amounts of concrete required. This operation was also hampered by the fact that it was snowing and that faults developed in both cranes on site at the same time. After reaching about 10 ft. (3 m.) the whole thing was stopped and we waited for better things. About four days later we commenced again and this time we rose continuously without any major hold-ups of any description to 250 ft. (76 m.). At this level, which was just below the boom of the tower crane, sliding was stopped while a connection was made from the crane mast to the core. The crane was then extended up to its full height of about 400 ft. (122 m.) and sliding recommenced up to a total height of 368 ft. (112 m.).

The large core now being finished, the small core commenced and rose in one continuous movement from bottom to top without any hold-ups whatsoever. Some statistics:

For the large core average rise per hour of sliding: 8 in. (200 mm.)
For the small core average rise per hour of sliding: 11 in. (280 mm.)

During the construction, working shifts were organized on a 12-hour basis and concrete cubes were taken every shift. The concrete control engineers supplied by the contractor calibrated the mixer and checked the concrete on every stage of its journey. We found that the curve supplied to us by the concrete consultant was remarkably accurate when we compared the 24-hour accelerated tests with the 3, 7 and 28 days strengths we were to get. With regard to the statistics previously mentioned, we found that the rate of sliding for the large core was largely dependent upon the speed of placement of concrete, as in the large core we were only dealing with reinforcement percentages of 0.2%. However on the small core, up to the first 100 ft. (30.5 m.), progress was dependent solely upon the speed of steel fixing as, due to tension developing under wind, we had steel at about 1½% of the cross sectional area.

Columns

The precast units forming the facade consist of columns at 13 ft. 6 in. (4.1 m.) centres with horizontal precast units acting as permanent shuttering up to fourth floor level. Above this level they consist of columns with full depth structural spandrel units which transfer the floor loads to the columns.

The columns are generally 4 ft. (1.2 m.) deep of which 3 ft. (0.9 m.) is outstanding from the building. As we have a very torsionally stiff outer skin above fourth floor level and a spindly structure below, we considered that the column joints below fourth floor level should be of a different design from that above, although the external structure was not considered to resist torsion or bending below this level. All column joints occur 3 ft. above floor level. At upper levels a shallow depression formed in the ends of the column is sealed by a circumferential neoprene gasket as one column is seated on another. The ½ in. (13 mm.) deep void thus formed is filled with non-shrink grout. The grout is fed in via a cast-in tube and returns via two other tubes into which the locating spikes fit. The locating spikes are designed to allow easy passage of grout. The arrangements are shown in Fig. 3(a). The lower column joints could be termed 'spigot' joints (Fig. 3(b)); the columns are cast with a tapering spigot on top and a corresponding void in the bottom. Clearances between the two concrete surfaces when the columns are mated are 1½ in. (38 mm.) on the long sides and 2 in. (50 mm.) on the short sides. This void is again filled with non-shrink grout.

The grout used is called *Embeco LL.62*. It is imported from the United States and costs about £5 for 0.7 cu. ft. (0.02 $m.^3$). The main expanding constituent is iron. It is capable of very high strengths up to 14,000 lb./sq. in. (96.5 $N/mm.^2$) and has very good non-shrink properties. Messrs. Sandbergs have carried out a series of tests for us on this grout which substantiate the maker's claims. We were forced to use this non-British material after many attempts to achieve the strength we required, i.e., 8,000 lb./sq. in. (55.2 $N/mm.^2$) for a 2 in. (50 mm.) cube at 28 days, using other products under the site conditions. We found that they would all give us 8,000 lb./sq. in. with no shrinkage under laboratory conditions but were unable to beat 6,000 lb./sq. in. (41.4 $N/mm.^2$) on the site.

Spandrels

The spandrel units are floor to floor units moulded to form two windows one each side of a vertical service duct with horizontal feeders to air conditioning units under the cill. The spandrels are fitted into the columns from the inside. They are supported at their head by temporary steel T's which are bolted to the spandrel and which sit on the precast columns. The arrangement is illustrated in Fig. 4. After levelling, the permanent connection is made to the column at the foot of the spandrel. This connection is formed by an outstanding shear key which fits into a chase in the column, the gaps left being dry-packed. The precast units are manufactured in the main contractor's precasting yard at Camberley. For the tower block they consist of 768 columns of various types together with 768 spandrels. The units are cast in steel moulds.

The aggregate-cement ratio is 4:1 and the water-cement ratio is .48. All aggregate is *Whitwick Granite*. Stearic Acid was added on the recommendation of the Cement and Concrete Association for waterproofing purposes and *Wargonin Compact* was used as a plasticizing agent. This became critical when trying to achieve a high degree of compaction in some of the areas of reinforcement congested by the spandrel units. From the moulds the units are taken to a grinding shop where sufficient grinding is applied to remove the external concrete paste film. The resulting finish could be termed 'reconstituted stone' and is of a very high quality. The majority of units (both spandrels and columns) weigh 5 tons (5.08 tonnes). This factor created difficulties on site.

Floors

The plant room floors are situated mainly at the top of the tower and are generally 15 in. (380 mm.) reinforced concrete cast in situ. The typical floors have coffered slabs at either end, in situ solid slabs at the corners and precast planks elsewhere; a typical floor plan is shown in Fig. 5. The precast sections of these floors are 9 in. (230 mm.) deep prestressed planks with 2 in. (50 mm.) of in situ structural topping. The planks are terminated 9 in. short of the bearing at each end and the junction is made with a full depth in situ joint, approximately 9 in. wide (Fig. 6.). Although on paper this joint appeared perfectly sound we had not

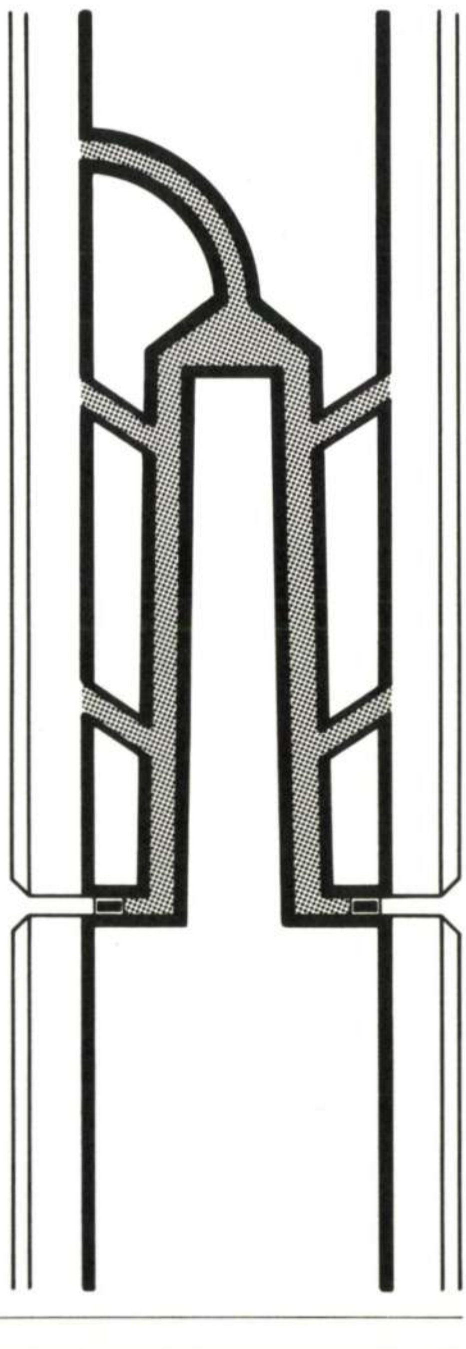


Fig. 3 (b)
Column joint,
below fourth floor
(Drawn by Susan Pickard)

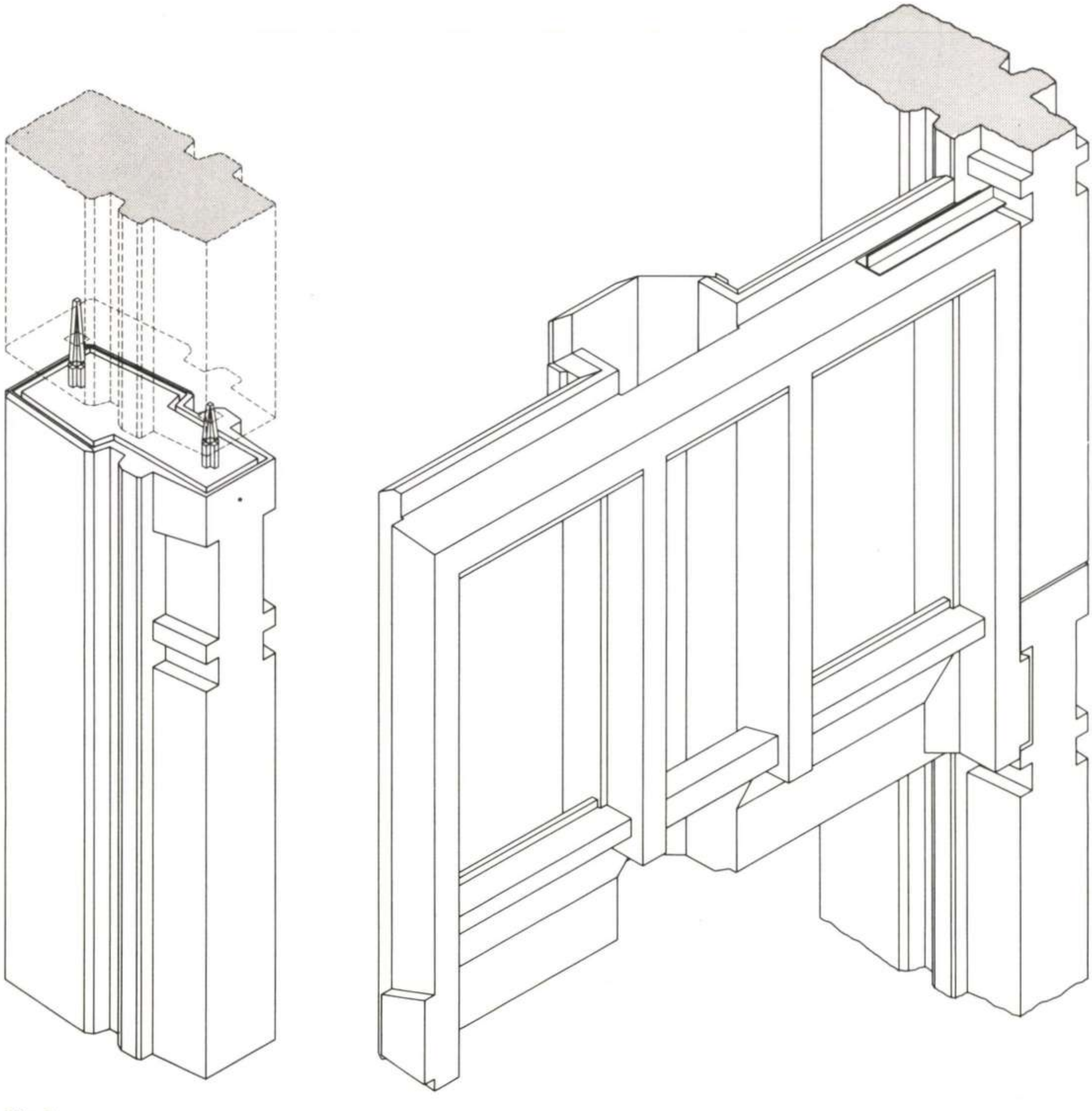


Fig. 4
Column/spandrel assembly.
(Drawn by Susan Pickard)

used it before and we had a full scale test carried out as part of the contract. It is worth noting that this testing cost approximately 1½% of the total cost of the precast flooring. Testing was straight-forward, the design load was maintained for 2½ hours and then loaded to failure. The slab was remarkably stubborn and refused to fail but destroyed our simulated support at about four times the design load. Needless to say we were reassured.

Special problems

The tower structure differs from most in that its plan shape is unsymmetrical and the perimeter columns protrude for more than ¾ of their surface area from the face of the tower. These two features pose problems of deflection, stability and temperature stresses.

Wind on the tower

The plan shape of the tower, shown in Fig. 5, with a large wind face of 172 ft. (52.4 m.) and an average core width of only 28 ft. (8.5 m.) did not evolve from structural considerations. However the core walls are pretty solid and it is they which resist the direct bending due to wind.

A statistical approach was used to evaluate the wind forces acting on the tower using observations made at Croydon over the last 27 years. A 100-year recurrence period was taken and a 15 sec. gust duration was assumed. Based on a 15 sec. gust, velocities of 82.8 mph (133 Km./h.) at roof level (325 ft. (99.1 m.) above ground) and of 51.6 mph (83 Km./h.) at 33 ft. (10 m.) above ground were found. These figures were higher than the mean hourly values and were used to calculate pressures. The resulting wind pressures were approximately 35% greater than those derived from the Code of Practice. The wind pressure profile is shown in Fig. 7. The core was checked for a 200-year recurrence period and found to be satisfactory; i.e., the resulting maximum stresses were found to be within the allowable increase of 25%.

Core walls 1 and 2 (Fig. 2) were assumed to act compositely in resisting wind, as were core walls 4 and 5, and wall 3 to act on its own. The wind moment of 208,000 kip.ft. (282,000 kNm.) at the level of the top of the ring beam (see July 1968 *Arup Journal*) was thus resisted by three groups of walls in proportion to their flexural rigidities. This moment was a maximum at this level reducing further down, due to the resistance of the surrounding earth to horizontal movement of the diaphragm wall. This reduction in moment was accompanied by tensile and compressive forces in the floor slabs in the region of the ring beam.

The maximum deflection of the tower due to direct bending was calculated at 3¼ in. (83 mm.). In the temporary conditions, after the two cores were slid and before any floor slabs were constructed, the smaller core was calculated to deflect 4½ in. (114 mm.) and tension stresses were developed over the lower 80 ft. (24.4 m.).

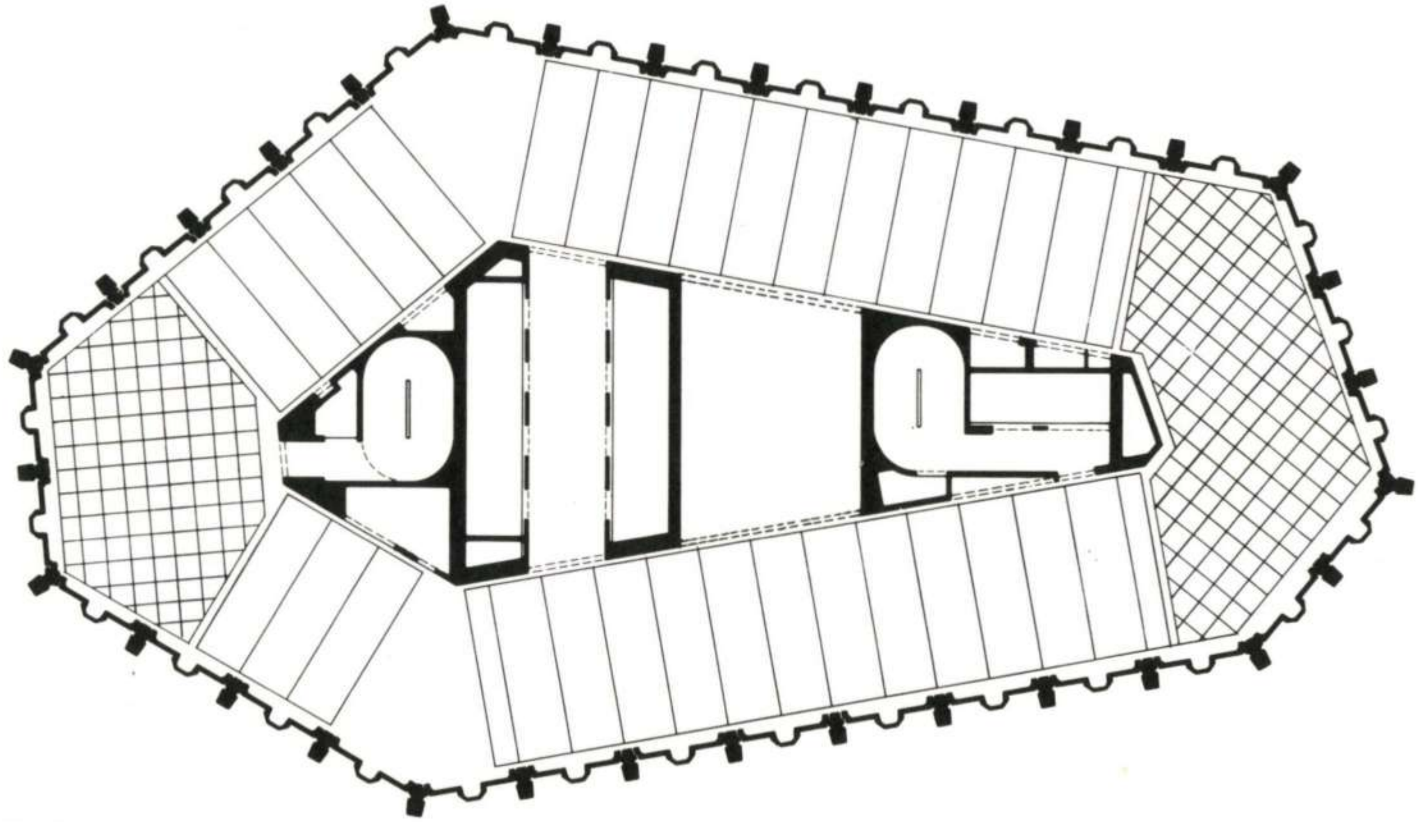


Fig. 5
Typical floor plan
(Drawn by Susan Pickard)

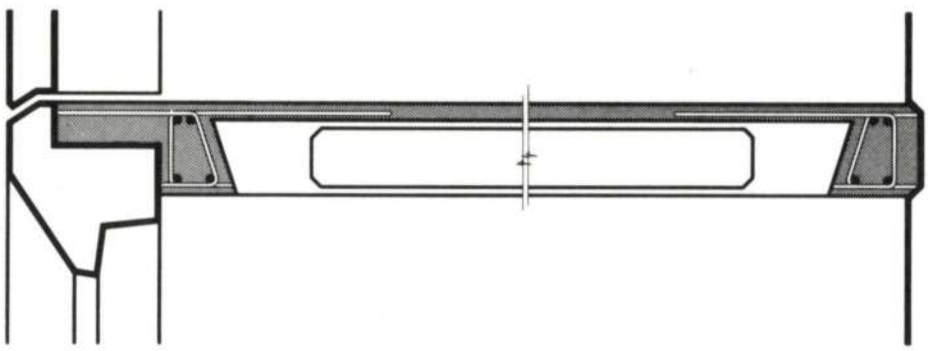


Fig. 6
Section through pre-cast floor
(Drawn by Susan Pickard)

Because of its unsymmetrical plan shape the tower twists as well as bends under wind loading. The maximum torsional moment at the raft was nearly 18,000 kip. ft. (24,400 kNm.), the wind force being eccentric to the shear centre of the core by 17 ft. (5.2 m.). The core walls are open sections and in themselves possess relatively little torsional rigidity. However, from fourth floor level to the roof, the precast spandrel units and columns form a large continuous ring around the tower as is indicated in Fig. 5. The spandrel bears on the column about 3 ft. (0.9 m.) below the spigot joint (Fig. 4). The floor slabs are cast onto the top of the spandrel covering the T section shown, and around the inside face of the column. There is a dry joint between the spandrel and column. Thus a complete ring is formed around the tower for each spandrel height and also there is continuity between one level of spandrels and the next. Hence the outside of the tower, from fourth floor level to the roof, acts as a 'cylinder' in resisting torsion. The torsional rigidity of this cylinder was found to be nearly 100 times that of the core walls when considered as separate units, that is not including the interaction of the beams and slabs. This interaction effect would not increase the effective torsional stiffness of the core by anything like a hundredfold and consequently the applied torsional moment above fourth floor was assumed to be taken entirely by the spandrels. The total angle of twist over this height was 2.02×10^{-7} rad. giving a maximum average circumferential movement to the rig of 0.17 in. (4.3 mm.).

Below the fourth floor level there are no spandrels present and so the torque has to be resisted by the core and outer columns. As the columns have negligible torsional stiffness as compared with the core, this means that the entire torque can be assumed to be taken by the core.

With this assumption the first approach was to consider the core walls as separate units but to allow for the increase in effective torsional stiffness of the walls due to the proximity of the fixed end at the raft level which prevents warping of the section at that level. This approach, which gave reasonably low rotations, did, however, result in unacceptably high bending stresses in the flanges of the core wall channel sections. Consequently, a more refined analysis was carried out.

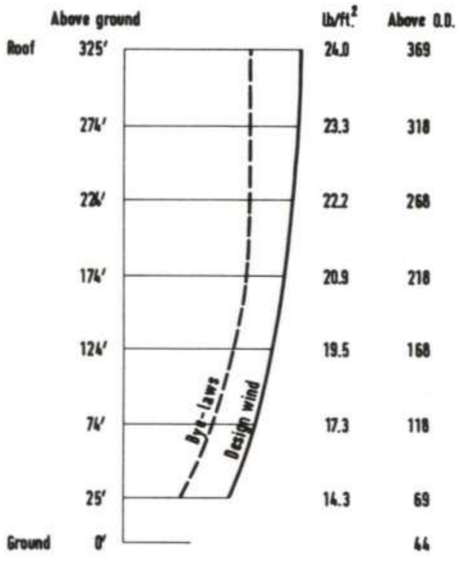


Fig. 7
Wind pressure envelope on tower
(Drawn by Susan Pickard)

The assumption was that the two largest core walls, walls 3 and 4, and their connecting 30 in. x 12 in. (762 mm. x 305 mm.) beams between the flanges acted compositely and resisted the entire torque. With this assumption the total rotation below the fourth floor was calculated as 8.4×10^{-4} rad. which was smaller than the value previously obtained and the bending stresses in the channels were small (less than 100 lb./sq. in. (0.7 N/mm.^{2})). The analysis was deemed to describe the twisting of the core reasonably well; the calculated rotation below fourth floor level is shown in Fig. 8. The interconnecting beams were reinforced to take the shear and double curvature bending due to warping of the walls, these values increasing from zero at the fixed end to a maximum at the fourth floor.

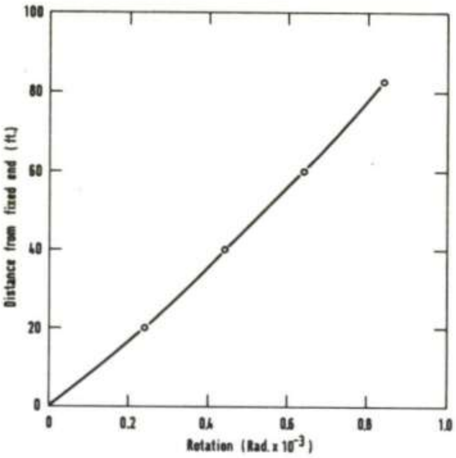


Fig. 8
Rotation of floor
below fourth floor level
(Drawn by Susan Pickard)

Fig. 11 above
View of tower
from Threadneedle Street, showing
construction stage at December 1968
(Photo: Sydney W. Newbery)

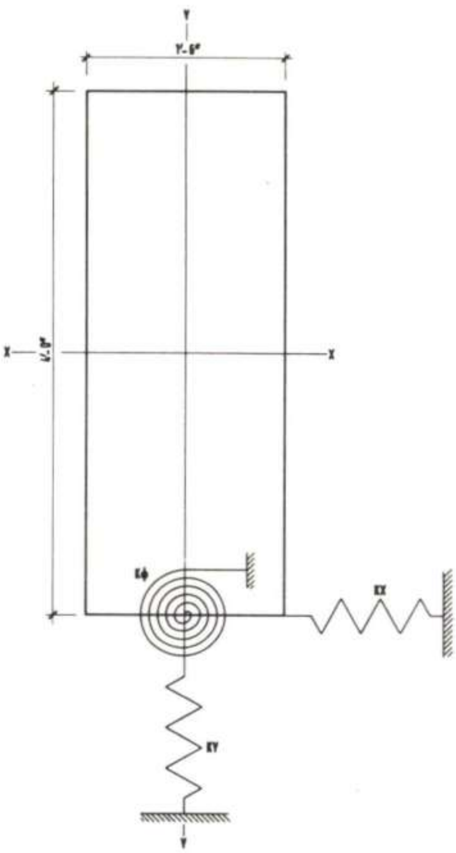


Fig. 9
Column restraints
(Drawn by Susan Pickard)

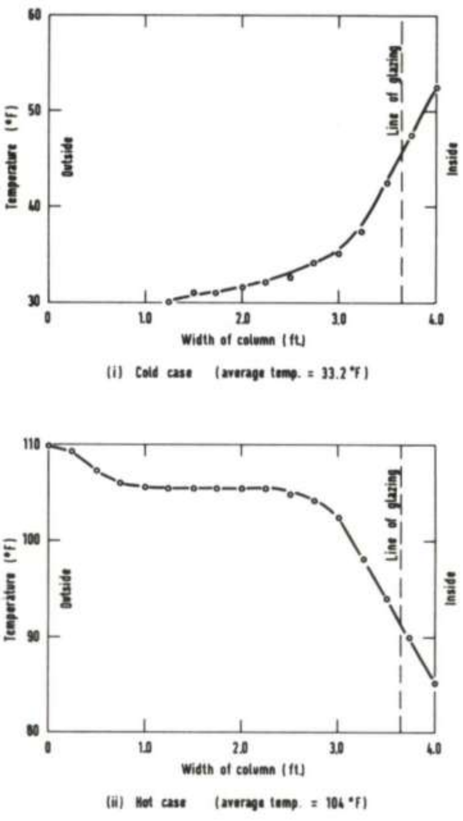


Fig. 10
Temperature distribution across column
(Drawn by Susan Pickard)

The maximum horizontal movement of the top of the tower relative to the base due to the combined effects of bending and warping was calculated as 4.1 in. (104 mm.).

Stability of the columns

The columns protrude 3 ft. from the spandrels from fourth floor level to the top; the columns over this height are 4 ft. (1.2 m.) long and 18 in. (460 mm.) wide. Hence these columns are 270 ft. (82.2 m.) long with three-quarters of their section free of any restraint.

The columns were investigated for stability and a simplified model of a rectangle with restraints at one end was analysed, see Fig. 9. The springs, K_x, K_y, K_ϕ shown represent the stiffnesses of the spandrels and floor slabs (or steel bars which tied the columns to the core at each floor level) against linear and rotational displacements. For such a section, with two axes of symmetry and where the centroid and shear centre are coincident and where the member is axially loaded and restrained as shown, two separate buckling modes are possible. They are:

1. Buckling about the major axis x-x.
2. Combined torsional buckling and buckling about the minor axis y-y.

The safety factor against mode 2 turned out to be nearly 100 but mode 1 was not very well represented in the analysis as continuous restraints had been supposed and the y-restraint only occurred at the floor slabs. As

Fig. 12
Precast column
and spandrel units
(Photo : Sydney W. Newbery)

mode 1 was independent of mode 2 it could have been examined separately and a more realistic model with spring supports (y) at discrete points, i.e., at each floor slab, was analysed for the major axis buckling case. The stiffnesses of the restraints were not sufficient to limit the half wave buckling mode to a storey height, but even so the safety factor against buckling about the major axis was more than 200.

Temperature effects
The columns are exposed for three-quarters of their surface area and the building is centrally heated, so there will be a temperature difference between the columns and tower, and hence a temperature gradient across the columns. The two basic effects resulting from column exposure are bowing and axial movement and when either of these is restricted, stresses will develop in the columns and in the resisting structure.

The columns are exposed aggregate on the outside but the inside face has 1 in. (25 mm.) covering of block board. The internal temperature was assumed as 70°F. (21.1°C.) and two external cases were considered :

1. Cold case with air temperature of 30°F. (−1°C.) plus a wind of 30 mph (48.3 km./h.).

2. Hot case with air temperature of 110°F. (43.3°C.).

The temperature distributions across the columns for the two cases are shown in Fig. 10. The resulting maximum bending stresses which develop, allowing axial movement but no bowing, are of the order of 500 lb./sq. in. (3.45 kN/mm².) for the two cases (compression for cold case, tension for hot) and the resulting moments are roughly 250 kip. ft. (340 kNm.). These moments are taken out at the base, and at the roof the moment can be distributed throughout the frame. Throughout the height of the structure the bowing affects the frame only where the section changes its depth ; otherwise equal and opposite moments occur at the slabs. The resistance to axial movement is provided by the slabs and partitions. The slabs are in the main precast as described and are simply connected and so provide only a little restraint to axial movement and can accommodate relative movements of their ends. Any restriction on axial movement would result in stresses in the columns which are additive to the bending stresses. However the restraints provided by the structure to axial movement were assumed to be negligible. The axial movements for the two temperature conditions were a decrease in length of 0.84 in. (21.3 mm.) for the cold case and increase in length of 0.78 in. (19.8 mm.) for the hot case. Hence any floor to ceiling partitions extending from core to column at the higher levels must be flexible enough to accommodate these movements.

Note
At the time of going to press, the cores have been slid, and the floors, columns and spandrels have reached the half-way stage.

Acknowledgements
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Llewelyn-Davies, Weeks, Forestier-Walker & Bor

Contractor : Trollope & Colls Ltd.
Quantity Surveyor : Gardiner & Theobald

Radio mast for Durham County Council Police Headquarters, Aykley Heads, City of Durham

Ken Anthony

The brief

The function of the radio mast at Aykley Heads is to enable the County Police, at their new headquarters on the edge of the City of Durham, to communicate with their forces and with other divisions. In order to meet these requirements the level of the top of the mast was fixed at 471 ft. (143.5 m.) above Ordnance Datum.

To preserve the setting of Durham Cathedral and its surroundings on the horizon, the City planning regulations normally require that no proposed development should exceed the general height limit of 370 ft. (113 m.) above Ordnance Datum, nor that any structure should create a point of interest or distraction on the horizon formed by the 'lip' of the basin in which Durham is situated. As the mast was to exceed this limit, the brief called for a design sympathetic to the environment, and a slender, relatively inconspicuous structure was envisaged.

The design

The operational specification required that the maximum angular rotation of the mast in the vertical plane be limited to less than ½° at a 50 mph (80 km./h.) wind speed. A steel lattice mast could, of course, have fulfilled this criterion but would not have been acceptable aesthetically. Initial sizing calculations based on this criterion led to the view that a concrete structure could be sensitive to the higher frequency components of wind loading, and a statistical analysis of local wind data was therefore undertaken, resulting in a one second gust velocity of 90 mph (145 km./h.) being adopted as the basis at an average annual probability of 0.01. It was more desirable to have the mast superimposed on a tripod as opposed to a pure shaft as this avoided high bending moments at ground level, and not only gave a higher natural frequency but also led to a more elegant form. On the basis of preliminary calculations, Yuzo Mikami developed a design to an architectural standard acceptable to both the client and the Royal Fine Art Commission.

With a drag factor of 2.0, the design pressure amounted to 41 lb./sq.ft. (1963 N/m²). A spectral analysis was carried out to determine the probability of a peak structural response occurring within the design wind speed range, but this was found to be very small and could therefore be discounted. The reason for this is the inherent stiffness of the mast, and hence its relatively high natural frequency (approximately 2.6 cps in the primary mode), the peak structural response lying in the tail of the spectrum where the energy input is small.

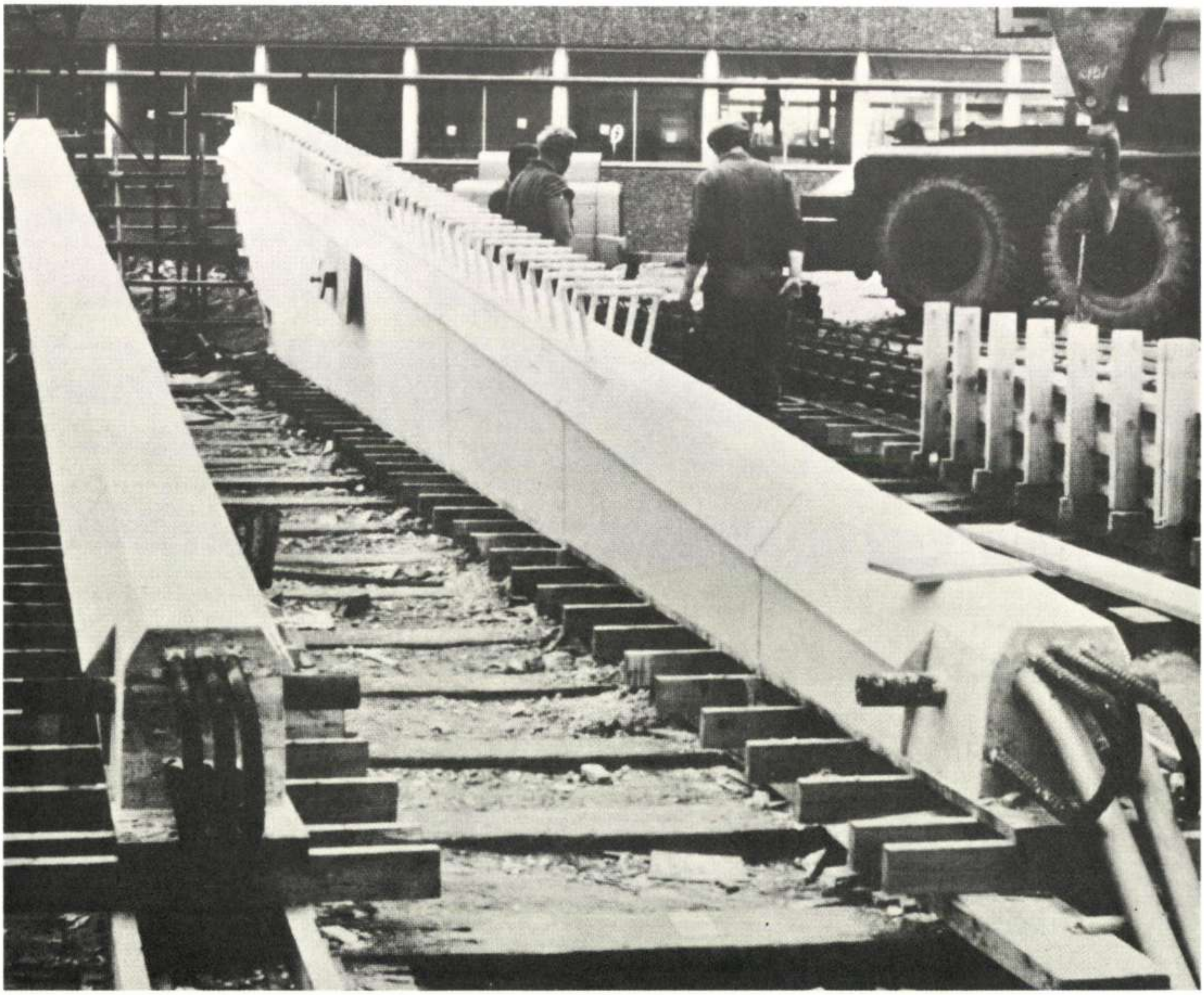
It was decided from the start that the structure should be precast in five elements, these being the mast unit, the key, and the three leg units, and that these should be cast on site immediately adjacent to their final erected position. In this way, full height scaffolding and formwork was avoided, as well as the need to make this temporary construction secure against wind effects.

All casting could then be carried out easily and safely at ground level. Apart from these advantages, it was felt that the use of the necessarily high capacity craneage for a very short period would be cheaper in the long run and more attractive to tenderers.

It was found that a prestressed concrete

Fig. 1

The leg units as cast on the site. Note the twin PVC ducts for aerial cables (Photo: Ken Shaw)



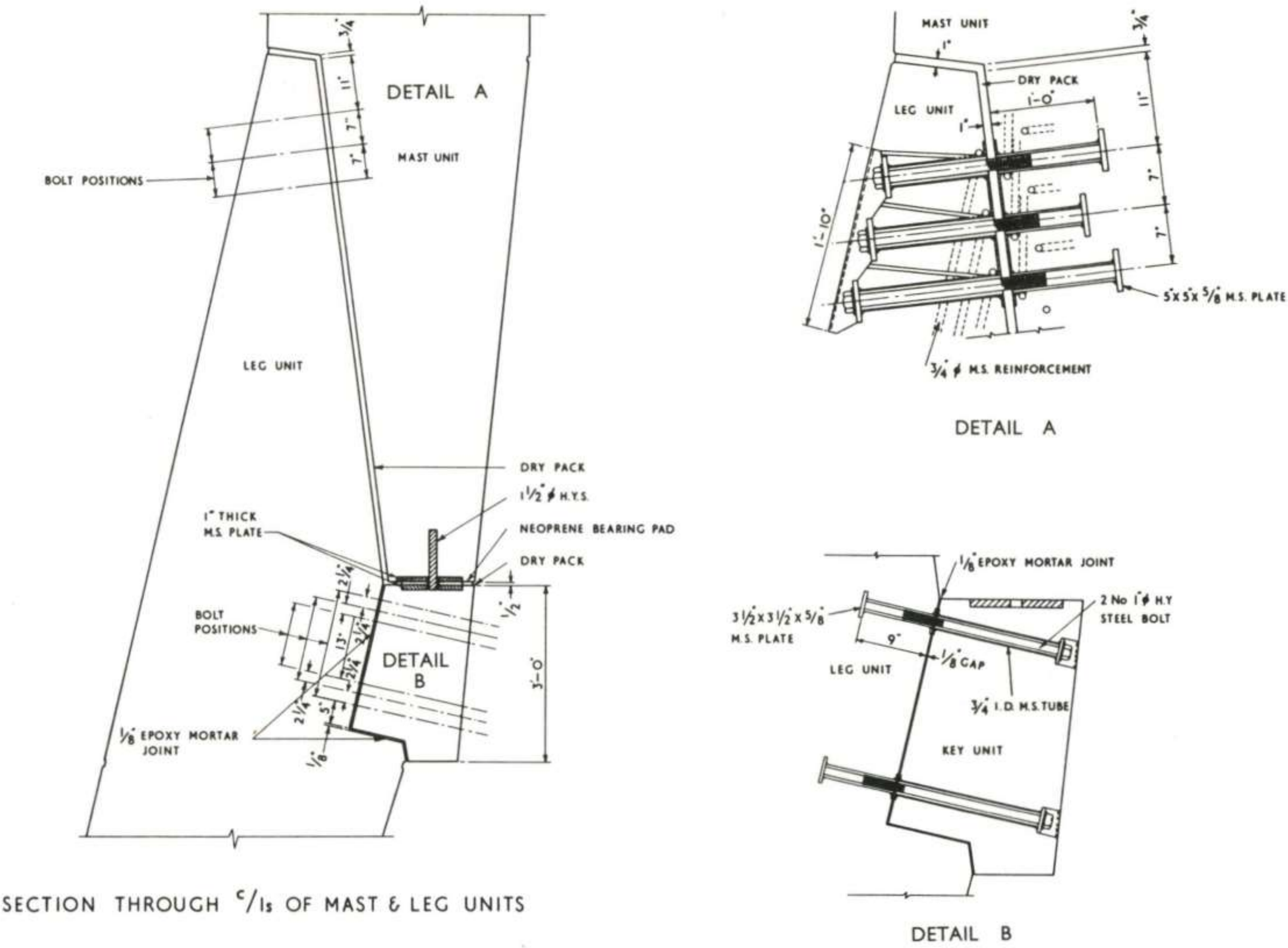


Fig. 2
Radio mast, Durham. Section and details

solution was not necessary as a reinforced concrete structure to this design limited the rotation sufficiently even at 90 mph.

The structural analysis was carried out with the aid of the computer, using a three dimensional frame programme assuming each concrete unit to be composed of several sections of varying flexural and torsional rigidity.

Construction

The foundations for the aerial comprised three in situ inverted T strip footings which formed the sides of an equilateral triangle and were monolithic with foundation blocks at each corner. Each of these blocks has a deep dovetail pocket with 'shelves' on which rest 2 in. (50 mm.) diameter High Yield steel pins cast into the foot of each leg, thereby allowing rotational adjustment of each leg prior to concreting the pockets. (See Fig. 1.)

The leg units were lifted straight from their casting beds, so that when fully supported by the crane sling they assumed an attitude slightly steeper than that required finally. The foot of each unit was placed in its foundation pocket and the entire member then lowered on to a light guyed scaffold tower about 60 ft. (18.3 m.) high.

The three faces of the key unit which would finally be in contact with the legs were primed and an epoxide mortar compound applied. It was then raised into position and each leg lowered against it and locked by means of six High Yield bolts. The shape of the tops of the legs and key unit provided a 'cradle' into which the mast unit could be lowered. (See Fig. 4.) Later, when the key joint had cured, the mast unit was lifted from its casting bed, which had been located so that the two 105 ton (107 tonnes) Coles *Centurion* cranes with 175 ft. (53.3 m.) jibs could manipulate the member from fixed positions, the base of the mast being supported by a smaller mobile crane. Each *Centurion* was carefully controlled so that their loads were equally divided. Upon reaching the vertical position, the mast was lowered on to a neoprene pad on the key unit. Small steel shims were placed between the mast spigot and the upper ends of each leg, and three High Yield steel bolts per leg tightened. The remaining 1 in. (25 mm.) gap was subsequently dry-packed with 1.3 cement-sand mortar. The mast lifting bracket was purpose-made of heavy steel welded plates and tested to twice the working load prior to use.

The aerial cables are accommodated in twin chases on the south side of the mast and lead down one leg until about 12 ft. (3.7 m.) above ground level where they are taken inside the leg in two 2½ in. (63 mm.) diameter reinforced PVC ducts to an underground chamber and thence to the police radio room. This avoids the necessity of covering surface fixed cables against accidental damage or vandalism.

The cables and aerials are serviced from specially formed 1 in. diameter zinc sprayed solid mild steel rungs. All aerial mountings are of zinc sprayed scaffold tube, and the lightning conductor is of aluminium to avoid concrete staining.

The height of the concrete structure is 162 ft. (49.4 m.) above ground level, and totals 165 ft. (50.3 m.) when the top aerial spigot mounting is included. The respective lengths of the mast and leg units are 119 ft. (36.3 m.) and 60 ft. (18.3 m.) and weigh 35 tons (35.6 tonnes) and 13.5 tons (13.7 tonnes) each respectively. The key unit weighs 0.35 tons (0.36 tonnes).

The concrete specified was 4,500 lb./sq. in. (316 kg./cm.^{2}) at 28 days with white ordinary Portland Cement, and the cube strengths attained were 5,250 lb./sq.in. (369 kg./cm.^{2}) at 2 days, 7,500 lb./sq.in. (527 kg./cm.^{2}) at 7 days and over 10,000 lb./sq.in. (703 kg./cm.^{2}) at 28 days.

The total cost of construction was £10,000. This, of course, is expensive for a mast of this height. A major portion of this cost was expended in the construction of the shuttering on which joiners rather than carpenters were employed because of the high degree of accuracy called for. Given sufficient repetition and a contractor who had done it before, there is no doubt that the basic cost per structure could be substantially reduced.

In this particular case, the appearance was a major factor. Although I don't think they should be, when the nature and the relative importance of the design criteria are different, cheaper one-off solutions are no doubt possible. It seems to be a continuing fact of life that good design costs money. Anyway, both we and the clients are very proud of the mast, and its success is due, in no small way, to Bierrum's, the contractors, who built what the *Guardian* called, 'a thing of beauty'.

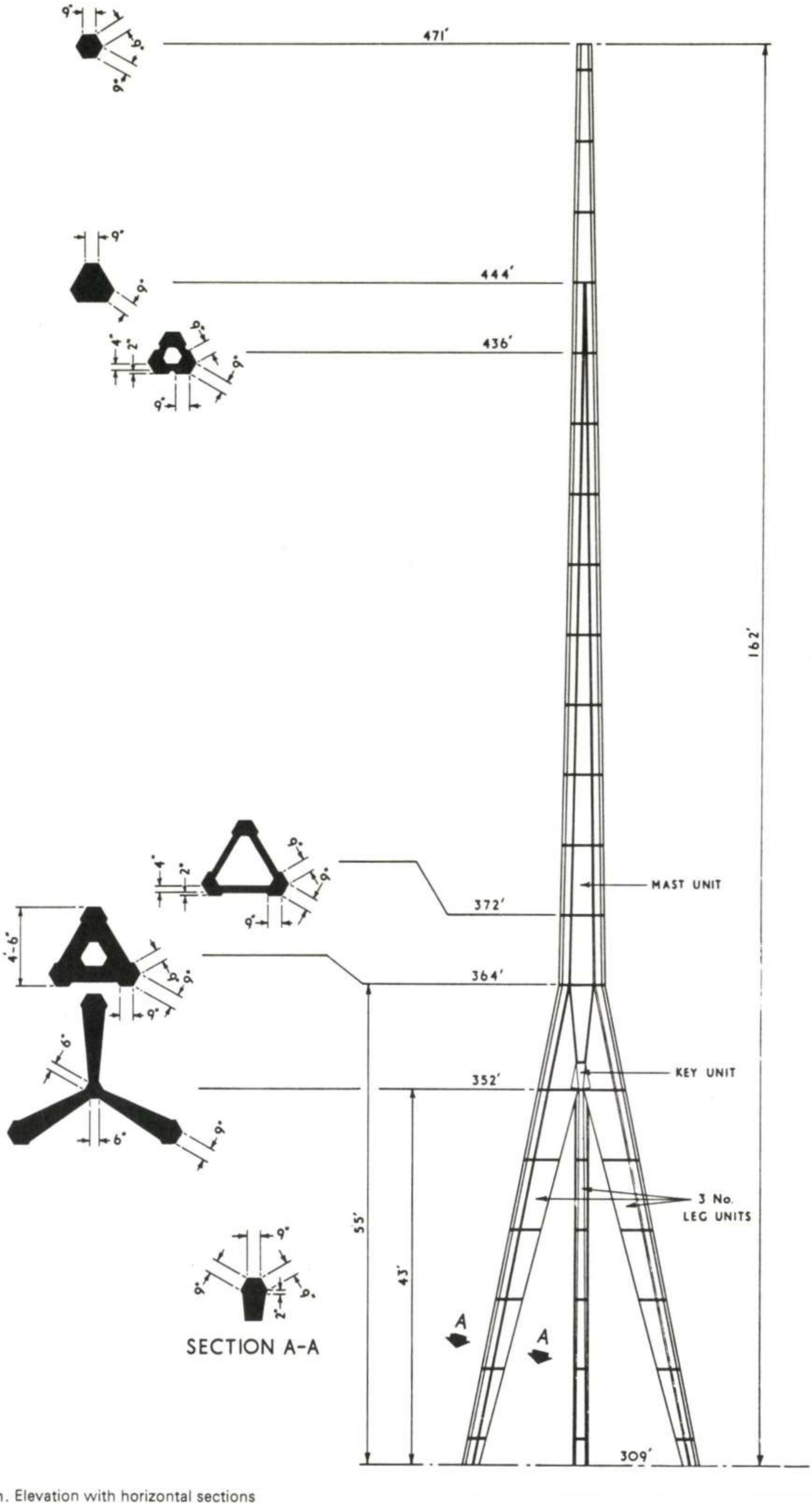
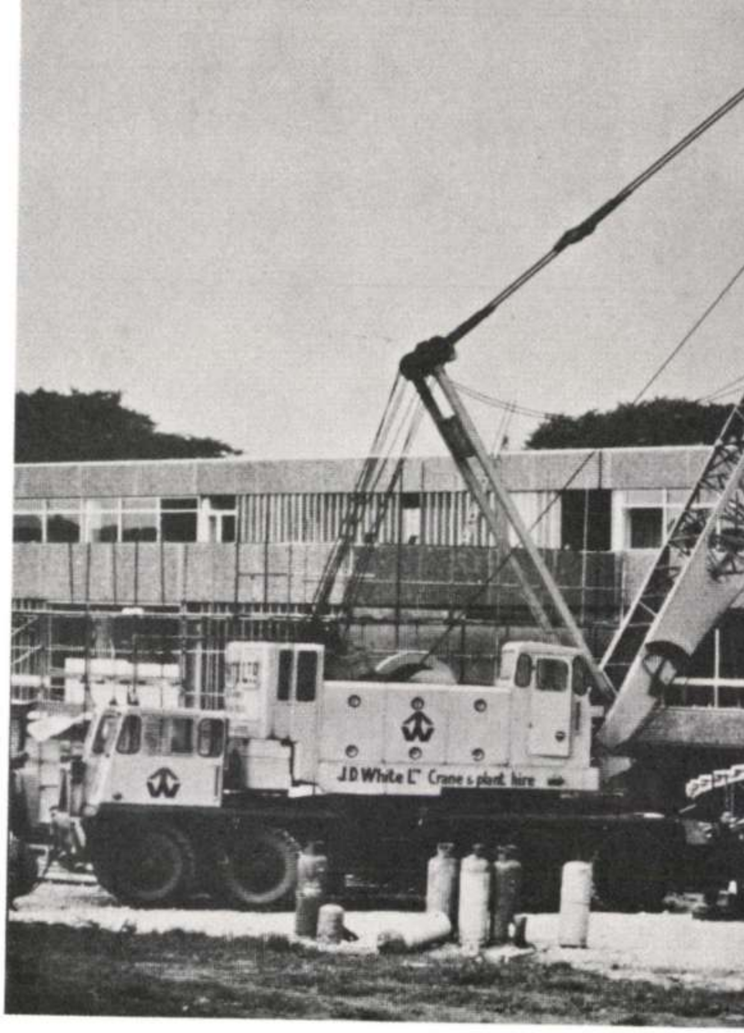


Fig. 3
Radio mast, Durham. Elevation with horizontal sections



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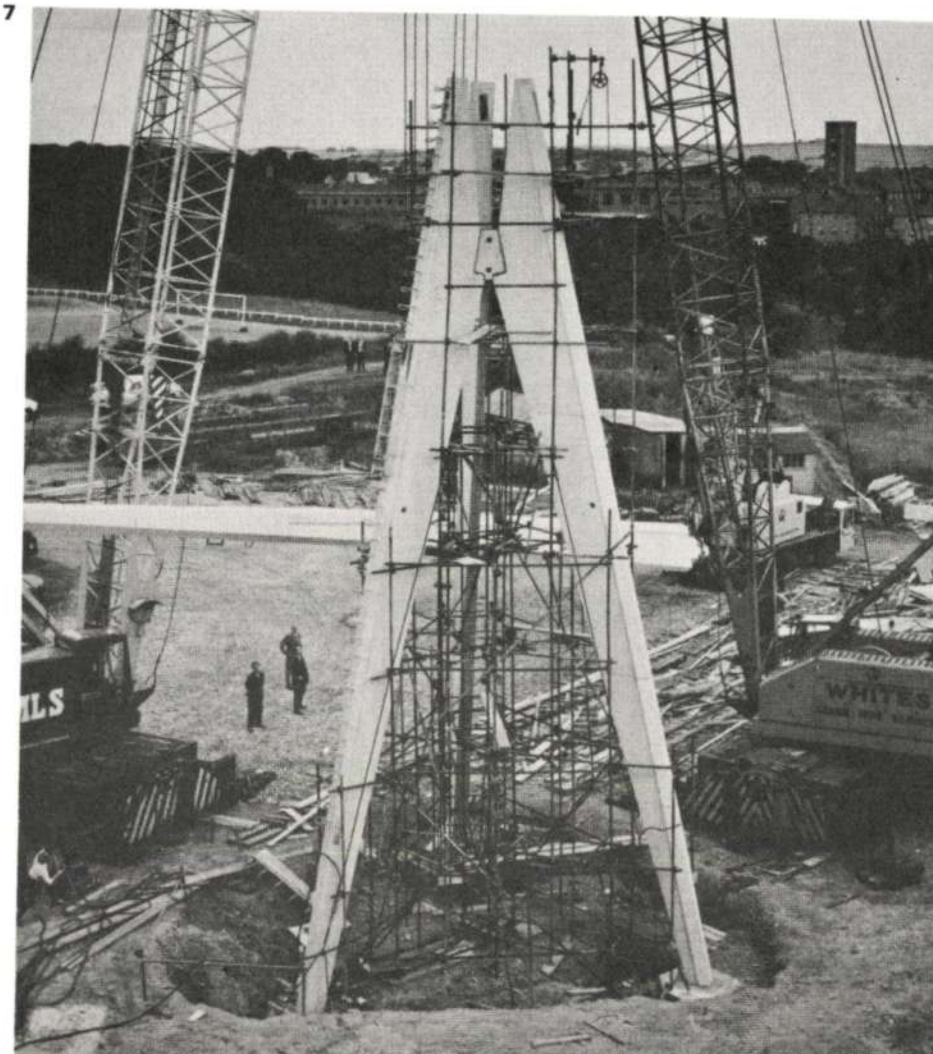

Radio mast - Durham: sequence of erection

Fig. 4
One of the legs being hoisted into position (Photo: Ken Anthony)

Fig. 5
The south leg being manoeuvred against the scaffolding (Photo: Ken Anthony)

Fig. 6
The Key unit (Photo: Ken Shaw)

Fig. 7
The completed tripod with mast unit on its way in the background (Photo: Turners (Photography) Ltd., Newcastle)

Fig. 8
The mast lifting bracket (Photo: Ken Anthony)

Fig. 9
A further stage. The main cranes have yet to slew towards each other. Note the smaller mobile crane supporting the lower end of the mast (Photo: Turners (Photography) Ltd., Newcastle)

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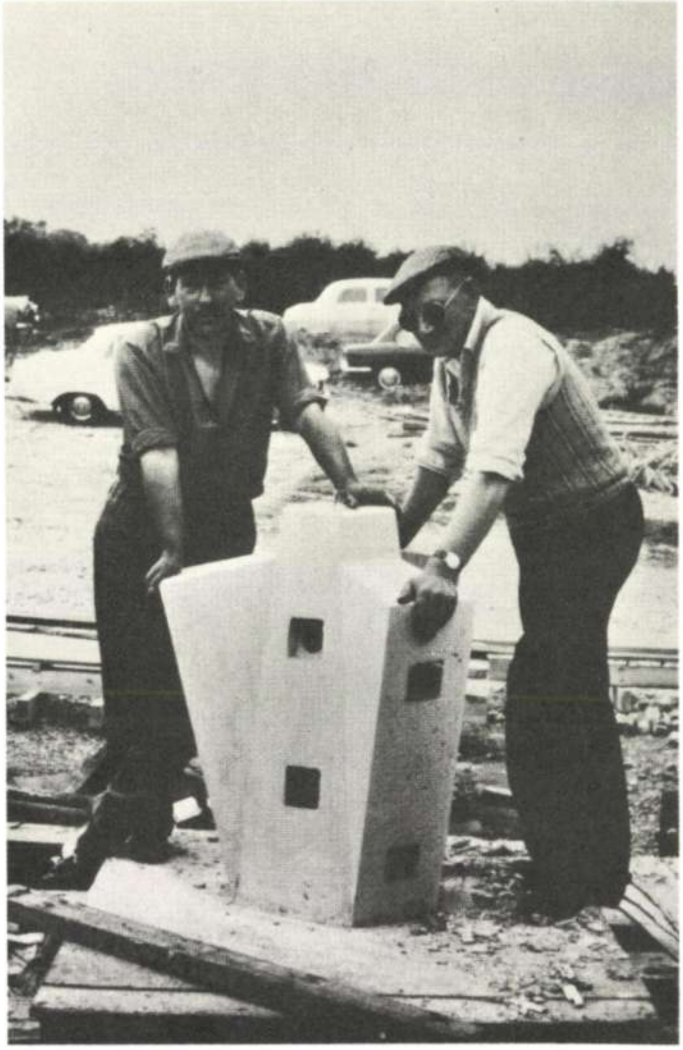

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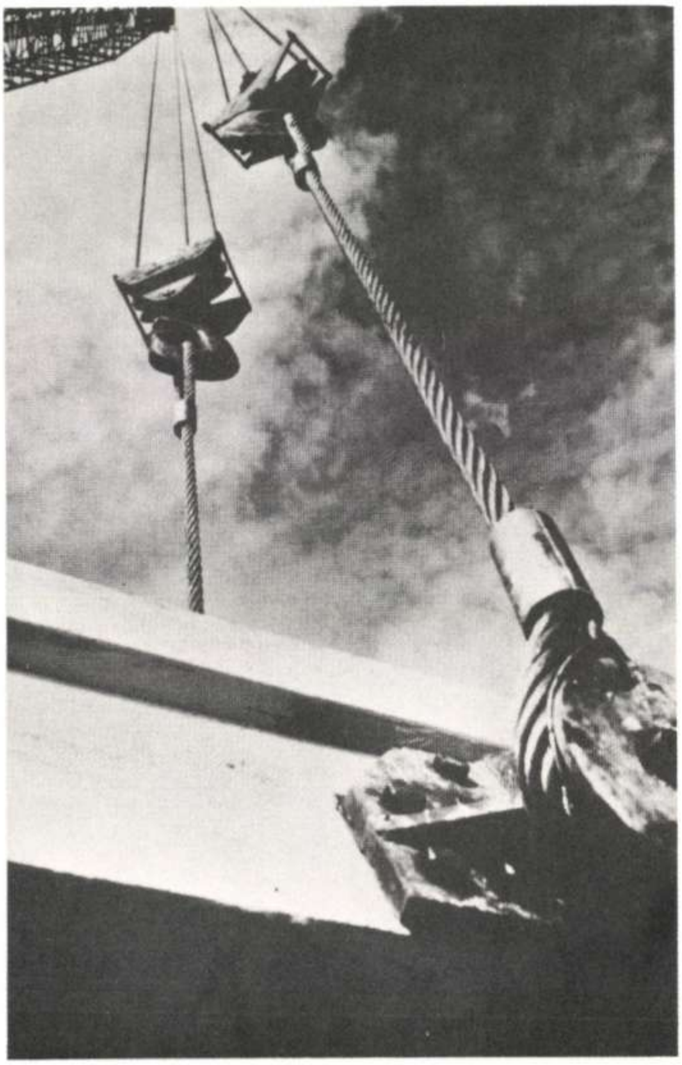

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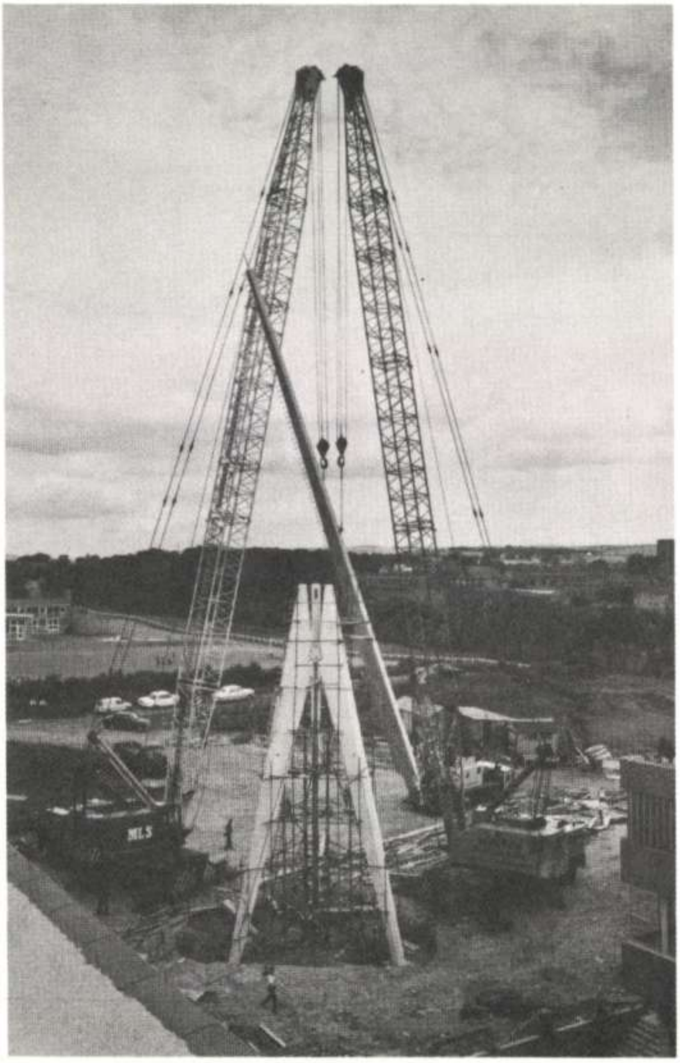

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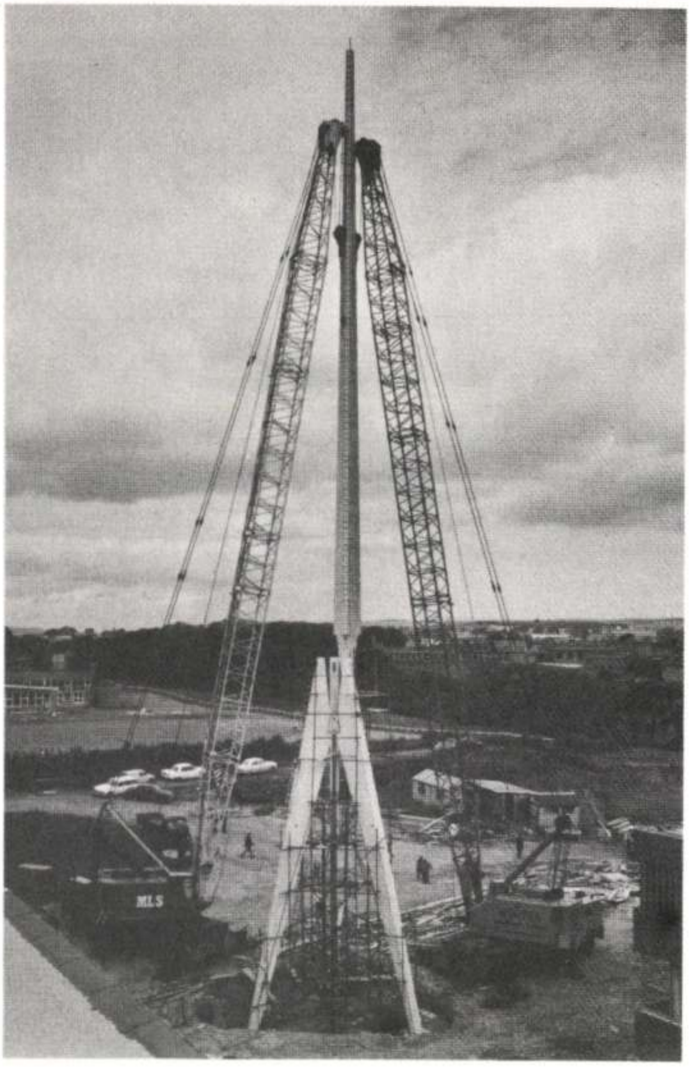

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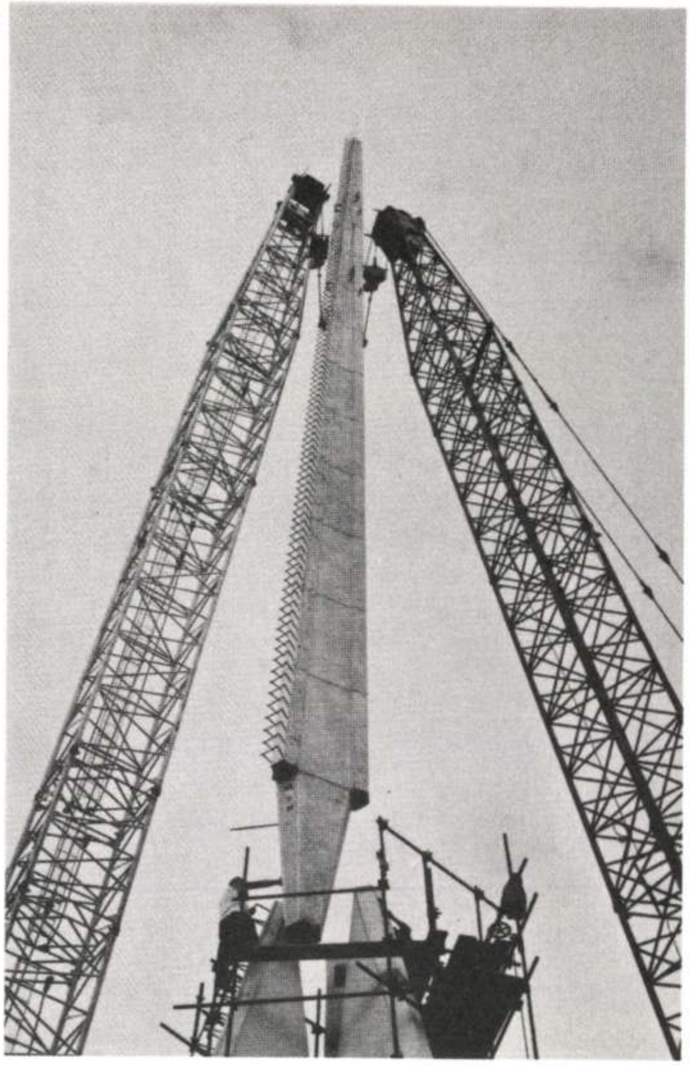

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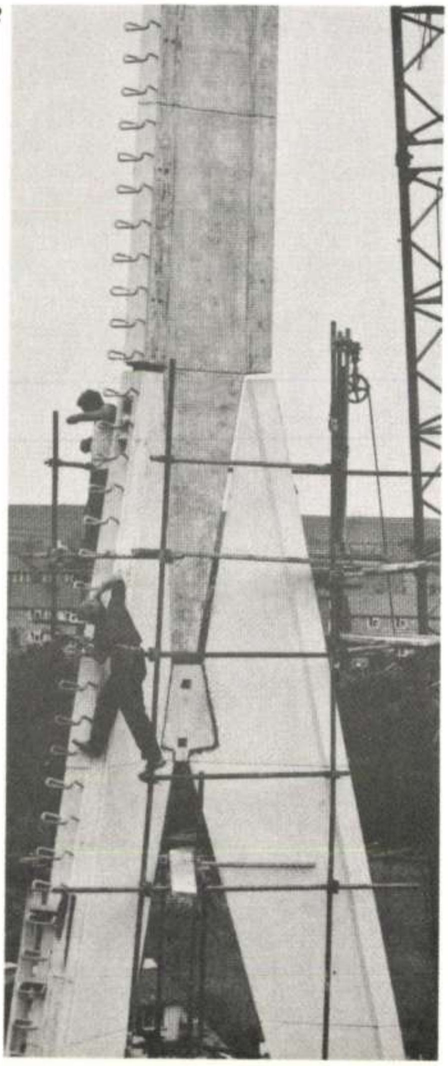

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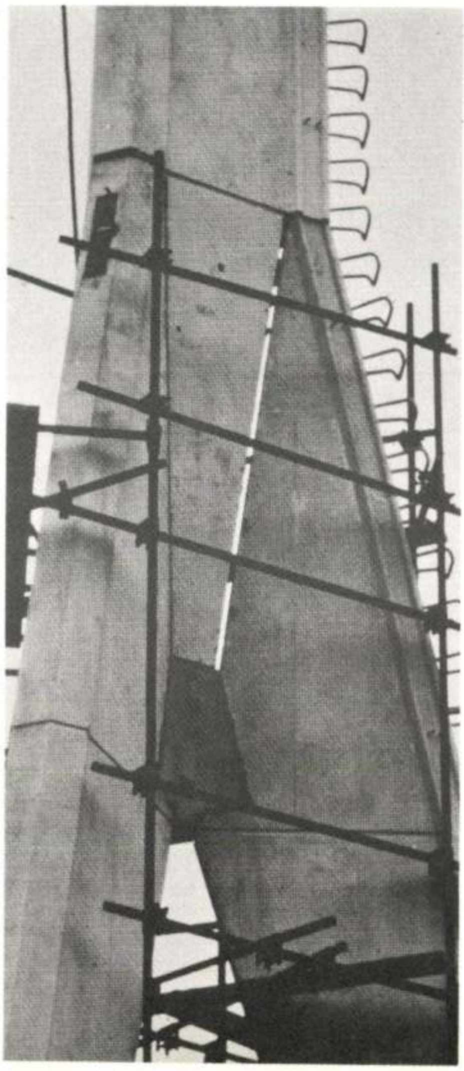

Fig. 10
Now in the vertical position, there is very little clearance at the heads of the cranes. (Photo: Turners (Photography) Ltd., Newcastle)

Fig. 11
The mast being given a helping hand (Photo: Turners (Photography) Ltd., Newcastle)

Fig. 12
The spigot end being located on its pin and neoprene pad. The mould oil and excess epoxide mortar were later cleaned off (Photo: Turners (Photography) Ltd., Newcastle)

Fig. 13
Now plumbed, the structure awaits the dry pack mortar at the cradle joint (Photo: Ken Shaw)

Fig. 14 on facing page
The finished job (Photo: Turners (Photography) Ltd., Newcastle)