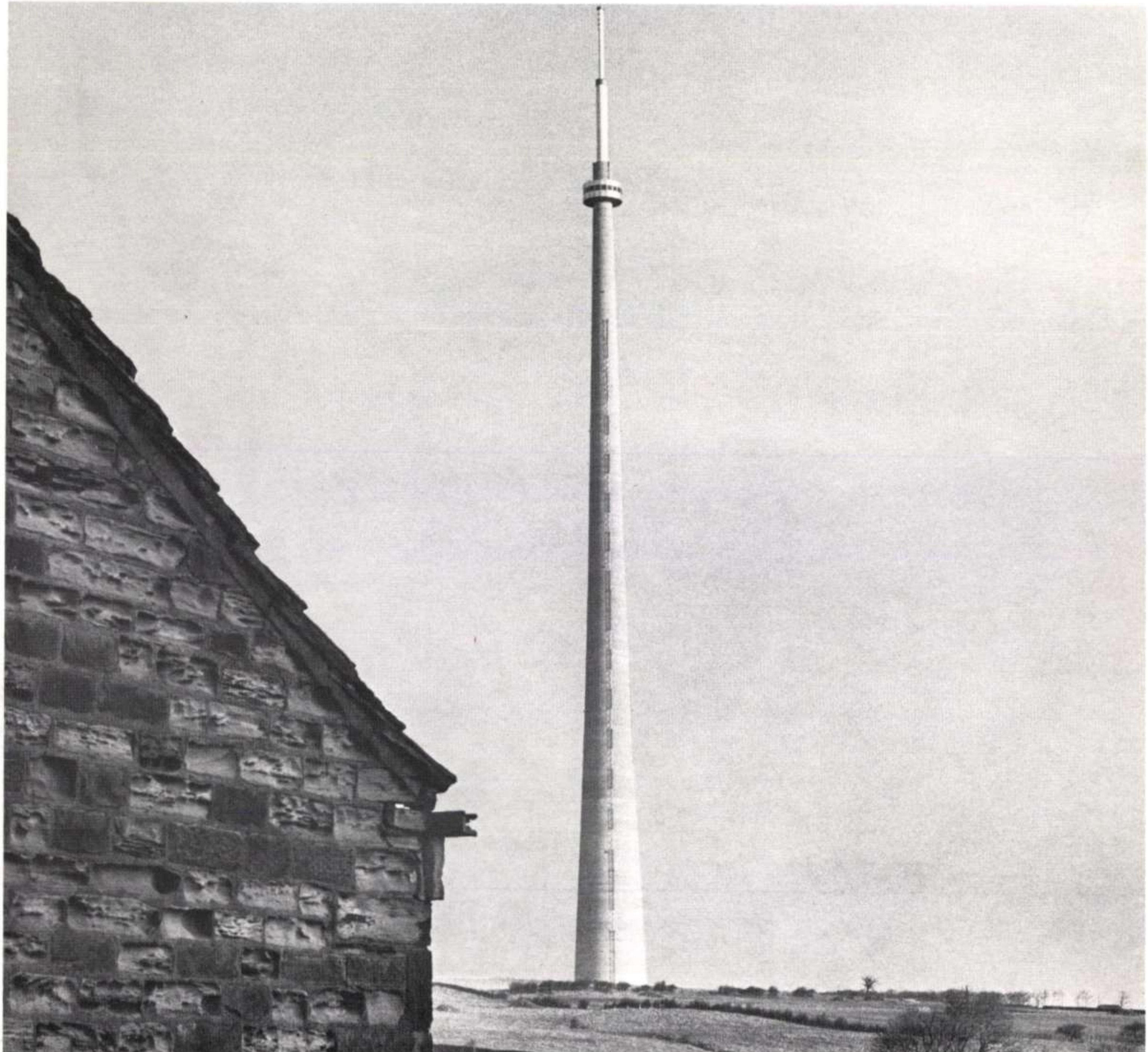
THE ARUP JOURNAL

MARCH 1972



THE ARUP JOURNAL

Vol. 7 No. 1 March 1972
Published by
Ove Arup Partnership
13 Fitzroy Street, London, W1P 6BQ

Editor: Peter Hoggett
Art Editor: Desmond Wyeth FSIA
Editorial Assistant: David Brown

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Front Cover: The Emley Moor television tower (Photo: Geoff Rooke)
Back Cover: Early 19th Century print of Manchester Market Place.

Future problems facing the designer

Ove Arup

This paper was given at the Royal Society Discussion Meeting, Building technology in the 1980's, *on 4 November, 1971.*

I am aware of, and grateful for, the honour you show me by asking me to address you on the subject of 'Future problems facing the designer'.

I am afraid, however, that you may have overestimated my powers. Like the Danish cartoonist Storm-Petersen I find it very difficult to prophesy – very difficult indeed – and especially about the future. But I can tell you straight away what I fear may happen: that designers in the future will even more than now be working under constraints which will make it impossible for them to give of their best. And I could add that even more than now their best may not be good enough because it is too narrowly based. This I think is by far the gravest problem facing designers, and it is a problem for all of us, unless present trends are reversed.

This is a somewhat pessimistic answer, and probably not what you expected. My opinion is of course based on my own experiences as a designer of sorts, and it is therefore a very personal one. Let's hope I am wrong.

Meaning of design

The word 'design', used as a noun or verb, can mean many different things. Here we are concerned with design as a link in the process of building and construction. Incidentally I make no distinction between the two; such distinctions have become obsolete with growing mechanization and factory production.

It would not do, however, to limit the discussion to structural design only, using the word in its narrower sense, for whereas everything we build or make must have structure to keep its intended shape, the structure itself is only a means to an end; its merit cannot be judged without reference to the thing of which it forms part.

Designing – and we could add planning – which is roughly the same thing, on a larger scale – plays a central and increasingly crucial role in technology; it is the key to everything which is made or built. For in modern construction everything is thought out beforehand, no unauthorized action is allowed. So any feature of the finished job which is not purely accidental must be due to a decision by somebody. As a matter of convenience, I call these decisions design decisions, and the sum of them the *total design*. The total design is seldom recorded *in toto*, it is more an idea than a reality. But the idea of total design implies that sufficient decisions have been made and recorded to enable others skilled in organizing such work to carry it out. The total design must, by definition, be viable, i.e. it must be possible to carry it out as intended. And it completely defines the finished job.

This cannot be said of all so-called designs. These are often only preliminary designs, sketches, plans or architectural perspectives showing certain aspects of the intended total design, but one cannot always be sure that they are viable, or practical, before the details have been worked out and all the implications considered. This leads to the often lamented gap between vision and embodiment. It is a wise precaution to bring the design to a stage when you know for certain that it can be built and what it will cost, before embarking on work on the site or in the factory.

Our designs, as executed, make our environment, and this in turn makes *us*, partly at least. Designing therefore assumes an importance not far short of that which belongs to scientific enquiry. In fact the two are intimately intertwined, one of them would get nowhere without the other. The whole of our technology is based on scientific knowledge, and scientific observation, and research depends in turn on sophisticated hardware. Both are complex mental activities, set in motion by a stated objective, and based on a stock-in-trade of knowledge and experience, augmented by fact-finding, classification, interpretation, analysis and synthesis in various proportions – a process where imagination, intuition and invention are of vital importance. However, the two differ in their objectives.

The *scientist* wants to *explore* nature, to find out how it works. He is looking for general laws. The motive may be pure curiosity – when we call it pure science – or he may hope that the acquired knowledge will help him to run the machinery himself. In this case it is not so pure, and approaches the kind of research which may form part of designs breaking new ground.

The *designer* wants to *change* nature to suit his convenience. He is trying to solve a practical problem here and now. But his problem is underdefined. There is not just one, but many solutions, good, bad and indifferent. He must choose. The bad solutions come easy to hand, the good he must search for, and work for. And *that is the designer's real métier*, that is his real problem, to find the *best* solution. Or, as you can never be sure that there isn't a better solution, to find at least a good solution, a solution of quality. And this is a problem in more than one sense – for *what* is *quality?* What is goodness?

The 'goodness' of a total design must be the same as the goodness of the finished structure, for the total design completely defines the latter. And the goodness of a structure, using this term about any product of the building industry, must be related to its purpose, and the consequences flowing from its being constructed in this particular location. The structure must obviously be fit for its purpose to be good. But that is not all that is implied by quality, as I hope to explain. The purpose behind the whole undertaking does not emanate from the designer, and may not even be disclosed to him. As far as he is concerned, it takes the form of a brief given him by his client and telling him in more or less general terms what is wanted. The designer then digests this information and disgorges it again in a form which will enable the builder to construct what the client wants.

This is the highly simplified version of the building process which is generally used when writing or talking about the subject. The reality is infinitely more complicated and we will come to that later.

Formula for excellence

The brief, supplemented by reference to the client if necessary, is obviously a very important document. If it could really explain exactly what was wanted, the question of quality would be solved. The best design would simply be the cheapest of those satisfying the brief. The formula measuring efficiency or excellence would be:

$$E = \frac{C}{P}$$

where C stands for commodity as defined by the brief and P stands for cost in pounds. And this is in fact what the whole of the modern obsession with cost-efficiency is based on . . .

All possible factors, even such imponderables as a saving in time for busy executives, are evaluated in money terms, and optima are produced by the computer and used as a basis for what are in effect political decisions. This is very crude, and potentially very dangerous, for it entirely ignores the quality of the product, which we do at our peril. Even the best of briefs cannot begin to define quality. The brief can be satisfied both by good and bad designs. You can specify that you want an elegant structure or a friendly house, or a town hall which is the envy of neighbouring cities, but that does not help you much. The client buys the cat in the bag. All he can do is to choose a good designer. That is why it is tempting to hold a competition for the best design, except that the submitted designs necessarily must be in the form of sketch-designs, which may not fulfil what they promise, and the assessors may not recognise nascent quality when they see it.

As the Danish author Piet Hein has said: 'Art is solving problems that cannot be formulated before they have been solved. The shaping of the question is part of the answer.' And designing is essentially an artistic process. Of course collection of data, research, analysis, calculating, quantifying, costing, etc., and especially previous experience, play their important part, and in engineering structures perhaps the most important part – and some of these activities can be eased by computerized information processing, analysis and mathematical simulation. But the essence of designing is to effect a harmonious synthesis of partly conflicting aims and obstinate facts. It is largely to find the right spatial arrangement of parts. Designing can be likened to the solving of a gigantic three-dimensional jig-saw puzzle – except that there is not one but many solutions, and not one but many designers. And they have got to find or shape the parts themselves, keeping always the object in mind.

A designer has his own standards. He is a professional, a craftsman, and if he is good himself, he knows when he has done a good job. It must be all of a piece, have wholeness, clarity, it must not be too strong at one point and too weak at another – but, as I said, it is useless to try to define quality. All we can say is that its emergence results from the involvement of the designer, from his passion for perfection, from the fever which grips him when he sees the chance of producing a really good job, and which makes him sustain the effort involved. I believe, perhaps naively, that such enthusiasm is a pre-condition for creating a structure or an environment which is not as cold and inhuman as much of our modern environment, but in which we can feel at home.

But enthusiasm is not enough. It can even be dangerous if too narrowly based. Designers – besides knowing their métier – must have an understanding of what other people need, and not just of what they want to give them, and sometimes perhaps what they need is a chance to build their own shacks – disregarding aesthetics.

Design in practice

I am aware that all this must sound a bit high-falutin' to you. What has all this to do with the client? He doesn't necessarily want an architectural or other kind of masterpiece. Maybe his object is to sell out quickly. Let's say a rural council wants a water tower. They have not much money – but they are in a hurry – for the matter has been debated for several years. They know what they want: so they go to an engineer who has designed water towers before and say:

One of these, please, so high, so many gallons, for so much money, or less. And we would like the tenders to go out in six weeks time. It can be done. A sketch design, approximate quantities which can be modified later, the contractor starts digging. But he hasn't had the working drawings yet. And so on.

The water tower stands in this village for a long time. It is its most prominent feature. You cannot avoid seeing it. A pity – but we must have water.

This is, on a small scale, what happens most of the time, more or less. But it is not the way to get the environment we like.

Of course most designs *are* more or less routine designs. They must be, we can't invent our technique afresh every time we build. But even if the bulk of what is built relies on previous experience slightly adapted to present circumstances, it is the fresh look at the evidence which initiates progress and improves quality. The natural instinct of a true designer is to ask himself: How can I do this thing better than it has been done before? Let's forget how it is normally done. By concentrating on the essence of what is needed and the most direct of all the ways of achieving it, perhaps I can find a simpler, cheaper and a better solution, fitting into surroundings better, pleasing the people I am serving. If these aims clash – as they will – perhaps if I try harder, some insight, some idea, will come to me in the middle of the night.

It may quite likely happen, of course, that the designer, after such an excursion, falls back on a traditional solution because it is in fact the best in the given context. He has wasted time and effort. But he has gained the satisfaction, valuable to any designer, of knowing that he has chosen rightly.

The brief, and the formula:

$$E = \frac{C}{P}$$

cannot even settle the dispute between alternative solutions, if they happen to cost the same. For instance, if the brief can be satisfied by using either structural steel, reinforced concrete or aluminium as the basic structural material, the three schemes will differ in many respects which cannot be measured with the same yardstick. Durability, thermal conductivity, ease of effecting alterations, cost of upkeep, weathering, suitability for possible mass production, use of local resources and labour and many more, all will vary. Even the shape would be affected, and they would certainly *look* different. We would have to modify our formula to:

$$E = \frac{\text{C plus EC plus D}}{\text{P}}$$

where EC stands for commodity in excess of that required, but still of some value, and D stands for delight, the artistic quality. EC and D cannot be objectively measured in money terms, yet they cannot be ignored either.

Brief, design, execution

The brief cannot define quality. But it can prevent quality being produced by the designer, and often does, either by being so vague that the designer has not really understood the client's needs and therefore comes up with the wrong answer, or more often by being too detailed, thus pre-empting design decisions which ought to be taken by the designer, or which at least ought to be integrated with the other design decisions and modified as a result.

It is wrong to treat brief and design as separate documents. The brief and the basic design decisions should result from a collaboration between client and designer. You cannot decide what to build without finding out what can be built for the money available, what the options are. The client's job is to explain his situation and his needs as fully as possible to his professional advisers, but not to propose, or at least not to dictate the solution. Just as a patient should explain his symptoms to the doctor, but should not suggest the cure, far less prescribe it, the very thought is preposterous in this case. The relationship should be one of openness and trust in both cases.

The same intimate relationship should exist between design and execution. The designer must know where he is going, he cannot design unless he can judge whether his design *can* be built, *how* it can be built and roughly how much it costs, at least enough to enable him to compare the cost of alternative solutions. Otherwise he proceeds blindfold. To complicate the execution through ignorance or neglect of the ways and means of construction is bad design. True economy demands that the design indicates a practical way of building.

I hope I have explained the need for integration of brief, design and execution. But there is much more integration to be done.

If we look at the real situation we find that jobs are getting larger and more complex and that the total design is split between dozens of different professions, experts, manufacturers and contractors, each at best pursuing his own particular kind of quality. Communication between them is inadequate, and not much concerned with a rational appraisal of the design. But although the quality of the overall planning services, the structural design, the architectural conception and detailing and the economic efficiency are all important, it is the synthesis of all these partly conflicting aims which constitutes the quality of the whole job. We need all-round or comprehensive quality, wholeness, which is really nothing else than a closer adaptation to human needs.

Total Architecture

But who is going to do the integration? The job is too big for one man. Team work is the answer, but is difficult when the members of the team so to speak live in different countries and speak different languages. It requires teamwork of a much higher order, whose members collectively embrace the experience needed for the job, understand each other and have the same desire to create what I elsewhere have called Total Architecture. In such a team enthusiasm can survive, and can even spread and flourish.

Total Architecture does not mean that cost is neglected. It only means that quality is not forgotten. The only hope I see of combining low cost with quality is to spend more time over the design. By more constructive forethought the time and money spent on the site can be reduced. Saving money is done by better design, better job organization and less waste, not by more accounting.

Post-design costing can be a useful check on a design before work is put in hand – but it comes too late – it means re-design if the design is too expensive. And in any case the present over-elaborate Bills of Quantities are a clumsy and time-wasting instrument for this purpose, as progressive quantity surveyors will admit.

But the present vogue and esteem for pre-design cost estimation, where definite sums are allocated to various parts of a non-existing design, rests on a curious delusion. Where does the estimator obtain his superior knowledge of what the right design should cost? It is really an insult and an obstacle to designers; their work is dismissed as of no importance.

Of course, designers may not be good enough – if you are strict you can say that of most designers. But before you bring in somebody else to do their job for them, you had better be sure that these others are better designers, for it is better designs we need, not better costs. A feasibility study by designers is the way to get an idea of costs beforehand.

No, costing must be an integral part of designing. We need cost-conscious design. And that is in fact what all engineering design is or ought to be – to find the cheapest solution to a given problem. This can be said to be the definition of engineering, and everything we build is engineering of one kind or another.

Cost-conscious design is the way as I see it – but the situation is not propitious for its adoption. Spending *more* on design goes against the whole trend, against official policy. We are trying to reduce the cost of design by mechanizing it, applying cost efficiency techniques, which cannot distinguish between good and bad design.

Naturally designers should work as efficiently as possible – but to prevent them from designing in the true sense is counter-productive, to put it mildly.

The client

The 'client' has changed as well. He has lost his identity and accessibility, being replaced by a board of directors responsible to their shareholders, or by government departments administering directives from on high, both represented by agents who can only administer official policy, no matter how inappropriate to the situation. But are these the true clients? Do we not design for the people who use the structures and live in the environment? This view has certainly been gaining ground among designers; it was one of the planks in the programme of the Modern Movement in architecture. This means that the critical look at the brief, for which we felt the need in the interest of wholeness, must penetrate deeper. When the client is the public, the brief must defend public interests. That means that we must take into account the wastage of scarce resources, the cost of preventing pollution, the spoliation of our environment, the harm done to fauna and flora – in fact all the things people have warned us about, and which are the pet theme of our more serious publications. We cannot afford to neglect them, even if only partly well founded. The total design must be truly comprehensive. We must add to our formula another *factor:* SP – the social price we have to pay, if we execute this design, and we had better put it below the line this time:

$$E = \frac{\text{C plus EC plus D}}{\text{P plus SP}}$$

You may object that the formula gets less useful for every addition, and you are right, for it is very difficult to put a value on EC, D and SP. But we have to do it, or at least we have to make decisions which take account of these items, and the sooner the better, if we can believe the prophets.

This is a matter for politicians rather than designers; it directly affects only the brief, and therefore the client's right to build what he wants. But, as I hope I have made clear, brief and design cannot be separated, and scientists and designers must be brought in as advisers, to decide why we build and *what* to build.

This is a much more difficult and controversial question than how to build. To get strong and concerted action by independent national governments in this matter seems beyond the reach of man, at least not until disaster stares us in the face. But at least people and governments have begun talking.

But that we must stop glorying in waste seems obvious. Waste can be glorious – but it leads to waste land. We must preserve our heritage, conserve our resources, build for permanence, not for scrap. We must reduce our production of unnecessary gadgets, simplify our lives where possible so that something is left over for those in need.

Does this invalidate the quest of quality in design? I don't think so. To do one's job well is good for one's self respect and good for one's fellow beings, and it can't do any harm as far as I can see even if it doesn't solve all the problems of mankind.

After all, it only means using our resources wisely.

The Emley Moor television tower

André Bartak and Mike Shears

This article first appeared in the February 1972 issue of The Structural Engineer *and is reproduced here by kind permission of the Council of the Institution of Structural Engineers.*

Introduction

The collapse of the guyed structure at Emley Moor on 19 March, 1969 resulted in the loss of both the ITA and BBC programmes in a large and densely populated area of Yorkshire. Although VHF monochrome services were quickly restored by various emergency measures including the erection of a temporary 216 m high guyed mast, full coverage of the area previously served by the UHF BBC2 colour transmissions could not be re-established by these means.

At the time, the Independent Television Authority were themselves in the midst of a programme of conversion to colour transmission, the opening of the new service being originally scheduled for the end of 1969, and therefore it is not surprising that when the ITA authorized a feasibility study for a new structure to support the aerials at Emley Moor, they specified that the UHF transmissions from the new structure should commence, if possible, by the end of 1970.

The initial briefing given on 16 May, 1969 requested solutions based on cantilever concrete structures surmounted by steel aerial supporting masts because:

(i) The local planning authority requested that any new guyed structures be located not nearer to public roads and dwellings than their height; the resulting increase in the distance from the existing station buildings making a guyed mast solution unacceptable.

(ii) It was believed that cantilever structures made entirely of steel could not be realized within the available time.

(iii) Past studies had indicated that steel cantilever structures did not have a definite cost advantage over concrete for the contemplated height range.

In order to optimize the overall cost, estimates were prepared for three alternative heights of the proposed tower as follows:

Height of concrete structure	*Height of aerial super structure*	*Total overall height*
213 m	55 m	268 m
244 m	55 m	299 m
274 m	55 m	329 m

In each case, the steel superstructure to carry the aerials was to consist of approximately 30.5 m of triangular section with 1.98 m sides and the top 24.4 m of triangular section with 0.99 m sides. The whole of the aerial superstructure was to be surrounded by fibre glass cylinders of 3.66 m in diameter around the 1.98 m triangular section and 1.52 m in diameter around the 0.99 m triangular section. These estimates were incorporated into the parallel studies made by the Authority's engineers, who concluded that the best overall solution was provided by the structure of the maximum height, i.e. 329 m. (Fig. 1.). The brief specifically excluded the provision for any public facilities such as a viewing platform or restaurant.

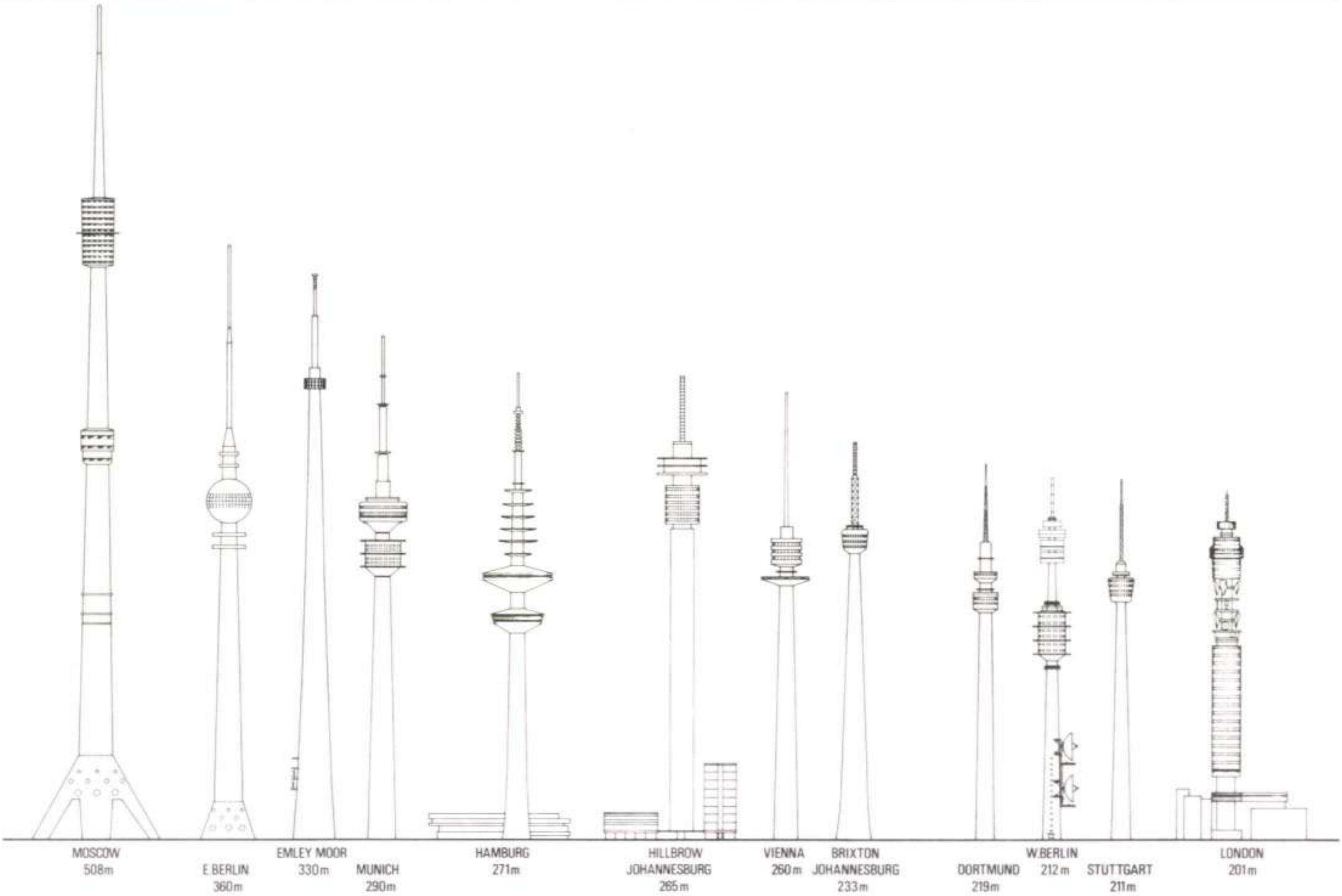
Early decisions and developments

Another decision which profoundly influenced the further development of the design was taken at this time. This was to erect the aerial supporting steel mast within the tower at the ground level, together with the aerials and upper section cladding, and hoist the assembly into its final position at the top of the concrete structure. The reasoning behind this decision was the desirability of eliminating the risk of delay which could result if conventional methods of erection were attempted, especially taking into account that the site had a history of severe weather conditions.

Also, in order to save time, it was decided to eliminate the period required for tendering and negotiate instead with a selected contractor. Six contractors possessing the necessary specialized experience were approached and after a series of interviews the choice was made. From this time onwards the contractor made available to the design team his considerable construction experience, thus making a very important contribution towards the development of the design which necessarily had to evolve around the mode of construction.

As previously indicated, time was a major factor governing the evolution and development of the design. A short list of dates which follows illustrates this point:

Initial briefing:	Mid-May 1969
Site investigation start:	End of May 1969
Commencement of work on the design:	Beginning of June 1969
Contractor on site:	August 1969
Foundation excavation start:	August 1969
Concrete structure completed:	September 1970
Aerial mast in position:	November 1970
UHF aerials in operation:	January 1971



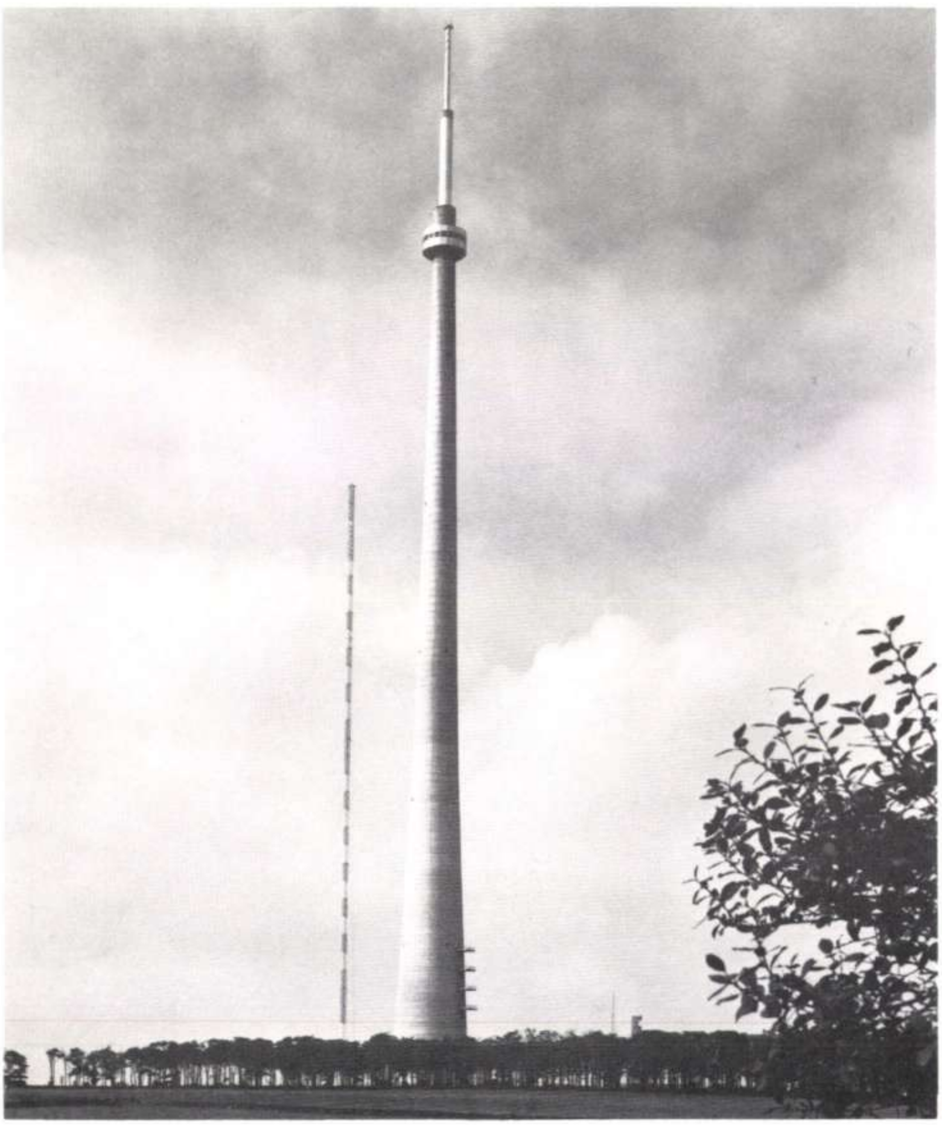


Fig. 1 above
Emley Moor Tower and other similar structures

Fig. 2 left
View of completed tower
(Photo: Greaves (Photographers) Ltd.)

Functional requirements

As the work on the design progressed, further functional requirements crystallized. Broadly, in addition to the transmitting aerials, the design was also to allow for the following features:

(i) Three 3.66 m diameter dish aerials for the Post Office to be carried externally at the 30.5 m level approximately. The Post Office was also to be provided with an equipment room situated within the base of tower.

(ii) Mounting of external broadcast dish aerials for both Yorkshire Television and the BBC. These were to be located as near as possible to the top of the concrete structure, preferably protected from weather. Equipment rooms associated with this function were to be provided internally, close to the aerials.

(iii) Provision for mounting future dish aerials at various points externally between the 61 m and 122 m levels.

Although a few minor changes to these requirements were made subsequently, in essence the above represented the full brief. The solution, resulting in the tower as built (Fig. 2), is described in this paper.

Site conditions

A desk study of the local geology suggested that a 15 m thick stratum of hard sandstone would be found 3 m below ground with seams of Green Lane coal, New Hards coal and Wheatley Lime coal at various depths down to 65 m. Two further coal seams were expected at 100 m and 150 m and faults had been recorded in the vicinity.

Two rotary cored boreholes were sunk and the successions shown in Fig. 3 were found. It was thought that the Green Lane seam might have been worked beneath the site.

A probing and grouting contract was let to investigate this possibility but the stratum was found to be in the condition of intact rock.

Superstructure and foundations

The general arrangement of the structure is shown in Fig. 4. The tower features a tubular concrete shell, 274.32 m high, surmounted by a steel aerial supporting mast which extends 56.08 m above the concrete. This is carried by two concrete slabs, 457 mm thick and 4.57 m apart, which transfer all of the bending and shear to the concrete shell by horizontal thrusts. The concrete shell is 24.38 m in diameter at the base with a wall thickness of 533 mm and reduces to 6.4 m in diameter at the top, where the wall is 350 mm thick. The profile, in the form of an exponential curve, was adapted to fit the method of construction by fitting in a series of straight tapers. The foundation is an annulus, 8.23 m wide and 4.27 m thick, which bears on sandstone approximately 6.1 m below ground level.

The specified minimum strength for the concrete of the shell wall was 40 N/mm² and for the foundation 26 N/mm².

The basic geometry of the aerial support mast was dictated by considerations of aerial performance and dimensions were therefore specified by the ITA. It is composed of two triangular sections, the lower having a side of 2.21 m and extending 30.48 m above the concrete, and the upper being 25.6 m high with a side of 0.99 m.

The legs of the mast were bent from plate varying in thickness from 38 mm to 16 mm.

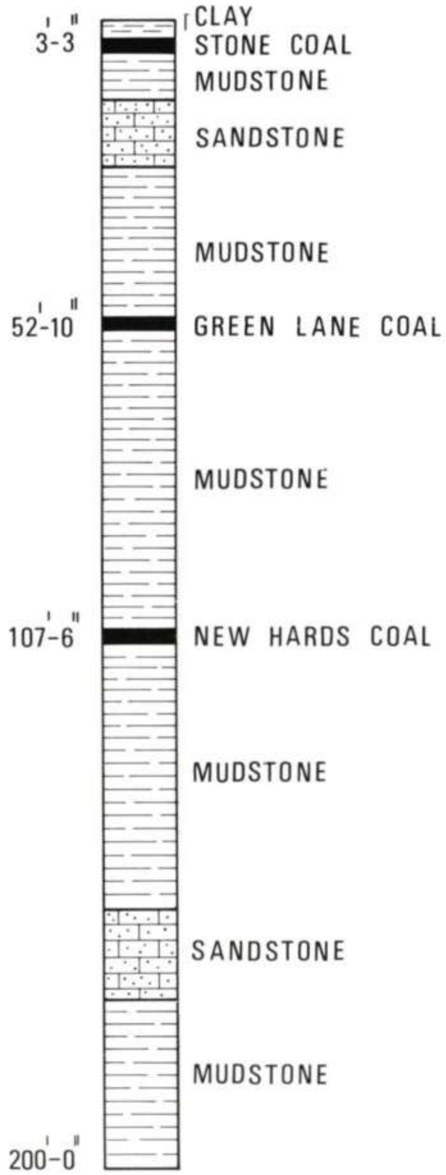


Fig. 3 above
Geological succession

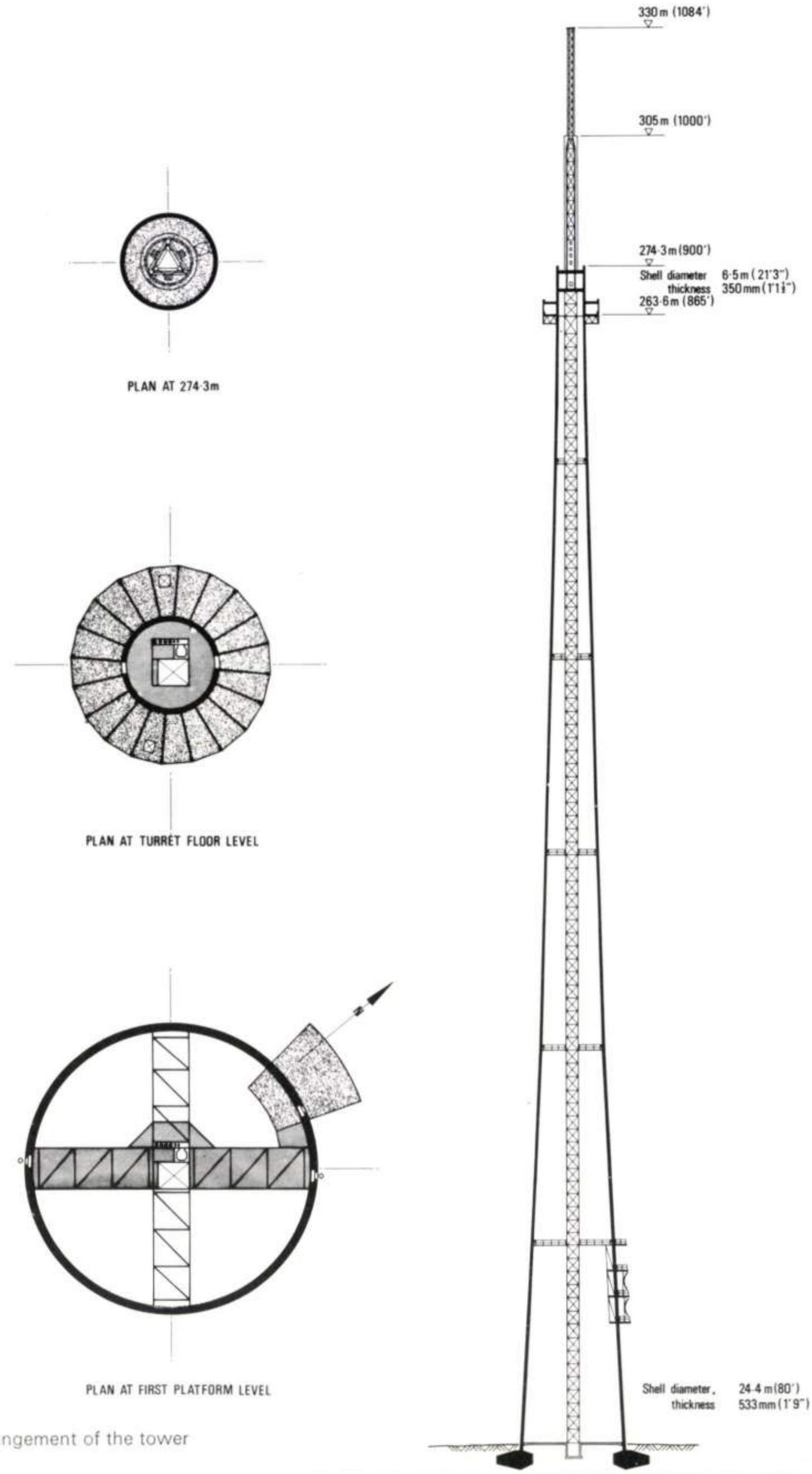


Fig. 4 right
Cross section and plans showing the general arrangement of the tower

The bottom 10.67 m has plate webs and above that the structure is latticed (Fig. 5). All connections were made with HSFG bolts and the steelwork was fully galvanized. Considerable thought was given to the problem of developing the friction in the faying areas. These areas were grit-blasted at the fabricator's works in order to remove the zinc but not to penetrate the alloy layer. No masking was used. After assembly of the steelwork at the site, the areas surrounding the connections were cleaned by further blasting and then painted.

For the protection of the aerials and of the men servicing them, the mast was enclosed in electrically translucent, cylindrical glass fibre reinforced plastic cladding 1.52 m in diameter over the upper mast and 3.66 m over the lower portion. Because of the high rigidity of the cylinders, all horizontal joints between units were designed to accommodate movement and the units were individually supported by the steelwork. The joints were sealed with one part polysulphide sealant.

Internal structure

Inside the concrete shell, a rectangular, latticed steel framework forms a shaft for a maintenance lift and supports the aerial feeder cables, the tower lighting, power and telephone facilities and incorporates an emergency cat ladder with rest platforms.

This framework which had also been designed to accommodate the aerial mast and to guide it during the hoisting, can be seen in Fig. 6. At intervals of 46 m the framework is braced to the concrete shell by platforms which also give access to the aircraft obstruction lights.

Aerial platforms

Four external platforms near the base of the tower, as shown in Fig. 7, will carry 3.66 m diameter microwave dish aerials. The platforms were cast on to steel beams projecting through the shell wall and braced internally.

Pockets which can be broken through were left in the concrete so that similar groups of platforms can be constructed in the future at any of the first three internal platform levels.

Doorways were also formed in the shell to give access to these platforms.

Fig. 5 right
The aerial mast partially assembled during trial erection (Photo: British Steel Corporation, Teesside Photographic Unit)

Fig. 6 below
Lift cage and internal platforms (Photo: Geoff Rooke)

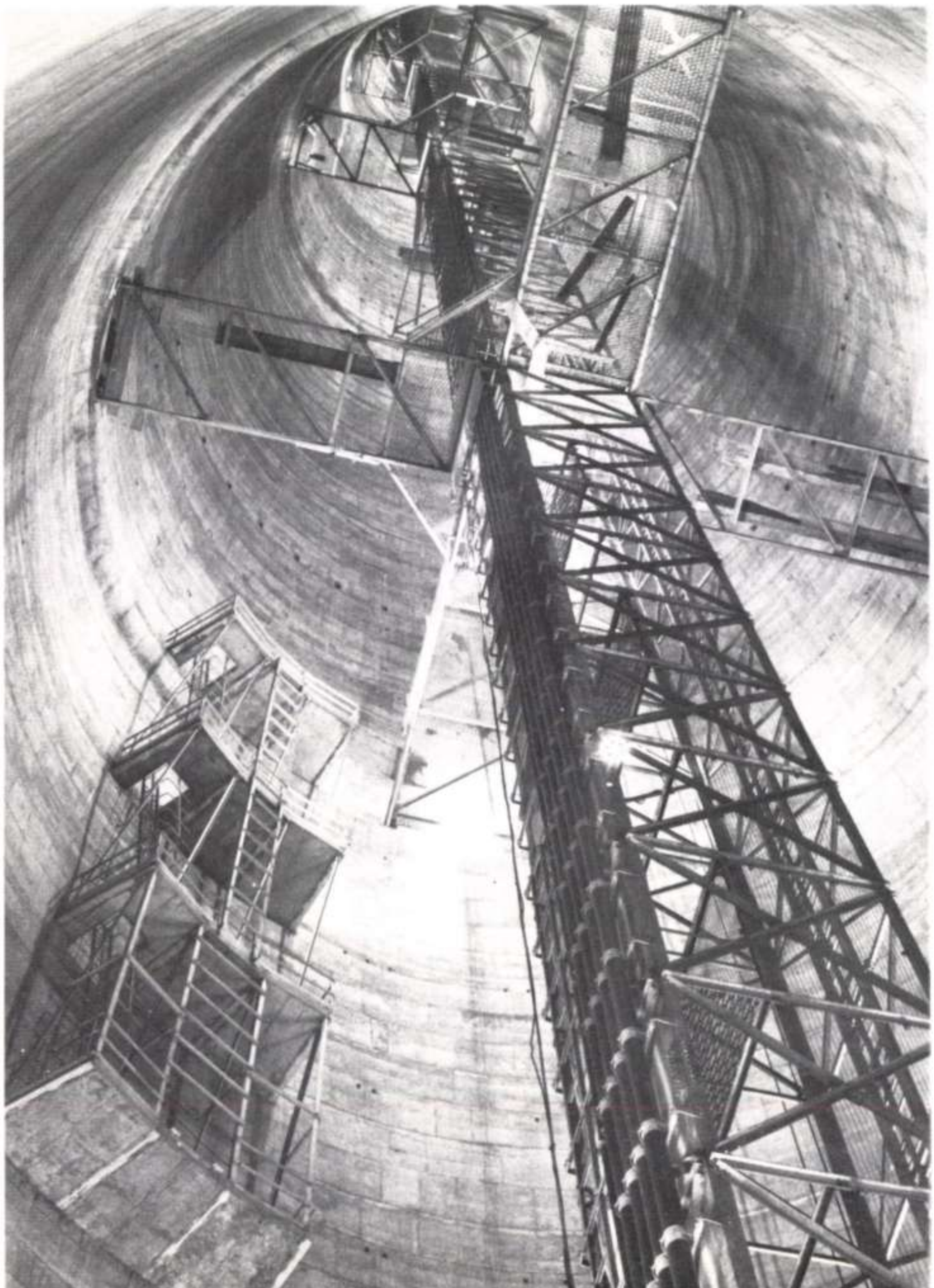


Fig. 7 above
GPO platforms (Photo: Geoff Rooke)

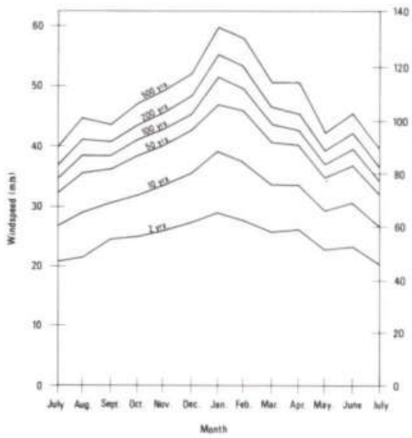


Fig. 8
Plot of predicted monthly wind velocities

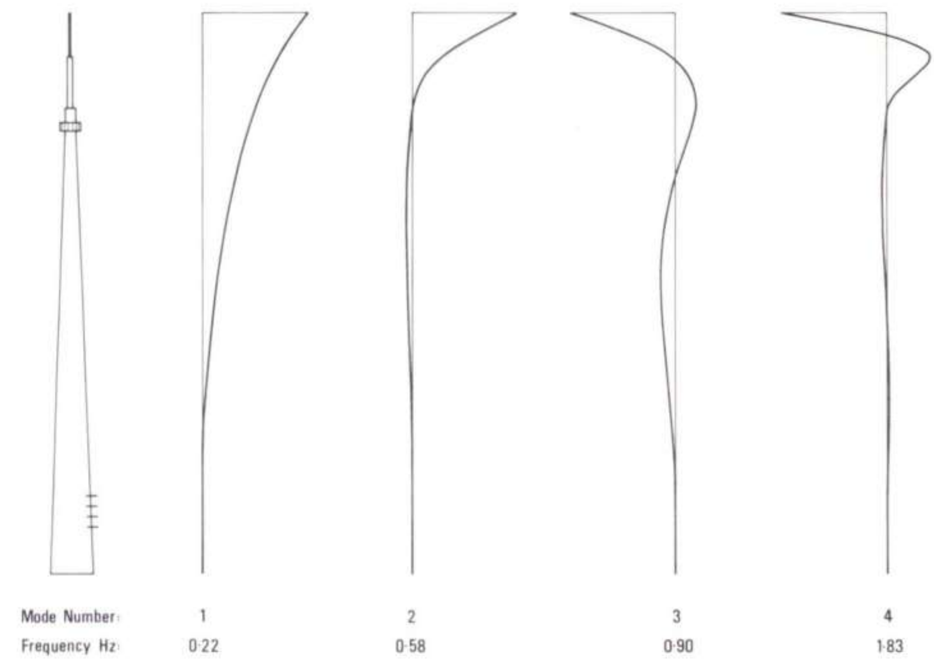


Fig. 9 right
First four natural vibration mode shapes of the tower

Turret
At the top of the tower an enclosed turret will house outside broadcast dish aerials and associated equipment. It was designed to be constructed after the aerial mast had been brought into service. The structure forms a collar around the tower so that only unbalanced live loads generate horizontal shear forces in the tower wall. It consists of 20 steel trusses, placed radially and cantilevering from a concrete compression ring and a steel plate tension ring. The roof is carried by steel beams supported by mullions carried at the ends of the trusses. The roof and floor are both of in situ concrete cast on permanent shuttering. The face is clad with insulated aluminium sandwich panels and the soffit is also clad in aluminium. The basic dimensions of the structure were determined by the need to provide 1.8 m square windows so that dish aerials of that diameter could be used in outside broadcast links. *Spectrafloat* glass was chosen for the windows to reduce solar gain. The client carried out tests to ensure that this glass would not degrade the aerial performance. To maintain the temperature within the range which can be tolerated both by personnel and equipment, heating and ventilating equipment is housed between the trusses beneath the floor.

Lightning protection
Four of the vertical reinforcement rods were welded continuously through the height of the tower to provide the down conductors for the lightning protection system. External copper bands around the tower at 91,182 and 274 m are connected to the down conductors and all steelwork and doors are bonded to the system. The aerial mast has lightning finials on top, a steel section of cladding at the change in section, and copper bands at the mid-height of each section. The system is earthed by four probes driven outside of the tower.

Other provisions
Electrical control gear for the tower lighting, power, telephone and lift is accommodated in a room within the base. A separate room is provided to house GPO transmitting equipment in the base of the tower.

Other transmitting equipment is housed in buildings remote from the site. Feeders to the aerials run underground to the tower perimeter and from there in a concrete duct to the central steel lift cage.

General design considerations
From considerations of the size and general proportions of the structure it was clear that the Emley Moor tower would be predominantly wind loaded and likely to be sensitive to wind gust fluctuations. Since the nature of wind loading is known to be highly variable, it followed that the estimation and description of the wind forces, and the prediction of the resulting structural performance, could be best assessed by statistical design procedures. Furthermore, the possible sensitivity of the tower to windspeed fluctuations suggested a statistical-dynamic, ultimate design procedure, rather than a quasi-static approach. It has been shown[1 2] that the more usual code type design based on a windspeed of return period equal to the design life can lead to inconsistent results for this kind of structure, since the risk of failure is not logically included in the design process and the resulting actual load factors against the assumed 'failure conditions' will not only be different according to the real exposure conditions of the site, but will also vary throughout the structure.

The decision to use a statistical-dynamic, ultimate design procedure is particularly appropriate in the case of a tall concrete tower, where the stresses in the shaft are not linearly related to variations in the wind forces, due to the effect of the high precompression in the tower shaft under self weight. Additional design checks were made, however, to establish the working load stresses and to compare the results with traditional design methods.

Meteorological investigation
The first problem in the design procedure was to establish an adequate description of the wind structure at the site in terms of the appropriate windspeed profiles and other ground roughness parameters required to define the gustiness of the wind, and to obtain estimates of the distributions of both extreme and regularly occurring windspeeds.

A statistical analysis was made of windspeed data recorded at several meteorological stations close enough to Emley Moor to be considered representative of the general wind climate and suitable for the assessment of conditions likely to be encountered at the site. The available windspeed data was of varied quality and duration but allowed fairly reliable predictions to be made of monthly and annual extreme mean hourly and maximum gust windspeeds for the site at various return intervals (Fig. 8). Windspeed frequency distributions required for the consideration of functional design requirements and problems associated with fatigue were also estimated, but with less confidence than for the extreme windspeeds.

As a comparison check, the once in 50 years maximum gust windspeed predicted by the estimated extreme gust distribution was 46 m/s, as indicated by the basic windspeed map in *CP3: Chapter 5: Part 2: 1970.*

Dynamic analysis procedure
The calculation of the structural responses, i.e. bending moments, direct forces, deflections, etc., of the Emley Moor tower due to the action of wind drag forces, was carried out using a statistical, dynamic analysis procedure. This procedure has been described in more detail elsewhere[1 2] but is outlined briefly as follows:

1 The wind velocity is assumed to be composed of a steady, mean component and a superimposed randomly fluctuating component to represent the gustiness.

2 The steady, mean component is the design mean hourly windspeed obtained from the estimated extreme windspeed distribution with a probability of occurrence chosen to

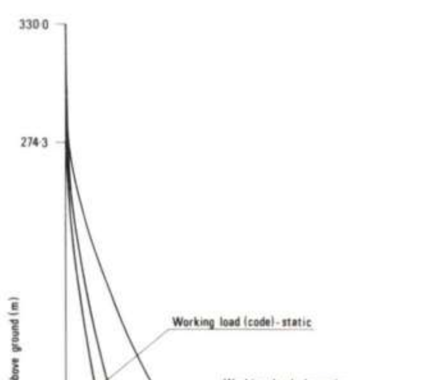


Fig. 10
Distribution of bending moments

suit the particular design requirement. The wind forces due to the design windspeed are applied to the structure in the normal manner, using appropriate force coefficients and taking account of windspeed variation with height. The resulting bending moments, direct forces and deflections are the mean responses of the structure.

3 The structure is assumed to vibrate about the mean deflected position due to the action of the fluctuating component of the wind, and an analysis is made to obtain the natural mode shapes and frequencies of vibration.

4 The random nature of the fluctuating gust component and the vertical distribution of wind gusts over the structure are assumed to be adequately represented by Davenport's cross-spectrum of longitudinal turbulence[3].

5 The dynamic contributions to the bending moments, direct forces deflections, etc. are evaluated using standard random vibration methods, the dynamic response value obtained being the largest peak value likely to occur during the design wind storm.

6 For a linear structure, the total design value of any response quantity is then obtained by direct superposition of the mean and peak fluctuating components.

Ultimate load design

For the overall design for structural safety of the tower it was decided to accept a 1% risk that the design windspeed would be exceeded in a period of 50 years, equivalent to a design wind return period of 5000 years. The resulting extreme mean hourly design windspeed estimated from the calculated extreme value distribution was 36 m/s at ground level, the variation with height being taken to be represented by the $\frac{1}{6}$ power law. A partial load factor of unity was taken for the ultimate load design at this extreme wind condition.

The steel aerial mast at the top of the tower is essentially a structural appendage, which may in principle have a different design classification to the concrete tower. Considerations of the function of the aerial mast and the economic consequences of failure, however, led to a comparable design basis for the entire structure.

The tower structure was analyzed for this and other wind loading conditions of interest in the drag direction, using a static and non-deterministic dynamic numerical procedure, all computations being performed on a CDC 6600 computer. In the computer program, the tower was represented by an assembly of one-dimensional beam-column elements interconnected at nodal points, utilizing a lumped mass idealization for the dynamic analysis.

In calculating the structural responses the secondary effects resulting from the elastic deformation of the structure were included in the analysis and the effect of cracking of the concrete under the extreme design load checked separately.

The drag coefficients were selected on the basis of the high Reynolds Number involved:

Circular tower and mast sections	0.7
Turret structure	1.0
Mast sections with helical strakes	1.3

The first four natural vibration mode shapes and frequencies obtained from the frequency-mode analysis, and shown in Fig. 9, illustrate an interesting feature of the Emley Moor tower in that the second and fourth modes are essentially motions of the aerial mast portion. It was found in fact that these modes of vibration corresponded very closely to the first and second modes respectively of the aerial mast alone. This clearly indicated that the fundamental mode of the tower, although providing the predominant contribution to the dynamic stresses of the concrete shaft, would not be sufficient to represent the overall oscillatory behaviour. Indeed, the second mode provided the most important contribution to the stresses in the aerial mast.

The structural damping values assumed for the various dynamic design conditions for the tower are given below for the lowest modes; damping was allowed to increase for higher modes.

Concrete shaft	0.01 – 0.02	Critical
Steel aerial mast	0.005 – 0.01	,,

The total probable peak (ultimate) bending moment distribution for the tower is shown in Fig. 10.

Working load design

To ensure that permissible material stresses are not exceeded under working load conditions, and to satisfy functional design requirements, a full dynamic analysis was carried out using the once in 50 years mean hourly windspeed. After the dynamic behaviour of the structure had been established, it was found possible to 'factor down' the ultimate load responses, i.e. moments, forces and deflections, for the working load designs. The wind loads acting on the tower were also derived from *BRS Digests 99 and 101*, and later from *CP3: Chapter 5: Part 2: 1970*, and applied as static forces for working load design comparisons using a building life factor (S_3) = 1.0.

The values for the drag coefficients used in the ultimate load analyses were retained for the working load design checks. The distribution of working load bending moments over the tower is also shown in Fig. 10 for comparison with the ultimate load moments.

Aerodynamic stability of aerial mast

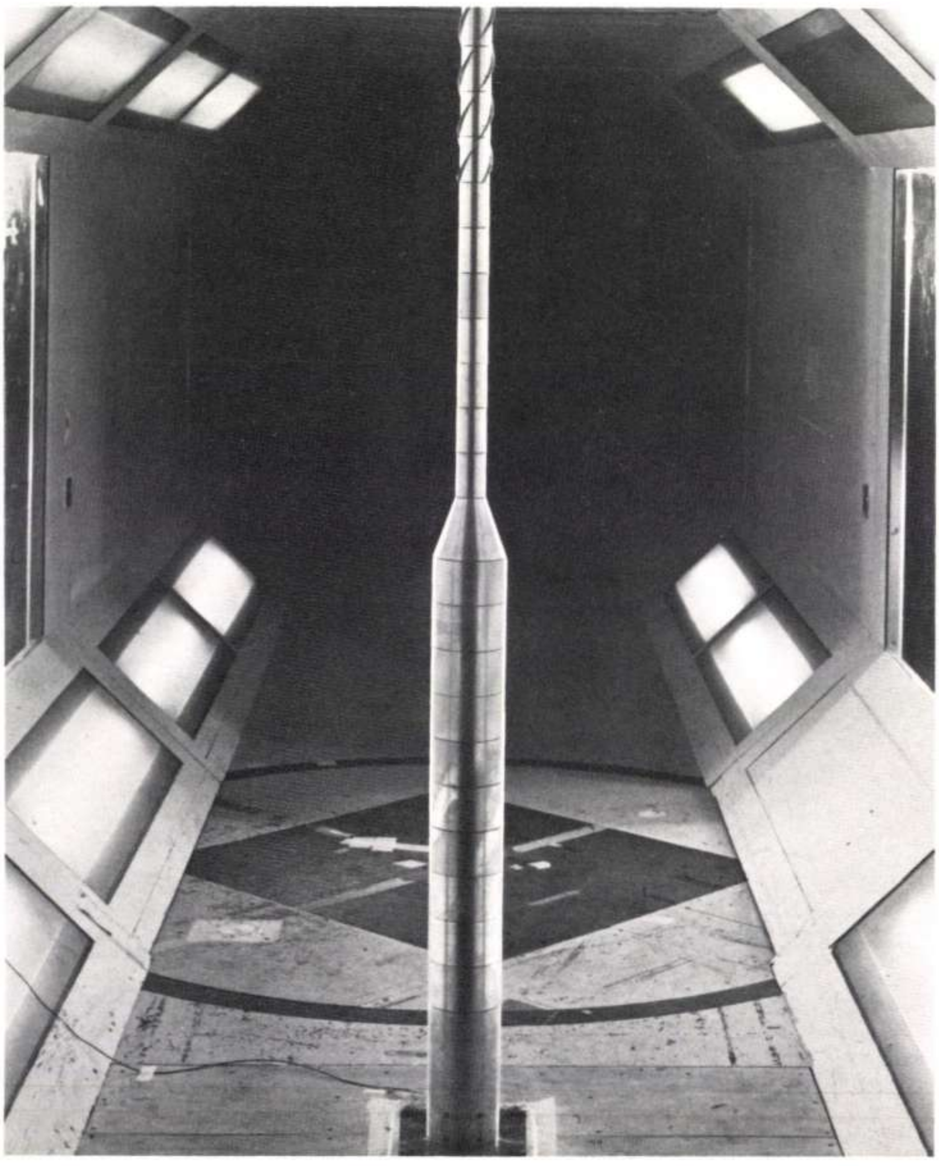
In addition to considerations of the effects of wind drag, the circular plan shape and very slender proportions of the tower made it necessary to study the aerodynamic stability of the tower as influenced by fluctuating aerodynamic forces that may result from the shedding of vortices from the various cylindrical portions of the structure.

Although vortex shedding over part of the concrete shaft was theoretically possible, consideration of the influences of shaft taper, the turret and end effects at the shaft head indicated that shedding would not be well correlated and cross-wind oscillations, if any, would be relatively insignificant. The more flexible steel aerial mast was expected to be more prone to aerodynamic instability, particularly in view of the lower structural damping inherent in the system. Furthermore, due to the operational requirements of the aerials themselves it was important to ensure the absence of regularly occurring, large amplitude oscillations of the mast.

Investigation of the vibratory behaviour of the mast showed that the natural frequencies at which vortex shedding effects would be most significant were 0.58 and 1.83 Hz, corresponding to modes 2 and 4 respectively, of the complete structure.

A series of wind tunnel tests was performed to study the aerodynamic stability of the aerial mast and the effectiveness of spoilers in suppressing any excitation. The tests were carried out at the National Physical Laboratory under the direction of Mr. D. E. Walshe[4] using an aeroelastic model of the aerial mast mounted in the atmospheric wind tunnel (Fig. 11), the model being designed so that the fundamental and second modes of the model were representative of the expected full-scale mast behaviour in modes 2 and 4 respectively.

Fig. 11
Model of the aerial mast in the wind tunnel (Photo by courtesy of the National Physical Laboratory)



Although there was evidence of excitation of each cylinder at both frequencies, pronounced instability occurred only in the fundamental mode, due to excitation of the upper cylinder within the critical windspeed range. This condition produced regular and well maintained oscillations and resulted in the largest amplitudes. It was found, however, that the addition of helical strakes to the top third of the upper cylinder suppressed this instability to an acceptable level.

Investigations were also made into the possibility of increasing the damping of the aerial mast structure by means of block dampers located at the support bearings of the mast within the concrete tower. It appeared that such devices could be designed to provide sufficient damping to effectively reduce the mast oscillations resulting from vortex excitation. In the event, however, it was found impossible in the time available to ensure that the theoretical performance of the dampers would be realized in practice.

Since construction, the aerial mast has been instrumented with anemometers and accelerometers in order that the actual vibratory behaviour of the mast may be studied, and it is hoped that data thus obtained will lead to a better understanding of full-scale structures of this kind.

Icing

Experience on site had shown that severe icing could occur and it was necessary to demonstrate that it was unlikely that large pieces of ice could become dislodged from the new structure. With conventional latticed and guyed steel structures this has presented at times a serious hazard to the users of the roads and buildings adjoining the site. Although the proposed solution, having mainly concrete surface and no exposed steelwork at all, was inherently more satisfactory in this respect, the designers made every effort to maintain a clean contoured silhouette, and to avoid incorporation of any features which could constitute future ice traps. Additionally, tests were conducted in the Climatic Test Chamber of the British Aircraft Corporation at Weybridge to simulate various freezing and thawing conditions. Sections of GRP cladding and concrete pipe (both 1.52 m in diameter) were used and it was clearly demonstrated that with the structure envisaged, the ice was more likely to thaw in place than fall off in lumps.

Foundation design

The foundation form as a reinforced concrete gravity annulus was adopted primarily in the interests of speed, both in design and construction time. The cross-sectional profile, with sloping underside surfaces to provide adequate horizontal key to the bed-rock while minimizing the rock excavation, and with sloping top surfaces to increase the concrete volume without the use of formwork, is placed eccentrically to the wall springing in order to minimize the concrete volume required to provide the necessary overturning resistance. The 'lozenge' section has a high torsional strength to absorb the forces created by the eccentric configuration. The general arrangement, including a typical cross-section, is shown in Fig. 12.

The equations used to determine the bearing pressure distribution below an annular gravity footing, subject to an increasing overturning moment, were those developed on a previous project[5]. The variation of extreme bearing pressure with applied moment (expressed as a proportion of the working moment – load factor) is plotted for the Emley Moor foundation in Fig. 13. The foundation dimensions were chosen so that:

(a) Under working load conditions the pressure under the windward toe is zero.

(b) A load factor greater than 2 against overturning is achieved with a maximum bearing pressure under the leeward toe of 1190 kN/m².

The bending and torsional reinforcement was determined, after computer analysis, as an equivalent grid framework under ultimate conditions.

Allowance was made in the reinforcement at the base of the shaft for the foundation deformations.

Concrete shaft design

As mentioned earlier, design at ultimate load is the logical procedure for overall safety, particularly in view of the large precompression resulting from the self-weight of the tower. Indeed the ultimate load requirements proved to be the most onerous conditions in determining the vertical reinforcement in the concrete shaft.

The dynamic analysis of the concrete shaft and supported steelwork determined the envelope bending and shear force diagrams under ultimate wind imposed loadings. Additional moments were included to allow for the secondary effects of eccentric dead weight in the deflected form under cracked conditions. The vertical reinforcement necessary to provide the ultimate bending capacity was determined in accordance with the commonly used assumptions for reinforced concrete under ultimate conditions. The governing equations for an eccentrically loaded annular section have been expressed in a convenient form for design purposes by Greiner[6] and they and the resulting design curves were used as a basis for the design of the vertical reinforcement.

Two reinforcement meshes, one at the outer and the other at the inner face of the annulus, were used throughout the height of the structure. The vertical reinforcement was detailed in lengths of two lifts of 4.6 m plus a lap length, half the required amount being placed in each lift to achieve a staggered lap distribution.

In terms of the steel area expressed as a percentage of the gross concrete area, a minimum value of 0.35% was considered to be appropriate for a structure of this kind. This value exceeded the strength requirement at both the base and top sections while the maximum value of 1.11% was reached at a height of 128 m.

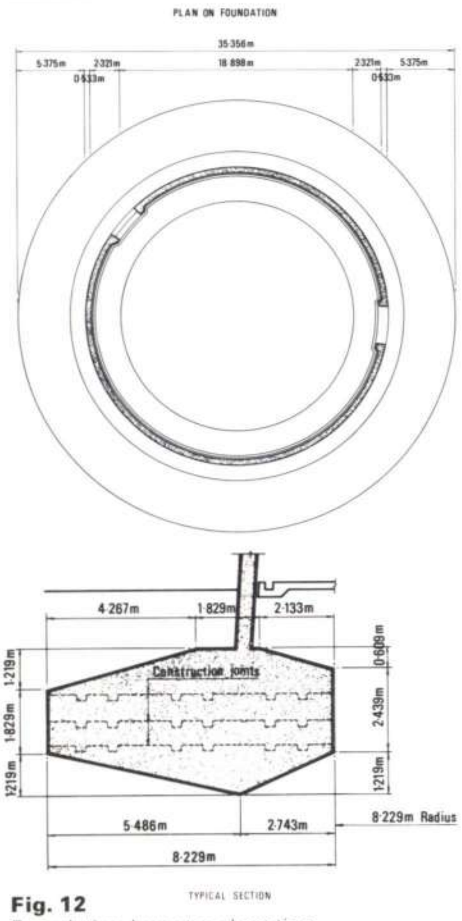


Fig. 12
Foundation layout and section

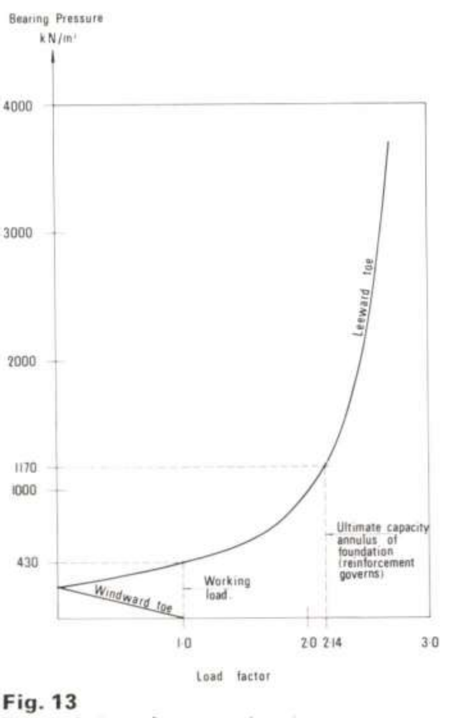
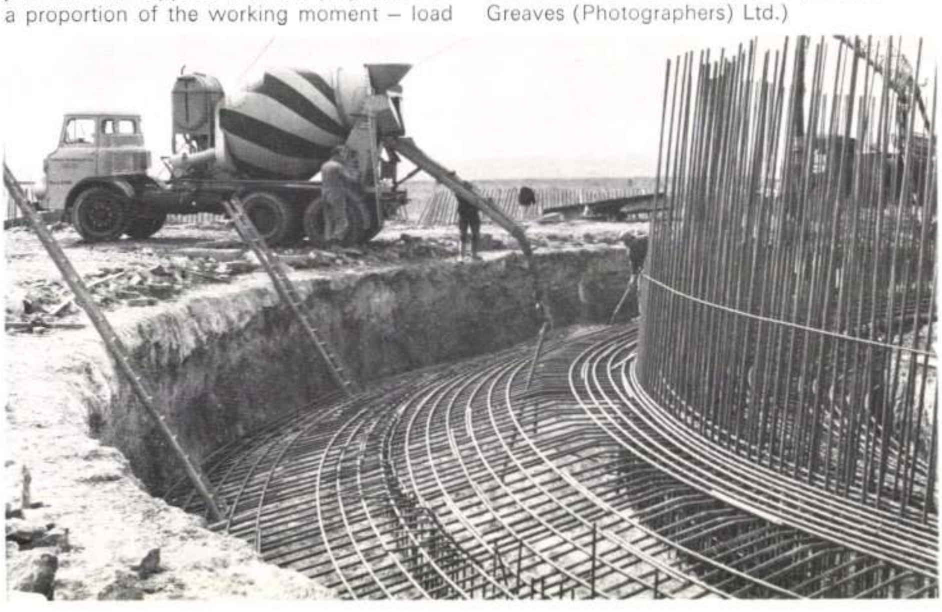


Fig. 13
The variation of extreme bearing pressure and its relation to load factor

Fig. 14
Foundation under construction (Photo: Greaves (Photographers) Ltd.)



The horizontal steel was determined by considering each section as an independent ring, loaded principally by positive and negative wind pressures according to the Roshko distribution. Additional loads were included where appropriate, arising from the horizontal component of the bending forces due to the vertical curvature of the shaft wall. The horizontal steel was detailed as closed rings, each made up of a number of standard mill lengths with a closing bar as necessary, all provided in straight lengths and sprung into position.

In the outer layer of reinforcement, the horizontal steel was placed outside the vertical steel to restrain the latter from springing outwards due to the vertical curvature when subject to bending tension. Similarly, in order to restrain pull-out tendency of the horizontal steel of the inner layer of reinforcement, the horizontal steel was placed outside the vertical steel. The cover at the outer face was 50 mm and, at the inner, 40 mm.

GRP cladding

The data relating to the GRP cladding panels, and the mechanical properties assumed, are given in Table A. The design requirements were stringent and careful analysis was required to obtain the optimum balance between face thickness and depth of ribbing. It was considered more economical to employ a single skin with polyurethane foam-filled stiffening ribs, than to use a complete sandwich panel.

The panels were made in GRP moulds formed from two timber master moulds. Very careful attention to panel edge details was required, as accuracy of fit on site was important.

Quality control tests were carried out on 460 mm square panels formed at the same time as, and in step with, the main panels. The tests included tensile strength, cross-breaking strength, elastic modulus in bend and resin/glass ratio.

The panels were required to have a very low surface spread of flame (Class 1 to *BS476: Part 1*). Normally this is achieved by the incorporation of large amounts of fire retardant fillers. However, these tend to have an adverse effect on strength and to increase the variability between panels, due to the fact that the resin material is more difficult to handle. Instead, on this project a new intumescent coating was applied to the inner face of the panels. This coating had been developed specifically for use with glass fibre reinforced plastics.

Table A: GRP Panel data

Panel data	*UHF cladding*	*VHF cladding*
No. of rings of panels	13	9
No. of panels per ring	3	6
Nominal height of ring	1.88 m	3.05 m
Ring diameter	1.52 m	3.66 m
Face thickness of panel	8 mm	10 mm
Depth of stiffening ribs	6 mm	127 mm
Depth of flanges	76 mm	127 mm
No. of ribs per panel	4	4

Mechanical properties of GRP laminate (BS 2782)

Tensile strength	125 N/mm²
Crushing strength	103 N/mm²
Cross-breaking strength	180 N/mm²
Elastic modulus in bend	5.5×10^3 N/mm²

Foundation construction

Excavation for the foundations down into the weathered sandstone was carried out between two mass concrete retaining walls. This minimized the amount of excavation and enabled ready mixed concrete trucks to discharge the concrete directly into place. Lorry mounted pumps were used for the remainder. Fig. 14 shows the foundations under construction.

Reinforced concrete shaft

The shaft was cast in lifts of 2.3 m using specialist equipment designed and developed by the contractor for chimney construction. The basic elements are illustrated in Fig. 15. The steel derrick was cruciform in plan and hung from inserts cast into the concrete. A rigid angle ring hanging from the head of the derrick supported the external scaffold and the external shutters. It was also used to adjust the diameter of the external shutter. Two internal working platforms hung from the derrick and these were served by three high speed rope guided hoists. The derrick was lifted for each new lift of concrete and from time to time both it and the working platforms were cut down to suit the reducing diameter of the tower.

A single set of steel shutters was used for the whole of the shaft. The external one was composed of a series of plates tensioned together and overlapping at intervals around the circle to take care of the variations in diameter and taper. The internal shutter was sprung towards the outer by a number of high tensile steel spirals clipped on to it. Adjustment for changing diameter and taper was made by interchanging sheets of different sizes (Fig. 16).

To limit the variation in diameter which the derrick had to accommodate, the bottom 23 m of the shell was cast from scaffold using ready mixed concrete and lorry mounted pumps. From 23 m up to 46 m the derrick was used in conjunction with the site batching plant, but the working platforms were provided by further scaffolding.

During the winter months shutters were insulated with polystyrene and the water for mixing concrete was heated. Construction continued in all weather conditions, only about 60 working hours being lost as a result of high winds and low temperatures.

The full height of the tower was cast in 122 lifts over a period of 44 weeks, the highest rate of casting achieved being five lifts per week. The concrete portion of the structure was topped out in September, 1970.

In all, the tower contains 7000 m³ of concrete

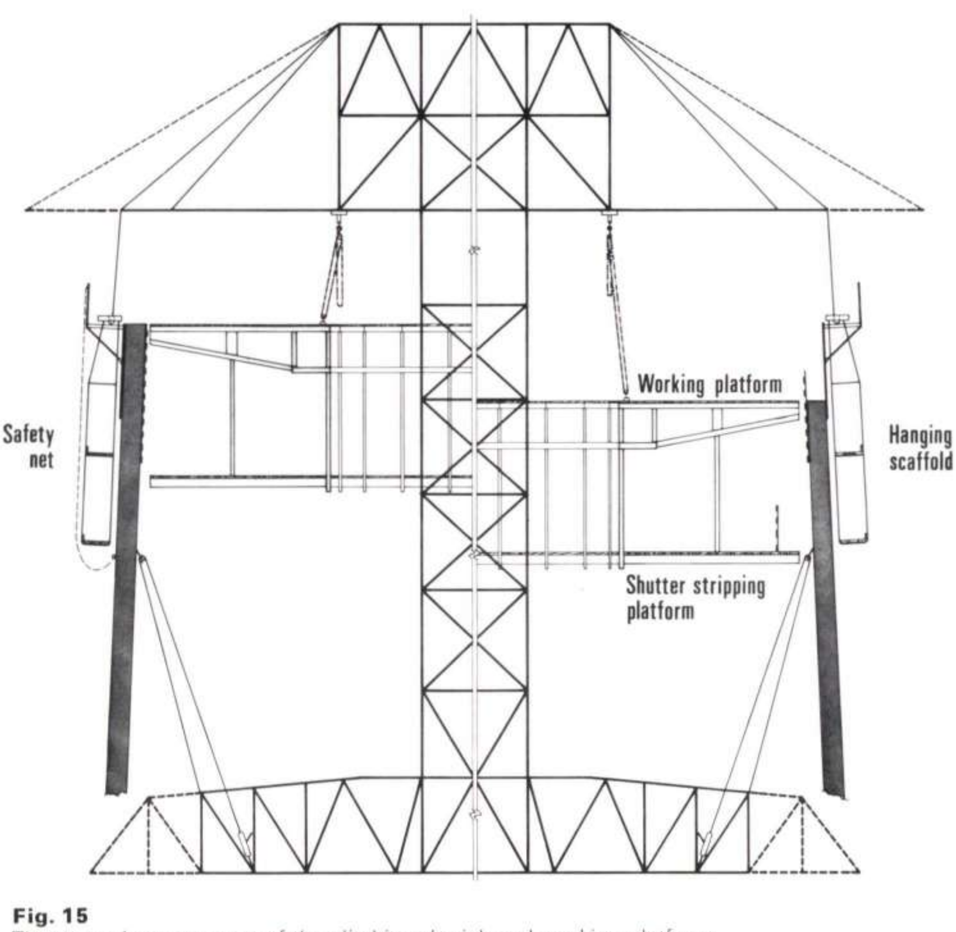


Fig. 15
The general arrangement of the climbing derrick and working platform

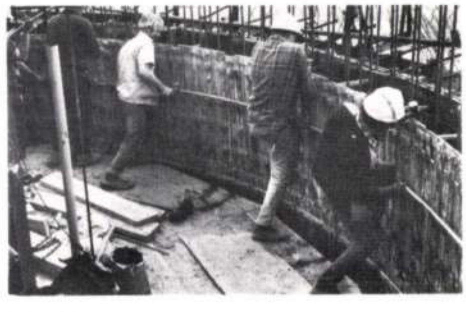


Fig. 16
Working platform (Photo: David Aldred)

and 660 tonnes of reinforcement. The lowest lifts of wall each contained approximately 76 m³ of concrete and were poured at the rate of 9 m³/hour. The topmost lifts contained approximately 15 m³ each and the rate of pouring was 4 m³/hour.

The centre of the tower was set out for each lift by using an autoplumb stationed at ground level and sighting an illuminated target. By the time construction reached 183 m, significant solar movements could be detected and setting out was usually carried out in early morning. Fig. 17 shows one plot which was made of this movement.

Hoisting aerial mast

The mast was first assembled into 6.1 m lengths outside the tower and these were lifted by mobile crane into the central lift cage (Fig. 18). When the mast was complete, the ITA's aerial contractors fitted the UHF aerials and GRP cladding.

The method of hoisting the mast, by means of hydraulic jacks of Swedish origin which act on special wire rope, was developed in close co-operation with the main contractor.

The total weight to be lifted was 62 tonnes and nine jacks were employed, three on each face of the mast, mounted on the slab at 269 m. The capacity of each jack was 12 tonnes and one in each group of three was used as a working spare. All jacks were controlled from a central console and could be operated independently to level the load. The lifting procedure is illustrated in Fig. 19. The wire ropes were attached to a lifting framework beneath the mast and stability was achieved by fixing rope guides to the mast at the centre and top. These were removed in turn as they reached the jacks.

Guidance through the lift cage was provided by timber skids. On the top slabs, heavy steel rollers were provided to bear on the corners of the lower section of the mast and to resist wind forces. The timber skids immediately below the top slabs were strutted back to the concrete since they were also required to resist wind forces.

Hoisting began on 18 November, 1970 and the mast was at the top by the evening of 20 November. During the following two days the strakes were fitted to the GRP cladding whilst it emerged slowly through the top of the tower. The third day was lost due to high winds. The mast was then raised so that approximately 30 m projected and a steel transition section in the cladding was fitted below the 1.52 m diameter GRP. The following day the mast reached its final position and brackets were bolted to each face above the 273 m slab. The mast was lowered back on to neoprene bearings, levelled in beneath the brackets.

The permanent horizontal restraints at the two top slabs are provided by two laminated rubber bearings placed between the mast and the edge of the slab on each face at both levels. Load was induced into each of the bearings by inflating a *Freyssi* flat jack placed behind it. Resin was used in the jacks and they were left to harden in position.

When the mast had been fixed, the ITA's aerial contractors raised the feeder cables and the UHF aerials came into service on 21 January, 1971. The lower VHF aerials and GRP cladding were erected from the outside of the tower using a cradle running on guide bonds. The work took three months and these aerials came into service on 21 April, 1971, just over two years from the collapse of the original mast.

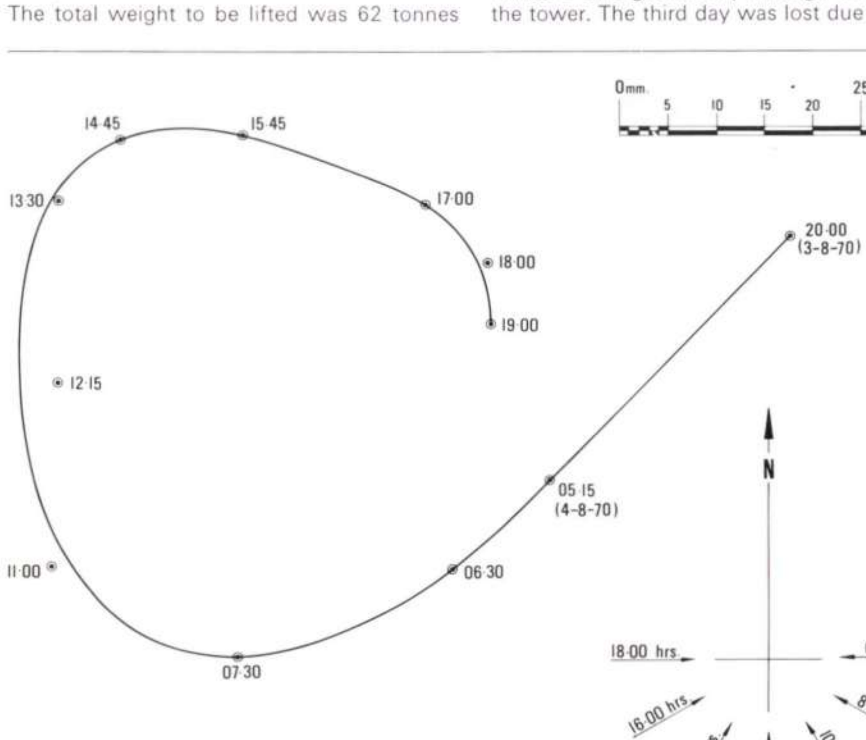


Fig. 17
Plot of solar movement at 257 m (840 ft) level

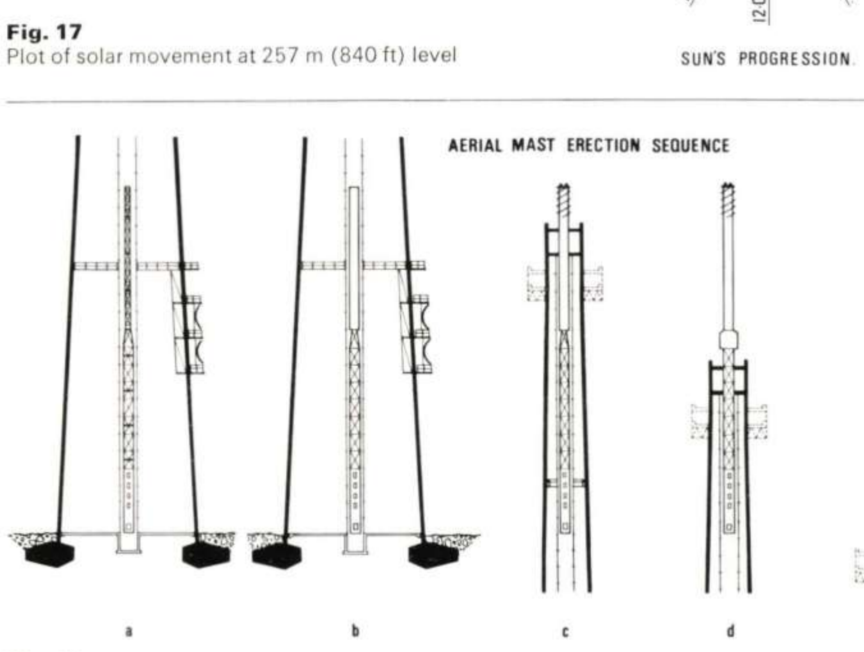


Fig. 19
Diagrammatic illustration of the hoisting procedure

Fig. 18
The aerial mast prior to the hoisting operation (Photo: Geoff Rooke)

Credits

The client for the project was the Independent Television Authority. Permission for the publication of this paper was obtained from Mr. Howard Steele B.Sc. (Eng.), ACGI, the Director of Engineering to the Independent Television Authority.

Main contractor: Tileman & Co. Limited.
Structural steelwork sub-contractor: J. L. Eve Construction Co. Ltd.

Advice on problems of vibration and fatigue, provided by Dr. T. A. Wyatt of Imperial College, London and the assistance received from the Meteorological Office at Bracknell in matters of weather information and wind data, are also acknowledged.

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Building 14 Russell Offices, Canberra

Mick Lewis and
Dan Ryan

This is an edited version of an article which first appeared in Constructional Review, 44(2), *pp. 18-23, 1971.*

The Russell offices are located at one of the points of the National Triangle, the basis of Canberra's Central Area planning. The focal point of the Russell group is the American War Memorial. Sited at one end of the group of buildings, Building 14 (job no. A164) is an important visual component creating a climax to the group particularly when viewed from the Kings Avenue axis. It is also the first building of significance to be seen as the city is approached from the airport. Distant sitings from points such as the proposed Parliament House site and the opposite shore of the eastern basin led to the design of a structure taller than the existing buildings which in no way detracts from the prominence of the War Memorial.

The existing Russell offices are all clad in grey granite and their façades are fairly flat in appearance. Precast concrete frame units were chosen to form the structural envelope of Building 14, with care exercised in the choice of surface finish to ensure harmony with existing buildings. The strong pattern of light and shade produced by the windows being recessed behind the precast frames contrasts with the smaller scale modelling of the adjoining buildings in the group. The recession of the window wall also provides sun protection and window cleaning facilities.

The building comprises 13 storeys, the first three containing offices, plant and computer centre with nine typical office floors above and the main plant room on the 13th floor.

The building's users are subject to security classification and required a large number of small offices with no false ceilings and a high degree of acoustic privacy. In order to fulfil these requirements a rectangular plan form was selected 50.3 x 17.4 m with a central core 27.4 x 5.5 m. The core contains toilets, lifts, stairs and strong rooms. A corridor surrounds the core leading to individual offices 4.1 m deep and planned on a 0.9 m module using demountable timber partitions.

Precast ribbed floor units with a soffit finish straight off the steel form were selected as the most satisfactory and economical means of fulfilling these requirements and providing a pleasing and uniform ceiling finish which could provide easy soundproofing of partitions.

The central core is cast in situ and provides the stiffening for all lateral loads and also the vertical support for the surrounding floors which consist of precast units. The precast column frames to the external walls transmit vertical loads but none of the precast elements are designed to resist the transverse loads on the building which are wholly accepted by the core.

This removal of the constraint to provide lateral stiffness to the building in the external column and floor element has enabled the introduction of a special connection between the precast units which provides for speed and ease of erection and for very accurate alignment along the building façade.

The central core walls were designed to be constructed by slip forming. This work proceeded simultaneously with foundations

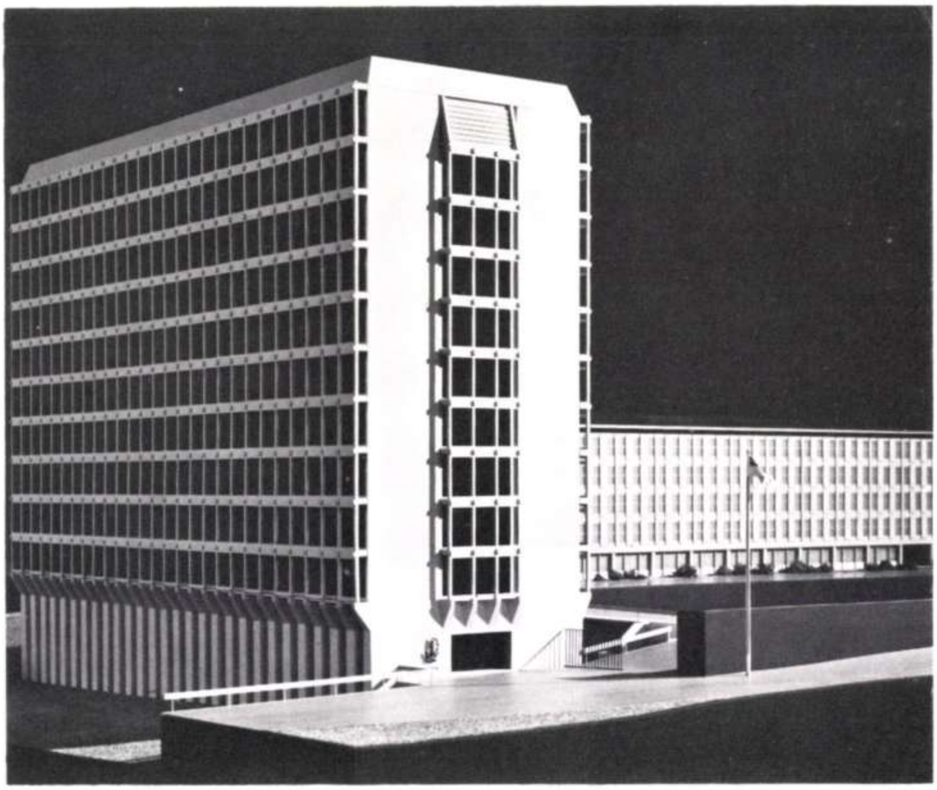


Fig. 1
Building 14: Model
(Photo: David Moore, Sidney)

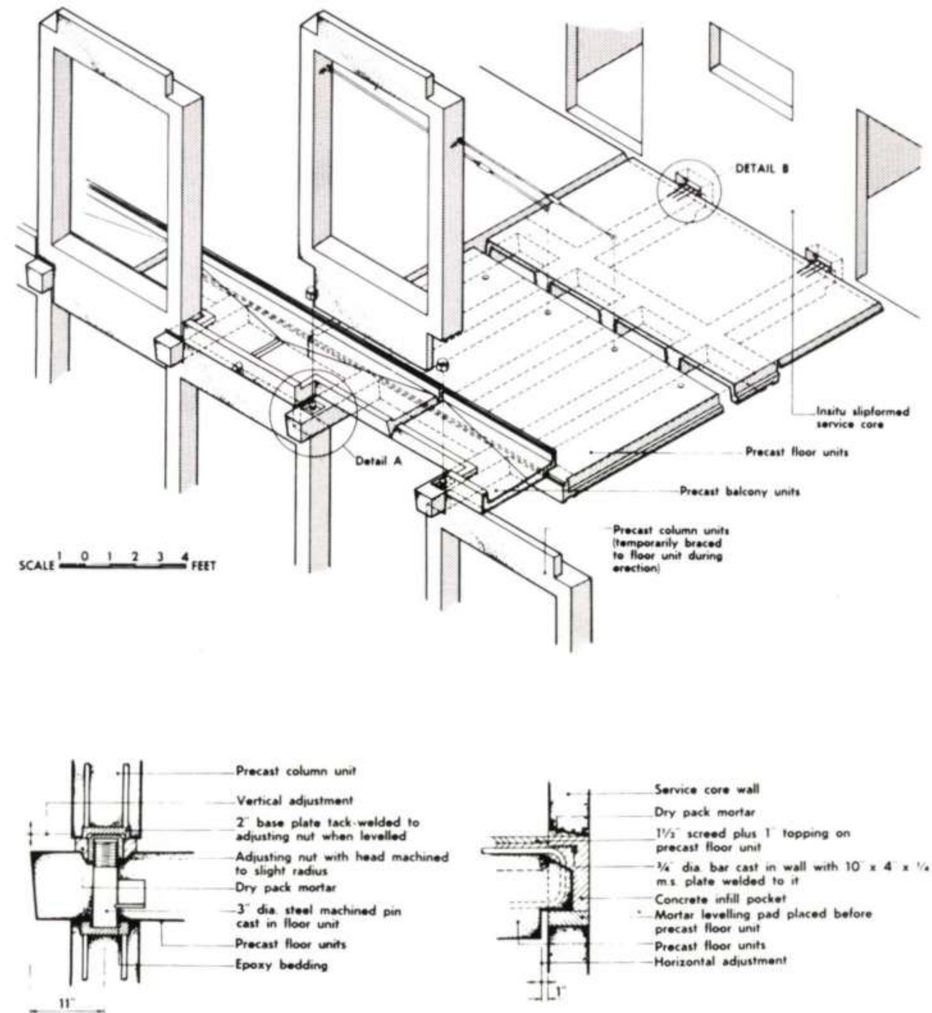
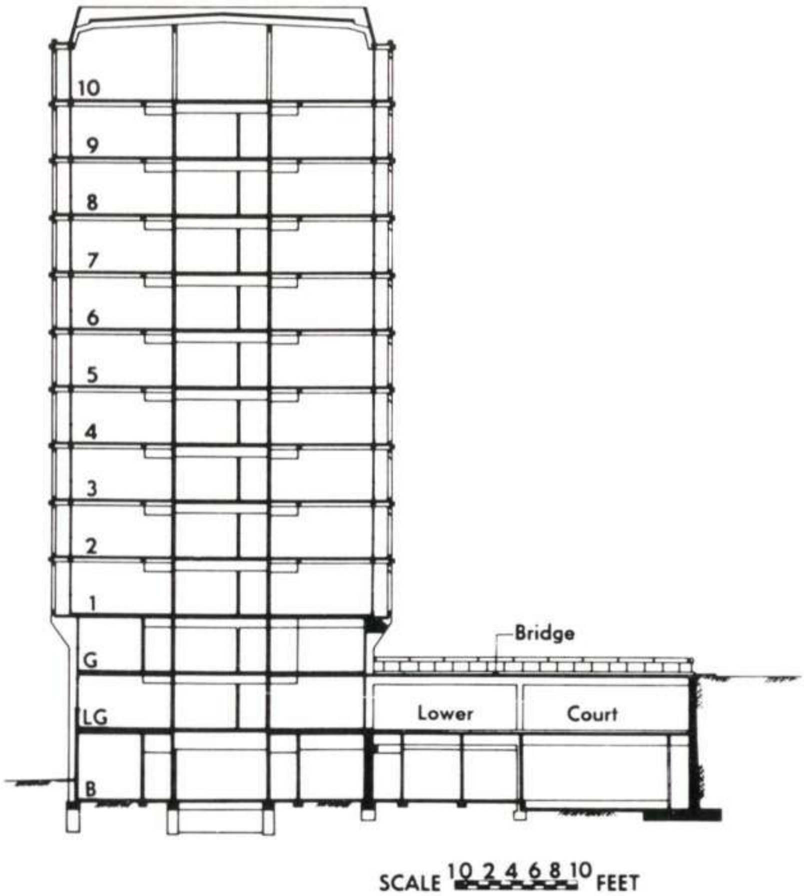
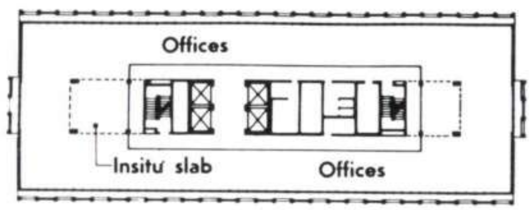


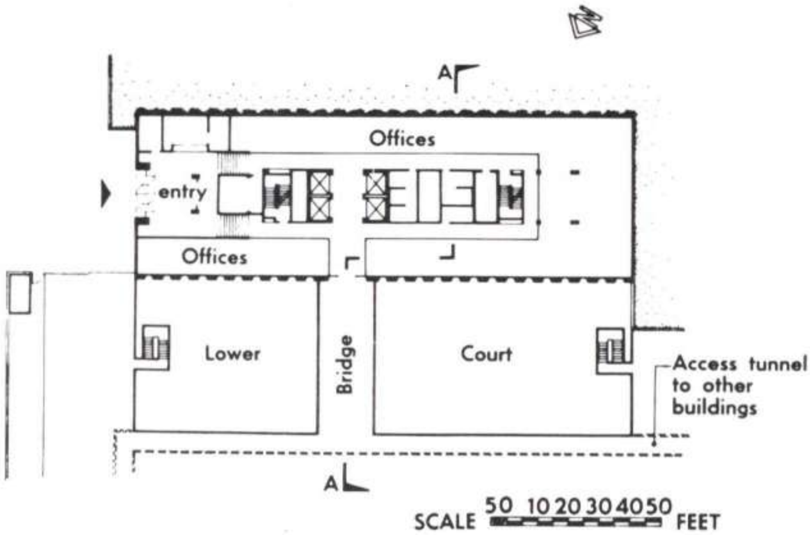
Fig 2
Details of precast units



SECTION A–A



TYPICAL FLOOR



GROUND FLOOR

Fig 3
Section through Building 14
and floor plans

for the perimeter columns and internal in situ columns. The construction cycle allowed for a lift to be completed every three or four days. On completion of the core walls the large perimeter columns were constructed, which marked the end of the 'wet' construction. These were cast to their full three storey height in one pour with reinforcement left projecting for the intermediate slabs to avoid horizontal construction joints.

At the heads of these columns, haunchings extend 1.1 m from the face of the columns. At the end of these haunches 230 x 230 mm pockets and pins carry the precast column frames above. Tolerances on these in situ columns in all planes were $\pm$ 3 mm.

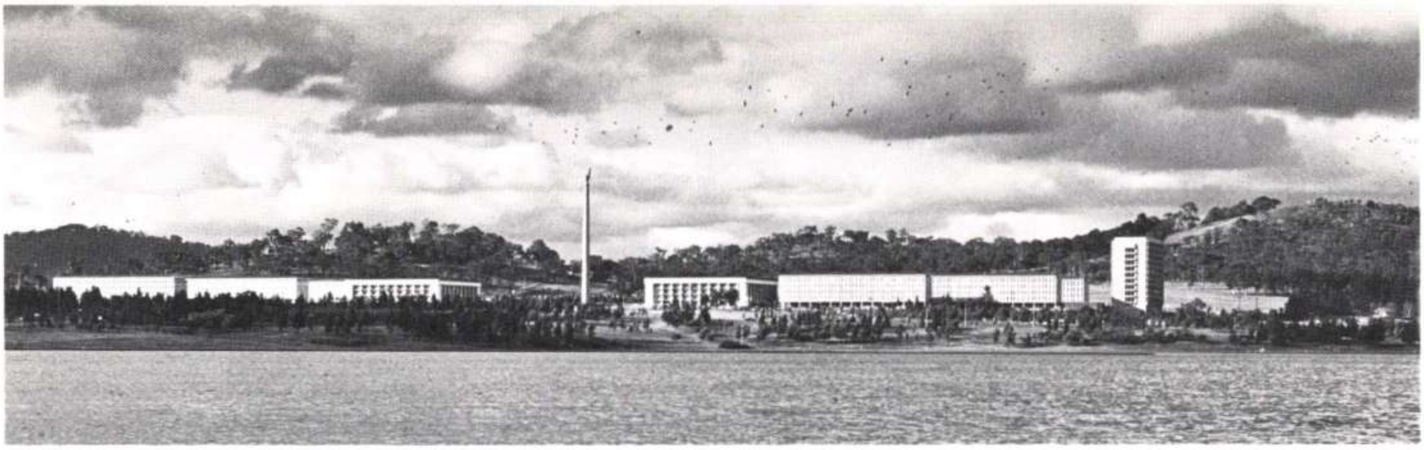
The quality of the plasticized plywood column formwork and concrete mix design was controlled strictly at all times to give a strip that ensured a high quality of finish.

Concrete was placed using a series of three tremmies and the mix showed no sign of segregation. Compaction was by internal vibration. Faces were bush hammered 10 to 28 days after placement and a very high standard of finish was achieved. While construction of the in situ slabs was proceeding, precasting had commenced and a sufficient number of units were stockpiled in readiness for erection.

The basic ribbed floor unit is 3.7 m wide by 7 m long and weighs 6.6 tonnes. Two main beams span 5.94 m at 1.83 m centres with an integral 64 mm slab. The main beams project 1.1 m beyond the slab. The overall depth of the unit is 320 mm and the depth of the projecting beams 254 mm. Intermediate beams between the main beams span between transverse beams coinciding with the window wall and corridor partition. These intermediate beams at 0.9 m centres provide for soundproofing the demountable partitions.

The main beams of the floor units sit within pockets boxed out of the core walls and on the precast column frame. The seating detail at the core line was kept the same for all support conditions. The support pin detail at the other end of the unit varied in size depending on the floor height, column loads being transferred through the pin. Three different pin sizes were used, 64 mm mild steel, 70 mm and 76 mm high tensile steel.

Fig 4
Close-up of precast units
(Photo: Harry Sowden)



The main column section of the frame unit is 267 x 256 mm tapering to 216 mm. The design allows for a reduction of 6 mm on three sides for bush hammering. The columns are reinforced with four 16 mm structural grade bars welded at each end to plates which transfer the load from the pin to the floor units. The section was designed to cope with the axial loads and moments induced by the maximum eccentricity possible at the extreme of construction and erection tolerances. A cage of galvanized high tensile mesh surrounded the main reinforcement in order to give a two hour fire rating.

External balcony units sat on the projecting beams of the floor units with the front face bush hammered and in line with the top of the frame unit. These serve as fire barriers between floors.

64 mm thick bush hammered blades were used for additional sun protection on the north façade. They were attached to the frame units after erection by site drilling threaded fasteners and bolting.

Load-bearing wall panels are used at each end of the building at typical floor levels. These are 3 m x 2.1 m x 150 mm and weigh 3 tonnes. They are bush hammered on the external face and joints are sealed with an elastomeric compound. They carry a 0.9 m strip of floor loading as the end floor units are supported continuously on these units. The external face of the floor unit edge beam is bush hammered to form part of the external wall treatment.

All floor units, frame units and typical balcony units and external roof beam units were cast in steel moulds. In special cases units were cast in timber moulds where no more than 12 were required.

Manufacture was based on a one-day cycle. Units were lifted in the morning, the forms cleaned out, checked for trueness of profile, reinforcement placed and concrete poured in the afternoon. The units were steam cured overnight for an average of six hours steaming at peak periods.

The large proportion of fines and the nature of the granite used in the mix caused some minor problems in steel trowelling the concrete in the forms. The suitable time for trowelling varied largely with weather conditions and it was found that a constant watch had to be kept.

Great care was exercised in supervision at the casting yards and the extremely high degree of finish obtained proves the worth of this operation.

In the main entrance to Russell 14, mounted on an armour plate glass security screen, there is a representation of the traditional Naval Fouled Anchor designed by the architect Max Collard. It is formed of a series of brass rods of different diameters, welded and polished. Immediately behind this screen on the rear wall of the entrance lobby are two vertical panels representing the traditional colours of the naval flag, executed in brilliant red and blue Duco paint, each with two tones of the one colour, making a vivid and interesting composition contrasting with the neutral colours of the concrete and natural timbers used in the foyer.

Russell 14 is a very happy addition to one of Canberra's more important groups of buildings. It complements the group and the sense of cohesion is further emphasized by the extension of the existing security walls of the site to tie in with the base of the building.

Fig. 5
The Russell complex, Canberra.
Building 14 is on the right
(Photo: Harry Sowden)

Credits

Client: The National Capital Development Commission.

Architects: Collard, Clarke and Jackson.
Quantity surveyors: Rider Hunt and Partners.
Main contractor: Mainline Constructions Pty. Ltd.
Precast concrete: Monier Ltd. (Canberra)

Fig. 6
Building 14: general view
(Photo: Harry Sowden)

The raising of The Wellington Inn

John Charge

Historical background

'When Queen Elizabeth I came to the throne in 1558, this building, now known as *The Wellington Inn*, was 230 years old. Long live Queen Elizabeth II.' This laconic claim appears on a poster on the outside wall of *The Wellington Inn*. Documentary evidence shows it to have been built in 1462 as three shops and dwellings, part of the meat market at the centre of old Manchester, then known as *The Flesh Shambles*.

The inn was the birthplace of John Byrom in 1692, author of *Christians Awake* and inventor of a phonetic shorthand, and was said to have been used as a recruiting office by Bonnie Prince Charlie during the 1745 Jacobite rebellion.

When first licensed as an inn in 1830, it was called *The Vintners Arms*. It is now owned by Bass Charrington and is the only timber box-framed building in Manchester.

The larger part of *Sinclair's Oyster Bar*, furthest from *The Wellington*, dates back to the 15th Century. Known as *John Shaw's Punch-House* in the 18th Century, it was a regular meeting place for the cotton merchants of Lancashire. The part adjoining *The Wellington* was only built in 1925.

Both buildings had remarkable escapes from the extensive bombing in the area during the last war, when such buildings as *The Bull's Head*, outside of which once stood the stocks, and *The Slip Inn*, were destroyed, leaving only this terrace of early buildings remaining from the old centre of Manchester.

The Wellington Inn was scheduled as an ancient monument by the then MPBW, now the DOE, and *Sinclair's* was listed as of historic interest. This being the case, the present development had their retention as the focal point of the scheme.

The development and planning

The development is by CWT Developments, (Manchester) Ltd., a subsidiary of Central & District Properties Ltd., in conjunction with Manchester Corporation. It covers 2 hectares, will cost £10m, and will contain shops, offices, two supermarkets, car parking for 700 cars, and includes a pedestrian walk system designed to link the development on both sides of Deansgate with future surrounding developments and with Market Street, which is to be pedestrianized. *The Wellington Inn* and *Sinclair's Oyster Bar* are situated in the centre of the east side of the development.

An underground road will service the entire scheme on the east side which, due to the road systems surrounding the development and overall planning, has only one available entrance and exit position. *The Wellington* and *Sinclair's* lie in a direct line with the new access position and are consequently above the best service road position.

As previously mentioned, the pedestrian walk system is designed to link Market Street with the upper levels of the scheme. This planning requirement necessitates a change in level of 1.46 m to the old buildings being retained, in order that their integration into the new immediate environment can be achieved.

The required service road levels dictated by the new ground floor levels and general construction depths, combined with a clear height requirement of 5.1 m in the road, gave a possible construction depth of 2.44 m under the buildings.

Fig. 1
Bowens: *The Wellington Inn* in the late 19th century

Fig. 2
The Wellington Inn and *Sinclair's Oyster Bar* before raising (Photo: Stan Parkinson)

The largest span of new supporting structure over the service road is 13.7 m which could be accommodated within the available depth. The decision to jack the buildings to the new levels and span over the service road was therefore accepted, being the only fully integrated and uncompromising solution to the many requirements. This was to be carried out prior to the main contract.

The development of the structural scheme

The planning requirements leading to the jacking up and spanning of the buildings as described were the major considerations in design.

The supporting beam positions are dictated by the existing ground floor vertical structure layout, and are generally central under the mean centreline of the existing ground floor supports and the new supports. These beams are carried on eight 'piers' founded on bedrock, the number and basic positions being fixed by the underground service road beneath *Sinclair's* and the access stairs and cellars beneath *The Wellington*.

The Wellington, although not obtaining physical support from the adjacent buildings, was sheltered by them. Its structural condition is very poor, particularly in respect of its stability.

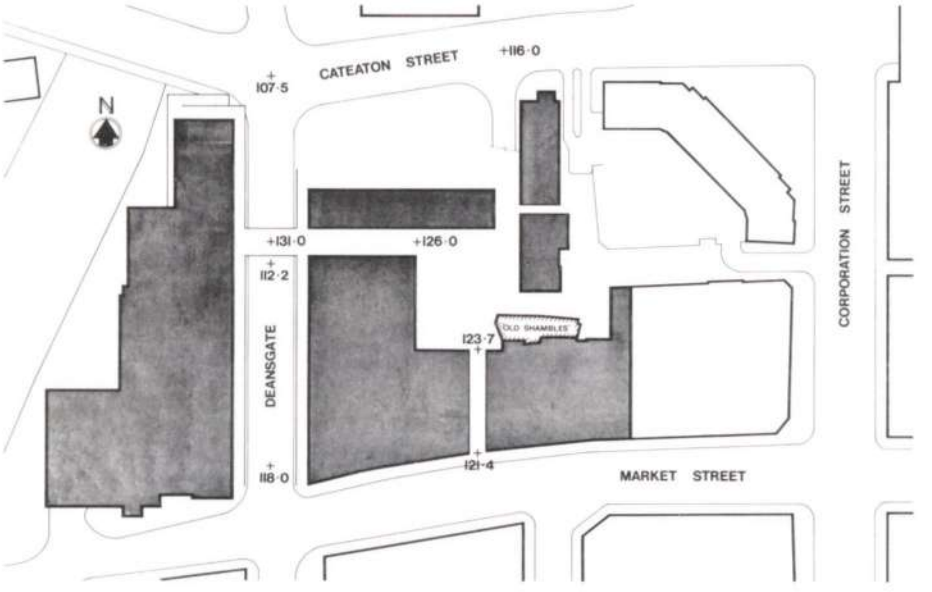


Fig. 3
Plan of Market Place Development

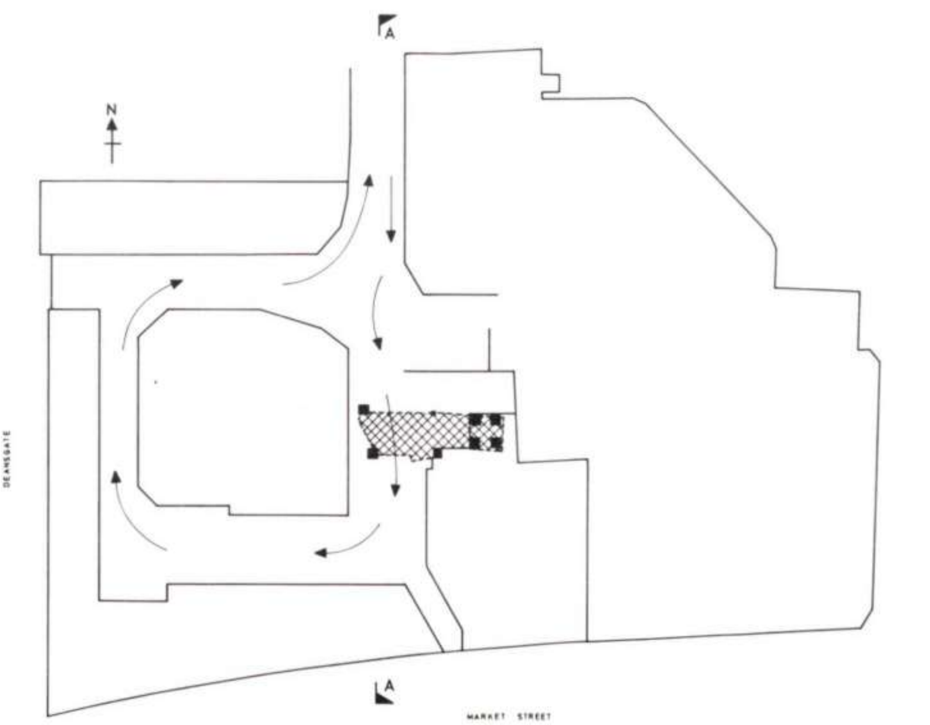


Fig. 4
Market Place Development: plan below ground

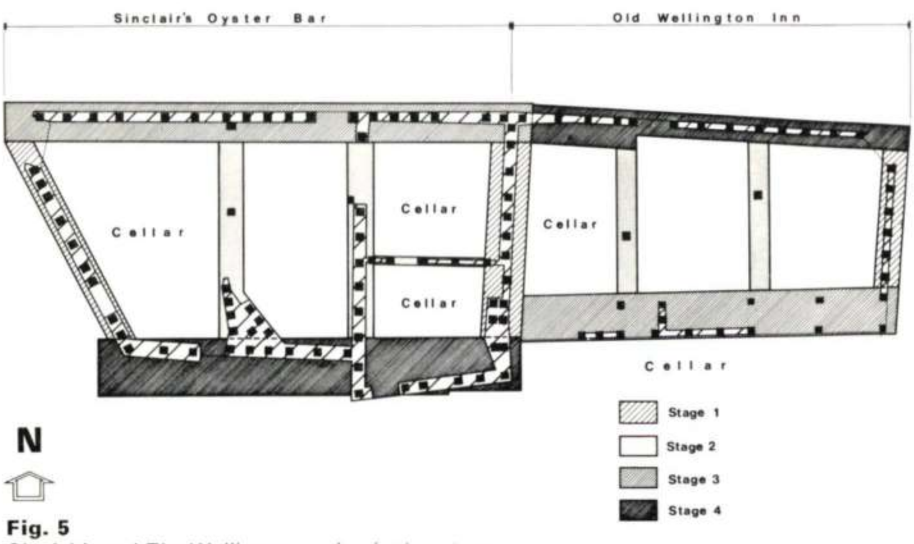


Fig. 5
Sinclair's and *The Wellington* underpinning stages

The temporary change of external environment during the course of the lifting and main contracts required the building to be fully braced. This bracing was installed as the first operation and is internal, as it had to be lifted along with the building.

Sinclair's, although not in such a poor structural condition as *The Wellington*, relied upon the Victoria Street terrace of buildings for lateral stability, its ground floor having no cross walls and the Victoria Street gable wall being largely window and doors. All ground floor openings under cross walls at 1st floor level, and the window and door openings in the Victoria Street gable, were cross-braced and dead shored off the jacking beams.

The contractor, Pynford Midlands Ltd., was chosen at the initial design stage in order that full advantage could be taken of their experience in the specialized field of underpinning, jacking and moving of buildings.

The planning as described dictated the possible maximum depths for horizontal structure and the basic number and positions of vertical supports down to bedrock, for the permanent works.

The premise was made that the final structure would be the structure used during jacking, and on that basis, the contractors were asked what their requirements for temporary works would be. The contractor indicated that this principle would in many instances require underpinning depths far in excess of normal practice, and considered the use of a maximum underpinning beam depth in the order of 0.9 m as more usual, and more feasible. The underpinning beams would then be used to jack the buildings against existing foundations and cellar walls and, where these were not sufficient, against temporary 'high' level pad foundations.

This principle of jacking was also consistent with standard practice, giving foundation pressures during jacking consistent with the pre-underpinning conditions, this being achieved by correct jack distribution.

The total dead load of the existing *Wellington* structure was calculated as 81 tonnes, with the new supporting structure giving a total of 149 tonnes to be lifted. The maximum allowable number of supports to the final structure under *The Wellington* was six, and calculations showed that the preferred underpinning beam depth was suitable for the final design conditions.

The total dead load of *Sinclair's* existing structure was calculated as 318 tonnes which, with the new supporting structure, gave 712 tonnes total to be lifted. The maximum allowable number of piers to the final structure under *Sinclair's* was four, giving a maximum span of 13.7 m. The preferred underpinning beam depth was not suitable for the final design conditions. The contractor therefore proposed that the final structure under *The Wellington* only should be used during jacking and that standard depth jacking beams and standard jacking procedure be used under *Sinclair's*. This meant introducing the final supporting beams, where required in addition to the jacking beams, into the space which would be provided by the building being lifted 1.46 m above its original position. The remaining supports down to bedrock were finally to be installed and pinned to the lifted structure. The first scheme was based upon this principle.

After finalizing this preliminary scheme, considerably more information concerning the development as a whole was available, and the establishment of the required contract time, together with additional items of work which had to be carried out in that time, necessitated a review of the construction sequence and programme.

A survey of the buildings was also now available, which showed that the inner line of cellar walls was well within the superstructure support lines, indicating that jacking off cellar walls may not be feasible. (Prior to this, access to the building was not available to the client).

The scheme to include all the final beams and supports prior to jacking, and thereby eliminating a planned sequence of construction, would clearly reduce both construction time and areas of possible doubt and delay. In the light of these factors it was decided to investigate further the original principle of jacking all final structure off piers down to bedrock.

A second scheme was therefore drawn up. This gave more calculable design conditions during jacking. Results of research and development work on a jacking system for this application led to this scheme being adopted as the basis of final design and pre-site planning.

The depth of beam was, for practical reasons, kept within the cellar depths below ground floor and became 1.98 m maximum.

Design

The design and control of the total free jacking system (that is after initial separation from the ground) was based upon the principle that all jacking points should lift at the same time and at a constant rate, as far as possible and therefore within specified tolerances of variation in level.

A basic system using on-off valves alone would only achieve this by an uneconomical and lengthy process of lifting each jacking point separately.

Various methods of control were investigated by the contractor and the method finally adopted was to introduce a hydraulic flow control valve into each of the six hydraulic lines to the individual jacking points. When calibrated and located to their best advantage these valves will maintain a uniform flow of hydraulic fluid regardless of varying pressures at each side of the valve. As used, they ensured that there would be a flow in all lines, not simply the line with least resistance, the rate of flow depending upon the valve calibration and variations in the system beyond the valve. The total area of jacks at each jacking point was determined from the estimated jacking load, assuming a fairly uniform working pressure for all jacks. The flow control valves were calibrated to allow for the variation in jack area at each point.

A pin joint was designed into the end of each longitudinal beam at the party wall where the *Wellington* beams are supported on a corbel off the main *Sinclair's* supporting beams. Under *The Wellington* the beams are of such stiffness that the pairs of supports to each longitudinal beam act as one support during jacking at the centre of pressure of their jacks. The resulting beam system is very simple, being basically a pair of simply supported beams in the east-west direction of each building, each carrying two cross-beams, with the party wall and two gable beams spanning basically onto the supporting piers. Free rotation is allowed to a large degree by the supporting jacks.

As the final structure is also the jacked structure, deformations within each building were less during jacking than at completion. The junction of the two buildings is the critical point during jacking, due to the possible effects of variation in level between jacking points. Owing to the flexible nature of the timber frame this possible distortion was considered acceptable, within specified tolerances, on the variation in level from datum.

The area of each pier had, for practical purposes, to be 1.83 m square. They were formed by hand excavation down to bedrock some 7.6 m below. Their exact location, following basic positioning from planning requirements, had to provide an off-set from the jacking beams in order that excavated material could be removed, and sufficient area off which to jack, as there were a maximum of nine jacks on any one pier.

The two vertical supports on the party wall line could not be 1.83 m square due to the circulation requirements beneath, the one on the north façade being restricted to 0.76 m x 0.46 m. At this position the jacks were on temporary supports founded on bedrock within the base excavation.

In the maximum exposed condition, during the main contract when the site excavation down to rock isolates the building, it will be approximately 21.3 m to the highest point from the excavated level. The pier sizes and positions are ideal for stability during this time.

The contract work

The work on site started on 11 November 1970 and was executed in seven principal stages, as follows:

1 Internal bracing and shoring
2 Demolition of adjacent properties
3 Stage underpinning and casting of supporting beams, in four phases
4 Stage excavation and casting of final vertical supports in four phases
5 Stage jacking and extension to piers
6 Removal of jacks and final underpinning
7 Final casting of ground floor slabs.

The internal shoring and bracing was installed prior to demolition due to the more exposed condition which the demolition creates. The bracing of *The Wellington* consists of two cross-braced steel towers for the full height, giving resistance to lateral loads only.

In *Sinclair's* the ground floor crosswalls and gable wall openings only were cross-braced and dead shored. The external walls were tied at ground floor level across the building's width during the 1.98 m deep underpinning of the *Shambles'* facade to give stability to the brickwork in this temporary condition.

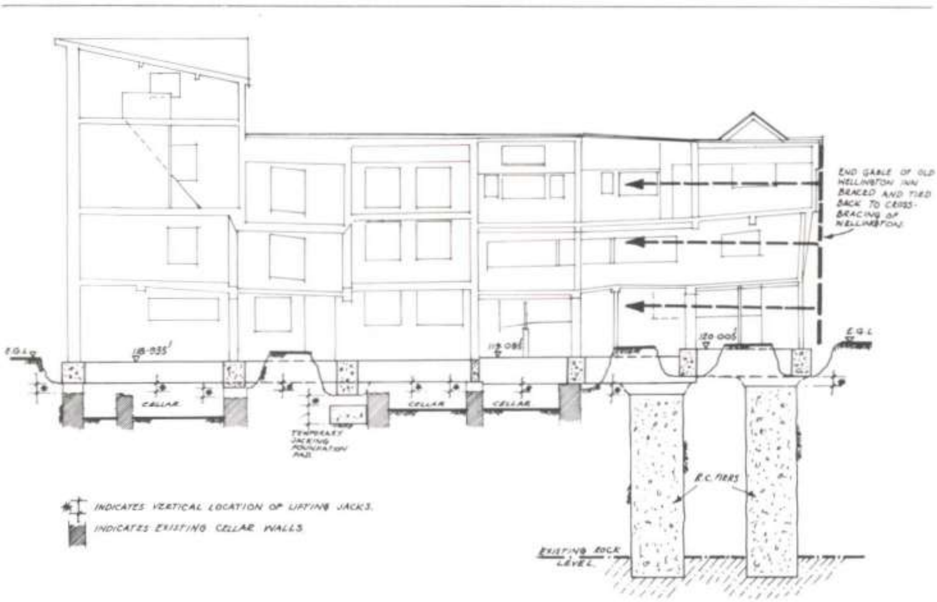
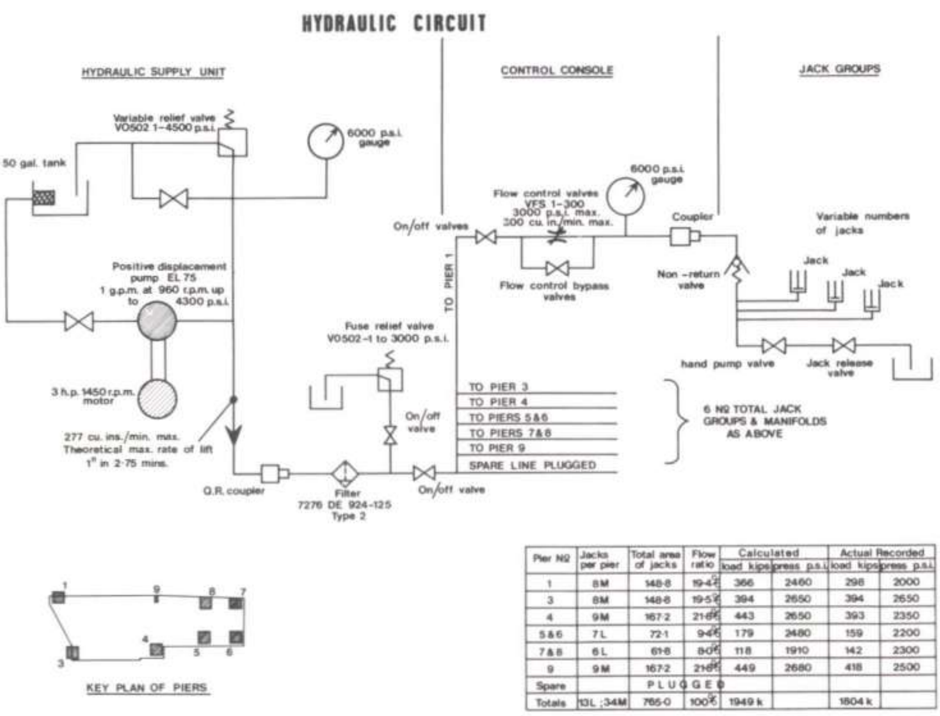


Fig. 6
Section AA, showing supporting structure before lifting operation



Pier No	Jacks per pier	Total area of jacks	Flow ratio	Calculated		Actual Recorded	
				load kips	press p.s.i.	load kips	press p.s.i.
1	8M	148·8	19·4%	366	2460	298	2000
3	8M	148·8	19·5%	394	2650	394	2650
4	9M	167·2	21·8%	443	2650	393	2350
5&6	7L	72·1	9·4%	179	2480	159	2200
7&8	6L	61·8	8·0%	118	1910	142	2300
9	9M	167·2	21·8%	449	2680	418	2500
Spare		PLUGGED					
Totals	13L ;34M	765·0	100%	1949 k		1804 k	

Fig. 7
Hydraulic circuit and plan of piers

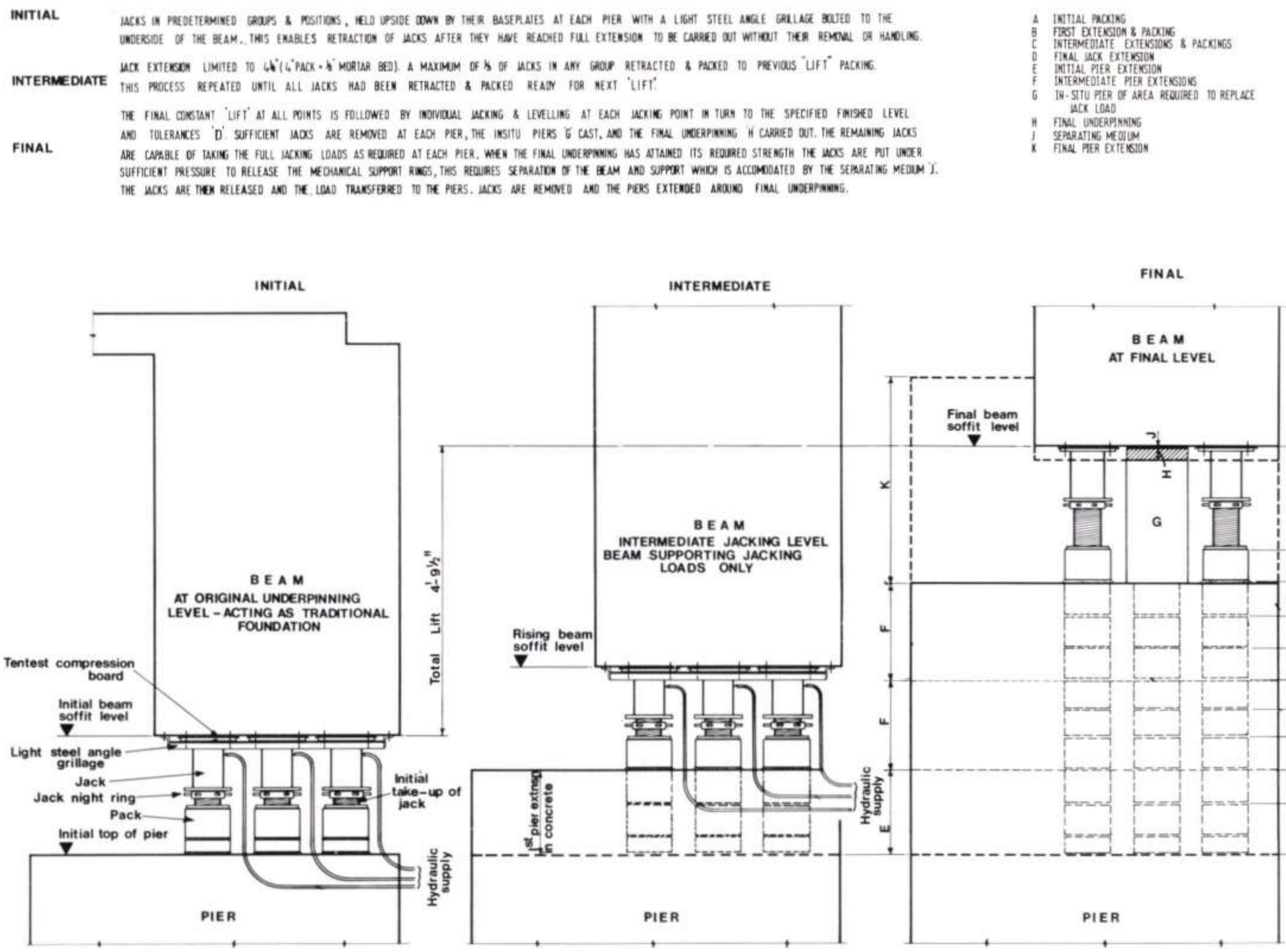


Fig. 8
Jacking sequence

Fig. 9
General Market Place Development: view prior to any contract work showing the site as it has basically been since the war (Photo: Stan Parkinson)

The demolition of immediately adjoining property followed on completion of the bracing, after which the underpinning works commenced. The underpinning was in four pre-planned stages ensuring overall and local stability at all times, each stage being completed before commencement of the one following. Lightly loaded posts were dead shored prior to being underpinned. Heavily loaded small lengths of brickwork and brick piers, where normal brickwork underpinning would cause unacceptable temporary stresses on the adjacent brickwork and foundations, were needled to relieve the load during the underpinning of that particular section. The work was done from within the old cellars where they existed.

There were two basic types of underpinning; the type used for standard depth beams being the contractor's standard, built-up, precast, concrete underpinning stool and, for underpinning beams in excess of 0.9 m deep, specially designed steel section underpinning stools. Both types of stool allowed reinforcement to pass through where required by the quantity of steel or by the variation in line of the supported brickwork relative to the centreline of the beam. Each stool is bedded onto a precast block, which gives the required cover to the steel stool. All underpinning was in pre-designed positions, the cover blocks being placed upon a separating medium at future beam soffit level which in turn rests upon a temporary underpinning foundation, or, where available and suitable, upon existing foundations or cellar walls. The location of stools and details of temporary foundations were such that pre-underpinning ground pressures were maintained as near as practically possible. Each stool was finished with a concrete cover block, the top of which was at the future beam top level, and the underpinning was completed with 1:3 dry mix mortar packed to the brickwork above.

All concrete for the underpinning beams was supplied ready-mixed and placed throughout by pump, due to the difficulties of access. The largest pour was in the 1.98 m deep *Shambles* façade beam of *Sinclair's*, which took 45 m^3. On completion of the underpinning beams, pier construction and jack installation were carried out in four stages, two piers at a time, remote from each other in order to minimize as far as possible the temporary increase in ground pressure accompanying, and adjacent to, pier excavation.

The excavation of piers was by hand down to a minimum of 305 mm into bunter sand-sandstone, or lower as required by the future main development contract sub-structures and drainage. Where the ground conditions were suitable, concrete was cast against the excavation with the exception of the top sections of each pier and the two piers on the party line of the buildings, which were required to be fair-faced. Each pier was cast up to the level required for jack installation. The jacks were located in predetermined groups and positions, and held upside down by their baseplates at each pier with a light steel angle grillage bolted to the underside of the beam. This method enabled jacks to be retracted when fully extended without the need to man-handle the jacks themselves.

A 13 mm pack of *Tentest* compression board was placed between the underside of the beams and the jack baseplates. When all jacks were in position in any one group the pier was extended up to a level approximately 150 mm below the jack, reinforcement being left projecting to provide continuity in subsequent pier extensions. 230 mm square precast concrete packs were placed and bedded into the spaces between the jacks and the extended pier. The jacks were then connected up to their group manifold and, using a hand pump, were extended to the top of the packs. Pressure was then applied up to 3.4 N/mm^2. This initial application of pressure was to compress and bed the *Tentest* board, to bleed and test the jacks and jacking circuit beyond the group manifold, and to relieve the temporary increase in local ground pressure associated with pier construction by taking load down the final support. Each jack has a screw-threaded mechanical stop called a night ring. This is used to guard against leaks or failure of the non-return valve when pumping ceases.

On completion of the piers and installation and pinning of the jacks, the remainder of the hydraulic jacking circuit and control was set up and bled free of air. The total hydraulic system was then tested. Power was supplied by a 1.5 kW motor driving a positive displacement pump, the rate of flow of the pump being 4.5 litres per minute at 960 rpm. At this flow the maximum theoretical pressure which could be developed by a 1.5 kW motor was 30 N/mm^2. The output was fed, via the main hydraulic line on-off valve, into a manifold with six hydraulic lines, one to each jacking point. Each of the six lines at the control panel

Fig. 10
Close-up of precast concrete underpinning stools (Photo: Stan Parkinson)

Fig. 11
Close-up of steel underpinning stools (Photo: Stan Parkinson)

had a line on-off valve followed by a flow control valve and its by-pass on-off valve for single line operation and finally a pressure gauge with a maximum capacity of 40N/mm². The rate of lift possible from the supply was 25 mm in $2\frac{3}{4}$ minutes. The flow in the line was however reduced by losses in the system and by use of the flow control valves, the actual average working flow achieved being 25 mm in 10-12 minutes.

At each of the six jacking points the supply was fed into a jack group manifold via a non-return valve. Each manifold had a hydraulic line to each jack in that group, the minimum being two and the maximum nine. At the end of each manifold were two valves for hand operation when levelling, and retraction and packing of jacks during lifting, one valve for the hand pump and one for venting off oil during retraction. At each jacking point a tape was rigidly fixed to the beams for levelling and control of lift.

Lifting procedure

The first lifting sequence was to release the beams from the ground stiction. The weight of the building and its new supporting beams had to be transferred totally from the ground, where the beams were acting simply as over-size footings, onto the lifting jacks, with the beams spanning freely. This was done by lifting all points together and measuring the extension at each jacking position simultaneously, the line to each of the six jacking positions being closed when a 3 mm lift could be measured, until all six lines were closed. The pressures during this initial lift exceeded later pressures, as anticipated, due to ground stiction. The pressures during the initial lift were restricted to the maximum beam reaction design pressures.

After all points had been raised a nominal 3 mm, the six permanent levelling tapes were checked in pairs using three fixed quickset levels and the five jacking points individually raised to the sixth and highest lift above datum. This process was repeated until the pressures of consecutive lifts were more consistent and it could be seen that the beams were all clear of the ground. (Five 3 mm lifts were used). The lifts were increased to 6 mm and the process repeated, including re-levelling, the flow control valve verniers being adjusted as part of the process, until the first full jack extension was reached. The jacks were then retracted and packed using a pre-designed sequence. This process was maintained for a further three 6 mm lifts following the initial full extension, but with the water level in operation. The latter was checked during these three lifts for accuracy and found to be functioning correctly. Lifts were then increased to 25 mm in 6 mm increments.

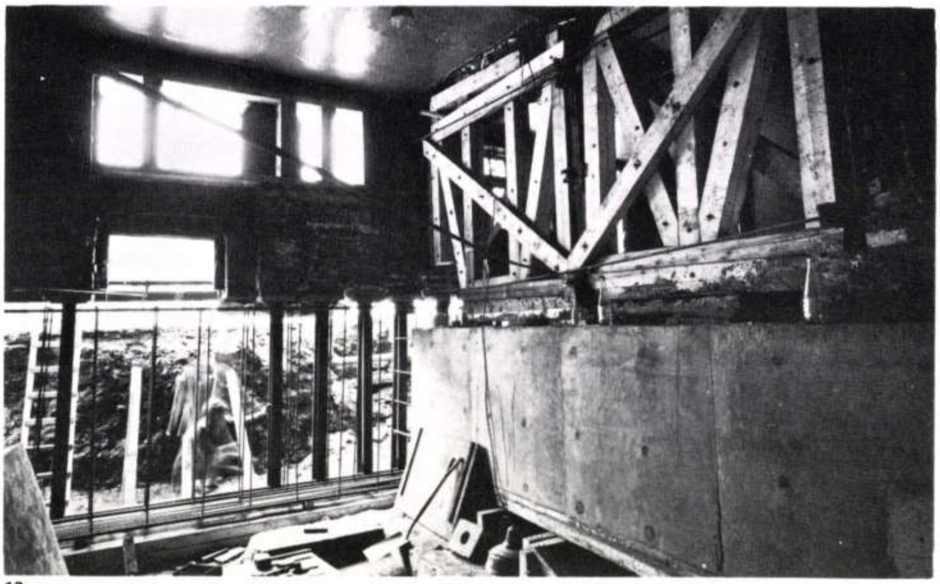
The water level

The water level had one vertical level tube fixed above each of the six jacking positions. This was supplied by two tubes, one down each of the long sides of the building, which were in turn connected to a control weir at a point away from the building.

The absolute water level is maintained by water constantly flowing over the weir, the level of which is manually controlled by a screw mechanism. The use of a weir eliminates problems associated with expansion or evaporation. The system is bled of air prior to use. Each of the six level tubes is rigidly fixed to the building alongside the levelling tape above each jacking point. Immersed into the tube are two electrical contacts completing a circuit containing a light. When the water falls below the level of the upper contact, the circuit is broken and the light goes out.

The water level acts as a control to the lifting in the following manner.

The upper contacts are lowered by means of a screw on a threaded rod until they just make contact with the water and complete the electrical circuit. All lights are now on. The weir is raised by the amount needed for the forthcoming lift and time is allowed for the water to flow through the system and come to rest. The amount of lift can then be seen as the distance between the level of water and the highest immersed contact. The jacks are operated and the building lifted. As the weir is away from the building the water level in each tube lowers in relation to the tube, staying constant with the weir, the actual change at each point being the actual lift at that point. As each upper contact breaks from the water the light at the position goes out and the on-off line valve to that point is closed. Hence the slowest point during lifting catches up with the fastest by continuing to



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14

Fig. 12 left
First stage underpinning
Wellington façade to Market Place complete and ready for reinforcing and concreting
(Photo: Martin Koretz)

Figs. 13 & 14
Third stage underpinning
Sinclair's façade to the *Old Shambles:* 2 m deep underpinning from inside with reinforcement being fixed. Fig. 13 shows cross beam second stage underpinning with formwork stripped and cross bracing and shoring.
(Photos: Martin Koretz)

close line valves until the last point has reached the required lift. Timing the closing of the valves gives the error in level during lifting which restricts the maximum following lift and indicates the necessary vernier adjustment on the flow control valves if an increase in lift is desired.

63 mm was the maximum continuous lift used but it was as efficient to maintain 25 mm lifts. The latter gave better control than that which was specified, and became the standard lift. The average rate was 10 minutes for 25 mm. The slight accumulation of variations in lift were corrected as required by taking levels at the six points after each retraction and packing sequence, and then levelling by jacking individual points.

The extension of jacks prior to retraction and re-packing was limited to approximately 115 mm, allowing for a 100 mm pack and mortar bed. A typical retraction and packing sequence was as follows.

Using a hand pump, sufficient pressure was fed into the jack group to enable the night rings on up to a maximum of $\frac{1}{3}$ of the jacks in that group to be released. The pressure was then released using the vent valve. By applying physical pressure, the free jacks were retracted, the jacks remaining being sufficient to carry the total load at that point, both under pressure and on their night rings. The new packs under the retracted jacks were bedded onto the previous pack and load applied again to the jack group by hand pump, thus loading up the new packs and enabling the next set of night rings to be released. This process was repeated at each jacking point until retraction and packing was complete. The next extension of jacks was then begun.

Where only two jacks formed the total number in a group, two additional independent jacks were used to carry the load whilst both lifting jacks were retracted and packed.

To maintain stability during jacking the number of packs allowed under extended jacks prior to stabilizing was limited to three. Therefore, each time a third set of unrestrained packs was placed a further section of pier was concreted.

Fig. 15
All jacks on pier location No. 1 in position and first three packed against pier.
Note Tentest board, angle support grillage, night rings, and continuity reinforcement.
(Photo: Martin Koretz)

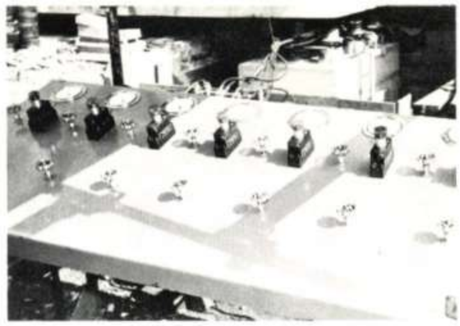

Fig. 16
Control panel in foreground showing a group of six lines, valves and gauges. In the background is pier No. 4 with its jack group and manifold, during jacking.
(Photo: Stan Parkinson)

Fig. 17
Water level control weir.
(Photo: Stan Parkinson)

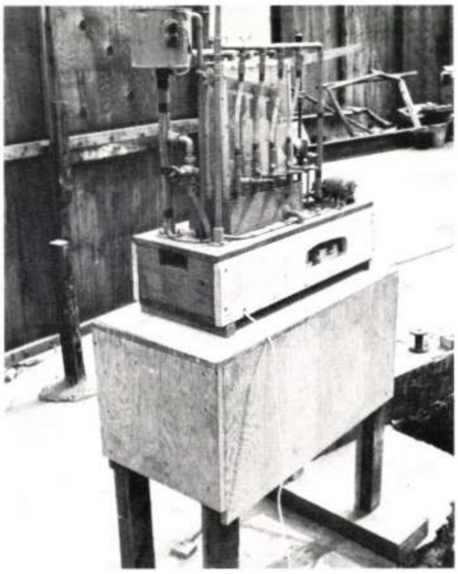

Fig. 18
Water level tube and control box for pier 8, and levelling tape.
(Photo: Stan Parkinson)

Fig. 19
View from north after main contract excavation had been completed
(Photo: Stan Parkinson)

The process of jack extension, retraction and packing, and pier extension was continued until the required lift of 1.46 m had been reached within the specified tolerances. Finally the levels were checked from the bench mark and the required adjustments to level were calculated for each pier position. Employing the hand pump the jacks were then used to finally correct all beam levels as required.

Sufficient jacks were then removed at each pier to enable final underpinning to be constructed. This underpinning consisted of an in situ pier cast in rapid hardening Portland cement of sufficient area to carry the full jacking load at each pier, cast to within 75 mm of the beam soffit. The gap was then dry-packed using a 1:3 mix beneath a separating layer of polythene. When this underpinning had attained its required strength sufficient pressure was put on the remaining jacks to allow their night rings to be released. The load was then transferred to the underpinning by releasing the pressure on the jacks, which were finally removed.

The final extension of the piers up to the beam and casting of remaining slabs followed, thereby containing the final underpinning.

Although the development will represent all that is most modern, functional and attractive in architecture, it will have an indissoluble link with the past.

The Old Shambles is to stay and those using the centre, whilst reflecting on the modernity of the city and the forward thinking of its people, will also be reminded of its centuries-old traditions.

Printed in England by Thompson & Tompkins Ltd.


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