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The new tower for the Independent Television Authority at Emley Moor, Yorkshire

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Synopsis

The paper describes the design and construction of the 330 m (1084 ft) high television tower built for the Independent Television Authority at Emley Moor in Yorkshire.

Account is given of the design process which was based on a statistical, dynamic ultimate load procedure. The problems of aerodynamic stability, and the related wind tunnel tests, are also discussed.

The method of construction, featuring the hoisting of the pre-assembled steel, aerial supporting mast to the top of the concrete tower, is described.

Introduction

Background

The collapse of the guyed structure at Emley Moor on 19 March 1969 resulted in the loss of both the ITA and the 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 (700 ft) 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 desired that the UHF transmissions from the new structure should begin 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 guyed mast solutions 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 superstructure*	*Total overall height*
213 m (700 ft)	55 m (180 ft)	268 m (880 ft)
244 m (800 ft)	55 m (180 ft)	299 m (980 ft)
274 m (900 ft)	55 m (180 ft)	329 m (1080 ft)

In each case, the steel superstructure to carry the aerials, was to consist of approximately 30·5 m (100 ft) of triangular section with 1·98 m (6 ft 6 in) sides and the top 24·4 m (80 ft) of triangular section with 990 mm (3 ft 3 in) sides. The whole of the aerial superstructure was to be surrounded by fibre glass cylinders of 3·66 m (12 ft) in diameter around the 1·98 m (6 ft 6 in) triangular section and 1·52 m (5 ft) in diameter around the 990 mm (3 ft 3 in) 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 (1080 ft) (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.

So as to save time it was decided to eliminate the period required for tendering and negotiate instead with

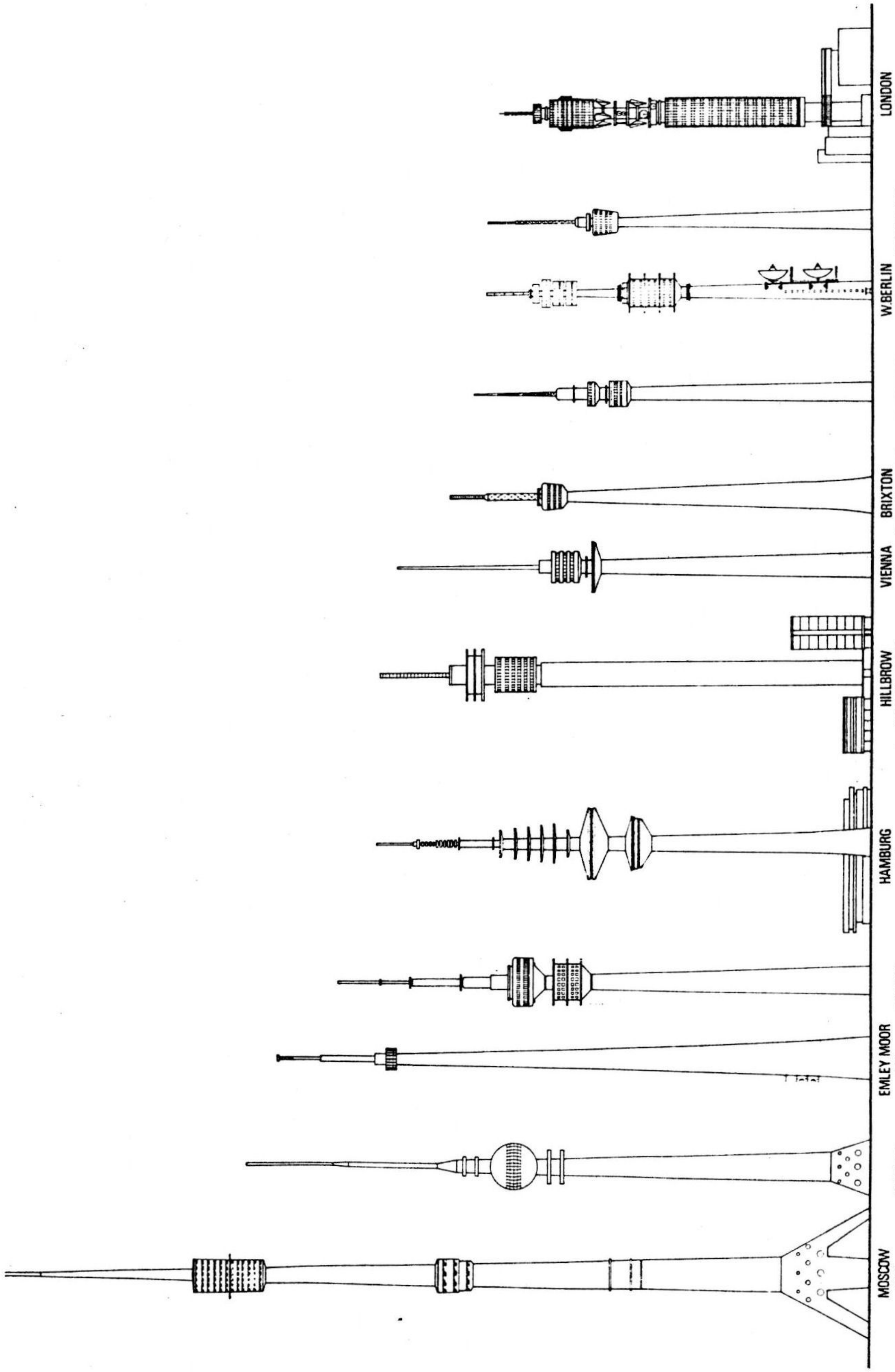

Fig 1. Emley Moor tower and other similar structures

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
Start 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

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 At approximately 30·5 m (100 ft) level the structure was to carry externally three 12 ft diameter dish aerials for the Post Office. The Post Office was also to be provided with an equipment room situated within the tower, at its base.

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 (200 ft) and 122 m (400 ft) 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 later.

Site conditions

A desk study of the local geology suggested that a 15 m (50 ft) thick stratum of hard sandstone would be found 3 m (10 ft) below ground with seams of Green Lane coal, New Hards coal and Wheatley Lime coal at various depths down to 65 m (213 ft). Two further coal seams were expected at 100 m (330 ft) and 150 m (490 ft) 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.

Description

Superstructure and foundations

The general arrangement of the structure is shown in Fig 4. The tower features a tubular concrete shell 274·32 m (900 ft) high surmounted by a steel aerial supporting mast which extends 56·08 m (184 ft) above the concrete. This is carried by two concrete slabs 457 mm (18 in) thick and 4·57 m (15 ft) apart, which transfer all of the bending and shear to the concrete shell by horizontal thrusts. The concrete shell is 24·38 m (80 ft) diameter at base with a wall thickness of 533 mm (21 in) and reduces to 6·40 m (21 ft) diameter at the top, where the wall is 350 mm ($13\frac{1}{2}$ in) thick. The profile, in the form of an exponential curve, was adapted to fit the method of

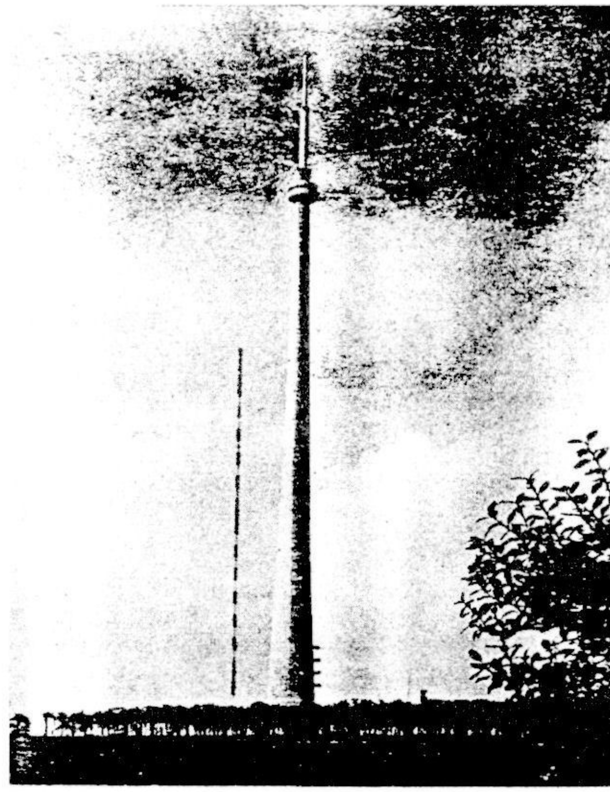

Fig 2. View of completed tower

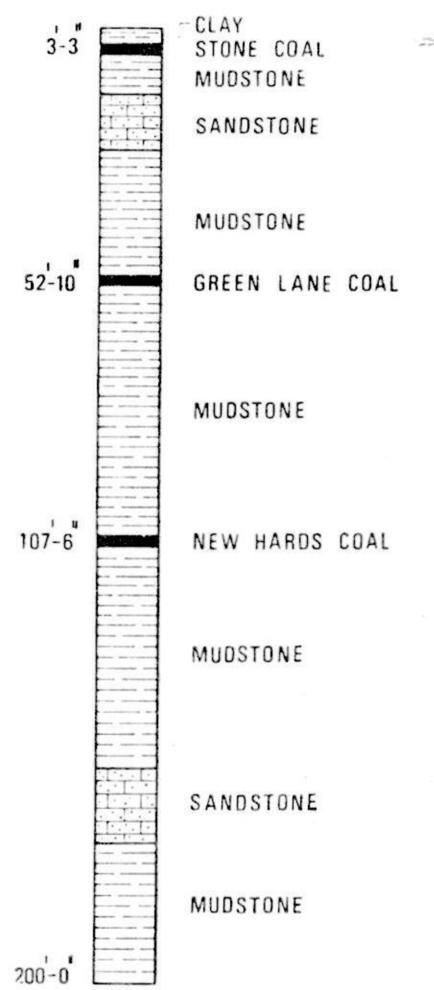

Fig 3. Geological succession

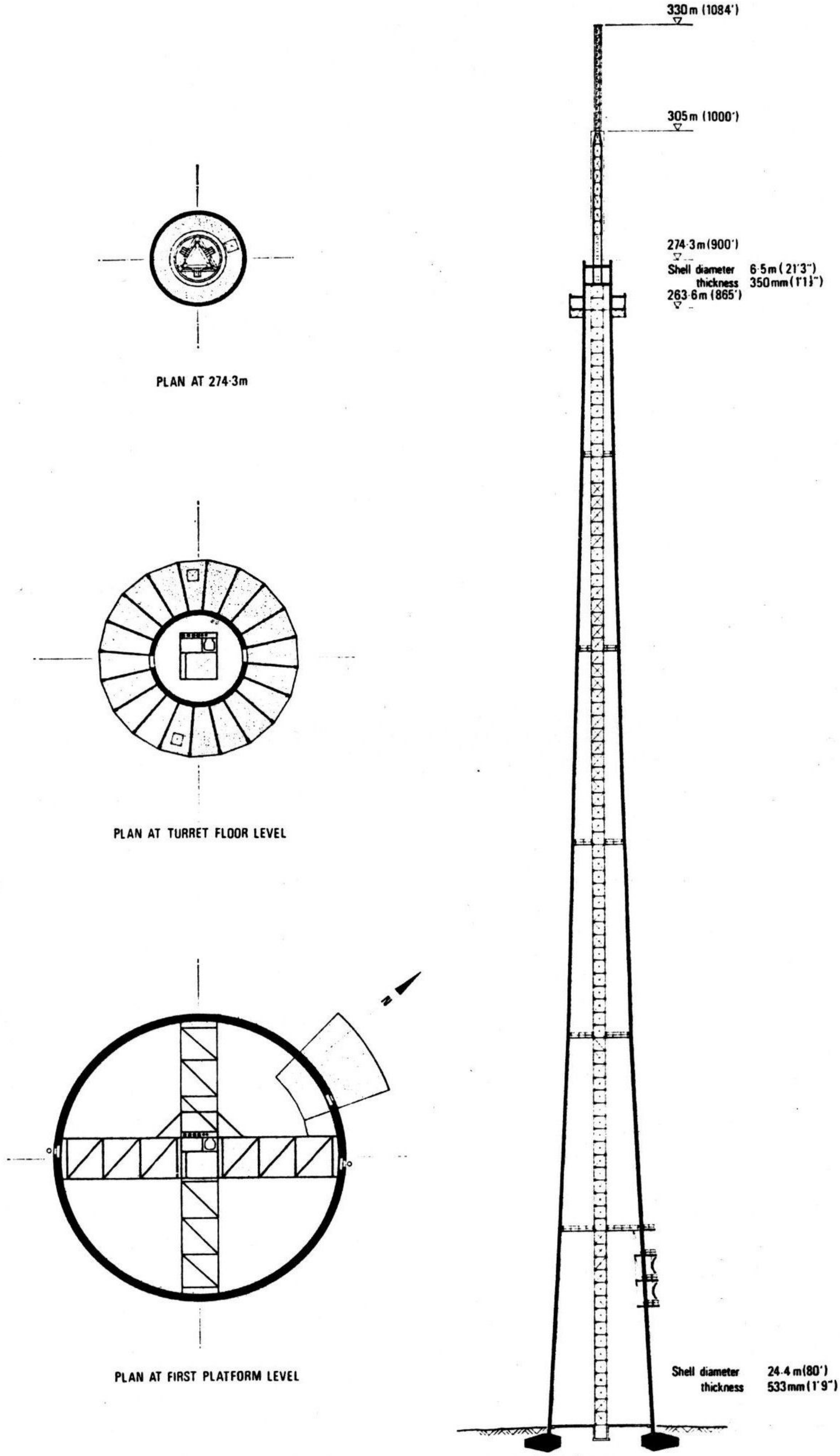

Fig 4. Cross section and plans showing the general arrangement of the tower

construction by fitting in a series of straight tapers. The foundation is an annulus 8·23 m (27 ft) wide and 4·27 m (14 ft) thick which bears on sandstone approximately 6·10 m (20 ft) below ground level. The specified minimum strength for the concrete of the shell wall was 40 N/mm^2 (6000 lbf/in^2), and for the foundation 26 N/mm^2 (3750 lbf/in^2).

The basic geometry of the aerial support mast was dictated by considerations of aerial performance and dimensions were therefore specified by the ITA. Comprising two triangular sections, the lower has a side of 2·21 m (7 ft 3 in) and extends 30·48 m (100 ft) above the concrete, and the upper is 25·60 m (84 ft) high with a side of 0·99 m (3 ft 3 in).

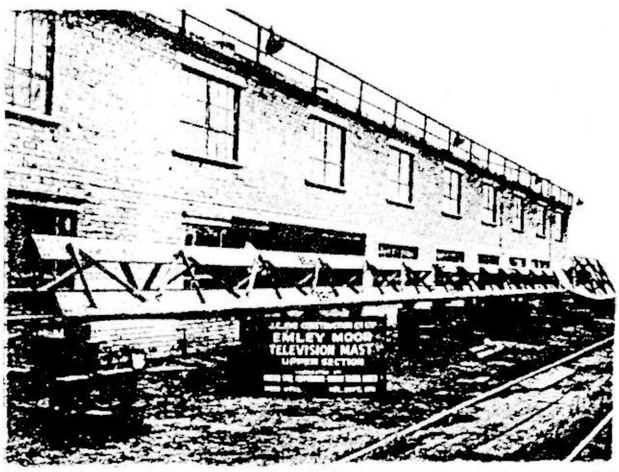

Fig 5. The aerial mast partially assembled during trial erection

The legs of the mast were bent from plate varying in thickness from 38 mm ($1\frac{1}{2}$ in) to 16 mm ($\frac{5}{8}$ in). The bottom 10·67 m (35 ft) 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 fabricators 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, areas surrounding the connections were cleaned by further blasting and then painted.

For the protection of the aerials and of men servicing them, the mast is enclosed in electrically translucent, cylindrical glass fibre reinforced plastic cladding 1·52 m (5 ft) diameter over the upper mast and 3·66 m (12 ft) 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 individually supported off 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, 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 (150 ft) the framework is braced to the concrete shell by platforms which also give access to the aircraft obstruction lights.

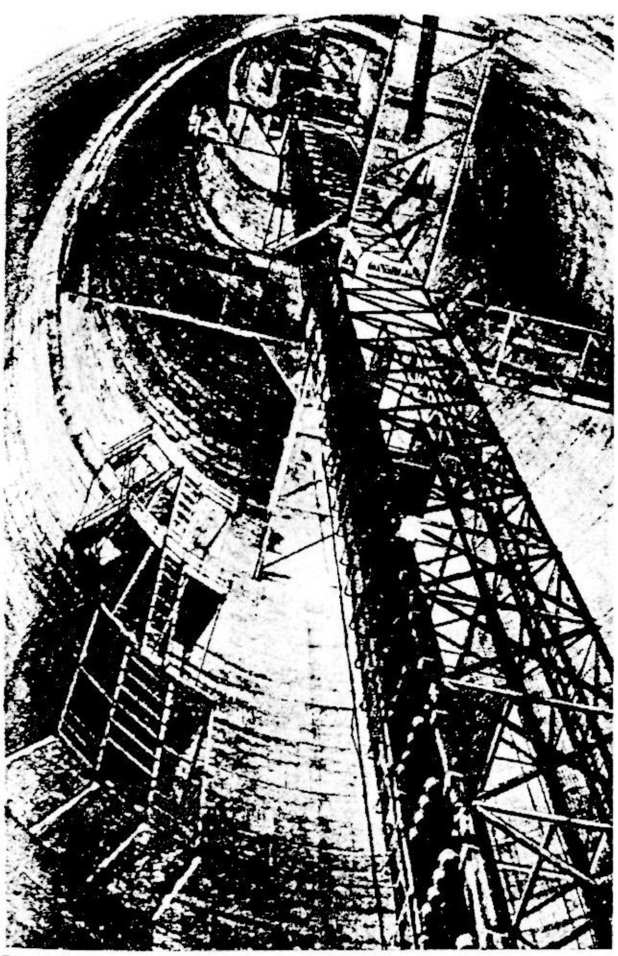

Fig 6. Lift cage and internal platforms

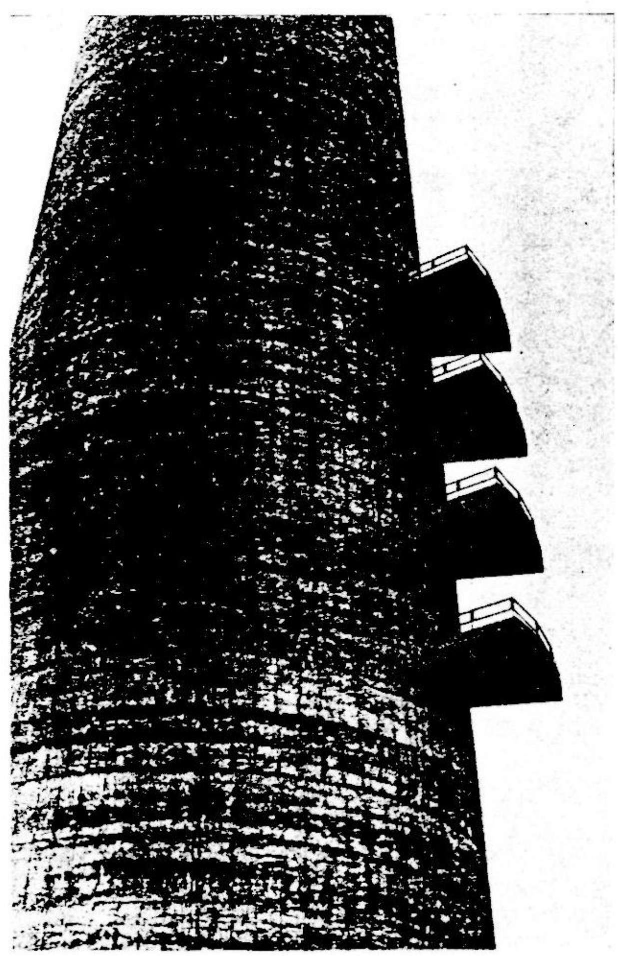

Fig 7. GPO platforms

Aerial platforms

Four external platforms near the base of the tower, as shown in Fig 7, will carry 3·66 m (12 ft) 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.

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 (6 ft) 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

Lightning protection is inherent in the structure, since the steel reinforcement effectively forms a Faraday cage.

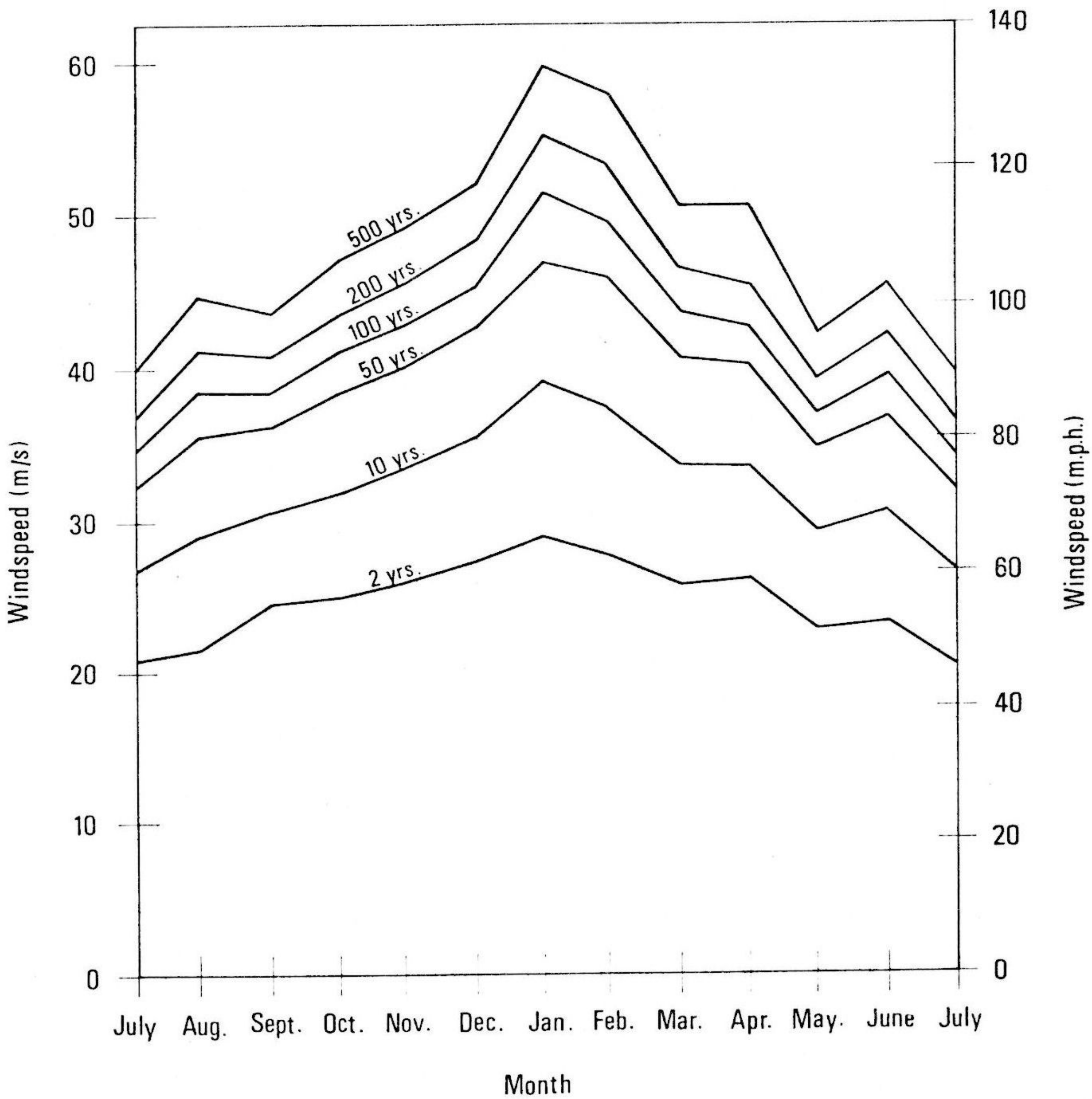

Fig 8. Plot of predicted monthly wind velocities

Peripheral coronal bands are provided outside the tower at vertical intervals of 91, 182 and 274 m (300, 600 and 900 ft) and additionally around the glass reinforced plastics aerial shroud. These bond the steel reinforcement, miscellaneous steelwork, doors, ladders, aircraft warning lights and the aerial mast into the protective system terminating in four driven rod electrodes outside 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 Post Office 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.

Design considerations

General
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. The more usual code type design if 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[1,2].

The decision to use an ultimate load method of design 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 (103 mile/h) as indicated by the basic windspeed map in CP3: Chapter V: Part 2: 1970.

Dynamic analysis procedure
The calculation of the structural responses, i.e. bending moments, direct forces, deflexions, 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 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 deflexions are the mean responses of the structure.
3. The structure is assumed to vibrate about the mean deflected position because of 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, deflexions, 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 one per cent 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 (82·5 mile/h) at ground level, the variation with height being taken to be represented by the one-sixth 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, lead to a comparable design basis for the entire structure.

The tower structure was analysed for this and other wind loading conditions of interest in the drag direction using static and nondeterministic dynamic numerical procedures, 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 inter-connected 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 Critical

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 deflexions, 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 V: 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 effect 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, because of the operational requirements of the aerials themselves

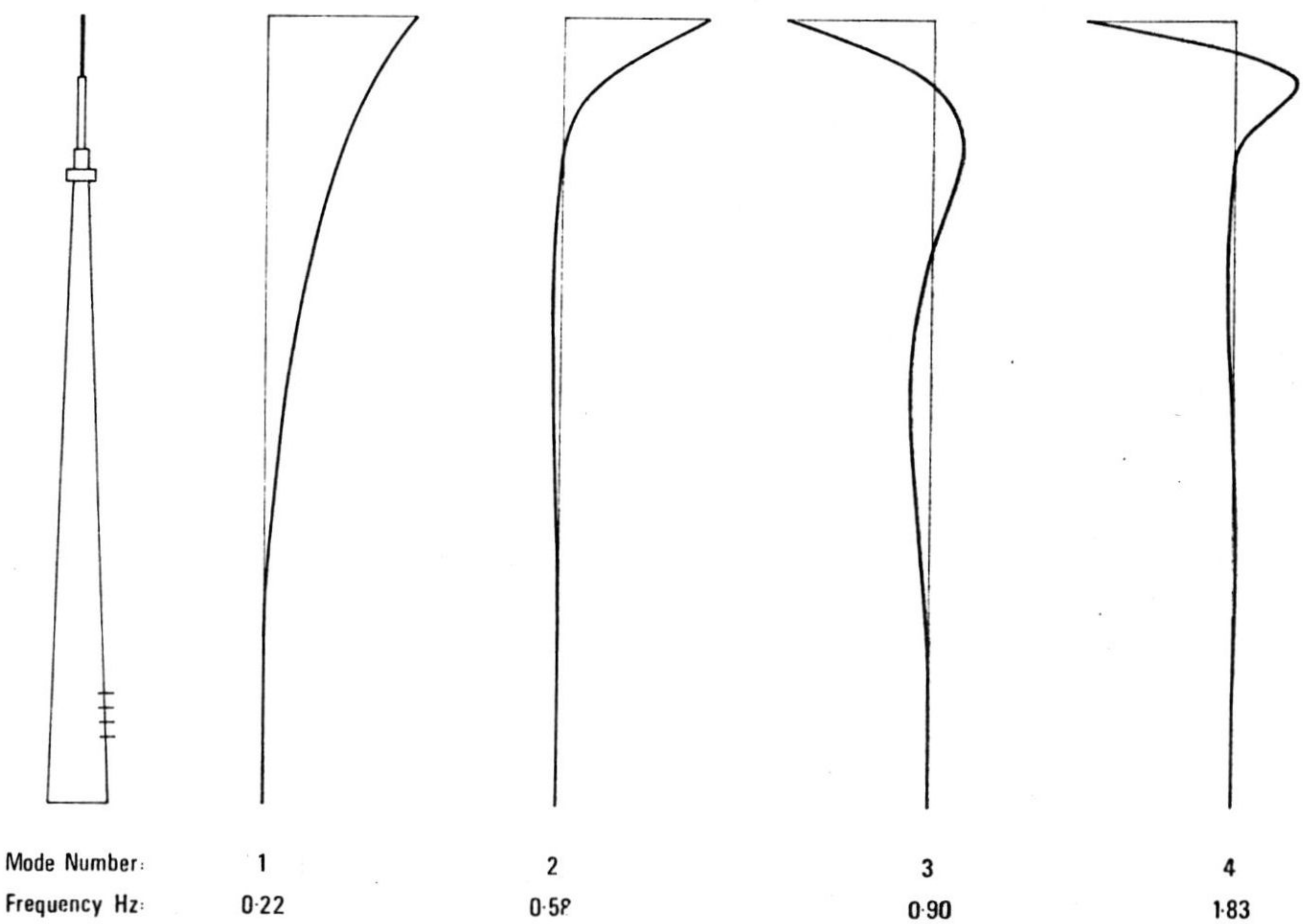

Fig 9. First four natural vibration mode shapes of the tower

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.

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 reduce effectively 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 so 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 surfaces 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, Weybridge, to simulate various freezing and thawing conditions. Sections of glass reinforced plastic cladding and concrete pipe (each 1·52 m (5 ft) 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 bedrock while minimizing the rock excavation, and sloping top surfaces to increase the concrete volume without the use of

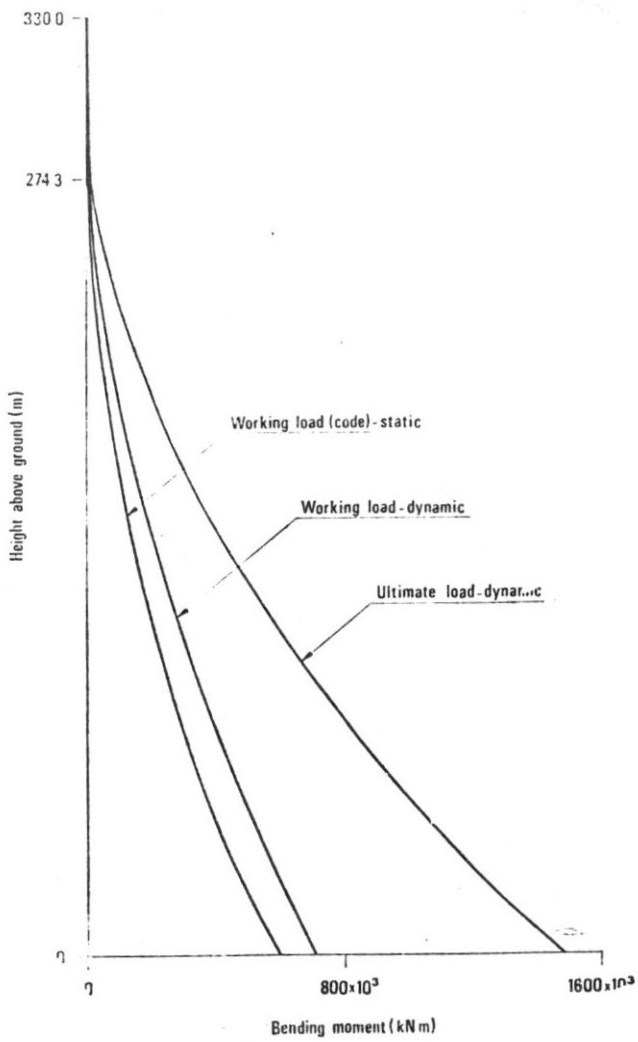

Fig 10. Distribution of bending moments

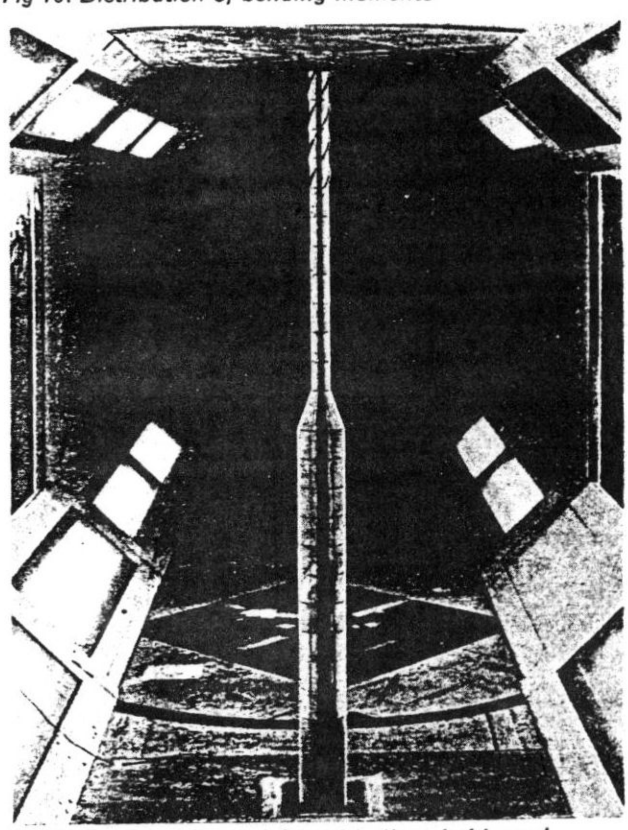

Fig 11. Model of the aerial mast in the wind tunnel

formwork, is placed eccentrically to the wall springing 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 windward toe pressure is zero;

b a load factor greater than 2 against overturning is achieved with a maximum leeward toe bearing pressure of 1190 KN/m² (25 K/ft²).

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.

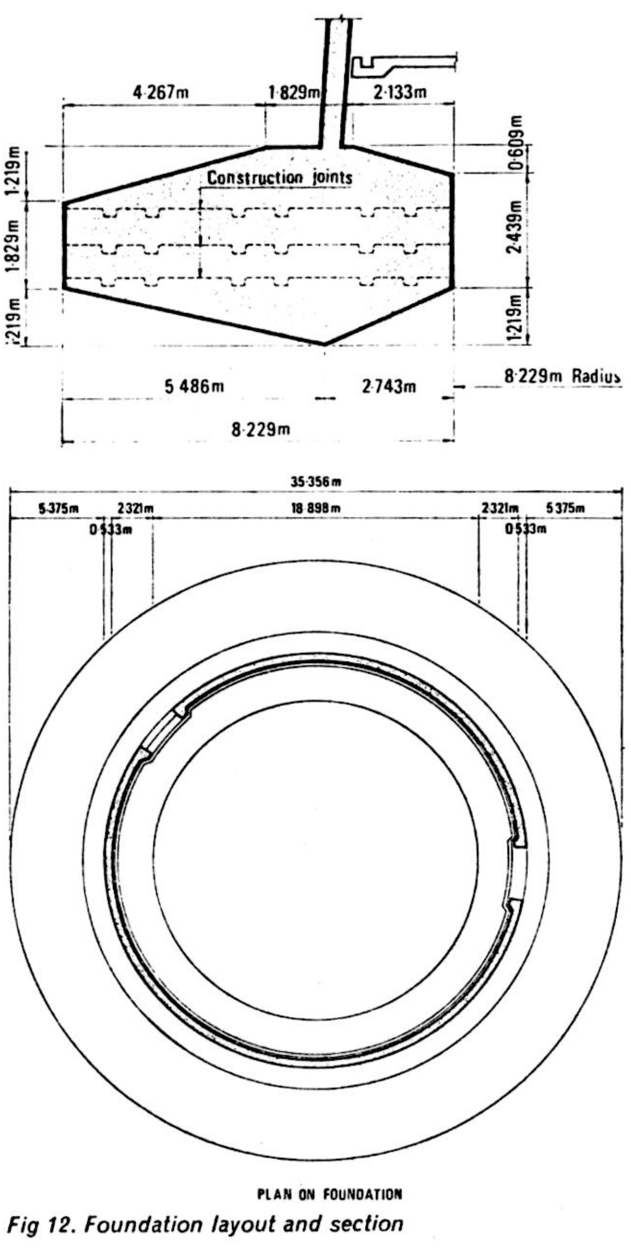

Fig 12. Foundation layout and section

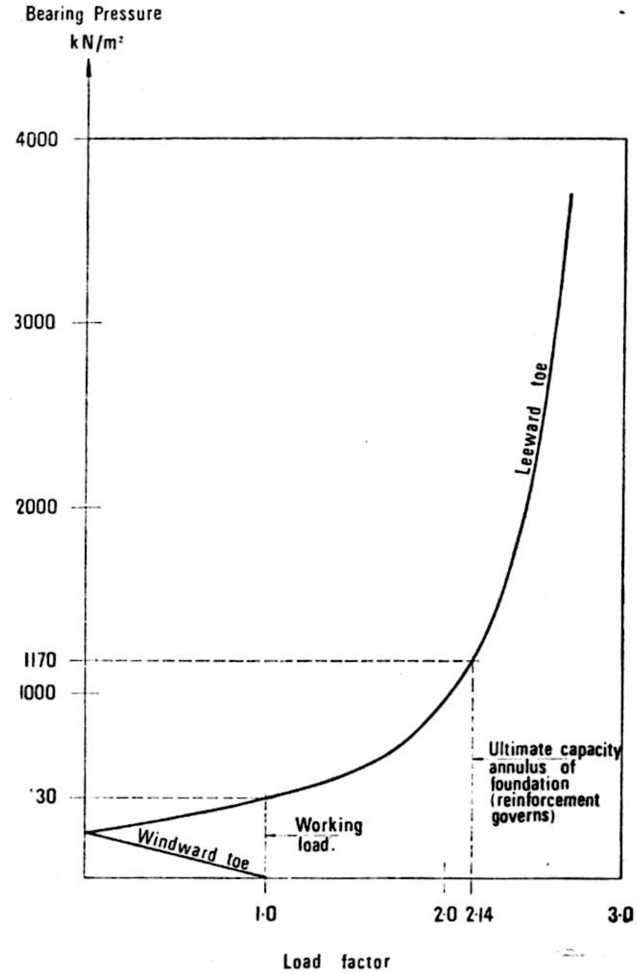

Fig 13. The variation of extreme bearing pressure and its relation to load factor

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 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 4·6 m (15 ft) 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 per cent 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 per cent was reached at a height of 128 m (420 ft).

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 because of 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 this steel from springing outwards because of 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 (2 in) and at the inner 40 mm (1½ in).

Glass reinforced plastics cladding
The data relating to the glass reinforced plastics (g.r.p.) 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 g.r.p. 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 18 in 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:Pt 1). Normally this is achieved by 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 because 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.

Construction

Foundations
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 mix concrete trucks to discharge the concrete directly into place. Lorry mounted pumps were used for the remainder. Fig 14 shows the foundation under construction.

Reinforced concrete shaft
The shaft was cast in lifts of 2·30 m (7 ft 6 in) 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.

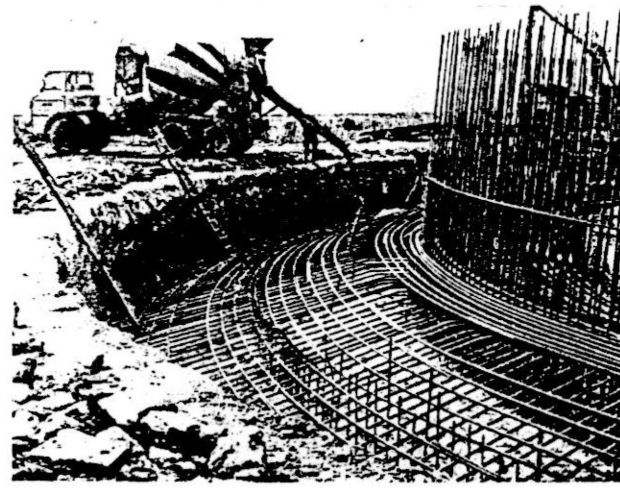

Fig 14. Foundation under construction

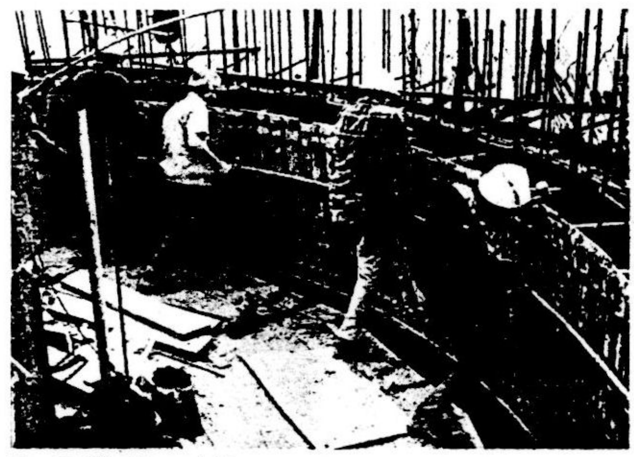

Fig 16. Working platform

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 (75·5 ft) of the shell was cast from a scaffold using ready mix concrete and lorry mounted pumps. From 23 m (75·5 ft) up to 46 m (151 ft) 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 60 working hours approximately 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 m^3 (9100 yd^3) of concrete and 660 tonnes (670 tons) of reinforcement. The lowest lifts of wall contained approximately 76 m^3 (100 yd^3) of concrete each and were poured at the rate of 9 m^3/hour (12 yd^3/hour). The topmost lifts contained approximately 15 m^3 (19 yd^3) each and the rate of pouring was 4 m^3/hour (5 yd^3/hour).

The centre of the tower was set out for each lift using an autoplumb stationed at ground and sighting an illuminated target. By the time construction reached 183 m (600 ft) 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.

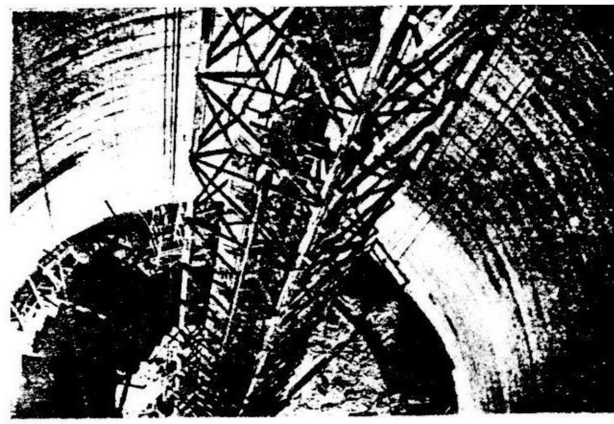

Fig 18. The aerial mast prior to the hoisting operation

Hoisting aerial mast

The mast was first assembled into 6·1 m (20 ft) 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 g.r.p. 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 (62 tons) and nine jacks were employed, three on each face of the mast mounted on the slab at 269 m (885 ft). The capacity of each jack was 12 tonnes (12 tons) 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

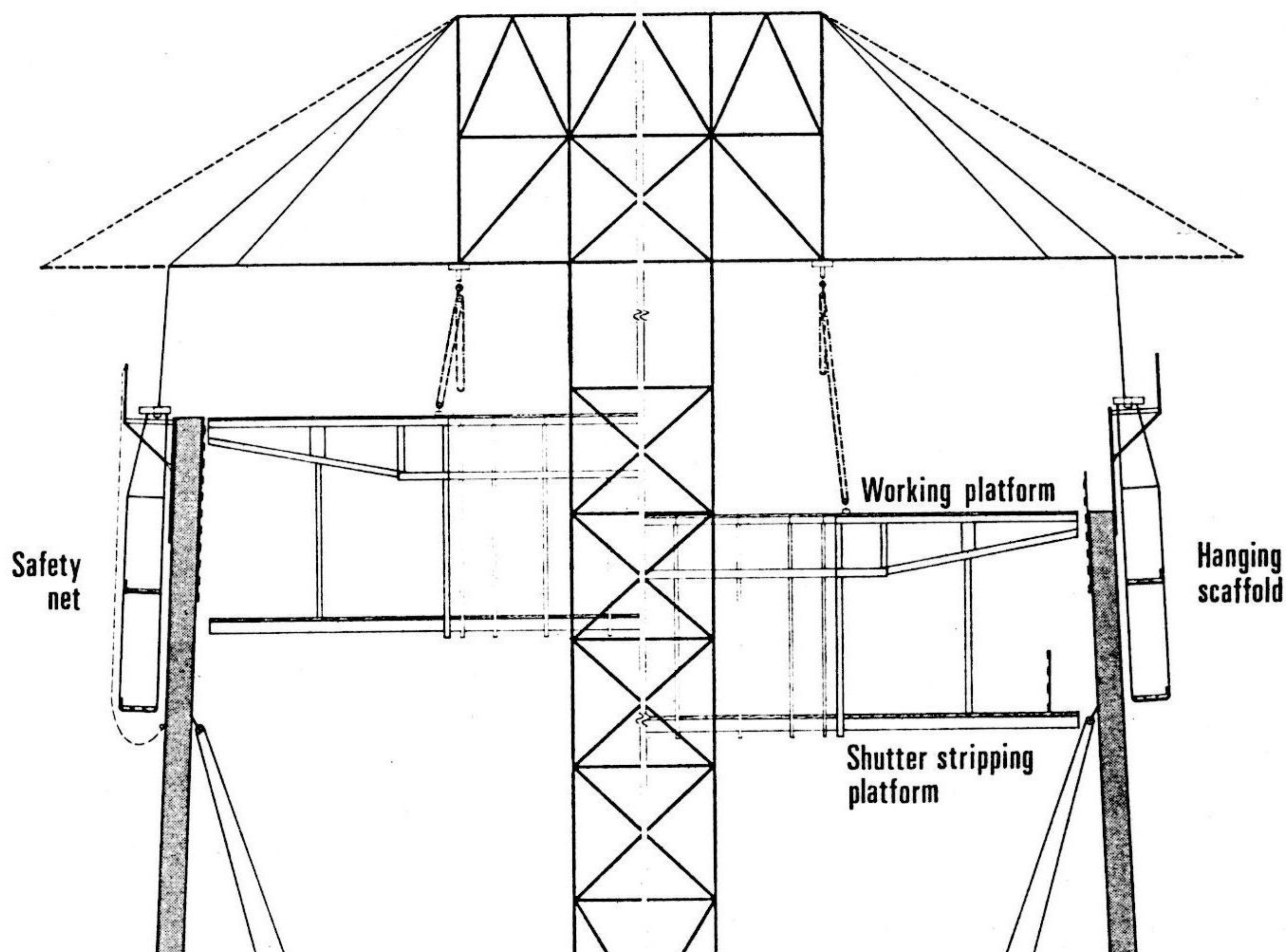

Fig 15. The general arrangement of the climbing derrick and working platform

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 mast and 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 g.r.p. cladding whilst the mast 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 (100 ft) projected and a steel transition section in the cladding was fitted below the 1·52 m (5 ft) diameter g.r.p. The following day the mast reached its final position and brackets were bolted to each face above the 273 m (900 ft) slab. The mast was lowered on to neoprene bearings, levelled 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 g.r.p. cladding were erected from outside 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.

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The 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.

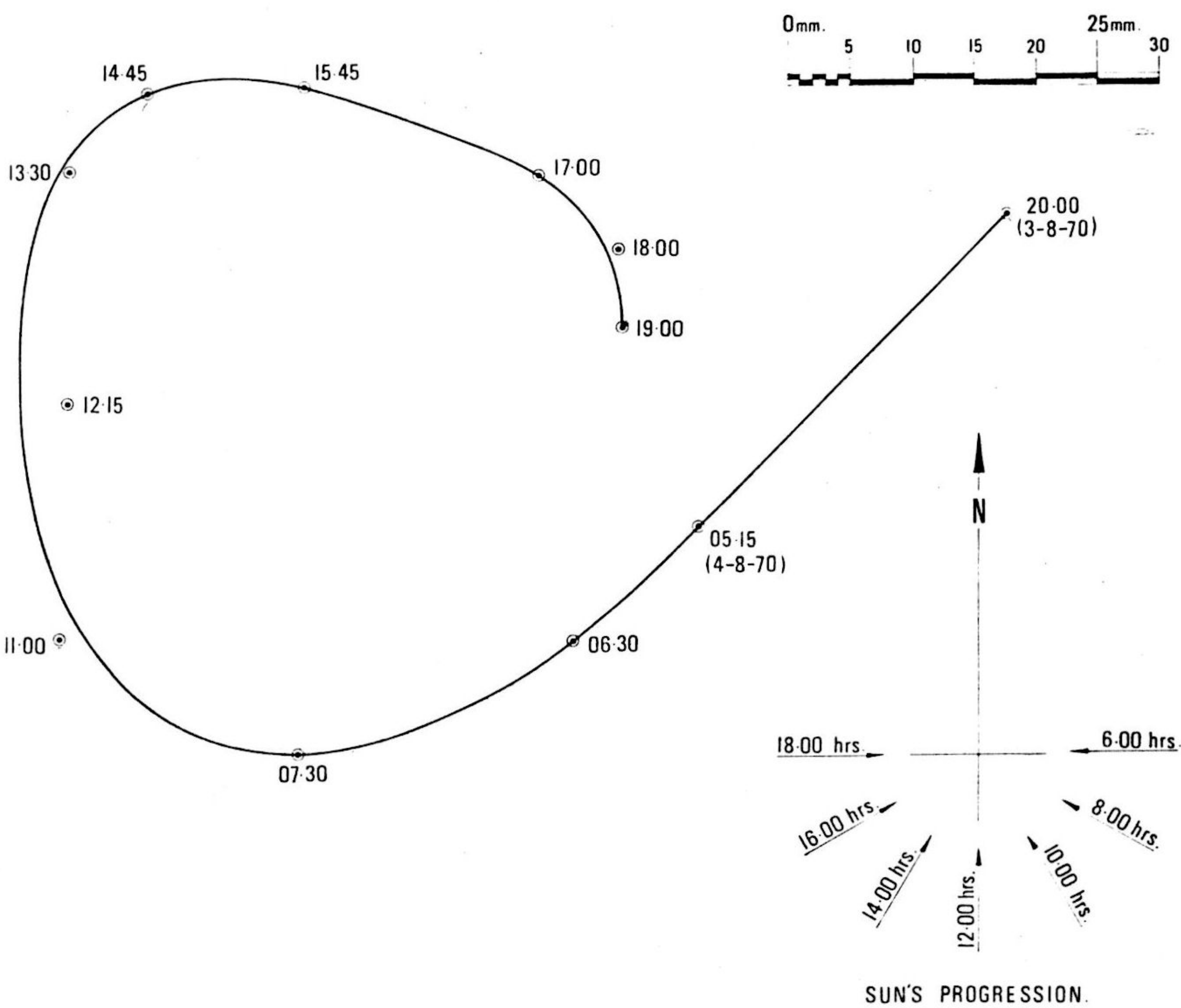

Fig 17. Plot of solar movement at 257 m (840 ft) level

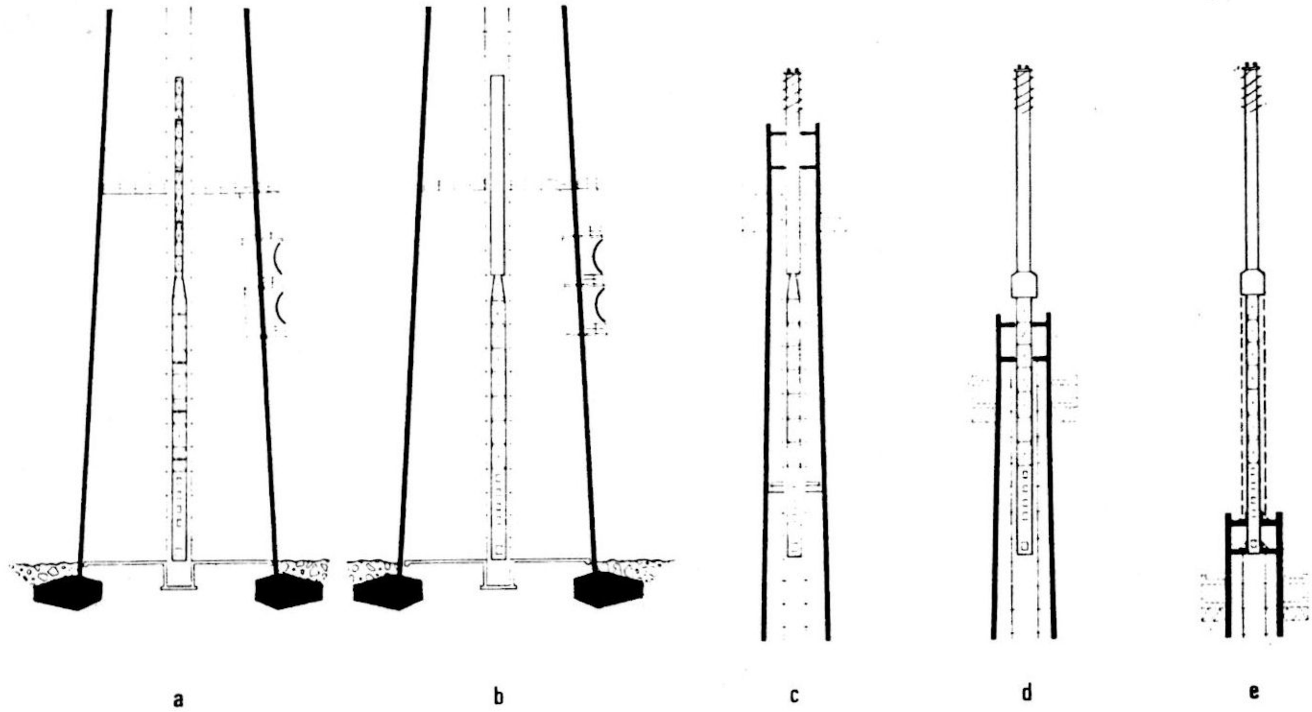

Fig 19. Diagrammatic illustration of the hoisting procedure

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TABLE A

Panel Data	UHF cladding	VHF cladding
No. of rings of panels	13	9
No. of panels per ring	3	6
Normal height of ring	1·88 m (6 ft 2 in)	3·05 m (10 ft)
Ring diameter	1·52m (5 ft)	3·66 m (12 ft)
Face thickness of panel	8 mm (0·3 in)	10 mm (0·4 in)
Depth of stiffening ribs	63.5 mm (2·5 in)	127 mm (5 in)
Depth of flanges	76 mm (3 in)	127 mm (5 in)
No. of ribs per panel	4	4

Mechanical properties of glass reinforced plastics laminate (BS 2782)

Tensile strength	125 N/mm^2 (18 000 lbf/in^2)
Crushing strength	103 N/mm^2 (15 000 lbf/in^2)
Cross-breaking strength	180 N/mm^2 (26 000 lbf/in^2)
Elastic modulus in bend	$5{\cdot}5 \times 10^3$ N/mm^2 ($0{\cdot}8 \times 10^6$ lbf/in^2)